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Racking Resistance of Bracing Walls in Low-Rise Buildings subject to Earthquake Attack (Volume 1 and 2)

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STUDY REPORT

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RACKING RESISTANCE OF BRACING WALLS IN LOW-RISE BUILDINGS SUBJECT TO EARTHQUAKE ATTACK

Volume 1

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This report outlines how wall bracing panels which develop slackness can rationally be used as a structural system to resist earthquake induced ground motion. It is published in 2 volumes. Volume 1 stands alone and contains all aspects and recommended amendments of both the test protocol and the evaluation method. Volume 2 contains the results and details of the experimental programme undertaken to justify these changes. Volume 1 is intended for use by technical advisers to building product manufacturers seeking to use their product as wall bracing elements within New Zealand houses. It provides the engineering rationale upon which a revision of the Wall Bracing Test and Evaluation Method, known as the BRANZ P21 test method, is proposed. As such it is also intended for use by Building Control Authorities, and Structural Engineers can utilise the bracing ratings derived for specific engineering design.

PREFACE

This study forms an investigation into the wind and earthquake racking resistance of timber framed bracing panels in New Zealand. It includes a revision to the current test protocol and a new test evaluation procedure. The latter is based upon a computer simulation procedure which visually matches the experimental elemental response to a mathematically equivalent element.

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Abstract

Lateral loads such as those produced by the effects of wind and earthquake can be resisted in buildings by cantilever action, by moment resisting frames, by shear walls, by diagonal bracing or a combination of these.

In New Zealand light timber frame construction, the resistance is provided entirely by shear walls. The total resistance of a wall is determined by summing the dependable strengths of individual full height panels located between openings. The standard method for assessing the racking resistance of wall bracing elements between openings, since 1978, has been the BRANZ P21 test. It has been known for some time that there are deficiencies with the P21 test and evaluation procedure with major problems being whether the test loading regime can adequately identify severely degrading elements, and in the assessment of wall ductility.

A detailed literature survey of wall racking tests carried out around the world and the factors which contribute to bracing panel behaviour is given. Taking this into account, a three phase experimental programme was carried out on bracing panels under various loading protocols, including monotonic and reverse cyclic loading. The end studs to the specimens were either fully held down with tie-rods, or restrained from uplift by the application of a vertical load or use of a partial restraint. A series of experiments was also carried out with no restraint to the end studs. The test specimens were lined with sheathings commonly found in New Zealand construction.

Both the onset of damage to the panels and the displacements at which a significant drop off in load occurred were investigated.

Methodology is presented in this report to enable an accurate computer model to be matched to the test element response. Once matched, the model may then be used to analyse the performance of the element under dynamic seismic loading and to generate seismic response spectra. The result from this analysis is quantification of the mass that the test panel can dependably restrain without the necessity to assess wall ductility.

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1. INTRODUCTION

The objective of this study is to review experimental methods used for determining the racking strengths of bracing panels, to critically review the current P21 test and evaluation procedure (Cooney and Collins 1978) and to identify any areas of deficiency. Finally a proposed revision of the P21 test procedure and evaluation is provided.

Lateral loads such as those produced by the effects of wind and earthquake are generally resisted in buildings by cantilever action, moment resisting frames, shear walls, diagonal bracing or a combination of the above. In New Zealand light timber frame (LTF) construction, the resistance to lateral load is provided entirely by shear walls. Within New Zealand, the magnitude of design actions for non-specific design are specified in NZS 3604 (SNZ 1990). These actions are resisted by the racking strength of these shear walls. The total resistance of a wall is determined by summing the dependable strengths of individual full height panels located between openings.

The standard method for assessing the racking resistance of wall bracing elements between openings, since the publication of NZS 3604 in 1978, has been the BRANZ P21 test. The procedure underwent a major change in 1990 to compliment the change in NZS 3604 (SNZ 1990) to a limit state format. Details of the (1991) P21 method are given in Section 12.1.1.

Several deficiencies with the P21 test and evaluation procedure have been recognised (Thurston and King 1992), the major ones being being the rational assessment of ductility in a degrading system, and also the degree of uplift restraint necessary to replicate the behaviour of the panel when it is an integral part of the wall. Both result in limiting the lateral load resisting capacity of strong ductile walls.

It was observed (Norton et al 1994) during the 1994 Northridge earthquake that a three-storey apartment building collapse was due to the failure of the main gypsum plasterboard lined shear walls, which carried high vertical load. Tests were carried out at BRANZ in order to simulate the earthquake loading on a replica of the apartment walls (King & Deam 1996). The failure observed was attributed to the separation of the wall linings consequential to pull-through of head failure of the fasteners once the wall had been loaded beyond its elastic limit. This resulted in sudden collapse. The current P21 test (King & Lim 1991) does not necessarily identify this limit of wall over strength capacity.

Thurston (1993) undertook a series of racking tests on long walls. He showed that the resistance of walls which contained large window openings was actually greater than that predicted using a standard P21 test configuration with supplementary end straps. He suggests that in such cases, the racking strength can be estimated assuming that the panel was fully restrained (i.e. no rigid body rotation) between openings. Conversely the actual strengths of panels between door openings were less than those using the P21 end restraint. Thurston recommended that end straps of 6 kN capacity be used where bracing panels terminated at or within two metres from a door opening.

1.1 Design Strength

Light timber framed buildings have been observed to perform well under severe earthquake attack. This can be attributed to the highly redundant nature of such buildings, the real contribution of non-bracing walls and the ability of the ceiling and floor diaphragms to distribute lateral load effectively between bracing lines.

In New Zealand the assessment of bracing resistance has been based upon mean test values, which appears satisfactory, when these effects are taken into consideration. If failure of an individual panel occurs, it is probable that load redistribution would occur and the excess load taken up between redundant members and bracing panels which may have over-strength capacity. Whilst such an approach is reasonable when there are many 'non-structural' walls present, problems may develop in the more open plan type house where bracing walls are fewer and shorter. In these instances each bracing element is stronger and will take high lateral load.

Failure of one such panel may be significant as high lateral loads would be expected to be redistributed to already highly loaded panels. The same degree of redundancy may not exist. Also, failure of a panel will produce additional forces on some of the remaining panels, due to torsion effects. The torsion effects are likely to be higher for small numbers of panels.

The question arises as to the determination of a design strength based upon the results of the destructive tests undertaken. Present (King & Lim 1991) procedure is to use the mean of six results (three test panels with data recorded in two directions). For future assessment it is proposed that each of the three matched test hysteresis loops (the weakest in each test) be evaluated through Phylmas (*see section 5.2.1*) and a restrained seismic mass for each determined. The design strength of the tested bracing panel is then the mean of the three results.

2. LITERATURE REVIEW SUMMARY

Racking tests are carried out to determine the resistance of bracing panels subjected to in-plane distortions such as those imposed by wind or seismic actions. In recent years a number of such tests have been developed around the world which have been adopted by the respective countries' approving authorities as a standard procedure for determining wall racking resistance.

A comprehensive literature review is given in Appendix B. It highlights the variations that exist in test methodologies and the difficulties there are in assessing the resistance of bracing panels in-service, from the results of panels tested in isolation.

The following outlines some of the key findings of the literature review:

- New Zealand is currently the only country which adopts a reverse cyclic test regime to determine the bracing ratings of panels to resist earthquake attack. The United States, for instance, use a monotonic loading regime, although the City of Los

Angeles is considering adopting a reverse cyclic test regime. In Japan a hybrid regime of monotonic and reverse cyclic loading is adopted.

- Criticism has been levelled at the monotonic load regime as a means of establishing a bracing rating of a panel subject to earthquake. This has led to studies in revising the protocol for the testing of bracing panels within the USA and Canada. Although these studies are incomplete, it is envisaged that specimens tested in these countries will be subject to loads based on 'Sequential Phased Displacement'. This consists of two load patterns, first to yield in which the elastic response is observed, and second post-yield in which the inelastic response is observed.

- 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 uplift restraint. This is presently achieved by using the standard P21 (1991) end restraint, which simulates the minimum restraint at the end of a bracing panel intersecting at a return wall. Other countries adopt either a full end stud hold down (Japan and the USA) by means of tie-rods or use a method which simulates as closely as possible the fixings used in-service (Australia and the U.K)

- The use of gypsum based plasterboard as a lining material for bracing walls has been questioned, especially since the 1994 Northridge earthquake. Current building codes within the USA allow gypsum plasterboard as a lining material for bracing panels, however bracing values are lower than similar bracing panels in New Zealand. Changes which are to be incorporated within a new (unified) Building Code in the USA will limit even further its use as a bracing material. New Zealand is therefore the only country in which gypsum plasterboard is allowed to act as the primary lining material for wall bracing elements which resist seismic attack.

- Generally it has been observed that little damage has been caused to houses as a result of inadequate wall bracing when subjected to wind or earthquake attack. More common forms of failure are due to poor sub-floor framing and/or inadequate fixings.

3. EXPERIMENTAL PROGRAMME

A comprehensive description of the experimental programmes and results are presented in Section 1 of Volume 2 of this report. The rationale behind the experimental programmes and the key findings are given below.

3.1 Experimental Phase I

This phase of the experimental programme was designed to determine an acceptable load/displacement protocol, one which adequately identifies the on-set of damage to timber bracing panels lined with degrading sheets. Different boundary conditions were used within the programme to help identify the most realistic method in which continuity effects can be simulated. Standard bracing panels with dimensions of 2.4 m high by 2.4 m long with a timber substrate were constructed and sheathed with a range of materials. Three identical panels were then each subjected to one of three load regimes, namely; monotonic, reverse cyclic under load control and reverse cyclic under displacement control. A further three panels were subjected to a reverse cyclic load regime with varying forms of end stud hold-down restraint. Finally a specimen 1.8 m long constructed with a door lintel at each end was tested in order to determine the restraint afforded by a lintel to the end of bracing panels.

Key Findings:

- The on-set of damage to timber bracing panels lined with degrading sheets can be identified by subjecting test specimens to increasing cyclic displacements.

- Using either a load controlled or displacement controlled load regime had no significant effect on either the maximum lateral load resisted or the related maximum reliable displacement attained.

- Degradation of test specimens is minimal under cyclic loading to loads up to 40% of peak load.

- The restraint afforded to a bracing panel terminating at a door opening is less than that achieved using the end restraint stipulated within the current P21 procedure (Cooney & Collins 1978 Amendment 2).

- 'Phylmas' analysis (refer Vol 2 section 3) indicated that the highest restrained mass on a bracing panel would be attributed to panels with a top plate gravity load, which partially counteracts end stud uplift. The panels which were restrained by the current P21 method could restrain greater mass than the equivalent panels which were restrained by tie-rods.

- The Serviceability Limit State (i.e. the on-set of cracking within the panel sheathing) in the lintel specimen was observed as beginning at lateral displacements of between 6 and 8 mm of the top edge of the panel.

3.2 Experimental Phase 2

The second experimental phase of the project was developed and conducted to further investigate and resolve some key issues that evolved during the first experimental phase, including the effect that panel length had on the seismic mass capable of being reliably supported. Panels were evaluated with enhanced end stud restraints (to simulate systems which end at taped and stopped wall junctions) and also with diminished end stud restraints (to simulate panels terminating at door lintels).

Panels evaluated within this phase of the project followed a loading protocol of increasing cyclic displacements. This was developed from the findings of Test Programme I and described in Volume 2 section 2.3.

Key Findings:

- The addition of the parent curves for panels W17 and W24 showed good correlation with the parent curve of the long wall tested by Thurston (1993). Also the sustainable seismic rating for the combination of Panels W17 and W24 was similar to that reported by Thurston for his long wall.

- The restraint afforded to a bracing panel by a standard door lintel, having no vertical load, is approximately 50% of that used in the current P21 test procedure.

- The reliable maximum displacements of short panels (1.2 m long) with the same sheathing were similar regardless of whether the end-stud restraint was provided using the standard P21 end stud restraint detail or a full tie-down rod restraint. The specimen sheathed with plywood achieved the greatest reliable displacement.

- The effect of bracing panel length on seismic rating was investigated and it was found that doubling the panel length increased the seismic rating by 190% (i.e. seismic rating ≈ panel length) provided the end stud is equally restrained.

4.0 DISCUSSION ON KEY ISSUES WITHIN THE DRAFT REVISION TO P21 WALL BRACING EXPERIMENTAL PROCEDURE

Concerns have been growing following recent post-earthquake experiences (Norton et al 1994) that the current P21 wall bracing rating procedure may not be identifying potentially brittle bracing systems. When the bracing panel is sheathed with a lining material which possesses sufficient strength to overcome the standard P21 restraint (resulting predominately in rigid body rotation of the panel) the panel has its assumed 'ductilty' based primarily on the arbitrary end-stud restraint prescribed in the P21 assessment method (Cooney & Collins 1978 with Amendments).

To overcome this problem it is proposed that a single specimen, having a full end restraint, is first subject to a monotonic load to failure and that the specimen must exhibit some ductile behaviour. A proposed revision to the Test Procedure, including the degree of ductility and how it is to be assessed is outlined in Appendix A.
This preliminary test will also be useful in:

- Determining the maximum loads and displacements to which the panel can be subjected.

- Determining the serviceability limit state with the on-set of cracking.

- Determining damage and failure mode.

4.1 Displacement Cycles

The purpose of the P21 racking test is to assign to a panel a racking strength which can be used in determining the total resistance of a building. It is therefore necessary to subject the test panel to a realistic load/displacement regime (i.e. one which represents wind and/or earthquake actions as closely as practical).

This is difficult since the actions imposed by wind and earthquake attack are fundamentally different, (wind action being essentially uni-directional and earthquake attack reverse cyclic). The reliable resistance of the bracing panel can reasonably be expected to also be different. However the industry is reluctant to bear the costs of evaluating systems under each loading case separately. It is therefore highly desirable that the loading regime applied to the panel be adjusted in such a way that both the wind and earthquake resistance can be assessed from the one set of results.

NZS 4203 (SNZ 1992) requires the performance of a structure to be considered at two limit states:

- Serviceability Limit State, in which the building is required to remain functional. Damage which requires repair should be avoided. The loads experienced at this limit state are expected to occur several times. Generally load associated with this limit state will have a 10 to 20 year return period.

- Ultimate Limit State, in which it is required to have sufficient strength to prevent collapse. Inelastic excursions, with residual damage to the structure may occur, but injury and loss of life is to be prevented. The load at this limit state is derived from an extreme event, having return periods of between 350 and 500 years.

4.2 Serviceability Limit State Cycles

Although both serviceability and ultimate limit states are required to be satisfied, the bracing ratings within NZS 3604:1990 (SNZ 1990) are derived from considering ultimate limit state loads only. The P21 evaluation method addresses this by requiring the ratio of resistance at the onset of damage (i.e. at serviceability limit state displacements) to the peak resistance (i.e. ultimate limit state) to exceed the ratio of limit state loads specified within the New Zealand loading standard, NZS 4203:1992. Where this ratio is not met, then the bracing rating is down rated to relate to serviceability limit state resistance rather than ultimate.

Experimental phase I involved consideration of a range of different wall linings. The onset of damage was never observed at displacements < H/400 for those systems, where H is the panel height. (The on-set of cracking at the lintel on specimen W17 occurred on the 6 mm, i.e. $^{H}/400$ cycle). This is consistent with BRANZ experience with other materials also. This experience is reflected by King (1996) who suggests serviceability limits for in-plane deflections of $^{H}/400$ for gypsum or plasterboard wall linings and $^{H}/600$ for masonry.

The deflection at serviceability limit state should be chosen carefully to reflect the possible damage to linings and the consequence of excessive lateral deflections on the other parts of the structure.

BRANZ has tested most systems in common use within New Zealand houses. Observation of these tests confirms that response is purely elastic (i.e. no degradation) at loads less than 0.4 of the ultimate load (Pu). Non-linear behaviour (i.e. the onset of damage) is typically observed at loads of approximately 0.6 Pu. It therefore seems reasonable to remove the smaller displacement cycles where the response is purely elastic as a means of simplifying the test procedure.

The draft revision to the test method proposes that the specimen be cycled under displacement control, so that the top plate is subjected to displacements of ±6 mm three times. A fourth cycle in one direction will be encountered during the first cycle of the ultimate limit state sequence.

4.3 Ultimate Limit State Cycles

Wall bracing panels are required to resist lateral loads generated either by severe wind storms or from earthquake induced ground motion. Whilst both these load cases impose lateral load, they are fundamentally different in nature.

Extreme wind loads are generally unidirectional in nature, although they fluctuate in intensity due to wind turbulence. Code compliance is satisfied provided the peak resistance of a bracing panel exceeds the loads resulting from severe wind loads the intensity of which is specified as the ultimate limit state load case for wind from the New Zealand loading standard, NZS 4203:1992. The current P21 evaluation method requires the specimen to be loaded in its weakest direction and imposes a 10% reduction on the peak resistance experienced as an allowance for the wind turbulent effect.

In contrast, earthquake induced loads impose full load reversal as dynamic shaking occurs. For rare events of this intensity, damage to the building is expected, but collapse is required to be prevented. The current P21 method assigns the dependable resistance to earthquake induced loads as being the average resistance experienced after three excursions to a nominated displacement. It provides no guidance as to what this displacement should be. The latest amendment removed previous acceptable ratios between the third cycle resistance and peak resistance (third cycle resistance to be not more 80% of the peak).

The draft revision proposes to subject the specimen to three load reversals at a series of incremental displacements with the intention of each specimen experiencing between three and four such displacement series before failure. An envelope of the third cycle residual resistance levels encountered at each cycle can then be developed with the maximum reliable displacement (MRD) being that imposed immediately prior to a resistance of 90% of the maximum peak resistance, this being deemed to be 'system failure'. If the loss of resistance is sudden (i.e. increasing from less than 90% to less than 75%, then the MRD is to be considered that experienced two cycles earlier.

In the draft revision, the loading protocol proposes that the displacement increments for the ultimate limit state assessment:

$$(\Delta max - \Delta s)/4$$

where Δmax = Displacement at peak monotonic load
Δs = Serviceability limit state displacement

Pre-peak cycles aim to replicate the response of the specimen when it is subjected to lower intensity earthquakes or to less severe ground motion from more distant severe earthquakes. As such they are applied to the test specimens as a form or preconditioning whereby the specimen response during the more intense incremental degradation cycles is more representative of in-service behaviour.

4.4 Number of Load Cycles

NZS 4203 (SNZ 1992) permits large reductions in the lateral seismic coefficient when structural systems have 'adequate ductility'. The code commentary expands this prescribing that the building as a whole should have the capability of deflecting laterally through at least eight load reversals without the lateral load carrying capacity being reduced by more than 20%.

The current P21 test method subjects each specimen to four displacement cycles at an undefined displacement limit intended to represent the ultimate limit state displacement. The dynamic displacement/time plot (Figure 1) for Panel W9 subject to NZA artificial earthquake record as derived from the NZS 4203 elastic response spectra. It can be seen that between three to five cycles result in displacements which are within 20% of the maximum displacement experienced. Inspection of many such displacement/time plots of other specimens show similar trends.

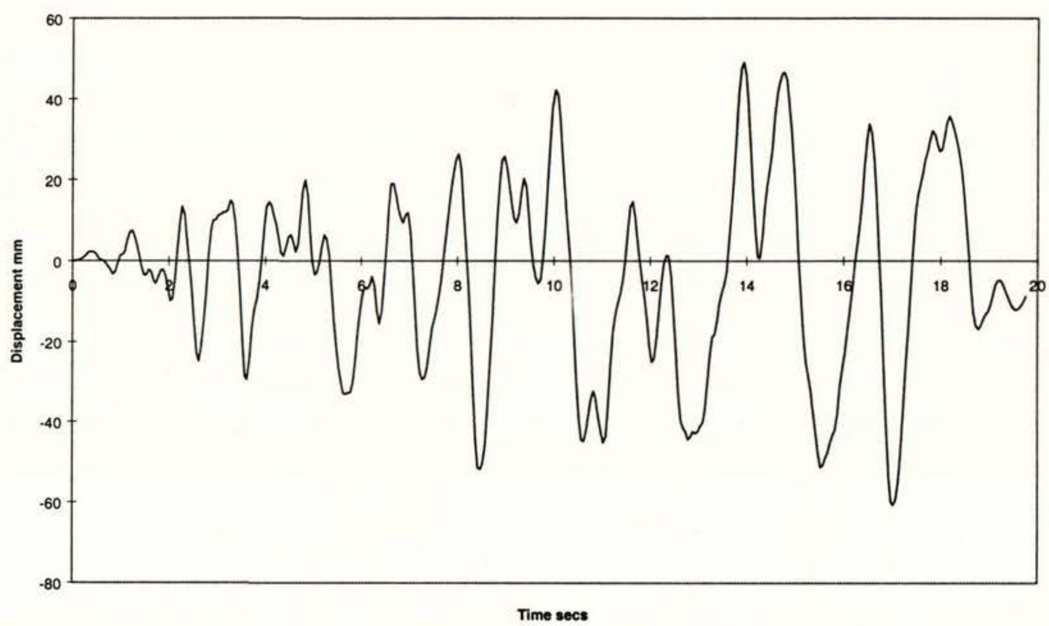

Figure 1 : Dynamic Displacement/Time for Specimen W9

Regarding the appropriate number of reversals appropriate for each displacement increment, the reduction in resistance experienced decreases with each cycle to the extent that the resistance observed during the third cycle is approximately the same as that observed during the fourth cycle. The draft revision to the P21 procedure proposes to retain the requirements of imposing three cycles to each displacement and to use the resistance experienced during the third cycle as the basis of determining the maximum reliable displacement.

4.5 Boundary Conditions

4.5.1. End Stud Uplift

It has been shown (refer Vol 2 section 4.2.1.1) that the uplift restraint afforded to bracing panels terminating at a wall return of length of 1.2 m, is equivalent to 12 kN, provided that the wall joints are stopped and taped. If they are not stopped and taped then the restraint provided by three nails in shear, as stipulated in the current P21 method (King & Lim 1991), is appropriate. This configuration is detailed in Figure 2 and is hereafter called the 'Current P21 restraint'.

Where panels terminate at a door opening, the restraint afforded by the lintel is approximately 3 kN (i.e. about half of that attributed to the current P21 restraint).

Although changing the degree of end restraint to replicate the wall behaviour at the lintel is simple (e.g. reducing the number of nails from three to one or two) some consideration as to the likely occurrence of this particular lower bound condition is necessary.

It leads to the question of whether the difference in the lintel and P21 type restraint (of approximately 3 kN) can be attributed to systems effects.

The standard method of construction for laterally supporting a top plate is to nail fix it at the intersection with framing members, such as floor joists, ceiling joists etc. If the framing members run parallel to the bracing panel, blocking pieces are fixed between the framing members and at right angles to the panel, at no more than 2.5 m centres.

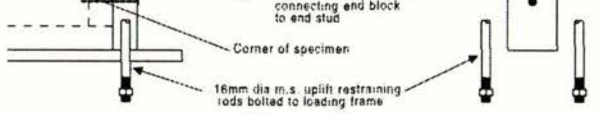

Figure 2 : Typical P21 End stud uplift restraint (for timber framed walls)

Taking the case of the lower storey in a house, the gravity load provided by the upper floor would generally be sufficient to offset the difference between the current P21 restraint and the lintel restraint. Where floor joists run parallel to the bracing panel, the top plate of the wall is supported by the blocking pieces and any uplift in the end of the panel would immediately attract additional gravity load. In some cases the bracing panel would also be supporting upper storey load bearing or non load bearing partitions. Similarly bracing walls in an upper storey or in single storey houses will at least have the top plate supported against face loading by blocking pieces. In such cases the weight of the ceiling would add to the panel self weight to provide resistance against uplift.

The draft revision therefore proposes to continue with the use of the current P21 end restraint to simulate continuity effects within light timber framed dwellings. If straps or any other ancillary tie downs are to be used in-service then they are to be included in the specimen configuration. Where the ancillary end stud hold down restraint consists of light gauge steel straps, these may be omitted in-service when there is a minimum length 1.2 metre length of wall beyond the extend of the bracing panel. Similarly panels which terminate at a free end or at a return of less than 1.2 metres require a hold down device of 6 kN capacity.

4.5.2. Bottom Plate to Foundation

The current P21 procedure prescribes that each specimen is be installed upon framed timber floors with the minimum bottom plate nailing pattern (as specified in Appendix A of NZS 3604:1990) when their bracing rating is being evaluated. This has been assumed as being a lower bound condition with the results being applied to panels installed on concrete slabs. In the latter case, the bottom plate was firmly fastened to the slab with little or no relaxation being possible at this location.

As part of the revision to the evaluation procedure, a more realistic assessment of the true panel response is input into a time-history analysis using the Phylmas procedure outlined in the draft revision to the evaluation procedure. A key finding from this work is that the Reliably Restrained Seismic Mass (RRSM) which a test panel can sustain is derived from the period coincident with the Maximum Reliable Displacement (MRD). Generally the RRMS increases in proportion with the MRD.

Thus, contrary to previous expectations, some systems which are more rigidly restrained may result in a lower Maximum Reliable Displacement and thus a lesser Reliably Restrained Seismic Mass. The ratio of RRSM for systems fixed to either concrete floors or to timber framed floors ranges from 1.1 to 0.8. The key component appears to be the relative strength and stiffness of the sheathing/fixing/substrate. It is thus impractical to predetermine a lower bound configuration until the material properties are known.

The draft revision suggests that either a preliminary test is undertaken to determine whether the bottom plate hold-down requirement is the lower bound result, or the conservative approach of a fully rigid bottom plate restraint be applied. A rational extension is to determine RRSM for each type of floor system.

The proposed test procedure is given in Appendix A.

5. DISCUSSION ON KEY ISSUES RELATING TO THE EVALUATION PROCEDURE OF THE LATERAL RESISTANCE RATING

The proposed test evaluation process fundamentally differs from that currently used, in that it is displacement based, utilising the Phylmas computer programme, rather than being force based.

The proposed method may prove to be too onerous for general use by testing agencies and a 'black box' approach is presently being undertaken in which it will be necessary only to feed in relevant data from test hysteresis loops to obtain the bracing ratings. However should the testing agency wish to carry out the loop matching and time history analysis phase then this option will also be available. To this end the effects that each of the different parameters within Phylmas have on the restrainable mass have been examined and a method of determining this mass without running Phylmas proposed. This information will be made available at a later date.

The draft of an Evaluation Procedure is outlined in Appendix A.

5.1 Determination of Dependable Resistance Under Wind Loads

5.1.1 <u>Ultimate Limit State Compliance - Wind Load</u>

The wind bracing demands published in NZS 3604 (SNZ 1990) were derived wind speeds published in DZ 4203 (SNZ 1988), being the then current draft of the 1992 revision of the Loading Standard later published as NZS 4203:1992 (SNZ 1992). The wind section of both the draft and finally the Standard was based extensively upon the Australian Wind Design Standard AS1170 Part 2 (SA 1989). Both the Australian Standard and DZ 4203 prescribe the ultimate limit state wind loads which correspond to wind speeds having a 5% probability of exceedance during a the building life of 50 years (i.e. a return period of 950 years).

Between the 1990 draft and the 1992 publication of NZS 4203 a limit state multiplier, M_{ls} was introduced which effectively brought the wind loads to be those which have been assessed as having a return period of 350 years for ultimate limit state. Thus the wind load was brought nearly into line with the return period of the ultimate limit state intensity earthquake load of 450 years. Thus wind zones specified within NZS 3604:1990 should be amended to bring them directly into line with the wind loads specified in NZS 4203. This needs to be addressed within any future revision of NZS 3604.

In each case the design wind loads are based on dynamic pressures based upon a peak 3 second gust (SA 1989).

The present P21 test and evaluation method imposes a strength reduction factor of 0.9 on the peak resistance attained under monotonic loading. This was introduced as an allowance for some strength loss associated with previous load excursions which may approach the ultimate limit state loading. This reserve no longer appears necessary since the probability of events which are likely to result in loads which approach the ULS intensity loads is sufficiently remote that it can be ignored. Furthermore the reserve strength present in other non-structural elements is expected to remain and provide supplementary resistance in these rare events. In addition, the draft experimental phase subjects the specimen to numerous cycles prior to reaching the peak test load, and sheet degradation has already occurred. The draft revision to the evaluation procedure therefore proposes that the maximum peak load experienced be used as the ultimate resistance of the panel subjected to wind load (i.e. the strength reduction factor equals 1.0 for ULS wind resistance).

5.1.2 Serviceability Limit State Compliance under Wind Load

For wind loads, the current P21 (1991) bracing ratings evaluation procedure requires that the serviceability limit state resistance to monotonic (wind) action be greater than 0.563 of the ultimate limit state (wind load) resistance. This was derived on the basis that the ratio of wind speeds between serviceability and ultimate limit state conditions was 0.75 and that thus the ratio of wind pressures, being proportional to the wind speed squared was $0.75^2 = 0.563$. These ratios were extracted from DZ 4203:1988 (SNZ 1988) which was available unofficially at the time the 1990 edition of NZS 3604 was being prepared.

Late changes to the draft however introduced an ultimate limit state multiplier for wind loads of 0.93. The correct modification ratio for wind pressures between limit states should therefore be 0.65 (being $(0.75/0.93)^2$). The draft revision to the bracing rating evaluation with respect to wind addressed this issue and recommends that the serviceability limit state resistance to wind be greater than 0.65, that assigned to ultimate limit state wind resistance.

5.2 Determination of Dependable Resistance under Earthquake Induced Actions

5.2.1 Ultimate Limit State Compliance

The fundamental drive behind this review is the recognition that real bracing systems used in houses do not display a bilinear elasto-plastic response when they are cycled beyond their elastic limits. Thus the engineering concepts of ductility and energy absorption break down and cannot realistically be applied to degrading systems. Yet it is known that such systems performance admirably under damaging earthquakes. The deficiency is therefore considered to be within the engineering models used to evaluate the reliable resistance of the system rather than the ability of the system to adequately resist earthquake induced actions.

The draft evaluation method proposes the following approach be applied as the basis of determining the seismic mass which may be dependably restrained by the system in question:

1. Develop an electronic element which has the degradation characteristics which are the same as that experienced by the laboratory specimen when subjected to reverse cyclic loading,

2. Install this element into a computer model and subject this model to a suite of simulated earthquake ground motion using engineering time-history analysis techniques.

3. Use the resulting most demanding displacement response spectra, in combination with the maximum displacement the experiment specimen was able to sustain without significant strength loss, as the basis of determining the maximum reliable period of the system.

4. Use the simple kinematic relationship between period and mass to translate this to a dependable mass capable of being sustained. This mass is to be used as the earthquake rating of the system under consideration.

The engine to accomplish this analysis is contained within a computer package specifically tailored for this task. Three phases of operation are included within this package.

1. An electronic equivalent model of the system under consideration requires to be developed. This is achieved by displaying first one of the load displacement plots of a specimen tested under full load reversal. Ten parametric adjustment features are available using slide switches. Each adjustment result changes the display of the electronic replica. This process continues until there is a close visual match between the results derived during the experimental phase and the electronic replica.

2. The electronic element is used as the basis for analysing the response of the system to simulated earthquake induced ground motion. Time history analysis techniques are used to impose this motion onto the element which, through the electronic match achieved above will exhibit degradation characteristics similar to those observed during the laboratory evaluation. Whilst several ground motion records will be available, the record derived from the design spectra presented in NZS 4203:1992 will usually be the most demanding and therefore control the response of the system.

3. Determine the ultimate limit state seismic mass, Mu, which can reliably be resisted by the specimen under consideration without exceeding the maximum reliable displacement when subjected to the simulated earthquake motion. The process is automated to provide the analysts with the solution directly (the default approach) but can also return either a force or displacement response spectrum if requested. The most demanding displacement response spectra will be used to determine the fundamental period of vibration at which the specimen remains within the maximum reliable displacement observed within the laboratory. The mass which results in a simple single mass oscillator of similar initial stiffness to respond at this period is considered as the maximum seismic mass which can be sustained. This mass is converted to Bracing Units (using the relationship 20 Bracing Units = 1 kN) and thus brought into line with the units of resistance used within NZS 3604:1990.

4. The analysis package used for the evaluation is based on the Phylmas (Pinched Hystertic Loop Matching Analysis System) developed by Deam (Deam 1994) modified to allow direct application as part of this evaluation technique. While several earthquake time history records will be included, the NZA artificial earthquake record derived by Carr (Andriono and Carr 1991) has been found to most commonly control and is usually used to determine the seismic mass upon which the earthquake rating is derived. Seismic mass ratings were derived as part of the various experimental programmes applied during the course of this research.

The results generally show an apparent reduction in the bracing ratings for panels evaluated using the new draft provisions compared with those derived using the current provisions.

There are several reasons for this namely:

1. The earthquake demands published within NZS 3604 (SNZ 1990) were derived from the response spectra contained in draft DZ 4203:1988 (SNZ 1988) assuming the fundamental period of the building was 0.6 seconds. This assumes that a lateral displacement of 60 mm can be reliably sustained by a mass supported on a 2.4 m stud wall. The experimental programme associated with this research (and reported in Volume 2), together with similar commercial work undertaken to determine system bracing rating on commercial systems, clearly indicate that most systems are unable to attain even half of this displacement without significant loss of resistance. Fundamental periods of between 0.2 and 0.3 seconds are much more likely (i.e. maximum reliable displacements of between 25 and 32 mm).

2. The NZA earthquake record was derived by Andriono and Carr at the University of Canterbury (Andriono and Carr 1991) from the elastic response spectrum for normal soil sites published within the draft New Zealand Loadings Standard, DZ 4203 (SNZ 1988). This spectra when used in combination with the system response factor, Sp, has remained unchanged and is now contained with the NZ Loading Standard, NZS 4203:1992 (SANZ, 1992). The spectra was derived using a uniform risk approach considering a suite of many earthquake records and a modified Katayama attenuation relationship to account for distant events. Although not strictly an upper bound envelope, the published spectra contains a much higher energy content than is present within any individual actual earthquake record and is thus usually more demanding than any individual earthquake record.

3. If a real earthquake record is used, for instance El Centro 1940 then there is generally a reduction in period and hence an increase in mass restrained. Inspection of the generated time-history plots shows that this mass increase is approximately 10%.

A more realistic earthquake record is presently being formulated at BRANZ which will overcome this anomaly.

A damping ratio of 5% was used throughout the test result evaluation. The effect of damping on the acceleration response spectra can be considerable, especially at the short period end of the spectrum, i.e. the period which is of concern for the tested bracing panels.

Figure 3 and Table 1 shows the effect of damping ratios on the displacement spectra for Panel W12. It is evident that if realistic masses are to be determined then a realistic evaluation of damping is necessary.

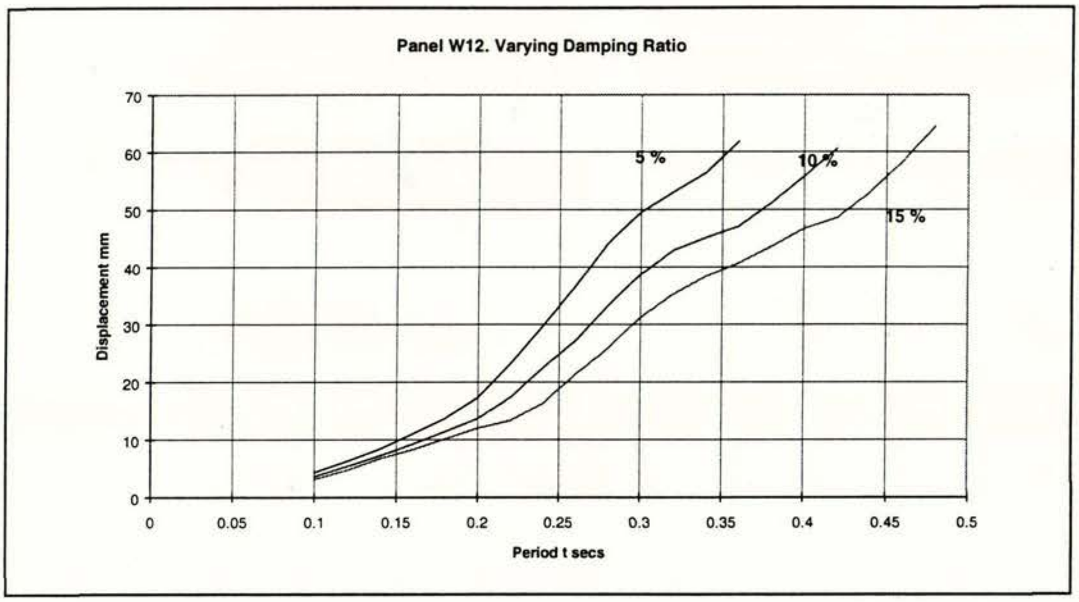

Figure 3 : Effect of Damping Ratio on Period

	Damping Ratio		
Displacement	5 %	10 %	15 %
10 mm	1400 kg (100)	1750kg (125)	1970 kg (140)
20 mm	2650 kg (100)	3210 kg (121)	4100 kg (155)
30 mm	3500 kg (100)	4430 kg (127)	5100 kg (146)
40 mm	4390 kg (100)	5830 kg (133)	7440 kg (170)

Table 1 : Comparison of Restrained Mass with Varying Damping Ratios for Panel W12 (The percentage variation is shown in brackets)

It is proposed that the seismic mass rating procedure is based upon the Phylmas loop matching and time history analysis procedure as described in Volume 2 section 3. The default damping ratio of 5% prescribed as the basis for the design spectra within the NZ Loading Standard (SNZ 1992) is recommended to be used until more information is available to warrant an increase in this figure. The draft revision proposes to nominate the Maximum Reliable Displacement (MRD), (being the maximum displacement at which stable panel response is observed during the experimental phase) as the displacement to which a specimen can be cyclically displaced prior to the successive 3rd cycle resistance dropping below 90% of the maximum recorded 3rd cycle resistance observed for that specimen. When the loss of resistance is sudden (i.e. it drops from above 90% of the peak resistance to below 75% of that peak within the three displacement cycles) then it is proposed to assign the MRD to be that imposed two increments prior. In this way the proposed approach should address the extent of degradation that may be experienced by the panel sheathing, this being an area of concern within the existing P21 evaluation procedure.

5.2.2 Serviceability Limit State Compliance - Earthquake Action

The intensity of ground motion assigned to serviceability limit state assessments is nominated as being $1/6^{th}$ of the elastic response spectra prescribed in NZS 4203:1992 (SNZ 1992). The draft revision of the evaluation method proposes that the elastic response spectra be scaled down by $1/6^{th}$ and the displacement limit equate to that associated with the onset of damage which requires repair, δs under the serviceability intensity earthquake.

The seismic mass used to derive the seismic bracing rating of the systems is the lesser of Ms or Mu. The limit state criteria which dictates the maximum sustainable seismic mass along with the bracing rating resulting from that mass will be output from the modified PHYLMAS analysis package.

6. DISCUSSION OF OTHER ISSUES

6.1 Elastic Recovery

Concerns about the prospect of permanent building offset (tilt) following serviceability intensity earthquakes are addressed within the current (1991) P21 evaluation procedure by the modification factor K1. The factor is dependant upon the serviceability limit deflection and the residual displacement present after the load is removed. Rebound of less than ± 40% of the serviceability displacement (i.e. residual displacements greater than 4.8 mm when δs = 8 mm) was deemed to be unsatisfactory and panels which exhibit this were disqualified from being used as bracing elements. Penalties were imposed by reducing the serviceability resistance when the residual displacement is greater than 3.2 mm.

The draft revision to the evaluation procedure addresses the onset of inelastic behaviour which requires repair. Residual deformations following load removal are a consideration when ascertaining the serviceability displacement. The need for further reduction because of residual deformation is therefore no longer necessary and is omitted from the draft revision.

6.2 Asymmetric Performance

The current P21 test evaluation procedure (King & Lim, 1991) allows for variation in the performance of a test panel in any particular load cycle, for both geometrically non-symmetrical and geometrically symmetrical test panels. In the case of geometrically symmetrical panels, the data obtained from racking in both the positive and negative directions for all three test panels in a set are amalgamated for evaluation purposes.

For non-geometrically symmetrical panels, a lack of symmetry in performance is acceptable. The resistance used in the determination of bracing ratings is taken as either the 3rd cycle residual resistance recorded at the ultimate limit state displacement cycle or 1.2 times the corresponding lesser resistance, whichever is least. The asymmetric performance criteria was continued in the P21 revision (King and Lim 1991). Although not explicit, the restriction imposed on the non-geometrical panels has (at least at BRANZ) in recent years been applied to geometrically symmetrical panels as well.

The draft revision proposes to use PHYLMAS as the analysis tool to ascertain the seismic mass which can be dependably sustained. Both the positive and negative responses of each specimen are to be matched when developing the electronic replica of the test specimen. The analysis undertaken will consider the least advantageous configuration when developing the response spectra and thus when assigning the dependable restrained seismic mass. Asymmetric response will thus be accommodated within the analysis procedure and does not require further specific consideration.

7. DISPLACEMENT COMPATIBILITY

7.1 Bracing Panels Within Parallel Brace Lines

Seismic forces generated by the building mass are distributed to each line of bracing through ceiling and/or floor diaphragms. The full scale testing of houses referred to in Appendix B 1.7 suggest that the ceiling diaphragm response is closer to rigid diaphragm action than flexible diaphragm action, in which case only limited differential movement between brace lines should be permitted.

Until more information is available on the flexibility of ceiling diaphragms, it is proposed that bracing panels within any one storey of a house should have maximum reliable displacements that vary by no more than ± 10 mm. Panels which fall outside this category can be used, however the rating of panels derived as an MRD which is more than 10mm in excess of the lowest MRD used within that floor will need to be down rated so that the displacement demand remains consistent within that building storey.

It will be necessary therefore to publish the displacement demand at which a particular panel type has been rated.

7.2 Bracing Panels Contained Within the Same Brace Line

NZS 3604:1990 places no restrictions on the acceptable deformation of bracing panels. While earlier versions of the P21 test procedure (e.g. Cooney & Collins, 1978) nominated both 8 mm as the working stress displacement limit to be applied to panels and required the capacity of the system to be checked at 32 mm displacement, the 1991 revision (King & Lim, 1991) to the P21 test removed this requirement. It is thus left to the discretion of the testing agency to nominate the ultimate limit state displacement at which the panels is to be rated. This information is not published with the resulting bracing rating. It is therefore impossible to ensure displacement compatibility between panels within the current evaluation procedure.

As the nature of determining displacement demand is not exact, it is proposed that the restriction on panels outlined in section 0 (i.e. displacement incompatibilities at a maximum of ± 10 mm) be imposed. The draft revision to the evaluation procedure would include the necessity to publish both the system bracing rating and the maximum reliable displacement at which the rating was derived. When this approach is incorporated into a future revision to NZS 3604 the acceptable limits for displacement compatibility would need to be included together with the basis upon which bracing rating adjustment can be applied to panels where their ratings are beyond the acceptable limits.

8. CONCLUSIONS

The current BRANZ P21 Test and Evaluation Procedure (King & Lim, 1991) has been critically reviewed and a draft revision proposed. The draft includes a revised experimental phase which includes cyclically displacing each specimen at nominated increments until the on-set of significant load degradation occurs. The behaviour of the panel at events beyond the design event are thus also included in the evaluation procedure thereby ensuring the premature development of unacceptable collapse mechanisms is avoided.

Bracing panel linings are required to possess some level of ductility regardless of the panel boundary fixings. Only by doing so can some ductility be utilised when complete panel restraint is effective in-service. This is achieved by evaluating the monotonic behaviour of a fully restrained panel.

Consideration has been given to the degree of uplift restraint included within each test. Although the restraint at a door opening has been found to be less than the 1991 P21 restraint (for panels without vertical load), the method is retained.

Where panels terminate at a return wall they can be assumed to be restrained. If straps, of maximum capacity 12 kN, have been used in tests to determine bracing ratings, they may be omitted when these panels are constructed on site.

It is suggested in the draft revision to the evaluation procedure that the rating be based on the maximum reliable displacement which the panel can sustain without significant loss of lateral load bearing capacity. It is proposed that the bracing rating will be derived using a computer simulation of the degrading system observed during the experimental phase. The derivation of the bracing rating itself will be contained within the computer procedure (PHYLMAS) which has been introduced to visually match the tested elemental response to a mathematically equivalent element.

Bracing ratings derived using the draft revised method have been compared to those determined by the current procedure. The results show that the current ratings are non-conservative and need to be down-rated. Explanations are provided as to why such reductions may be required.

9. FUTURE WORK

Work is underway at BRANZ to further verify the PHYLMAS computer programme. This involves pseudo-dynamic experimental techniques of simulated earthquake induced displacements being applied to various bracing wall configurations. While initially this is limited to single degree of freedom systems, it is proposed that this will further develop into multi-degree of freedom evaluations.

Work is also progressing on the development of a cut-down version of PHYLMAS which will permit testing agencies to match the experimentally observed system behaviour with that of a synthetic electronic element and allow the analysis package to return the bracing rating directly. Such a 'Black Box' approach aims to negate the necessity for testing agencies to learn or understand the intricacies of executing a time-history analysis in order to determine bracing ratings.

Work is required to develop a procedure which will enable the behaviour of systems tested to the current (King & Lim, 1991) procedure to be updated and brought into line with the procedures contained in the proposed revision. Whilst advantage may be gained by re-testing to the draft revision, a translation of existing test results to derive a revised rating will be developed.

Bracing panels are at present not restricted in their uplift capacities nor in their lateral load capacities. Initial tests undertaken on typical timber flooring (Refer Appendix B, Volume 2) indicate that failure of some parts of the floor must be anticipated at uplift loads of 12 kN. As the resistance of typical flooring and foundation systems was outside the scope of this study, further work in this area is required to assess the effects of the combined uplift and lateral capacities of these systems so that compatibility with the present ratings of bracing panels can be assured.

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APPENDIX A : DRAFT REVISION TO THE P21 TEST PROCEDURE AND EVALUATION METHOD FOR WALL BRACING

1. Scope and Use

This test and evaluation procedure is intended to establish the dependable racking resistance of wall bracing panels when they are subjected to lateral loads resulting from either extreme wind or severe earthquakes.

2. Referenced Documents

NZS 3604:1990 Code of Practice for Light Timber Framing Buildings

NZS 4203:1992 Code of Practice for General Structural Design and Design Loadings for Buildings

3. Definitions

Acceptable Structural Bracing System	A system which does not exhibit brittle failure when subjected to large deformation cyclic displacements.
Bracing Units	A unit of force such that 1 kN = 20 Bracing Units (BU)
Brittle Failure	The development of a collapse mechanism which does not involve the sheathing-to-substrate fasteners
Cyclic Tests	A series of three tests which subject each specimen to incrementally increasing fully reversing in-plane racking displacements (see Figure A-4)
Maximum Reliable Displacement	The displacement which the specimen can cyclically sustain without significant strength loss.
Phylmas	An integrated time history analysis programme the elements within which can be generated to match the inelastic response curves of observed experimental results.
Preliminary Test	A simplified cyclic test undertaken on a single specimen. Used to establish the suitability of the system to be assessed as a bracing material (by avoiding brittle failure) either for rigid (concrete slab) or flexible (timber frame) floor anchorage points. (see Figure A-3)
Significant Strength Loss	Significant Strength Loss is deemed to have occurred when the 3rd cycle resistance (in either the positive or negative direction) reduces by more than 10% of the maximum 3rd cycle resistance established during earlier cycles.
Supplementary Uplift Restraints	Devices attached to each specimen edge to provide a level of uplift restraint which could reasonably be anticipated in service.

System Failure	The system is deemed to have failed when	

a) during the preliminary test, the lateral resistance of the panel is less than 50% of the maximum lateral resistance;

b) during the cyclic tests, the lateral resistance of the third displacement cycle of a nominated displacement is less than 90% of the maximum third cycle resistance encountered during previous series.

4. Notation

Rmax	=	The maximum third cycle lateral resistance which can be sustained by the panel.
Δs	=	The displacement at which signs of damage sufficient to require repair are observed (either within the test panel or within elements which rely on the bracing system for support)
Ps	=	The average of the +ve and -ve resistance measured during the cyclic test phase at displacement Δs.
Δb	=	The basic first cycle displacement (either to be nominated or Δs + Δi)
Δi	=	The displacement by which the series is to be incremented during the cyclic test protocol. Δi should be selected upon the basis that at least two series will be imposed prior to the Maximum Reliable Displacement being reached.
Pult, Δult	=	The peak lateral resistance and associated displacement applied during the preliminary test.
T_u	=	The equivalent response period of the structure which is the period associated with the Maximum Reliable Displacement and is determined from the displacement response spectrum derived for each test.

5. Sampling

Sheathing materials supplied shall be representative of those being used to attain the bracing resistance attained. Framing members shall be of a grade and have dimensions which are the minimum of that to which the rating is to be applied.

6. Test Specimens

The specimen used for each series of tests shall be conducted on bracing panels built in accordance with construction and installation specifications stipulated. The resulting bracing rating shall apply only to bracing panels built according to those specifications. The panel specification is to apply to the panels themselves. Items not particularly specified but required to construct the specimen, are to be in accordance with local trade practice using minimum acceptable grades and fastenings which are compliant with that practice. Levels of workmanship employed in building the specimen shall be of a standard similar to that which may be expected in the field.

{Note: For framed systems, the specification is expected to nominate the sheathing parameters including the sheet thickness and orientation, whether the sheathing is installed on one or both sides of the substrate, the grade, type and characteristics of the substrate, the fastener type (fastener geometry including head and shank diameters and shank length) and fastener spacing. Where sheets joints are to be stopped, the method of stopping is to be included within that specification with regards to stopping compounds, their application method and the joint reinforcement (if any). Any supplementary end stud hold down restraints which are specific to the system being assessed are to also be included within the panel specification and within the test specimen.}

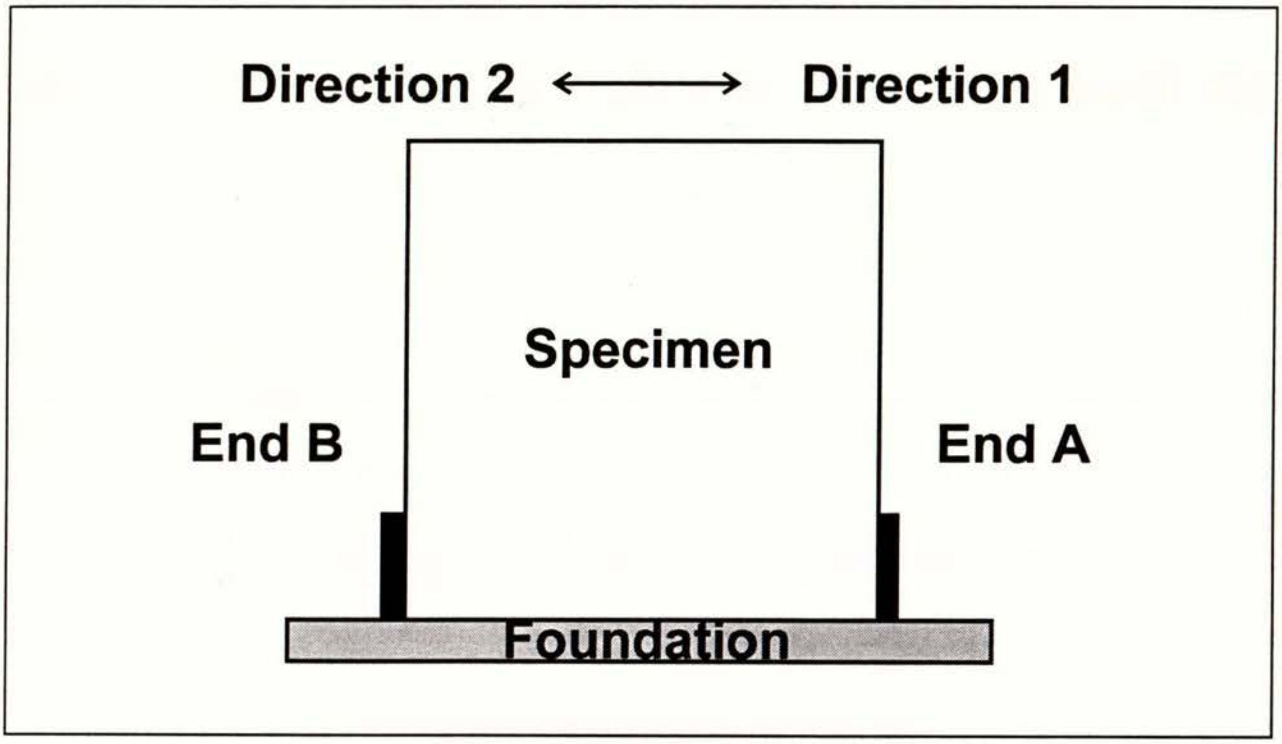


Figure A -1 Specimen Configuration

7. Apparatus

{Note: This section covers the loading rig, together with any ancillary fixings or restraints provided to hold the specimens in place within the rig. The minimum instrumentation required is also specified within this section.}

7.1 General

The test rig shall be

a) Of sufficient size that full scale specimens (with a minimum height of 2.4 m) and ancillary supporting elements (e.g. foundation beams etc.) can be installed.

b) Of sufficient in-plane rigidity that it will not distort more than 3 mm when subjected to the maximum lateral load imposed on the specimen with this load being applied along the top edge of the specimen.

c) Of a configuration which enable actions to be imposed along the top of the specimen which result in in-plane distortion of the specimen.

d) Include a base connection system suitable for connecting the base of the specimen and any supplementary end stud anchorage devices.

e) Have provisions for a restraint mechanism to prevent out-of-plane distortion along the top of the specimen without providing any supplementary in-plane restraint.

7.2 In-plane rigidity

Distortion of the specimen is to be achieved through the application of a horizontal load applied to the top edge of the specimen. Where asymmetry may be present within the test specimens the first loading cycle is applied in the weakest direction. The loading mechanism is to be such that the load rate is in the range 20≤load rate≤120 (mm/min). The load is to be measured to an accuracy of ±1% of the maximum load imposed. The connection between the loading device and the specimen is to be such that, throughout the complete deformation range, the load application does not interfere with the system response, and the load is applied appropriately to each portion of the specimen.

{Note: The connection between the load application device and the specimen should be at midway along the length of the top of the specimen thereby minimising eccentric effects being introduced by the rig loading mechanism during full cyclic loading.}

7.3 Base fixity

The base fixing system is to be provided with sufficient rigidity that the end restraint of the specimen does not displace more than 0.05 mm at any stage during the test. The degree of fixity between the specimen and the rig is to be such that it reflects the most rigid fixity the panel is expected to encounter in service consistent with the intended scope of application.

{Note: Where the bracing panel is intended to be used in both a timber foundation system and within a concrete 'slab-on-ground' construction, the base plate is to be rigidly fixed to a concrete or steel foundation beam using 10 mm diameter bolts @ 600 mm centres. When the bracing panel is only to be used within a timber foundation system, then flooring grade particle board not less than 18 mm thick shall be present over 2/150x50 abutting floor joists with the base plate being fixed to the foundation beams in accordance with the local trade practice (for NZ; in accordance with the nailing schedule specified in NZS 3604 (SANZ 1990)).}

7.4 End uplift restraints

The edges of the specimen are to be supported in a manner which provides a level of restraint which is consistent with that expected to be present in normal service.

{Note: The test regime aims to determine the dependable lateral resistance of each bracing panel as it performs within a complete lateral resisting system. It is thus considered reasonable to provide a level of uplift restraint which is consistent with that normally available rather than those which represent the true lower bound conditions. Thus although individual free standing bracing panels may be present within a house, the majority of panels will either have door (or window) lintels present, or will abut return walls. The level of restraint normally provided to the panel within the test set-up is consistent with that provided by door/window lintels. The test configuration will therefore need to include the effects of door lintels (i.e. be 'T' shaped), unless some alternative can be justified. For panels which use timber framing as their substrate, a supplementary 400 mm long timber block with the same cross section as the framing members

is to be nailed to the outer face of each end stud with three 100 x 3.75 mm flat head nails and held down rigidly as this has been found to provide a similar level of uplift restraint to that experienced by 400mm deep door lintels (refer Figure A2).}

For the Preliminary Test, the specimen is to be restrained from uplift at the tension end, by providing a full tie-down to the end stud.

{Note: Full tie-down restraint is similar to that provided by two 16 diameter mild steel rods placed vertically either side of the specimen and attached to a load skate at the top of the end stud. At the bottom end the rods are to be rigidly fixed and then hand tightened.}

If a known permanent dead load is supported by the panel in service then this load may be applied to the test specimen. This load is to be distributed along the length of the specimen such that it is representative of the actual loading condition.

The mechanism used to control out-of-plane distortion of the specimen, as it is laterally displaced, will be normally located along the top edge of the specimen. The mechanism should not restrain the specimen from in-plane movement, but should be sufficiently rigid that it will not permit the specimen to distort more than 3 mm out-of-plane when subjected a force equal to 10% of the maximum in-plane load imposed on the specimen.

{Note: Horizontal rollers or tie backs loosely fitting onto either side of the top plate near each end achieve this requirement.}

7.5 Displacement Measurements

Displacement measurement devices shall be capable of reading to an accuracy of 0.1 mm. The <u>minimum</u> displacements to be recorded are the horizontal deflection of the top edge of the specimen relative to the base of the specimen, and vertical movement at each end of the specimen. Each specimen is to be constructed and installed into the test rig in such a way that the test apparatus does not impinge onto the specimen to provide artificial restraint or to impose unintentional deformation throughout all loading cycles.

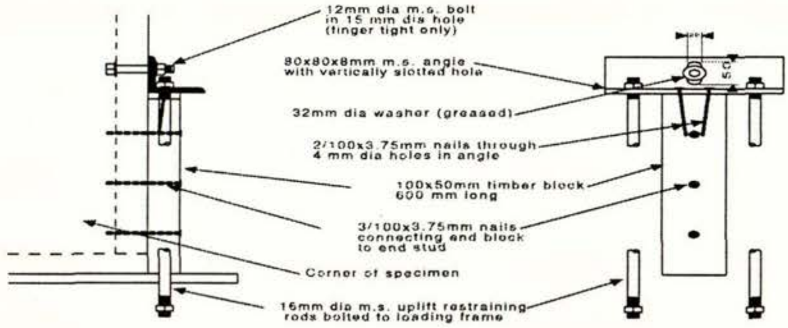


Figure A-2 'Standard' Supplementary Uplift Restraints (for timber framed systems)

8. The Preliminary Test

8.1 Objective

The preliminary test is undertaken to establish the ductility characteristics of the specified configuration with particular reference to the shear resisting element (e.g. the sheathing of framed systems).

{Note: The Preliminary Test may be omitted if prior knowledge of the bracing panel is available and the key parameters are therefore known.}

8.2 Specimen Configuration

The specimen is to be constructed in accordance with the construction specification applicable for the system under consideration.

{Note: The panel construction includes the overall dimensions, sheathing, fixings, fixing spacing and substrate elements.}

8.3 Specimen Installation

The specimen is to be fixed along its base to the test rig in the most rigid manner consistent with the scope of application of the bracing panel.

The boundary element at End A is to have a supplementary 'semi-rigid' uplift restraint, and at End B a 'standard' flexible uplift restraint.

{Note: For timber framed systems a 'semi-rigid' uplift restraint can be achieved either by providing top-plate hold-down tie rods with appropriate rollers to facilitate lateral displacements or alternatively by using 12 No. 3.75mm diameter 100 mm nails to connect the standard timber end-block to the outer stud [cf the 3 nails for the standard uplift restraint (refer Figure A-2)].}

8.4 Preliminary Test Loading Protocol

The preliminary test specimen is to be displaced as follows:

1. To 20 mm in direction 1
2. Return to zero displacement and displace 20 mm in direction 2
3. Return to zero displacement and displace to 30 mm in direction 1
4. Return to zero displacement and displace to 30 mm in direction 2
5. Return to zero displacement and displace in direction 1 until 'completed'

except that the test is deemed to be 'completed' when the resistance encountered falls below 50% of the maximum resistance in the direction of loading.

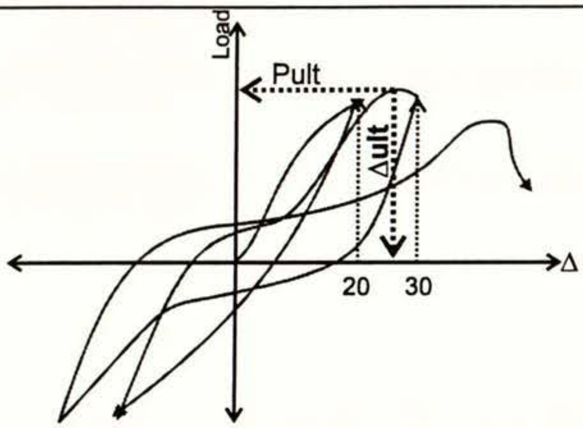


Figure A-3 Preliminary Cycle Loading Protocol

8.5 Required Results and Observations

The following data and observations are to be recorded during the preliminary test and included in the test report:

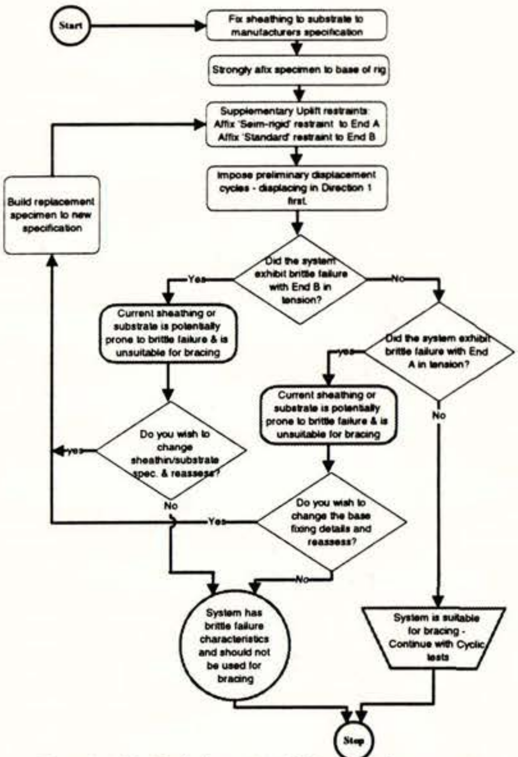
1. The maximum load and the associated displacement imposed on the specimen in either direction.
2. The peak loads experienced during each cycle and the associated displacements.
3. The failure mechanism which develops at and beyond the maximum imposed displacement are to be observed, recorded and photographed.
4. The ductility characteristics assessed for the panel are to be recorded.

8.6 Assessment Criteria

The system is deemed to be suitable as a structural bracing panel when it possesses ductility. Panels which fail through the development of a brittle (i.e. non-ductile) failure mechanism are inappropriate as structural bracing panels and cannot be assigned a bracing rating.

{Note: For framed systems, limited ductile systems involve the development of a failure mechanism which includes the connectors and/or the sheathing at or near the connectors. Brittle failures are those which involve flexural failures of the substrate framing elements, base-plate splitting, sheathing buckling or tensile failure of the specified hold-down straps.

If brittle failure is experience in a configuration which is rigidly connected along its base (i.e. as may be expected when installed on a concrete slab), the option exists to limit the scope of application of the bracing to timber floors only and to use nail fixings through the panel base.}



Flowchart for Preliminary Test & Ductility Assessment

9. Cyclic Testing

9.1 Cyclic Test Objective

To establish the cyclic loading characteristics of the specified configuration needed to assign a dependable bracing rating.

9.2 Specimen Configuration

The specimen is to be constructed in accordance with the construction specification applicable for the system under consideration.

{Note: The panel construction includes the overall dimensions, sheathing, fixings, fixing spacing and substrate elements.}

9.3 Specimen Installation

The specimen is to be fixed along its base to the test rig in the most rigid manner consistent with the scope of application of the bracing panel.

The boundary element at each end of the specimen is to be provided with supplementary restraints which reasonably reflect the degree of uplift restraint which could be considered present during 'normal in-service conditions'.

{Note: For timber framed systems a standard uplift restraint fixed with 3 No. 3.75 mm diameter 100 mm nails to each outer stud is considered appropriate.}

9.4 Pre-test Nominations

Prior to undertaking each cyclic test assessment, the following information requires to be specified:

1. The serviceability displacement, Δs, for systems in which the panel is to be used.

2. The basic displacement, Δb, target being the first set of displacements to which the specimen is displaced beyond its serviceability displacement.

3. The incremental displacement, Δi, being the incremental displacement to be imposed with each incremental cycle.

(Note: When determining the serviceability limit state deflection, (being that associated with the onset of damage requiring repair) either knowledge is required as to when the onset of damage to linings or lintels beyond the test panel may be expected, or a special study undertaken to establish such displacements. For systems which contain gypsum based lining materials, a lateral drift limit of height/300 (or 8 mm for 2.4 m panel height) is deemed to be an appropriate serviceability limit state racking deformation.

The basic (first cycle) displacement is somewhat arbitrary. As a guide it is suggested this should be between 1.5 Δs *and* Δs + Δi.

The incremental displacement, Δi, *should be selected so that the specimen is cycled to its serviceability cycle plus at least two additional cycles prior to attaining it maximum dependable displacement. With this in mind, it may be appropriate to reassess both* Δb *and* Δi *after each specimen of the series has been tested.}*

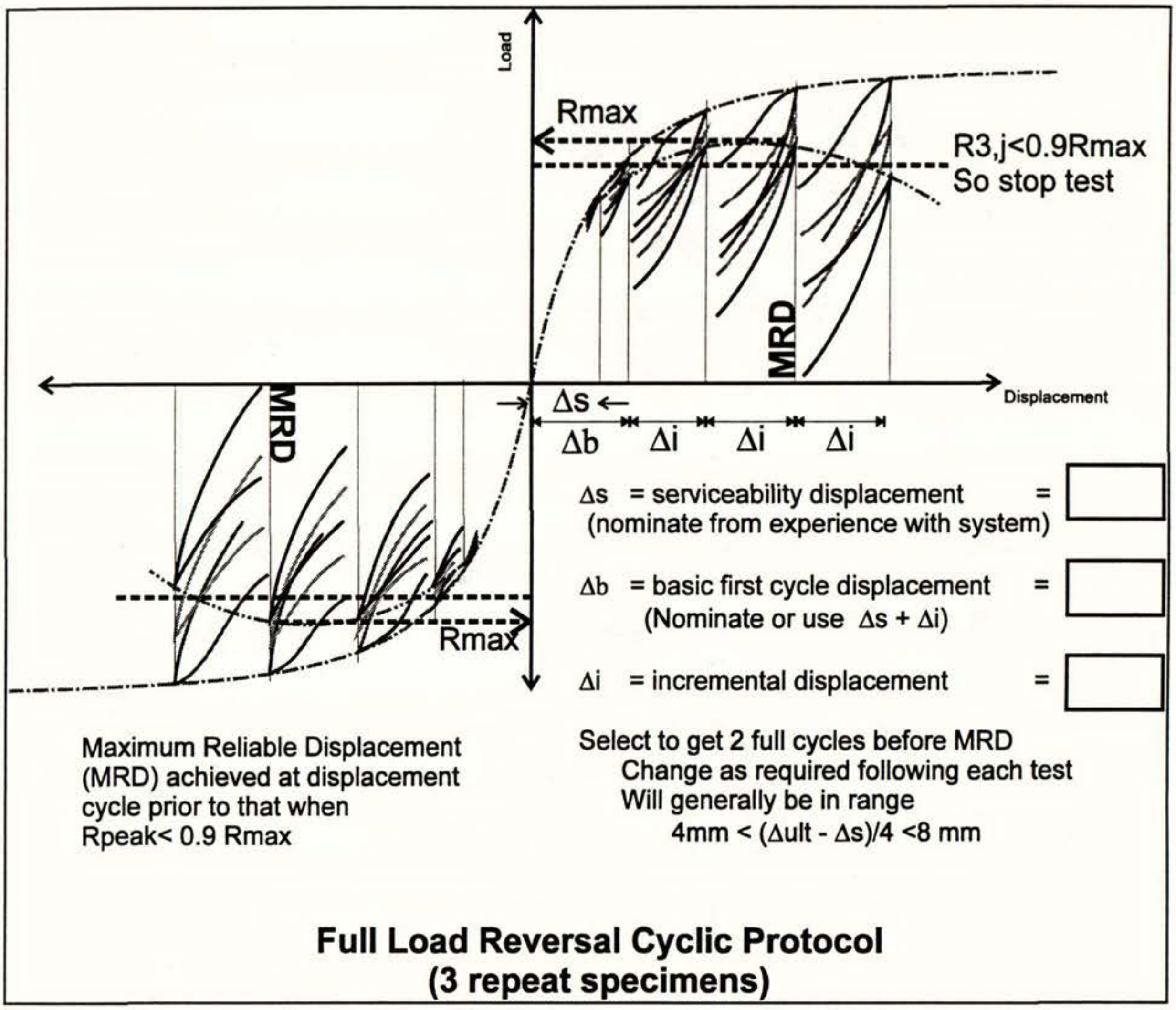


Figure A-4 Cycle load protocol

9.5 Cyclic Test Loading Protocol

Each specimen subjected to cyclic testing shall be displaced as follows:

1. Set target displacement $\Delta t = \Delta s$
2. Displace to Δt in direction 1
3. Return to zero displacement and displace Δt in direction 2
4. Repeat 2 and 3 two additional times (i.e. three cycles to displacement $\pm \Delta t$)
5. Set target displacement $\Delta t = \Delta b$
6. Return to zero displacement and displace to Δt in direction 1
7. Return to zero displacement and displace to Δt in direction 2
8. Repeat 6 and 7 two additional times (i.e. three cycles to displacement $\pm \Delta t$)

9. Set target displacement $\Delta t = \Delta t + \Delta i$

10. Return to zero displacement and displace to Δt in direction 1

11. Return to zero displacement and displace to Δt in direction 2

12. Repeat 10 and 11 two additional times (i.e. three cycles to displacement $\pm \Delta t$)

13. Go to step 9 and repeat until failure.

{Note: The cyclic loading protocol has been developed to subject the panel to a reasonable number of extreme load reversals whilst avoiding fatigue failures of fasteners or fixings, a phenomena not commonly experienced in the field. Thus the first cycle of the protocol is established as being nominally above the serviceability displacements, and the subsequent cycles developed to give moderate increments between steps, whilst enabling system degradation to proceed. It is thus quite acceptable to estimate the incremental displacements provided at least two cyclic excursions have been imposed before the MRD is attained. The failure criteria is based on the residual strength left after three excursions to each displacement limit.}

9.6 Data Recording

For each test the following shall be recorded:

1. Sufficient readings to enable a graph of the lateral load to be plotted against the lateral displacement at the top edge of the specimen.

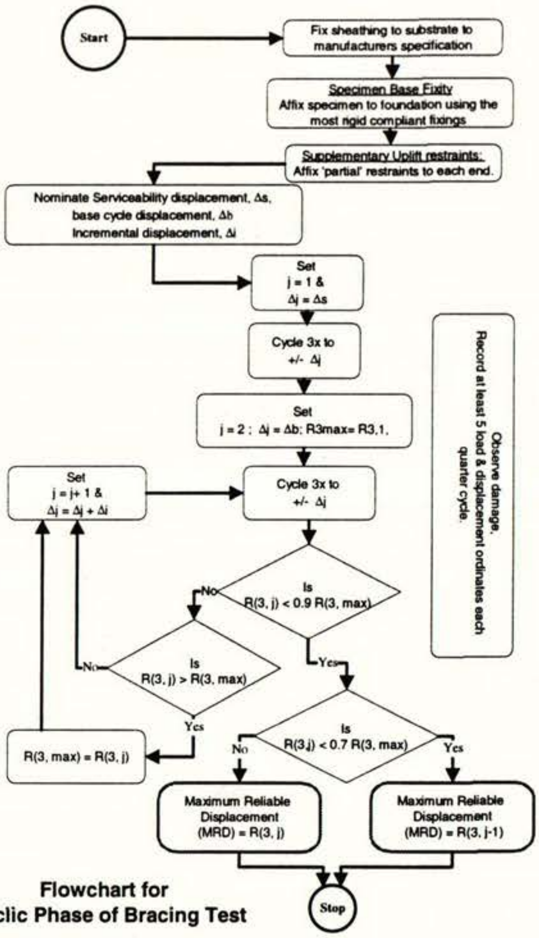
2. Uplift deformation of each end of the panel at the corresponding top load and displacement peaks.

3. Shear slip between the bottom plate of the specimen and the test foundation at the corresponding top load and displacement peaks.

4. Complete construction details including specimen dimensions, the moisture content of the frame (if present), curing time of plaster joints if appropriate, boundary conditions.

5. Photographs of the specimen before and after the test.

6. Mode of failure and description of damage to the specimen observed during the test.



Flowchart for
Cyclic Phase of Bracing Test

10. Bracing Performance Evaluation

The following evaluation procedure is to be used to ascertain the dependable lateral load resistance of degrading bracing panels subjected to extreme winds or severe earthquake attack.

The resulting lateral resistance rating is to be applied to bracing panels constructed in accordance with the test panel construction and installation specification. The results may be extrapolated on the basis of panel geometry within the following limits:

a) The lateral mass dependably restrained per metre length of panel may be applied directly to walls up to 600 mm less in height than the test panel height, and may be reduced in accordance with the ratio of test panel height to actual panel height for panels up to 600 mm greater than the test panel height.

b) Where test panels have a height to length aspect ratio > 1, the lateral mass dependably restrained per metre length of bracing panel is to apply to walls of actual length La provided the L<La<2 L where L = the test panel length.

c) Where test panels have a height to length aspect ratio ≤ 1, the lateral mass dependably restrained per metre length of bracing panel is to apply to walls of any length greater or equal to the test panel length.

Provided that the specimen exhibits ductile behaviour during the preliminary test, the average results of the cyclic tests can be used to determine the panel bracing rating.

10.1 Evaluation of Dependable System Lateral Resistance under Earthquake

{Note: The determination of the dependable racking resistance provided by wall bracing panels which develop slackness as they degrade under simulated earthquake action is commonly grossly mistreated in that the slackness is overlooked and simple conventional dynamic oscillator response is applied. Such modelling is incorrect and does not in any way represent the true response of the system. The alternative and correct representation involves developing electronic replications of the observed system response and applying simulated earthquake induced ground motion such as those available through integrated time history analysis. In this case the elements being analysed accurately reflect the degradation of the panels and the true response can be modelled. The resulting reliable resistance can then be used either as an engineering basis for design or as a means of compliance with non-specific compliance standards such as New Zealand's light timber framing standard, NZS 3604:1990.}

1. Develop a modelling element within the Phylmas computer programme which matches the experimentally generated load displacement loops for a given test.

   {Note: Where significant asymmetry is apparent either as a result of an asymmetric element or from other reasons, the Phylmas parameters should match the weaker set of results, with the results then being a lower bound and applied to the complete system.}

2. Subject the single degree of freedom oscillator model which has these characteristics to a computer time history analysis using a suite of suitable earthquake records.

   {Note: for New Zealand conditions, the synthetic earthquake record NZA has been derived. This has been matched to the design response spectra published in NZS 4203 and is thus equivalent to that used for specific engineering design of buildings. Evaluation of test displacement loops of bracing panels typically used in light timber frame construction using the NZA earthquake record has shown to produce the greatest response and hence conservative results.}

3. Produce response displacement spectra for the matched model.

4. From the test hysteresis loops determine the Maximum Reliable Displacement (MRD).

   {Note: For systems in which the sheathing and fasteners exhibit reasonable levels of ductility (as determined from the Preliminary Test) then the MRD being the displacement to which the specimen can be cyclically loaded prior to the successive 3rd cycle dropping to less than 90% of the maximum recorded 3rd cycle load. If the drop in load is greater than 75% then MRD is to be taken at the displacement cycle two incremental displacements prior. For systems for which the Preliminary Tests indicate non-ductile behaviour, then the MRD is the lesser of that derived from the cyclic test as described above, or the maximum dependable displacement experienced during the Preliminary Test.}

5. Use the spectral displacement plot to determine the equivalent response period of the structure, T_u, whereby the Maximum Reliable Displacement is not exceeded.

6. Determine the Mass able to be restrained by the test specimen by:

   Mass Restrained $M_u = 25300 \cdot k \cdot T_u^2$................ *(A1)*

   Where k = The initial stiffness parameter (from PHYLMAS, (kN/mm))

   T_u = Period in Seconds

   M_u = Mass restrained in kg.

   {Note: Derived from the Rayleigh equation for the determination of the dynamic response of a simple single degree of freedom oscillator namely:

   $$T = 2\pi\sqrt{\frac{m}{k}}\ \}$$

7. Elastic Recovery.

   $K_1 = 1.4 - C/\Delta_s$

   where C = Residual Displacement on the serviceability cycle

   Δ_s = Serviceability Limit State displacement

   and $K_1 \leq 1$

   {Note: Some non-recoverable offset associated either with minor inelastic behaviour or tolerance take-up is acceptable during the serviceability load cycle. A limit of H/666 (or 3.6 mm for a 2.4 m high panel) is to be applied and reflected in the K_1 factor above. Permanent sets greater than this may be acceptable and will require special consideration.}

10.2 Conversion of Dependable Lateral Resistance to Bracing Units

The maximum Earthquake Resistance Capacity R_{ce} is equal to M_u.

$$\text{Earthquake Resistance in Bracing Units} = \frac{R_{ce}}{29}$$

Where R_{ce} is in kg

{Note: Bracing units have been introduced as a simple means of converting forces (in kN) to near whole numbers known as Bracing Units and used in New Zealand within our Non-specific design standard NZS 3604. A simple conversion is 1 kN = 20 Bracing Units (or BU).

For earthquake considerations, an additional conversion coefficient of 0.176 needs to be applied. This is derived from the combination of the lateral force coefficient of 0.22, used to define the Bracing Units required within NZS 3604 and the Zone factor of 0.8 as prescribed in NZS 4203.

$$\textit{Load effect: } \frac{Mass x 9.81 x 0.176 x 20}{1000} = B.U,s \textit{ required.}$$

$$\textit{i.e. } Mass \times 0.0345 = B.U,s$$

$$\therefore B.U's \approx \frac{Mass}{29}\ \}$$

10.3 Determination of Dependable Panel Resistance Rating under Wind

The following procedure is to be applied to determine the panel bracing rating for wind loading:

- Determine the average maximum peak force resisted P_u by the test specimens and the average force resisted at the serviceability displacement P_s.

- Maximum Wind Resistance Capacity R_{cw} is the lower of P_u or P_s x K_1/0.65

  {Note: The 0.65 factor is derived from the ratio of serviceability to ultimate limit state wind speeds i.e. = 0.65}

- Then the Wind resistance in Bracing Units. = R_{cw} x 20

11. Reporting

The report shall contain the following information:

1. The testing agency controlling the tests.
2. The location and dates over which the testing was undertaken.
3. The identity of the Agency/Engineer responsible for the test.
4. Description of specimen construction, including the trade-name of the sheathing, the jointing compounds (including reinforcing used if any) and the curing times of setting compounds used.
5. Details of the fixing details of the bottom plate to the foundation.
6. Details of the means by which uplift is controlled at the ends of the specimen.
7. Details of the observed mode of failure of both the Preliminary and Cyclic tests.
8. Load vs top edge horizontal deflection plots.
9. Bracing Rating and the maximum reliable displacement.
10. Photographs and Drawings.
11. Sheathing Density and Thickness (if applicable).
12. Time of plaster joint curing (if applicable).
13. Time for the concrete/mortar to achieve target strength (if applicable).

12. References

Standards Association of New Zealand (SANZ) 1990. Code of Practice for Light Timber Frame Buildings not Requiring Specific Design, Standards Association of New Zealand, NZS 3604, Wellington.

APPENDIX B : LITERATURE REVIEW

1. Review of Wall Racking Tests

The following review highlights the variations that exist in test methodologies and the difficulties that exist in assessing the resistance of bracing panels in-service, from the results of panels tested in isolation.

1.2 New Zealand

The majority of wall bracing elements used in New Zealand house construction, are assessed using the BRANZ P21 test (Cooney and Collins 1978). The test method was adopted in NZS 3604 (SNZ 1978). The associated bracing evaluation method assigned a racking resistance strength in Bracing Units (with 20 Bracing Units, B.Us = 1 kN) to a bracing panel depending on the force resisted when the panel is cyclically loaded to a nominated working stress deflection limit. It was then subjected to a translational displacement of four times that nominated displacement with the requirement that each specimen was able to sustain 80% of the peak lateral resistance during four cycles to plus/minus this nominated displacement.

In 1990, a major philosophical change to NZS 3604 (SNZ 1990) occurred whereby loads and load combinations became based upon the principles of limit state capacities. The revision was followed by a similar revision to the P21 test which was published as BRANZ Technical Recommendation TR 10 (King and Lim 1991).

The 1991 amendment required three panels to be tested to pseudo-static reverse cyclic displacements in $\pm$ 1 mm increments, up until the serviceability limit state displacement δ_s. This is usually taken as 8 mm (corresponding to a displacement of H/300 for the standard 2.4m high test panels). The panel is then racked to a target displacement, δ_u, and cycled four times at this displacement.

The system's resistance to wind load action was assessed (in King and Lim 1991) as being 0.9 times the average peak resisted load for the two test directions. The 0.9 factor has been applied to allow for some degradation of the lining which could be expected to occur if the load was applied in a fluctuating manner resulting from the effects of wind turbulence. The resistance to seismic action is taken as the average load at the fourth cycle to the ultimate limit state displacement. This resistance force may be reduced if the panel ductility (found from the test) is less than four.

Strong but flexible walls may be governed by the serviceability limit state considerations in which case the bracing rating is based upon the load which can be applied without exceeding the serviceability displacement.

In recognition that the individual panel is provided with supplementary restraint, the test configuration during the P21 test allows each end stud to be partially restrained from uplift by fixing an additional block using three nails. The additional block is then firmly fixed to the test rig to prevent any uplift of the block. In effect the end studs are restrained by the three nails in shear. The degree of restraint is based upon work carried out at BRANZ (Gerlich 1987) and found to be the minimum restraint at the end of bracing panels which intersect at return walls. The 1991 P21 test method allows for the use of additional uplift restraints if they are to be used in practice. It also allows the addition of top plate vertical loads if they are known to be permanent dead loads.

Bracing ratings are applicable to walls of the same length as the tested panel or to walls up to twice the panel length using the same per metre rating. They can not be used for walls which are shorter than those tested.

1.3 U.S.A.

The most common forms of light timber frame construction prior to the second world war were with timber board linings or with timber diagonal bracing.

Tests on these construction types were carried out in the 1930s and 1940s and formed the basis of acceptance standards of the FHA (Federal Housing Administration) and HUD (US Department of Housing & Urban Development).

The test methods served as the basis for an ASTM Standard E72, first published in 1947 and codes for racking resistance are generally still based upon the results of this test.

ASTM has defined two racking test procedures; ASTM E72 (ASTM 1980) and ASTM E564 (ASTM 1976).

The E72 Standard procedure requires a vertical hold down rod to be applied at the tension end of an 8′ x 8′ specimen which is then racked monotonically. As full restraint is provided, the E72 procedure provides a means of measuring the relative performance of individual linings. The results do not assess the in-service panel performance.

Conversely, ASTM E564 aims to evaluate the performance of the panel in service rather than that of just the lining. The specimen is installed only with the actual uplift restraints specified for each system. No supplementary hold down is provided. The panel is thus assumed to act independently from its neighbours or from the frames into which it is installed. Loading is uni-directional. The test method specifically excludes its use for evaluating the effects of cyclic loading. A minimum of two tests are carried out on identical panels, with a third necessary if the results do not agree within 10% of the lower value. The second test is run with the specimen orientation reversed with respect to the load application used in the first test. Loading is to failure and the ultimate shear strength is simply the maximum load/length of panel. The unloading sequence occurs at one and two thirds of the estimated shear strength.

Allowable shear resistances allocated to panels in the Unified Building Code (ICBO 1991) are based on tests carried out to ASTM E72. Questions have been raised (e.g. Oliva and Wolfe 1988; Griffiths 1984) as to the suitability of this test method to panels which are subjected to seismic (and hence cyclic) loads.

The current test methods are presently being reviewed (Skaggs & Rose 1996). Dolan (Dolan 1996) has indicated that cyclic testing on shear walls has been carried out recently to a protocol known as Sequential Phased Displacement which was originally developed by the TCCMAR group, a joint American-Japanese research group. The new loading consists of two displacement patterns. First to yield, in which the elastic response is observed, and the second post-yield in which inelastic response is observed.

Further developments in wall racking testing have been undertaken by the Structural Engineers Association of Southern California (SEASOC). Their test method (Skaggs and Rose 1996) uses a load regime based upon the Sequential Phased Displacement procedure. The relationship between the different test methods is also under review. It is interesting to note that current USA building codes allow the use of gypsum plasterboard as a lining material for bracing panels, although its use is discouraged by assigning such systems low allowable bracing ratings.

Restrictions on the use of gypsum plasterboard as a bracing material in conventional construction may also be expected when the new unified Building Code is introduced in 2000. So further penalising these systems. (Dolan. 1996)

1.4 Australia

The Australian timber framing standard, AS 1684 (SA 1992), specifies the number of bracing panels required within Australian timber framed houses. The bracing panels are designated as either Type A or Type B bracing panels and are stipulated as, having a working stress racking resistance of 2 kN and 4 kN respectively (with a load factor of two used). Two deemed to comply solutions (for plywood and Masonite) are prescribed within the standard, and 20% of the lateral resistance of the building is assumed to be resisted by unrated walls.

Although AS 1684 is silent on the test method by which the lateral bracing is to be determined, Technical Record TR 440 (Experimental Building Station 1978) has been used as the basis for determining the racking resistance of panels which are subjected to cyclonic winds. TR 440 requires test panels to be representative of those used in service with respect to construction details, point of load application, and the use of auxiliary panel hold down devices such as cyclone bolts.

The TR440 test procedure subjects the bracing panel to a monotonic displacement regime. At first the panel is racked to a test serviceability load which is the lowest load to cause a net horizontal displacement of the top plate of H/300 (or in the case of masonry and concrete, to a displacement which causes the onset of cracking). Panels are then unloaded and the irrecoverable displacement measured. The panels are then subjected to either what is termed a Static or a Repeat Cycle. The Repeat Loading Cycle is used to investigate fatigue effects that may occur under dynamic wind load. Panels are racked in one direction to 0.625D (where D = the working stress strength design load), then unloaded. This is then repeated 400 times in each direction. The panel is then subjected to 100 cycles at 0.75D and a further 100 cycles at 1.0D. It is then finally pushed to failure. The test is therefore more representative of a fatigue load more commonly associated with claddings and their fixings. The number of load reversals is however much reduced since the bracing element is only indirectly exposed to the wind effects.

For the Static displacement cycle the serviceability limit deformation is imposed first in one direction and then the other. The lesser of the resulting two loads is deemed the 'Test Serviceability Load'. The panel is then displaced in the direction of least resistance to failure, the failure load being defined as the "Ultimate Static Load Test Failure Load".

1.5 Japan

The test methods for evaluating panel racking resistance are described in A1414 (JIS 1973). These methods are modifications to the ASTM E72 tests (ASTM 1976), outlined in section 2.1.2, except that the use of tie rods or omission of tie rods is permitted. The maximum shear load applied is considered as the maximum load resisted by the panel or by the load resisted as a rotation limitation, with the deformations not exceeding 0.015 radians when the tests are carried out with tie rods or 0.02 radians without tie rods. Although test protocol is prescribed, there is no standardised evaluation method. Tests carried out in Japan (e.g. Yasumura et al 1988; Sugiyama 1988) suggest that tie down rods are commonly used.

The following method and evaluation procedure was described by the Building Research Institute of the Ministry of Construction. (Yasumura 1996).

Four test panels, 2.4m (H) x 1.8m (W) wide are used with tie-down rods included. The first is subjected to monotonic load until failure, with peak load Pmax and displacement at peak load δmax recorded. Two panels are then subjected to monotonic load and unload cycles to an increasing percentage of Pmax until failure. The final specimen is subjected to the same loading levels but applied in a bi-directional loading regime. Three results are taken from each test; the load at an actual (corrected) shear deformation of H/300; the load at 2/3 Pmax and the load at δmax/2. The average results for each of the above three tests are obtained and the shear resistance of the panel is taken as the minimum of the three values.

According to Sugiyama (1988), Article 46 of the Building Standard Law Enforcement Order provides the minimum value of total effective lengths of all shear walls for each direction. He also notes that the distribution of shear walls, throughout a building is not prescribed and is left to the discretion of the designer/builder.

1.6 U.K.

The standard procedure for determining the racking resistance of panels by testing is set down in BS 5268 (BSI 1988) and is based upon work carried out at Princes Risborough Laboratories and the University of Surrey. The loading protocol aims to replicate lateral loads applied from wind action.

No definitive hold down restraint is used, however the test panels are to be fixed to the foundation by methods which simulate as closely as possible the fixings that are to be used in-service. Test panels are 2.4 m by 2.4 m. Where vertical loads are known to occur in service then they can be used in the test. Where a panel is intended to be used under a range of vertical loads a minimum of two panels are tested, one being tested with the assumed maximum vertical load and the other under the assumed minimum vertical load. In addition, stiffness tests are carried out on each of the two test panels.

For the strength test, racking loads are applied unidirectionally, either continuously or incrementally until failure of either the lining or frame. The loading rate adopted is slow, with racking deflections of no more than 15 mm in any five minute period permitted.

For the stiffness test a racking load is applied until a deflection of 0.002 times the panel height is reached. The test is repeated four times.

The 'Test Racking Strength Load' and 'Test Racking Stiffness Load' are taken as the test load factored to take account of the number of tests conducted. In addition, limitations are placed on the residual racking deflections after the first and third cycles.

'Test Racking Design Load' for a particular vertical load is then given by the 'Test Racking Strength or Stiffness Load' divided by a factor of safety. Other modification factors (shown below) are applied to the design load to account for panel height and length.

Dimension range	Modification factor
Length, L, 0 to 2.4 m	L/2.4
Length 2.4 tp 4.8 m	$(L/2.4)^{0.4}$
Length >4.8 m	1.32
Height, H, between 2.1 m and 2.7 m	2.4/H.

Generally, plasterboard is only permitted to be assigned 50% of the resistance provided by other materials.

1.7 European Code

The test methods used to determine racking strength of structural wall panels is specified within the European Standard Pr EN 594 1991 (CEN 1991).

Standard panels are 2.4 m x 2.4 m lined either one or both sides with (normally) a wood-based lining. The frame bottom plate is held rigidly to the foundation by means of four No. 10 mm diameter bolts, with the leading bolt between 100 mm and 200 mm from the panel edge. Unidirectional load is applied to the top plate and the load is applied to 0.4 times the estimated maximum racking force, maintained at constant load for 30 seconds then unloaded. After two minutes of unload the panel is racked to failure. Vertical loads may be applied during the test, spread equally to each of the studs.

1.8 Canada

According to Dolan and Madsen (1991) racking tests carried out in Canada follow the test procedure outlined in ASTM E72 and are as described in Appendix B section 0 above. As discussed in that section, a revision to the test procedure is being worked on within the United States and it is envisaged that this will be adopted for use in Canada also.

APPENDIX C : PRINCIPAL FACTORS AFFECTING PANEL BEHAVIOUR

The response of a bracing panel to earthquake or wind load is complex. Attempts to relate the strength of a panel tested in isolation to that of a similar panel in-service is notoriously difficult and should be done with caution. Many of the factors which influence panel behaviour extend beyond the configuration of the panel itself and include the framing into which the panel is to be installed, the degree of uplift available to each panel as installed and degree of interaction between panels and their neighbours.

The following is a review of work which has been undertaken to assess the influence that various factors have been found to have on bracing panel performance.

1. Panel Uplift Restraint

The methods used to connect test panels to foundations to restrict panel uplift are varied. The current BRANZ P21 test method (King & Lim 1991) allows the addition of a partial restraint to the outer boundary members of the test panel. The requirement is to apply 'realistic levels of uplift restraint' which are 'appropriate for the system configuration being considered'. For panels installed within timber framed walls, this additional restraint is provided by end blocks fixing with three nails in shear to the end studs (Gerlich 1987).

Kamiya et al (1980) conducted several tests on 2.4 m high panels of varying lengths lined with plywood. The method of tie-down of the frame also varied from no tie-down through to the use of tie-rods and a vertically imposed load on the top plate. They found that the apparent shear strain (or Total Panel Rotation) was greater for tests using the tie-down method than either of the other two methods. However, the influence of the test tie-down method on the actual shear strain (i.e. the 'lozenge' effect) was small. They concluded that the 'lozenge' effect was independent of the manner in which the panel was tied-down and that the total panel rotation increased and approached the lozenge rotation as panel lengths increase. It was therefore appropriate to establish the dependable racking strengths on the basis of the lozenge rotation.

Sugiyama et al (1988) carried out lateral load tests on individual panels as a lead up to testing a two storey 'Post & Beam' traditional Japanese timber framed house. The lateral resistance of diagonal cut-in timber braces (105 by 53 mm timber section) were first assessed. The external walls were then clad with exterior grade calcium silicate sheets. The internal bracing panels remained unlined. Failure developed within one of the internal walls where the diagonal bracing only was present. Of relevance of this study was that the initial stiffness of the whole house was 50% greater than that assessed from the individual bracing elements.

Leiva (1994) studied the shear racking resistance of a series of tests on 2.4 x 2.4 metre square panels, which were lined one side with plywood, with varying numbers of anchor bolts as part of his investigation into the racking behaviour of timber shear panels. Three tests were carried out with the sill (bottom) plate fixed to the foundation beam using two, three or four No. 8 mm diameter bolts. No other restraints were used. He found that the sill plate exhibited large bending deformations when only two bolts were used and that there was no observable deformations when four bolts were used. Each specimen was subjected to monotonic displacement. The racking resistance and stiffness decreased with reduced bolt numbers. He also noted that a reduction from three to two bolts led to a decrease in racking resistance of 44% while an increase from three to four bolts saw an increase in racking resistance by 20%. Leiva was thus able to conclude that the degree of base plate fixity had a marked influence both on the lateral resistance of the panel and also on its fundamental response characteristic with greater base plate fixity resulting in a more brittle sill plate splitting mode or sill plate flexural rupture.

Oliva and Wolfe (1988) investigated the influence of panel end restraint by conducting two wall racking test series. The first series used the ASTM E72 (ASTM 1976) tie-down rods and the second series superimposed actual vertical load to the panel throughout the test. They concluded that the ASTM E72 standard procedure did not provide a sound basis for judging shear wall performance.

2. Linings on One or Both Sides

Tissell (1989) conducted a series of tests using frames lined one and two sides with plywood. He confirmed that walls identically lined on both sides have double the strength of a single sided wall.

Leiva (1994) carried out tests on twenty-eight 2.4 x 2.4 m specimens under static monotonic and reversed cyclic loading to evaluate the influence of anchoring conditions using variable nail spacings and linings on one or two sides using plywood. He showed that panels with linings on two sides resulted in increases in racking strength per metre of 83%, with initial stiffnesses increasing by 94%.

Robertson and Griffiths (1981) reviewed test results carried out by various organisations in the UK, using both the ASTM E72 (ASTM 1976) restraint and the UK method of applying vertical load to the studs as described in Section 0 (BS 5268, BSI 1988). Faces of the panels were lined either with the same material, or with material of differing stiffnesses such that the stiffness ratios varied between 50:50 and 12:88. Racking strengths of panels lined both sides increased over those lined on one side only from between 84% (for plywood) to 104% (with a 12:88 lining stiffness ratio).

Patton-Mallory et al (1984) conducted a series of racking tests on small scale panels using plywood and plasterboard linings. They concluded that racking strength of frames lined on both sides was the sum of the single sided strengths.

3. Panel Length

Oliva & Wolfe (1988) carried out a series of tests, using plasterboard linings on 8′ x 8′ (2.4 m by 2.4 m) panels. The panels were tied down using 6 mm diameter steel cable and were subjected to monotonic, cyclic and high speed dynamic loading. They found that the racking strength increased as panel length increased, from 140 lb/ft (2.04 kN/m) for the 8′ long panel to 150 1b/ft for the 16′ panel and 170 lb/ft (2.48 kN/m) for the 24′ panel (i.e. increases of 5% to 17%).

Patton-Mallory et al (1984) compared wall racking strengths and effective wall length using a number of small scale tests 2 ft high specimens with lengths of 2 ft, 4 ft, 6 ft and 8 ft (i.e. 0.6 m, 1.2 m, 1.8 m & 2.4 m), and compared them with full scale, 8 ft high specimens with lengths of 8 ft, 16 ft and 24 ft (i.e. 2.4 m, 4.8 m and 9.6 m). They concluded that ultimate racking strength with plasterboard linings was proportional to wall length for aspect ratios between one and three. Racking strengths of plywood panels were proportional to wall lengths for aspect ratios between one and four.

Robertson and Griffiths (1981) showed that with no vertical superimposed load, panel lengths increasing from 2.4 m to 3.6 m resulted in an increased racking strength of 120%. Increasing the length to 4.8 m showed an increase of 180%.

4. Effect of Transverse Walls

To determine the effects of cross walls on the lateral stiffness of buildings, Suzuki (1990) carried out an experimental study on a one-third scale model of a 4.5 m x 4.5 m single storey light timber frame house using plywood sheathing. He found that although the cross walls carried only a small percentage of the lateral load, they prevented rotation of the bracing walls and as a consequence contributed significantly in providing stiffness to the building. This partially explained the difference in performance of a building under full-size testing and the contribution obtained by simply considering the wall bracing elements.

Yasumura et al (1988) carried out tests on a full scale three-storey timber framed house to investigate the behaviour of shear walls and diaphragms compared to theoretical calculation. The specimen was racked in three stages:

1. without linings to floors and transverse walls
2. without linings to transverse walls and finally
3. all walls and floors lined.

The results showed that the torsional deformations observed in case 2 were far greater than in case 3, indicating that the torsional moments caused by the difference in longitudinal wall shear stiffnesses were carried by the transverse walls, with a consequential reduction in floor torsional displacement.

5. Panel Orientation

Wolfe (1983) found an increase in ultimate strength of 50% when paper faced plasterboard panels, tested in accordance with ASTM E72 (ASTM 1976), were orientated horizontally rather than vertically. This was attributed to the fact that in the horizontal orientation the top and bottom of the sheets were confined by the paper facing which inhibited core loss following fracture around the nail head. With the sheet in a vertical orientation this did not occur due to edges being cut.

Dolan and Madsen (1991) performed dynamic tests using plywood and waferboard linings and found that the orientation of the linings did not affect the response of the panel provided joints were fully blocked and nailed.

6. Ceiling Diaphragm Action

Engineering design procedures commonly used in New Zealand to apportion the lateral load shared between lines of adjacent shear walls is usually based upon the mass within the floor area between adjacent lines of shear resistance (i.e. the tributary floor method). The assumption is that the roof and ceiling diaphragms are flexible relative to the shear walls and therefore the lateral load attracted is proportional to the area of diaphragm supported. Based on this assumption differences in wall stiffness are of little consequence.

Several tests conducted on full scale houses suggest this over-simplifies the actual building behaviour and could affect building performance. Stewart et al (1988) conducted tests on two manufactured 14 x 66 ft (4.2 m by 20 m) full scale houses each containing five shear walls under simulated wind load. The test involved the individual application of concentrated loads at eaves and bottom plate levels of each shear wall to evaluate their in-plane stiffness. In addition a uniform in-plane lateral loading was applied to the entire side wall until failure. (The roof was constructed of timber trusses with metal roofing and a fibreboard ceiling.)

Results showed that racking deflections were greater in the stiffer end walls, even when concentrated loads were applied to the internal shear walls. The deformation profile of the roof diaphragm was compared to that of the shear walls and indicated a high in-plane bending stiffness of the diaphragm relative to the walls. Also the lateral deflections of the diaphragm were small, 0.264 inch (6.7 mm) being recorded at mid span (span being 66' (20 m)) for a concentrated load of 3600 lb (78 kN) in the same location and only 0.44 inch (11 mm) under the uniform loading of 75 p.s.f. (3.6 kPa). They concluded that the roof diaphragm - shear wall system behaved as a very stiff beam on elastic foundations with the shear walls acting as springs with different stiffness. Interestingly there was no apparent structural failure when the uniform in-plane load was applied to one side of the building, even at a load as high as 75 p.s.f.

Mahaney and Kehoe (1988) presented a structural model for the seismic analysis of multi-storey buildings with wood diaphragms and shear walls of varying construction. They evaluated the results of numerous tests undertaken in the USA on plywood sheathed diaphragms with aspect ratios from 1 to 5 and proposed a computer programme semi-rigid diaphragm analysis procedure. The procedure was demonstrated on an example two-storey building and compared to a rigid diaphragm and tributary area method. The analysis indicated that some walls are over-designed and some under-designed when comparing the flexible and rigid analyses methods.

Phillips et al (1993) conducted racking tests on a full scale house with dimensions of 16' x 32' (4.8 x 9.6 m) long. The house consisted of four shear walls, each having a different lateral stiffness, constructed with various linings of plywood and plasterboard and different nailing patterns. Roof trusses spanned the length of the house and were lined with external ply roof sheathing and a plasterboard ceiling. Concentrated loads were applied to the top of the shear walls in four stages; when only one side of the walls had been lined; when both sides had been lined; with linings added to the transverse walls and finally with the complete roof system added. The first two stages were carried out as individual racking tests on each of the four shear walls. Results of that test clearly showed that the roof diaphragm possessed sufficient rigidity to produce load sharing between shear walls. Phillips et al concluded that the rigid diaphragm response was closer to actual roof diaphragm behaviour than that predicted by flexible diaphragm response and that load distribution was a function of shear wall stiffness and wall location.

7. Vertical Load

Robertson & Griffiths (1981) showed that within the normal range of design loadings the racking resistance of panels increases as vertical load increases. However, the rate of increase reduces with increase in vertical load, the greater reduction occurring in the weaker lining materials. For this reason they recommended that tests should include the full range of working conditions and that results are not to be extrapolated to vertical loads greater than those used in the test.

8. Openings in Shear Walls

Typically shear walls which contain openings are considered to have a resistance being equivalent to the sum of the resistances of individual components between openings. Continuity of the panel either above or below the opening is ignored in the resistance evaluation.

Line and Douglas (1996) reviewed the development of a shear wall design method which allows for openings. The design method is based upon the empirical equation proposed by Sugiyama (1981), and involves adjusting the unit shear capacity of a full height wall by coefficients which are determined by the size and number of wall openings.

Johnson and Dolan (1996) conducted racking tests on ten shear walls, 2.4 x 12 m long, sheathed in plywood. Specimens were subjected to monotonic and sequential phased displacement until failure. They concluded that the empirical equation proposed by Sugiyama can conservatively predict the capacity of a shear wall with openings.

Thurston (1993) studied the behaviour of several compound wall bracing systems. These included combinations of long and short bracing panels lengths along with door and window openings. He concluded that the presence of window openings could generally be ignored provided they did not intrude below 800 mm above the panel base and the head lintel was not less than 400 mm in depth. Conversely the presence of door openings provided a severe dislocation which significantly affected the lateral shear of the system. The degree of uplift restraint offered by the head lintel over doors was less than that assumed by the P21 end restraint device. Thurston suggested that in such cases, a supplementary up-lift restraint should be provided as a matter of course. Where return walls are present, such restraints were however unnecessary and in Thurston's opinion supplementary end straps could be omitted without significantly reducing the system performance.

9. Performance of Houses Subject to Wind and Earthquake

Many writers have commented on the structural performance of Light Timber Framed houses when subjected to wind and earthquake forces and the resulting damage is well documented. Such damage that has been observed has been attributed to an inability to adequately tie the elements within the building together (i.e. a lateral load path was lacking) or to poor construction detail (failure to provide adequate hold-down or high torsional irregularity). There have been few instances where damage has been attributed to the provision of inadequate racking strength. It should be remembered that, apart from the Edgecumbe earthquake (Pender et al 1987), there have been few events which have even come near to imposing design intensity lateral loads. During the Edgecumbe earthquake, most houses built to modern standards performed well. Damage did occur in instances where fault dislocation extended directly through buildings. This is to be expected however and even in these cases the structure did not collapse.

The modern New Zealand practice of using large span metal plate connector timber trusses to span between external walls, has resulted in more open plan housing layouts with a consequent reduction in the number and length of internal walls. This, together with the trend for larger window openings, has resulted in modern houses having less structural redundancy and potentially greater torsional (twisting) response.

There are several modern construction trends which make extrapolation of system performance based on historical behaviour a somewhat delicate art. These trends include the use of panelised wall system to provide greater flexibility of use within the house which often reduces the number of structural bracing panels available. Adhesives are replacing nails as the means of attaching wall linings to their substrate. The necessity of a 50 year durability on structural components and the lack of reliable data on the ability of these adhesives to perform in the long term is preventing the use of adhesives within structural panels unless they are also nailed. Air driven gun nails are replacing their hammer driven counterparts. The influence of such a change is sometimes difficult to predict. Similarly the use of fine gauge staples can have durability implications which need to be carefully assessed.

Cooney (1979) identified the principal weaknesses in the traditional New Zealand house as inadequate bracing within the foundation and subfloor regions, poor connection detailing and more seriously the poor performance of chimneys. Highly irregular vertical or horizontal building profiles were acknowledged as introducing a higher level of torsional response than the traditional more regular counterparts.

Moss (1991) found that considerable damage has been caused by earthquake and to a much lesser extent by wind. He cites the case of older houses being renovated by the removal of internal walls and the consequent reduction in strength and stiffness as a potential problem area.

Shepherd et al (1990) comments on the effects of the Loma Prieta earthquake on various structural forms. They found that low rise timber framed buildings generally performed well even when subjected to severe ground shaking. The failures tended to be due to the collapse of subfloor framing or where little lateral resistance was afforded due to the structural configuration.

McDonald (1991) identified an extensive list of publications on damage to buildings in the USA due to wind loads. Weak connections between both the roof and superstructure and the superstructure and the foundations or subfloor are cited as the major contributing factor in the failures observed following severe wind storms in the USA.

Diaphragms and shear walls are seldom identified as being the principal failure mechanisms.

Allen (1984) reported that both old and new house constructions in Canada were damaged in tornadoes and attributed most of the damage to construction defects such as inadequate nailing of joints and lack of anchorage to foundations. Similar observations were recorded by Lux (1990).

A report by Goers (1976) on the damage recorded after the San Fernando earthquake concluded that failures were mainly due to the lack of walls with enough strength to resist horizontal forces, or poor stud to bottom plate nail connection.

Gupta and Stalmaker (1991) report on the cost of damage to house due to wind and earthquake in the USA in monetary terms. They reported that common failures were due to the lack of wall hold down anchorage and foundation anchorage, with the most common feature being the failure of wall connection nails, whereby nails are pulled out of the sheathing and framing or sheathing is punched out near the nails.

BRANZ P21 Assessment & Evaluation Software

The proposed revision to the P21 bracing panel test and evaluation procedure requires integrated time-history analysis in order to assign a bracing rating.

Software has been developed to automate the process of characterising the test specimen responses and assessing their response to the level of earthquake ground motion required by the NZ Loadings Standard, NZS 4203:1992. BRANZ is making this software available to the industry with a nominal fee to cover copying, distribution and the maintenance of a register of users.

The software was developed as part of a research programme funded by the Building Research Levy, the Public Good Science Fund adminstrated by the Foundation for Research Science and Technology, and the Earthquake Commission.

The software requires (at least) a 100 MHz, '486 PC with 16 Mb of memory operating Windows 95 to run the software.

A beta version of the software is currently available and the final version will be released when the test and evaluation method is adopted by the industry. The final version will be distributed to all registered users free of any additional charges at that time. The research team would like to be alerted to problems or suggestions for improvement by users before the final version is released.

The research team has subjected the software to rigorous testing and has interpreted the Loadings Code requirements with care but BRANZ accepts no responsibility for the resulting data nor for any designs undertaken using this data.

Copies of the software may be purchased directly from BRANZ at a cost of NZ $50 (inclusive of GST & P&P). The software and will be made available to a registered user via the web, e-mail or posted on 1.44 Mb floppy disks.

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STUDY REPORT

No 78 (1998)

RACKING RESISTANCE OF BRACING WALLS IN LOW-RISE BUILDINGS SUBJECT TO EARTHQUAKE ATTACK

Volume 2

Experimental Program Derivation & Assessment

P.D. Herbert and A.B. King

This is the second of two volumes which combine to report on how wall bracing panels which develop slackness can rationally be used as a structural system to resist earthquakes. A summary of findings and a proposed revision to the current P21 test method and related references are contained in Volume 1. Volume 2 focuses on the experimental programme which underpins the recommended changes. It is intended that the combined report will be used by technical advisers to building product manufacturers seeking to use their product as wall bracing elements within New Zealand houses. It provides the engineering rationale upon which a revision of the Wall Bracing Test and Evaluation Method known as the BRANZ P21 test method is proposed. As such it is also intended for use by Building Control Authorities and Structural Engineers can utilise the bracing ratings derived for specific engineering design.

PREFACE

This volume contains the experimental results for the project. The completed experimental programme was undertaken in three phases, each of which is covered separately within this volume. In all cases the behaviour of timber framed bracing panels under wind or earthquake racking is examined with a view to addressing anomalies such as the degree of end restraint and the onset of premature brittle failure associated with lining degradation. The findings from this experimental programme are used to support the proposed revision to the test and evaluation method contained in Volume 1.

ACKNOWLEDGMENTS

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RACKING RESISTANCE OF BRACING WALLS IN LOW-RISE BUILDINGS SUBJECT TO EARTHQUAKE ATTACK

BRANZ Study Report SR 78 **P.D.Herbert and A.B.King**

Reference

Herbert, P.D and King, A.B. 1998. Report on Racking Resistance Test and Evaluation Procedure. Building Research Association of New Zealand, Study Report SR 78, Judgeford, New Zealand.

Herbert, P.D and King, A.B. 1997. Report on Racking Resistance Test and Evaluation Procedure. Building Research Association of New Zealand, Study Report SR 78, Judgeford, Wellington, New Zealand.

Abstract

Lateral loads such as those produced by the effects of wind and earthquake can be resisted in buildings by cantilever action, by moment resisting frames, by shear walls, by diagonal bracing or a combination of these.

In New Zealand light timber frame construction, the resistance is provided entirely by shear walls. The total resistance of a wall is determined by summing the dependable strengths of individual full height panels located between openings. The standard method for assessing the racking resistance of wall bracing elements between openings, since 1978, has been the BRANZ P21 test. It has been known for some time that there are deficiencies with the P21 test and evaluation procedure with major problems being whether the test loading regime can adequately identify severely degrading elements, and in the assessment of wall ductility.

A detailed literature survey of wall racking tests carried out around the world and the factors which contribute to bracing panel behaviour is given. Taking this into account, a three phase experimental programme was carried out on bracing panels under various loading protocols, including monotonic and reverse cyclic loading. The end studs to the specimens were either fully held down with tie-rods, or restrained from uplift by the application of a vertical load or use of a partial restraint. A series of experiments was also carried out with no restraint to the end studs. The test specimens were lined with sheathings commonly found in New Zealand construction.

Both the onset of damage to the panels and the displacements at which a significant drop off in load occurred were investigated.

Methodology is presented in this report to enable an accurate computer model to be matched to the test element response. Once matched, the model may then be used to analyse the performance of the element under dynamic seismic loading and to generate seismic response spectra. The result from this analysis is quantification of the mass that the test panel can dependably restrain without the necessity to assess wall ductility.

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TABLE

1. EXPERIMENTAL PROGRAMME PHASE I

1.1 Objective

The first phase of the experimental programme was designed to:

- Determine a load/displacement protocol which enables the onset of damage of bracing panels to be identified.
- Determine the effect that the alternatives of either load or displacement control had, if any, on the peak resistance of the bracing systems.
- Determine the effect that different boundary conditions (ie end stud uplift restraints and also the degree of sill plate fixity) had on panel strength and stiffness.
- Determine the amount of restraint afforded by lintels.
- Provide data to verify the Phylmas computer programme. Details of the programme can be found in Section 3.

1.2 Description of Test Specimens

All frames were constructed from machine stress graded 90 x 35 mm kiln dried radiata pine studs and 90 x 45 mm kiln dried radiata pine top and bottom plates and machine stress graded to grade F5. All frames, except for panel W17 described below, were 2420 mm high and 2435 mm long. Frame studs were centred at 600 mm nailed together using two 90 x 3.15 mm gun nails. Panels W4-W6 had one row of noggins at mid height. All other panels were devoid of noggins.

Bottom plates to panels W1-W16 were nailed to a 150 x 90 timber foundation beam, through a 20 mm particleboard strip, using pairs of 100 x 4 mm flat head nails at 600 mm centres. The particleboard strip represents typical flooring. The foundation beam was bolted rigidly to a purpose built steel framed test rig. The typical test set-up is shown in Figure 1.

Panel W17 was uniquely constructed to simulate a short section of wall between two door openings. The bottom plate of the panel was fixed to a 300 x 100 mm timber foundation beam using pairs of 100 x 4 mm flat head nails at 600 mm centres. The foundation beam was firmly bolted to the laboratory strong floor and had a 20 mm strip of particleboard fixed to its upper face. The door lintels were constructed using 90 x 35 mm F5 kiln dried radiata pine and were checked 15 mm into the studs. The top plate was continuous. Hold down restraints were provided at the extreme ends of the specimen by means of 16 mm diameter tension rods clamped to the lintel and fixed directly to the strong floor. This prevented uplift at the tension end of the panel during load cycling but did not prevent vertical downward movement of the end stud in compression. Ten kN load cells were used to measure the uplift in the restraining rods. (see Figure 2).

Various linings were used in the tests. Linings were used on one side only, with the exception of panels W16 and W17 which were lined both sides. All linings were fixed using galvanised clouts.

Variations in the lining material, the end boundary restraint and the loading regime are identified in Table 1 below.

Panel	Lining	Restraint	Load Regime
W1	PB	P21	Monotonic
W2	PB	P21	Cyclic/Load Control
W3	PB	P21	Cyclic/Disp Control
W4	FC	P21	Monotonic
W5	FC	P21	Cyclic/Load Control
W6	FC	P21	Cyclic/Disp Control
W7	PY	P21	Monotonic
W8	PY	P21	Cyclic/Load Control
W9	PY	P21	Cyclic/Disp Control
W10	BL	P21	Monotonic
W11	BL	P21	Cyclic/Load
W12	BL	P21	Cyclic/Disp Control
W13	BL	Tie Rod	Cyclic/Load Control
W14	BL	Vert Load	Cyclic/Load Control
W15	BL	None	Cyclic/Load Control
W16 [1]	BL	Tie Rod	Cyclic/Load Control
W17 [1,2]	PB	Refer Figure 2	Cyclic/Disp

Table 1 : Test Specimen Configurations

1 Lined both sides
2 Lintel Specimen. Refer Figure 2

The four proprietary linings used were:

PB - Nominal 9.5 mm standard grade paper faced plasterboard[1] fixed with 30 x 2.5 galvanised clouts at 150 mm c/c to panel perimeter. 300 mm c/c at lining join and pairs of clouts at 300 mm c/c on intermediate studs.

FC - Nominal 7.5 mm thick smooth faced fibre cement sheet[2], fixed with 40 x 2.5 galvanised clouts at 150 mm c/c to panel perimeter, internal studs and noggins.

PY - Nominal 7.5 mm thick plywood sheet with three laminates, measured thickness was 7.8 mm and density 4.1 kg/m^2 fixed with 30 x 2.5 galvanised clouts at 150 mm c/c to sheet edges and at 300 mm c/c to intermediate studs.

BL - Nominal 9.5 mm enhanced paper faced bracing plasterboard[3] fixed as per the plasterboard pattern above but with purpose made washers beneath the nails around the panel perimeter.

1 Winstone Wallboards Ltd standard Gib® plasterboard of measured thickness of 9.5 mm and density 6.7 kg/m^2.

2 James Hardie Harditex® with a measured thickness of 7.6 mm and density 10.3 kg/m^2.

3 Winstone Wallboards Ltd Gib® Braceline with a measured thickness of 9.5 mm and density 8.46 kg/m^2.

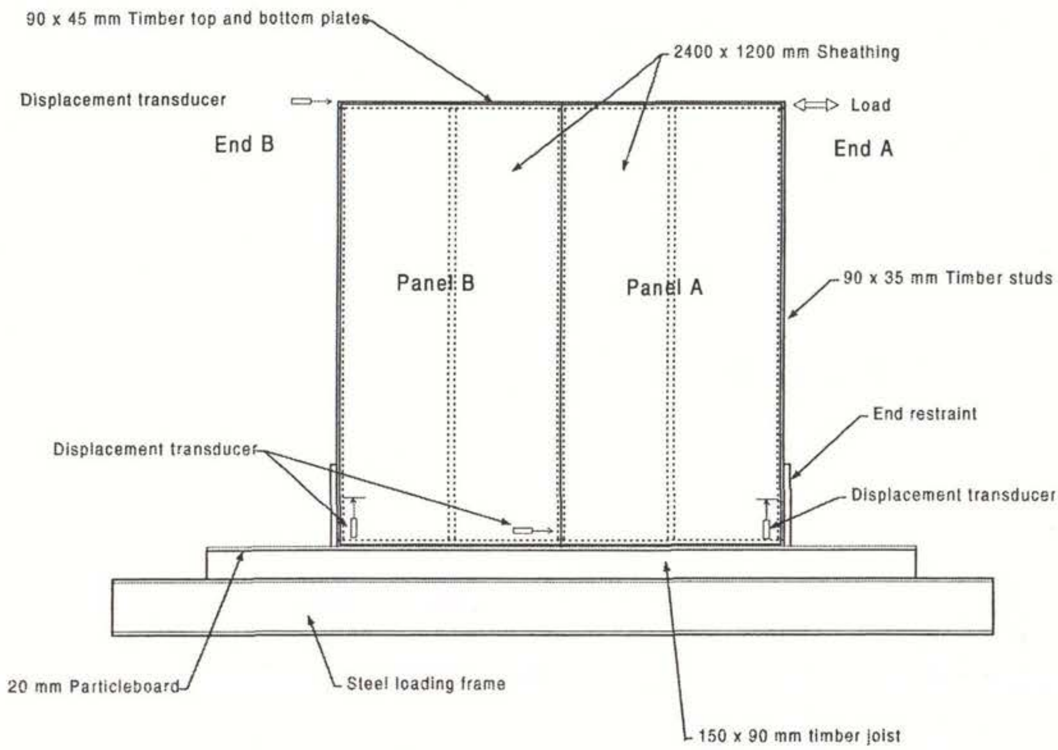


Figure 1 : Typical Test Set-Up

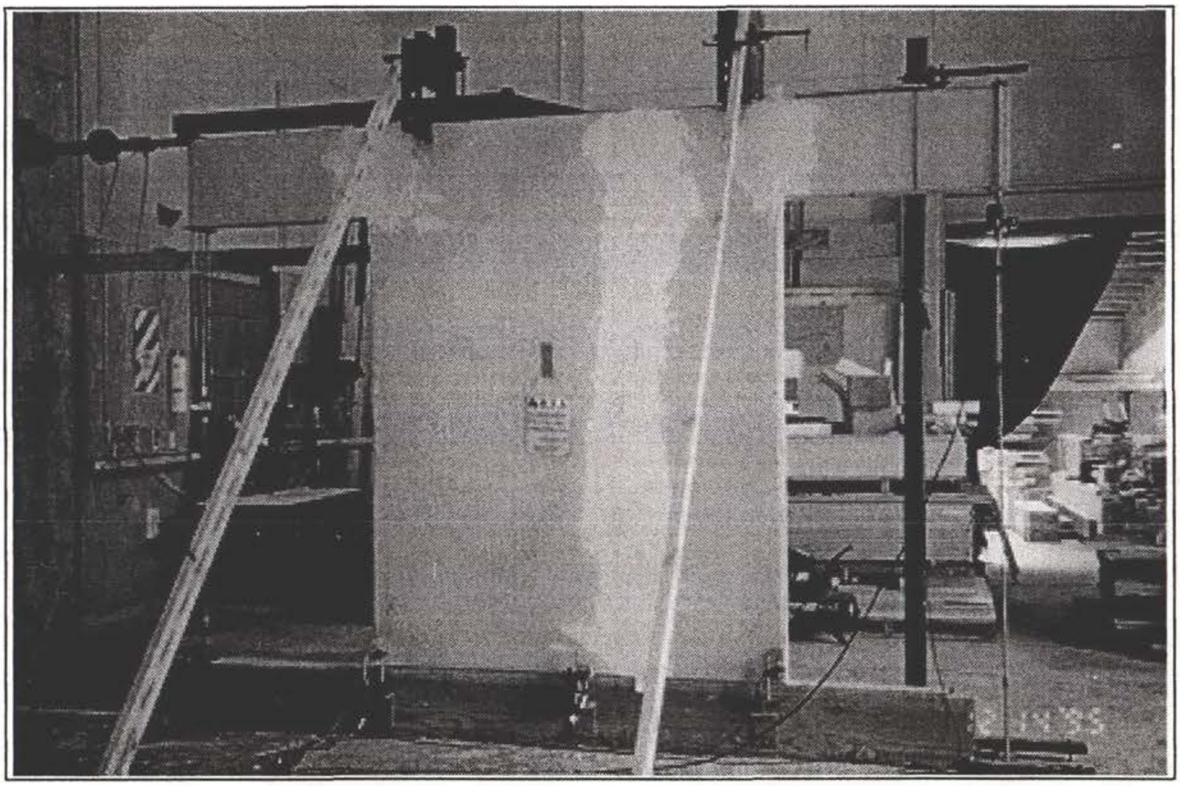


Figure 2 : Panel W17 Configuration

Both sides of the panel were lined with 9.5 mm plasterboard, with joints fully stopped and taped. A 25 x 1 mm (nominal) galvanised high tensile steel strap was used to form a diagonal brace within the main body of the specimen.

The panel was similar to a section of wall tested by Thurston (1993) so observations and results from each of the tests could be readily compared.

1.2.1 End stud uplift restraints used

The effect of varying degrees of uplift restraint were investigated in panels W1 to W16. Four levels of restraint were imposed as follows:

i) ***P21 Restraint*** - Consisting of a portion of stud fixed to each end of the test specimen using three No. 100 x 4 mm flat head nails fixed horizontally. The stud portion was restrained from uplifting by bearing against a mild steel angle which was bolted down to the test rig. Refer Figure 3. Reference to the P21 restraint is made throughout this document and refers to this detail.

This method of restraint is the one currently employed for the BRANZ P21 test method (King & Lim 1991) and is considered as representative of the minimum restraint afforded by the intersection of cross walls at bracing panel ends.

ii) ***Tie Rods*** - Four 16 mm diameter mild steel rods were placed in pairs at each end of the specimen. The rods extended the full height of the specimen and bolted rigidly at the bottom to the test rig. The top of the rods were bolted and hand tightened to a mild steel angle which was placed across the specimens over the end stud. The mild steel angle was mounted onto a load skate to allow the specimen to move horizontally without restraint. Refer Figure 4.

The vertical restraint provided a full tie down and prevented specimen rigid body rotation. It is an adaptation of the method used in ASTM E72 (ASTM 1976) as described in Vol I section C.1.2

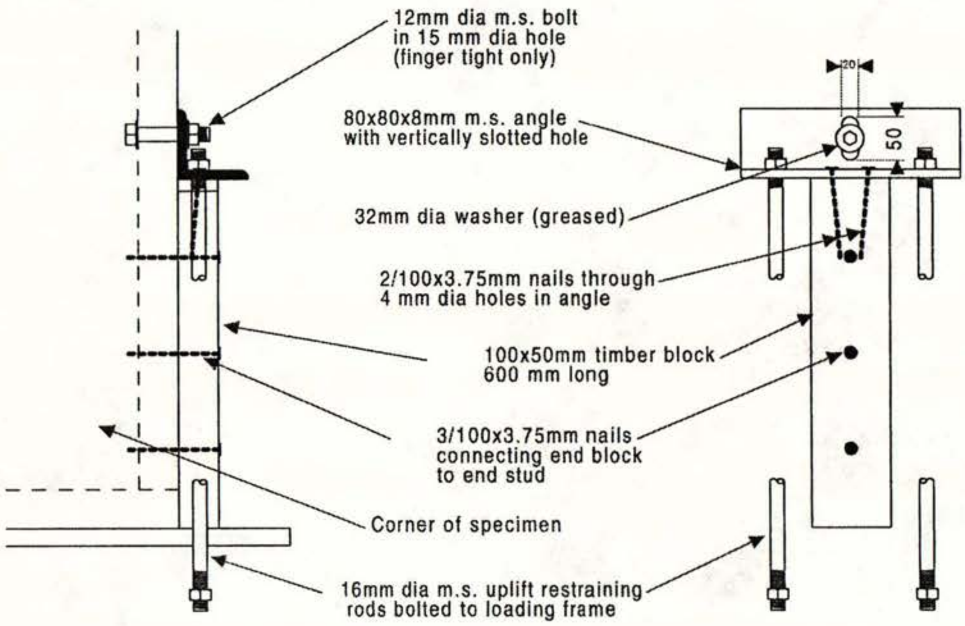


Figure 3: P21 End Restraint

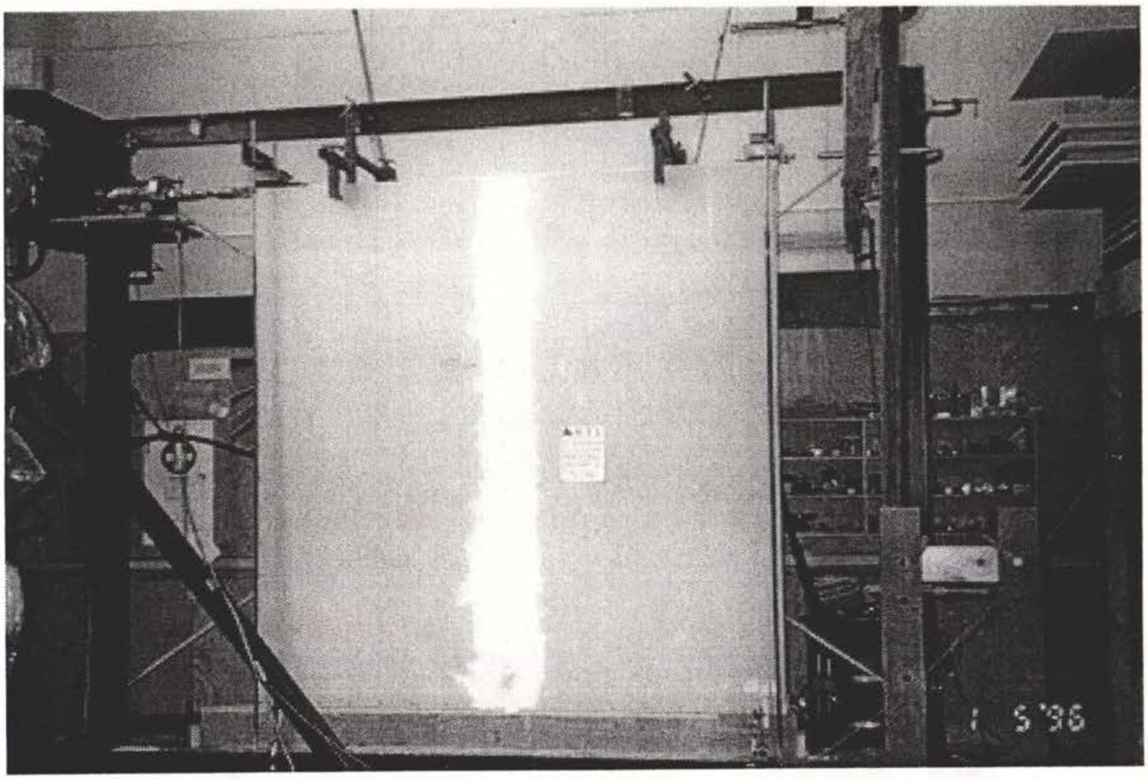


Figure 4 : Test Specimen with Tie-Rod Restraint

iii) ***Vertical load*** - A vertical load was applied to specimen W14 by means of two 125 kg weights suspended from the end of a cross piece bearing onto the top plate. The other end of the cross piece was tied down to the test rig using a 16 mm tie rod. Refer Figure 5. The resultant vertical load imposed through the lever arm of the cross piece was equivalent to a vertical mass of 1000 kg. No other restraints were used.

The load represented a lower storey panel carrying 8 m width of lightweight roof and 3m width of floor. The load was transferred to the studs via a steel spreader beam which was packed off the top plate at the stud positions.

iv) ***No restraint*** - This test was carried out without any additional restraints or gravity loads.

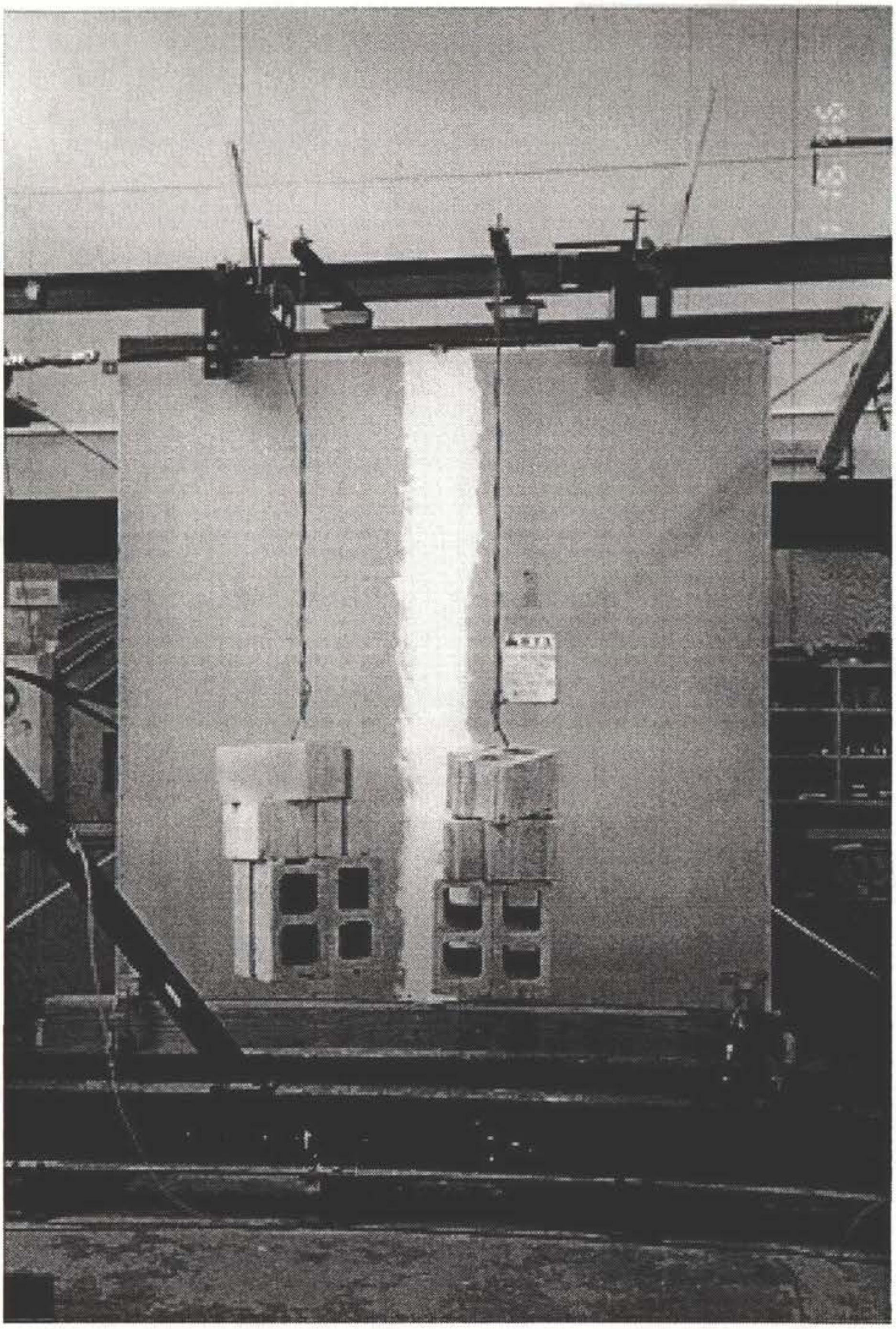


Figure 5: Test Specimen with Vertical Load

1.3 Phase 1 Experimental set-up

Panels W1-W16 were tested in a vertical orientation in a rigid steel loading frame. Horizontal load was applied to the specimen top plate with a 30 kN closed loop electro-hydraulic ram and measured with a 20 kN load cell.

Panel W17 was tested on the strong floor with the horizontal load applied with a 90 kN closed loop electro-hydraulic ram reacting against a strong wall and measured with a 100 kN load cell. A linked pair of steel channels was screwed to the top plate, (but not over the lintels) which transferred the horizontal load to the panel.

Steel rollers were used to prevent out of plane movement of the top plate. Load cells were selected such that they were accurate to within 1% at the peak loads encountered. Linear potentiometers, reading to an accuracy of 0.1mm, were used to measure:

- horizontal deflections of the top and bottom plates
- vertical uplift of the studs at either end of the specimen
- lining slip relative to the frame.

Test load and displacement measurements were recorded using an IBM compatible PC running a proprietary software programme to record the data.

1.4 Phase 1 Experimental Procedure

Three load/displacement protocols were used:

i) ***Monotonic*** - Horizontal load was applied to the top plate at an approximately constant rate of 5 kN/min in one direction only. The specimen was pushed until failure.

ii) ***Cyclic / Load Control*** - The maximum load reached during the monotonic test (i) above, was noted and used as the basis of controlling the loading cycle. The basic cyclic test regime is shown in Table 2.

Load kN	No. of Cycles
± 0.5 Pu*	3
± 0.6 Pu	3
± 0.8 Pu	3
± 0.9 Pu	3
± 1.0 Pu	3

Table 2. Specimen Cyclic Test Regime - Load Control

* Pu = Peak load recorded in Monotonic Test.

iii) ***Cyclic / Displacement Control*** - The displacement Δu reached at the Peak load Pu during the monotonic test was used as the basis of controlling the displacement cycles. Refer Table 3.

Displacement	No. of Cycles
$\pm 0.6\ \Delta_u$	3
$\pm 0.8\ \Delta_u$	3
$\pm 0.9\ \Delta_u$	3
$\pm 1.0\ \Delta_u$	3
$\pm 1.1\ \Delta_u$	3

Table 3 : Specimen Cyclic Test Regime - Displacement Control

1.5 Phase 1 Experimental Observations

1.5.1 General

The lining commonly experienced local distortions at the fastener as the imposed displacement increased. The zone of greatest distortion was generally along the bottom plate, particularly at the extreme corners during initial cycles, but progressing along the full extent of the bottom plate and eventually along each end stud.

In the following section the terms "nails working" or "working hard" (when distortions were more severe) are used to describe the observation that the nail heads were embedding into the lining material. If a nail head pulled through the sheet, this is referred to as "nail head pull through".

The following observations were made at the corresponding horizontal top plate displacements. Observations for specimens subjected to cyclic loading under both load and displacement control were similar and have been grouped together under the one heading. The general description of displacement relates to the horizontal top plate displacement.

The ends of the specimen described as end A or B are shown in Figure 1.

1.5.2. Monotonic Displacement Protocol with P21 Uplift Restraints

1.5.2.1 PB - Specimen W1 (see also Figure 6)

@ Displacement mm	Observations
10	Bottom corner nails were 'working hard', with the remainder of bottom plate fixings "working".
15	Damage increased and top plate nails were observed to be 'working'. Bottom corner fixings had 'pulled through'. This coincided with the maximum resistance.
15-30	Resistance reasonably constant
30	All nails to bottom plate and some of the top plate fixings had 'pulled through'. Large load drop off. Very little uplift of the end studs was observed.

1.5.2.2 FC Specimen W4 (see also Figure 6)

@ Displacement mm	Observations
20	Generally, the lining experienced little damage throughout the test. 'Twisting' of the bottom plate occurred at displacements greater than 20 mm.
30	Bottom plate fixings to foundation at end A almost totally withdrawn. (ie had lifted some 50 mm).
55	At 55 mm displacement the three nails of the P21 restraint were severely bent. The lining however remained largely undamaged.

1.5.2.3 PY Specimen W7 (see also Figure 6)

@ Displacement mm	Observations
15	Fixings began to 'work' along the bottom plate.
18	The tension end stud uplifted from bottom plate approximately 4 mm, equalling the uplift observed between the bottom plate and the foundation.
30	Peak Load recorded
45	Fixings to the lining/bottom plate continued to 'work hard', with nails at both end studs beginning to 'work'.
70	The two plywood sheets were rotating independently of one another. There was little drop off in resistance from the peak at 30 mm, to displacements in excess of 70 mm.

1.5.2.4 BL Specimen W10 (see also Figure 6)

@ Displacement mm	Observations
15	Bottom plate fixings were 'working slightly', and the bottom plate began to lift off the foundation beam. There was no apparent separation of the end stud from the bottom plate.
15-26	Resistance increased with the fixings 'working hard'.
26	The bottom extreme corner fixings were pulled through.
40	Little increase in load but the fixings at the bottom of the end stud began to 'pull through' at each end. The bottom plate was observed to be 'twisting' with the lined side pulling up higher than the unlined side. There was a drop off in load at this point.
55	The fixings down the centre joint between sheets were 'working hard'.
70	The tension end stud was observed lifting from the bottom plate approx 10 mm, and the same distance was noted between bottom plate and foundation. The top plate had separated from the end stud at end A by some 30 mm.

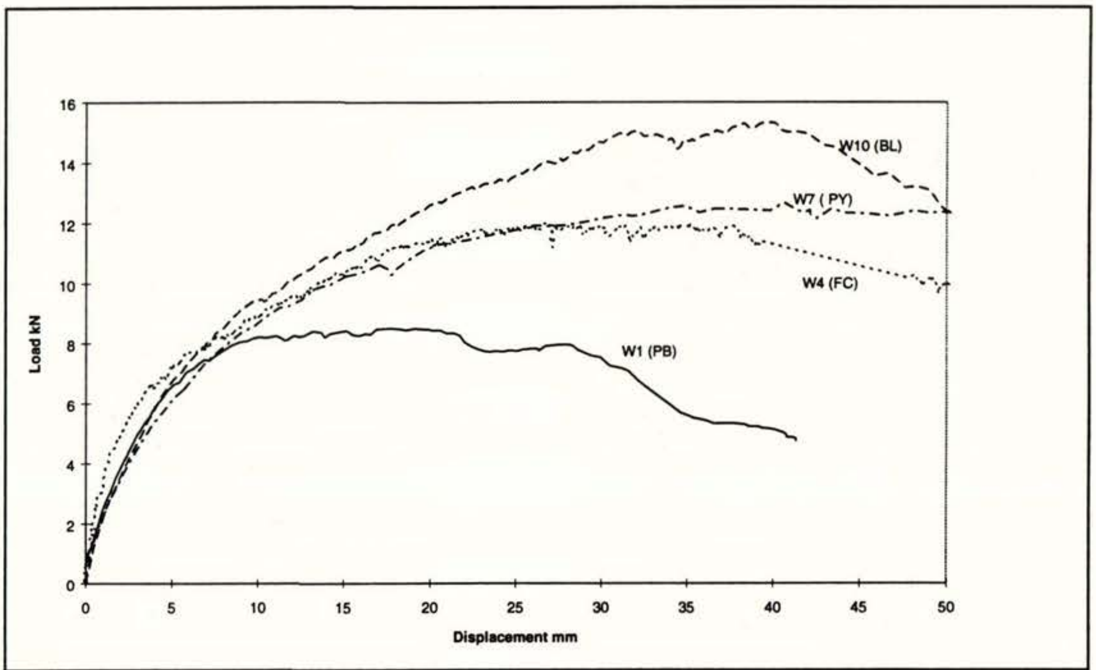


Figure 6 : Load Displacement Response for Specimens subjected to Monotonic Displacement Protocol

The initial stiffnesses (up to 4 kN) are similar, approximately 1.7 kN/mm with the exception of the fibre cement lining which had an initial stiffness of $\approx$3.3 kN/mm.

The plasterboard specimen (W1) attained a peak load of approximately 8 kN at which time there was a marked stiffness degradation. The panel was able to sustain this load over a wide displacement range; up to around 30 mm when there was a drop off in load carrying capability. The stiffness degradation for the other panels was not as severe as W1 and they were able to attain peak loads 50% higher. (Around 12 kN for both W4 and W7 and 14 kN for panel W10).

At the end of the test on panel W10 it was found that the bolts used to secure the angle restraint were very tight, normally they would be only tightened by hand. This may have led to the specimen being over-restrained and hence the higher peak load values. Previous tests carried out indicate that the monotonic curve would usually be similar to those obtained for fibre cement board or plywood because the hold-down strength usually limits the strength of the panel.

In all cases, the panels were able to sustain their load carrying capabilities during large inelastic deformations.

1.5.3. Cyclic Load and Displacement Protocols with P21 End Restraints

1.5.3.1 PB Specimens W2 & W3 (see also Figure 7)

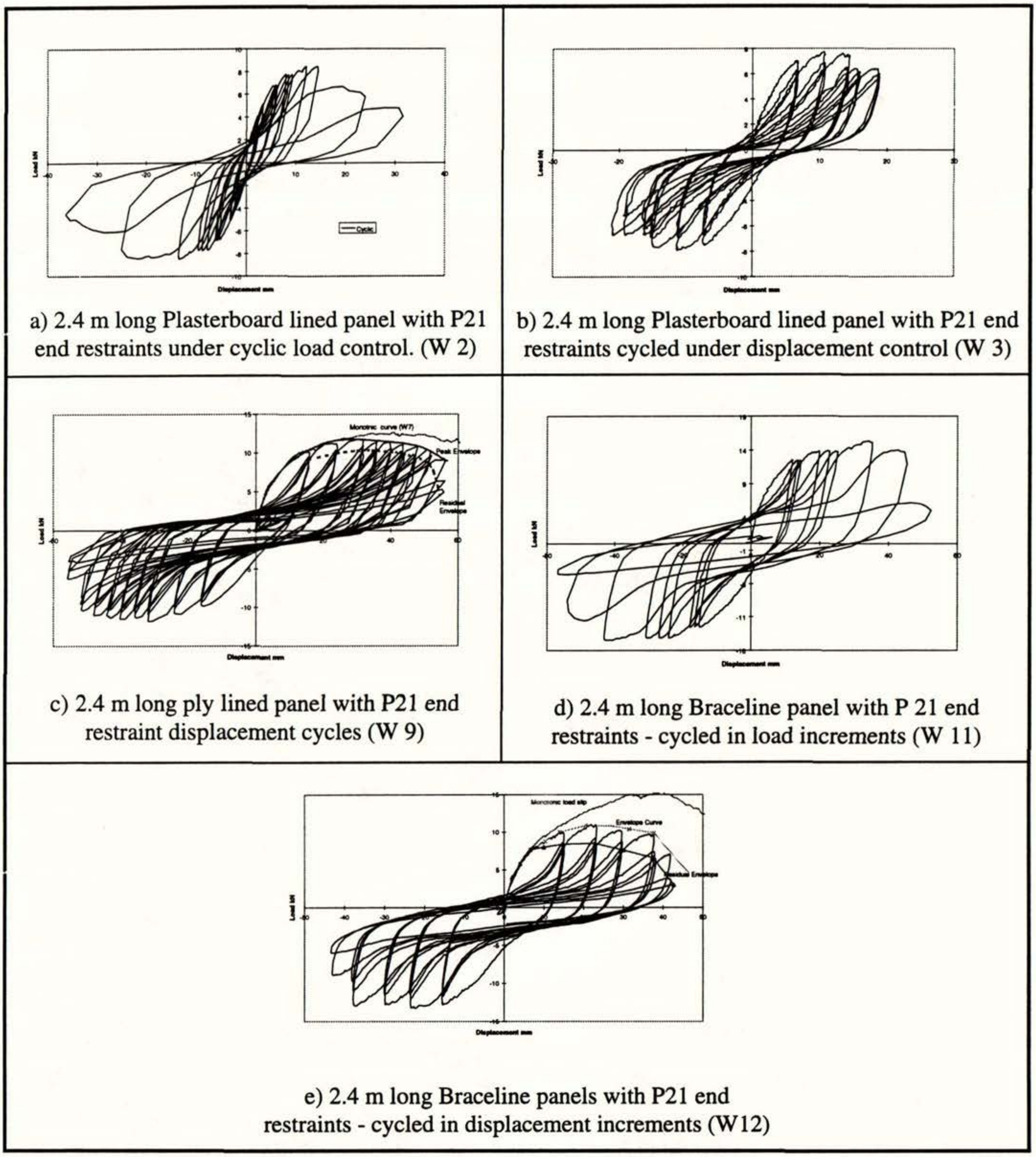
@ Displacement mm	Observations
5	The bottom corner fixings were seen to be 'working'. There was little increase in displacement recorded prior to the bottom corner of the sheet fracturing. As displacements increased the top plate fixings began to work.
11	Complete 'pull through' occurred at the bottom corner fixings and the load dropped off in subsequent cycles. There was no uplift of bottom plate from the foundation beam.

1.5.3.2 PY Specimens W8 & W7 (see also Figure 7)

@ Displacement mm	Observations
24	The bottom plate had lifted from the foundation beam by 2 mm and the end studs were just beginning to separate from the bottom plate.
36	The two sheets were observed to be rotating separately and the amount of differential movement across the join was approximately 10 mm. Bottom nails were 'working hard' at this point.
48	During the 48 mm cycles the bottom fixings and centre nail fixings began to withdraw from the framing and nail heads embedded into the linings.
56	There was a drop off in load as several nails along the vertical join sheared through completely, due to fatigue failure and the bottom plate fixings withdrew from the frame. The bottom plate remained securely attached to the foundation beam whilst the end studs separated from the bottom plate. On completion of the test the loose clouts were removed from the lining and a slight ovaling of the nail hole and crushing of the lining under the nail head was observed. Otherwise the plywood lining was undamaged.

1.5.3.3 BL Specimens W11 & W12 (see also Figure 7)

@ Displacement mm	Observations
9	Nails of bottom corners were 'working hard' and began to embed into the paper face.
15	The end stud at end A was seen to be lifting from the bottom plate by approx 6 mm, although no sheet slip was noticeable nor any uplift of the bottom plate from the foundation observed.
23	The bottom corner fixings had experienced complete pulled through. The peak load was recorded at this point
30	Stud uplift increased to approximately 12 mm. No observable movement of the bottom plate had occurred at end A, however the bottom plate was lifting some 5 mm during the reverse cycle at end B.
38	Nail pull-through occurred along each end stud during the tension cycle for a distance of 600 mm up from the bottom plate.



a) 2.4 m long Plasterboard lined panel with P21 end restraints under cyclic load control. (W 2)

b) 2.4 m long Plasterboard lined panel with P21 end restraints cycled under displacement control (W 3)

c) 2.4 m long ply lined panel with P21 end restraint displacement cycles (W 9)

d) 2.4 m long Braceline panel with P 21 end restraints - cycled in load increments (W 11)

e) 2.4 m long Braceline panels with P21 end restraints - cycled in displacement increments (W12)

Figure 7 : Cyclic and Monotonic Load Regime for 2.4 m long panels with different linings (Observe that plot scales may differ)

Cyclically loading panels W2, W5, W8 and W11 (which were identical to the panels tested monotonically), produced similar results in each case. The first cycle followed the monotonic curve almost identically. Subsequent cycles to the same displacement showed a load degradation, which was greater between 1st and 2nd cycles than between subsequent cycles. On the first push beyond the initial displacement (for small displacements at loads less than the peak load) it again reached the monotonic curve. The monotonic curve and the parent curve of the cyclically load test specimens were reasonably well matched up to peak loading. (In all cases specimens under cyclic loading regime did not quite reach the peak loads attained during the monotonic test regime). Cycles beyond the peak load showed an increasingly large variance from the monotonic curve. The variation in load degradation for successive cycles is shown in Figure 8. The plot shows the percentage of load degradation in each cycle from the monotonic curve at various displacements for panel W10 (monotonic) and W12 (cyclic/displacement). Figure 7 graphically shows the effects that cyclic loading has on panels with respect to similar panels subjected to monotonic load only.

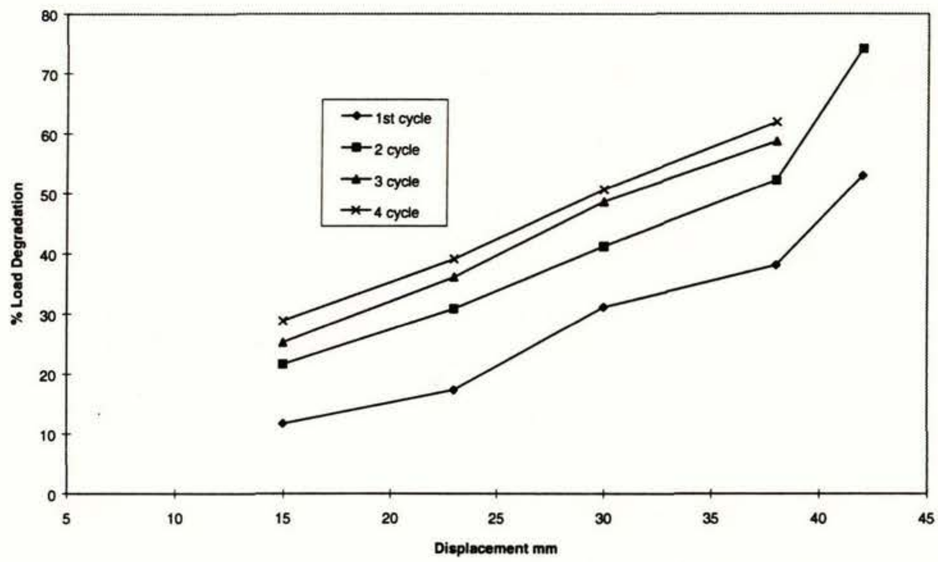


Figure 8 : Percentage Load Degradation from Monotonic Curve (BL) Specimens W10 and W12

There are two features of the test results which are worth highlighting. Both occurred for the load controlled and displacement controlled regimes:

(i) Little degradation occurs during the cyclic regime at low loads/displacements. Figure 9 shows the displacements at varying percentages of ultimate load (derived from the monotonic test) for 1st to 4th cycles for specimen W6 (plywood and cyclic/disp control). It shows that for loads up to 0.5 of the peak load, little load degradation occurs during cyclic loading. The value of Pu was taken for both the positive and negative cycles. Similar trends were observed for other panels.

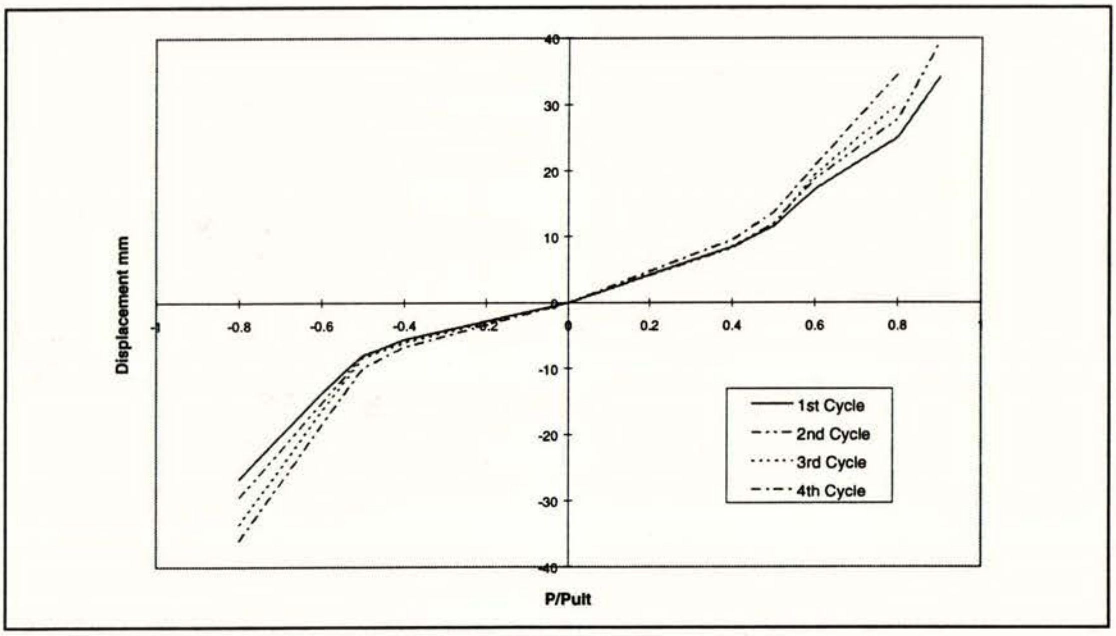


Figure 9 : Displacement at Successive Cycles as a Ratio of Peak Load

(ii) The residual (ie the displacement at zero load) after each cycle is a function of the maximum displacement of that cycle. Table 4 shows the ratio of maximum displacement to residual displacement for specimens W2 and W5, each of which was tested with P21 end restraints and under cyclic/load control. The results show a reasonably constant ratio for all loading cycles up to peak load.

Panel	Load	Displacement (mm)	Residual (mm)	Disp/Residual
W2 (PB)	0.5 Pu	2.3	0.9	2.5
		2.3	0.8	2.9
		2.4	0.9	2.8
	0.6 Pu	3.3	1.3	2.6
		3.4	1.2	2.8
		3.4	-	-
	0.8 Pu	5.6	2.4	2.4
		6.0	2.4	2.5
		6.2	2.5	2.5
	Max	12.0	6.0	2.0
		14.4	7.0	2.1
W5 (FC)	0.5 Pu	8.4	4.9	1.7
		8.2	3.7	2.2
		8.4	5.0	1.7
	0.6 Pu	17.2	10.5	1.6
		18.8	12.0	1.6
		19.4	12.0	1.6
	0.8 Pu	25.0	16.2	1.5
		27.8	18.9	1.5
		30.1	19.9	1.5
	Max	34.1	23.2	1.5
		39.5	26.0	1.5
		42.7	31.1	1.4

Table 4 : Ratio Of Displacement To Residual Displacement

1.5.4 Cyclic Load and Cyclic Displacement Test Regime Using Various End Restraints

The influence of different supplementary end restraints was investigated by conducting a further three tests on panels identical to W10 (ie bracing grade plasterboard) using different end stud restraints.

1.5.5.1 Tie Rod Restraint Specimens W13 & W16

Displacement mm	Imposed Load kN	Observations Panel W13
8	9	Little damage was observed until the 9 kN cycle when both bottom plate corner fixings began 'working'.
9	10	During the 1st of the 10 kN cycles, all the bottom plate fixings were observed working, with those in the extreme corners 'working hard'.
10	-	During the 2nd cycle the stopped joint fractured at the top of the specimen. The fracture extended down a third of the height of the joint. There was a subsequent drop off in load. There was no apparent 'working' of the end stud fixings. Subsequent cycles saw the nails along the central joint between the sheets working and starting to show through the stopping. Fixings along the top plate were also 'working'.
Displacement mm	**Applied Load kN**	**Observations Panel W16**
14	20	Little damage was observed until the 20 kN load cycle, at which time the bottom fixings were 'working'. The sheathing around the both sets of bottom corner nails broke away suddenly.
20	22	Nails along the bottom edge were 'working very hard' and 'nail head pull through' was observed at all corners.
30	24	'Nail head pull through' continued to increase in extent during the 24 kN cycle and the fixings along the centre stud began to 'work.' At this stage no damage was observed along the vertical edges of the panel and the fixings were only 'slightly working' along the top edge.
40+	26	The bottom fixings completely 'pulled through'. Nails along the centre stud up to 300 mm were observed to be 'working hard'.

The load/displacement plot for Panel W13 is shown in Figure 10. The initial stiffness of the panel was higher than the monotonic load curve (using the P21 type end restraint) at approximately 2 kN/mm. Maximum loads on each cycle followed the monotonic curve up to a peak value of 10 kN, at which point there was failure, by nail head pull through along the bottom plate, with a consequential drop off in load carrying capability.

The plot demonstrates the relatively brittle performance of a panel which was restrained from end stud uplift and lined with a degrading material. Some ductility is normally afforded by the partial relaxation of the end restraint, however in this instance, these inelastic deformations were prevented. All the load was taken through the sheeting fixings into the sheet which degraded rapidly and eventually exhibited a brittle type failure.

It would seem prudent when evaluating this type of panel to assume that it is an elastically responding element and assign it with a structural ductility factor of 1 in accordance with NZS 4203 (SNZ 1992). However, under the current P21 evaluation method (which assigns the on-set of inelastic behaviour to occur at a displacement at which half the maximum load occurs) (King & Lim 1991), specimen W13 would be rated with a yield displacement of approximately 3mm. As the maximum load was at a displacement of 12 mm, the ductility of the panel would be 12/3 = 4.

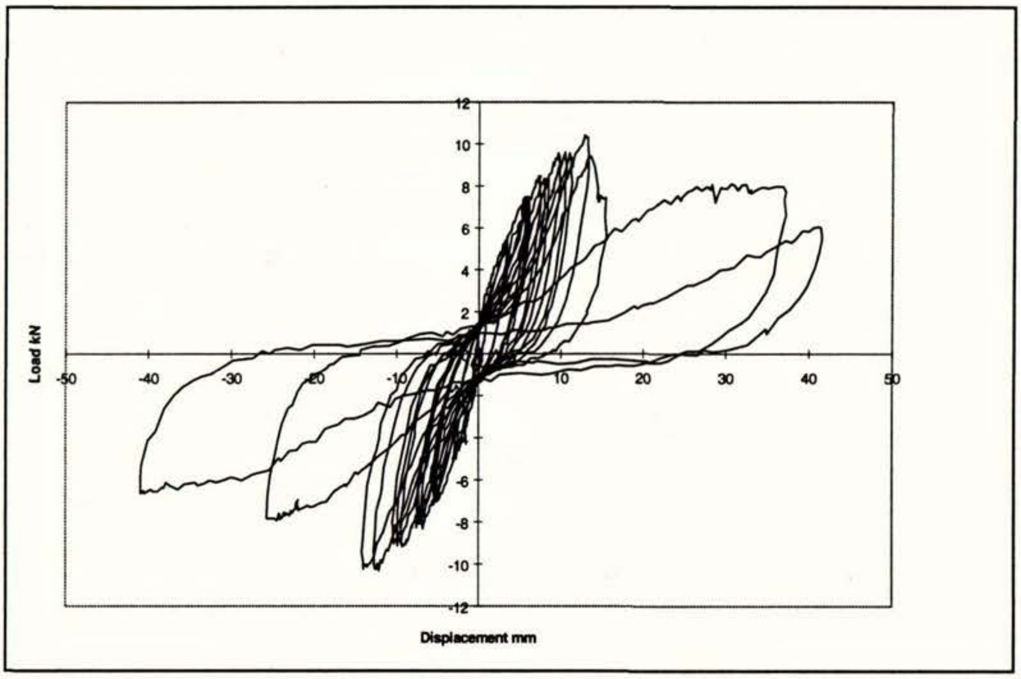


Figure 10 : Hysteresis Loops for Panel W13 - Specimen with Tie-Down Rod Restraint

1.5.5.2 Vertical Load Used as Uplift Restraint Specimen W14

Displacement mm	Applied Load kN	Observations
8	9	There was little uplift of the end stud - approximately 1 mm at the 9 kN cycle.
10	≈10	During the first excursion to 10 kN a majority of the fixings along the bottom plate 'pulled through', however there was little drop off in load carrying capacity.
50		Bottom fixings 'pulled through' the lining completely and the nails along the end studs were 'working very hard' up to 600 mm from the bottom plate.

The load/displacement plot is shown in Figure 11. Up to peak load, the hysteresis loops were similar to those using the tie down restraint. At the peak load however (≈10 kN and 10 mm deflection) there was no drop off in load carrying capacity. Subsequent cycles were able to reach the nearly the same peak load even with large in-elastic deformations. Peak loads of ≈10 kN were attained at top plate displacements of between 10 mm and 45 mm, producing stable 'fat' hysteresis loops. A single cycle at approximately 60 mm resulted in a drop off in resistance of around 20%.

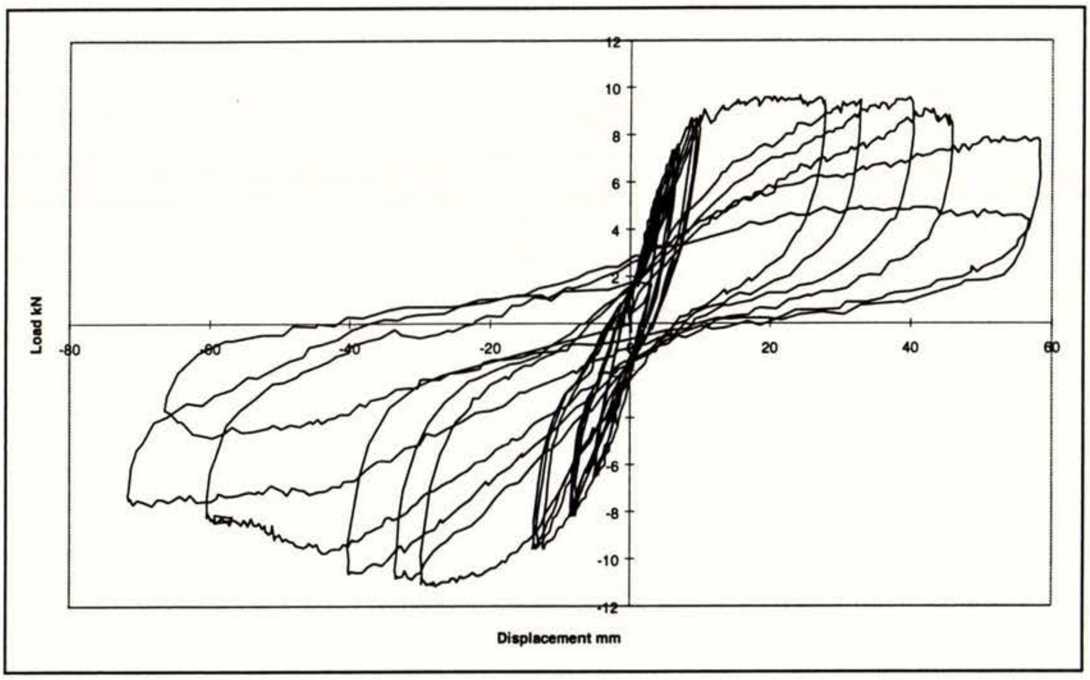


Figure 11 : Hysteresis Plot for Specimen W14 - Vertically Loaded Top Plate

1.5.5.3 No Uplift Restraint Specimen W15

Displacement mm	Applied Load kN	Observations Panel W15
6	6	The bottom plate lifted approximately 3 mm from the foundation beam.
12	≈7	The separation increased during the 8 kN cycle with the bottom plate only partially re-seated during the reverse cycles.
25	9	Separation of the bottom plate from the foundation increased until the whole of the bottom plate was 'floating' 10 mm above the foundation beam. There was no observable damage to the linings at the end of the test.

Initial stiffness was lower during the cyclic test than for the monotonic load regime. A peak load of just under 7 kN was attained, and with little reduction sustained over large displacements. Residual displacements were high when compared to other types of end restraint conditions.

1.5.5 Panel W17 (Lintel Specimen)

Displacement mm	Applied Load kN	Observations
6	5	During the 6 mm cycles fixings along the bottom plate were beginning to 'work'. A hairline crack appeared at both lintel joints on line with the outer stud.
16	8	Bottom plate fixings began to 'pull through'. Crack widths at the lintels increased and extended the full depth of the lintel.
24	8	The lintel joints were observed to buckle out of plane and the stopped joint fractured at the junction with the panel sheathing line. There was approximately 10 mm bottom plate uplift and 3 mm separation between stud and bottom plate at end B of the panel. However at end A, the uplifts were 8 mm and zero respectively. The fixings along the top plate over the openings were 'working' but this was not observed to be occurring over the main body of the panel.
36	7	Both lintels buckled out of plane and remained so when the load was removed. The bottom plate was observed to have uplifted 20 mm at end B of the panel but not at all at end A. All the bottom plate fixings had 'pulled through' and fixings 300 mm up the end studs were 'working hard'. There was little drop off in load.
40	7	The top plate fractured at the point where the metal strap diagonal brace was cut into that member (ie a notch type failure of the plate) and the test was terminated.

The nature of the hysteresis loops produced in this test was consistent with the results obtained for panels tested using the standard P21 end restraints. A comparison was made between the uplift restraining force that could be attributed to the P21 end restraint and the effective restraint of the lintel.

Thurston (1993) found that there is a close relationship between the wall racking resistance, Pw, and rigid body rotation displacement, Δ w (determined by correcting the system deflection by the panel rotation) obtained during a P21 test. He plotted a number of test results and used best-fit curves matching techniques to develop a relationship between resistance and panel rigid body rotation. For a 1.8 m long wall with no end straps the relationship was given by:

$$Pw = (8.7 \times \Delta w) / (1.3 + \Delta w^{0.95})$$

This is plotted on Fig 11 for rigid body rotation displacements up to 30 mm.

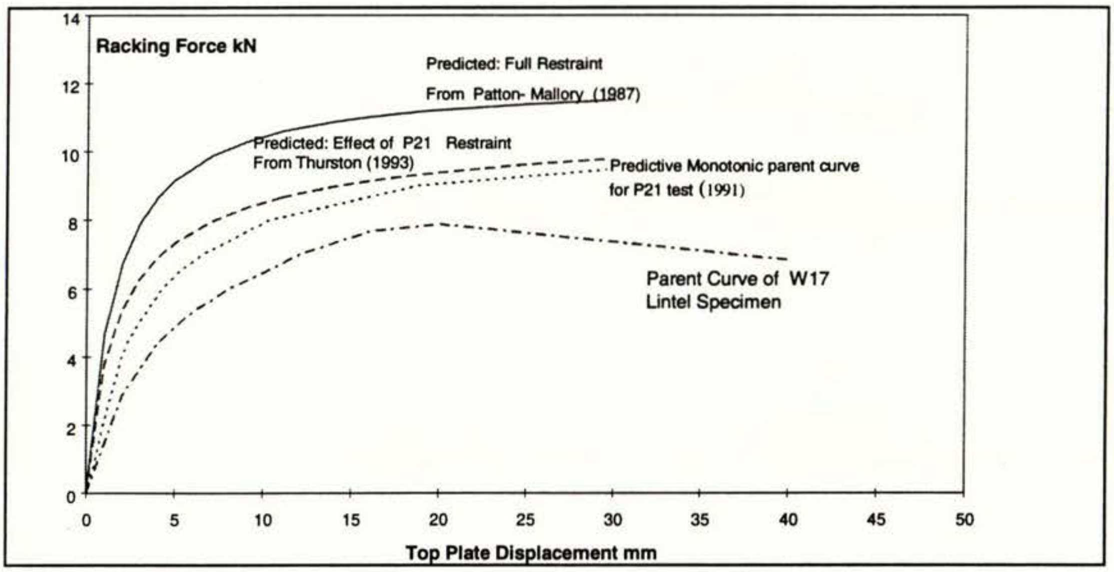


Figure 12 : Predicted Monotonic Parent Curves - Panel W17

The predicted full restraint parent curve shown in Figure 12 was calculated using the theory published by Patton-Mallory and McCutcheon (1987), assuming a panel length of 1.8 m and that the panel was lined both sides with standard 9.5 mm Gib board. Nail slip curves used were those established by numerous past tests carried out at BRANZ. To this was added the parent curve as determined by Thurston (1993) of racking load and rigid body rotation displacement for a standard P21 end restraint with a 1.8 m long panel. The result is the predicted cyclic parent curve for a P21 test on a 1.8 m long panel of height 2.4 m. This curve can be compared to the parent curve of the first cycle of wall W17, (the lintel specimen) and shows the lintel restraint as being less than the P21 restraint.

The amount of restraint required to replicate the lintel can be determined as follows:

From Thurston (1993) the top plate horizontal deflection of say 10 mm due to rigid body rotation on a 1.8 m long panel, using a standard P21 restraint is caused by a racking load Pw of :

$$Pw = (8.7 \times 10) / (1.3 + 10^{0.95}) = 8.5 \text{ kN}$$

From simple statics this corresponds to an uplift force of the end stud

$$= 8.5 \times 2.4/1.8 = 11.4 \text{ kN.}$$

From a series of small scale tests (as described in Appendix A) the average uplift capacity of the P21 end restraint was found to be 6.5 kN.

$\therefore$ The bottom plate contribution / foundation fixings = 11.4 - 6.5 kN = 4.9 kN

At the same displacement of 10 mm, the racking force on the lintel specimen is 6 kN i.e. an uplift force on the end stud of 6 x 2.4 / 1.8 = 8 kN.

The additional restraint required of the end stud over and above that afforded by the bottom plate to foundation fixings is therefore:

$$= 8 - 4.9 = 3.1 \text{ kN}$$

$$\text{i.e. an equivalent of } \frac{3.1}{6.5} \times 3 \text{ (no. nails)} = 1.4 \text{ nails}$$

Therefore it is anticipated that a partial restraint of one or two 3.75 mm ϕ flat head nails would best represent the restraint at door openings. Further testing was carried out in Test Programme II to confirm the restraint required to simulate the lintel.

1.5.6. Description of Light Timber Frame Bracing Panel Hysteretic Behaviour

Figure 7 shows the typical hysteretic behaviour of the lined panels. In the first part of the curve the lining fixings bend elastically and the nail shank crushes the lining material and timber. As the lateral shear load increases the nails deform plastically with the yield point being near the timber/lining shear interface. As deformation continues, axial tension is established within the nail as the nail head bears against the external face of the sheathing. Partial withdrawal of the fixing typically results provided the compressive strength of the lining is sufficient. Alternatively the lining may crush beneath the fastener head. Up until this point, the system is largely elastic and full deformation recovery is usually experienced upon load removal.

With the application of further panel deformation, the nail curvature increases and to maintain nail head/shank geometry, either the nail head becomes embedded into the lining material and /or the fasteners withdraw further from the timber frame. The third possibility of the tension yielding of the fixing is very rare and can be disregarded.

From the onset of inelastic behaviour, when the load is reduced the gradient of the unloading curve is generally similar to that of the initial loading curve.

At zero load there is a residual displacement due to inelastic deformations at the fixings. Loading in the negative direction produces similar hysteresis loops.

During the next cycle to the same displacement, the nail shank is unsupported due to the previous crushing of the lining and timber. Hence the load is resisted by the cantilever action of the nail, with the support point of the nail being at the intersection with the uncrushed timber.

When the panel displacement approaches the previous cycle displacement the nail shank once again becomes supported by the crushed material and there is a corresponding strength and stiffness increase. However, the maximum load on the second cycle does not reach the maximum load of the first.

Subsequent cycles show that the hysteresis curve follows that of the second cycle, with little stiffness degradation. Once displacements reach that of previous cycles the nail shank again bears onto the uncrushed material and there is an increase in strength and stiffness. There is also additional nail withdrawal from the timber. Load increase occurs up to approximately the parent monotonic curve.

Note that no nail withdrawal was observed for Plasterboard (PB), Fibrecement board (FC) or Braceline (BL) linings.

1.6 Conclusions to Experimental Programme Phase I

- The on-set of damage to timber bracing panels lined with degrading sheets can be identified by subjecting test specimens to increasing cyclic displacements.
- There is no significant difference in either the maximum lateral resistance or in the related maximum reliable displacement to which a specimen can attain by adopting a load controlled or displacement controlled load regime.
- Degradation of test specimens is minimal under cyclic loading to loads up to 0.4 of peak load.
- The boundary restraint of a bracing panel which is taped and stopped and which terminates at a return wall can be conservatively replicated by a 12 kN hold down.
- The restraint afforded to a bracing panel terminating at a door opening is less than the current P21 method (King & Lim 1991). It is anticipated that a partial restraint equivalent to 3 kN would be more appropriate.
- The Serviceability Limit State (i.e. the onset of cracking) in the lintel specimen was observed as occurring at a top lateral displacement of 6 mm.
- The ratio of residual displacement after each cycle to the maximum displacement of that cycle is reasonably constant up to peak load.

2. EXPERIMENTAL PROGRAMME PHASE II

Test Programme I indicated that the restraint afforded by door lintels is significantly lower than the P21 end restraint. Further tests were carried out in Phase II to ascertain the extent of this reduction observed in Phase I.

The degree of restraint afforded by taped and stopped wall junctions was also to be greater than that provided by the standard P21 end restraint. A 2.4 m long panel (W24) was tested using additional end stud restraint to simulate this condition and assess its significance.

The results for specimen W24 were eventually added to the results from W17 (lintel specimen) and compared to an equivalent long wall tested by Thurston (1993).

All panels were tested in accordance with the revised test regime described in Section 2.3.

2.1 Objective

This experimental programme was designed to:

- Determine the effect that different lining materials and panel lengths had on the panel resistance rating using P21 end restraint and tie down rods.
- Determine whether the proposed method of simulating in-service boundary conditions was valid for door openings and/or wall junctions.

2.2 Description of Test Specimens

The panel configurations studied are shown in Table 5.

Panel	Lining	Restraint	Length
W18	BL	P21	1.2 m
W19	BL	Tie Rod	1.2 m
W20	PY	P21	1.2 m
W21	PY	Tie Rod	1.2 m
W22	PB + BL*	P21[1]	1.2 m
W23	PB + PB*	P21[2]	1.8 m
W24	PB + PB*	P21[3]	2.4 m

Table 5 : Test Specimen Configuration

* PB + PB denotes a panel lined both sides with plasterboard etc.

1. Standard P21 restraint plus a 25 x 1 mm galv. steel strap used at each end.
2. Standard P21 restraint one end. Only 1 No. 100 x 4 FH nail other end.
3. Standard P21 restraint one end. 12 kN end restraint other end.

Frame construction and test set-up was as for Phase I and described in section 0. Nailing patterns for the different lining materials was identical to those used in Phase I for each sheathing type.

Panel W22 had two galvanised steel uplift restraining straps fixed to each end stud and to the foundation beam using six 30 x 2.5 nails to both the stud and foundation beam.

Panel W23 had a 25 x 1 mm (nominal) galvanised high tensile steel strap to form a diagonal brace within the main body of the specimen. A comparison could then be made of the observations and results with those of the (isolated) lintel specimen (Panel W17) and the panel tested by Thurston (1993) both of which had a diagonal brace.

2.3 Displacement Protocol

The following loading regime was adopted:

Displacement	No. of cycles
± 8 mm	4
± 15 mm	3
± 20 mm	3
± 25 mm	3
± 5 mm increments	3

Table 6 : Test Load Regime for Test Programme II

2.4 Observations

2.4.1 General

The lining commonly experienced local distortions at the fastener as the imposed displacement increased. The zone of greatest distortion was generally along the bottom plate, particularly at the extreme corners during initial cycles, but progressing along the full extent of the bottom plate and eventually along each end stud.

In the following section the term "nails working" or "working hard" (when distortions were more severe) are used to describe the observation that the nail heads were embedding into the lining material. If a nail head pulled through the sheet, this is referred to as "nail head pull through".

The following observations were made at the corresponding horizontal top plate displacements. The general description of displacement relates to the horizontal top plate displacement.

The ends of the specimen described as end A or B are shown in Figure 1.

2.4.2 Panel W18 - BL with P21 Restraint

@ Displacement mm	Observations
15	Bottom corner nails began 'working hard'.
20	During the 1st cycle corner nails began to 'pull-through'. All bottom fixings were 'working'. Each tension stud uplift approx 5 mm above foundation.
30	Bottom fixings 'working hard'. 'Nail head pull through' of bottom corner fixings. Top fixings 'working'.
35	'Nail head pull through' of bottom fixings.

2.4.3 Panel W19 - BL with Tie Down Rod Restraint

@ Displacement mm	Observations
0-30	Damage to bottom fixings were similar to Panel W18. Top fixings experienced the same extent of damage as the bottom fixings.
35	End stud fixings began "working hard'.
40	Virtually all the top and bottom fixings experienced 'nail head pull through'.
40+	End stud fixings experienced 'nail head pull through' for a length of approx 400 mm from top and bottom plates. Studs experienced significant minor axis flexural deformation.

Comments

Hysteresis loops produced for these panels were very similar. Peak loads resisted were both just under 6 kN at approximately the same displacement of 25 mm. The degradation in load carrying capacity on the 4th cycle was also very similar.

Panel W18 showed some asymmetric performance with the pull cycle being somewhat weaker (ie a peak of 4 kN) than in the push direction.

Maximum reliable displacements for the two panels were both determined to be 25 mm. Initial stiffnesses were 0.8 and 0.6 kN/mm for W18 and W19 respectively.

2.4.4 Panel W20 - PY with P21 Restraint

@ Displacement mm	Observations
15	Bottom corner fixings began 'working'.
20	Bottom plate at end A split at the junction with the nails into the foundation beam, approx 75 mm from the end of the plate. The split extended to the end of the plate.
30	All of the bottom fixings and 300 mm up each end stud were 'working'.
40	Bottom plate split at end B in a similar manner to that which occurred at end A. The bottom plate extreme corner nails had noticeably withdrawn. Stud uplift was measured at 8 mm.
50	Bottom plate nails over 300 mm from each end showed withdrawal. Further splitting of the bottom plate was apparent.

2.4.5 Panel W21- PY with Tie Down Rod Restraint

@ Displacement mm	Observations
	Observations were generally the same as those for panel W 20. Fixings along the top plate behaved in a similar manner to bottom plate fixings throughout the test.
45	By the 45 mm cycles the end studs were observed to be experiencing minor axis flexural deformation, similar to that experienced by Panel W19.
55	On the 55 mm cycles virtually all of the top and bottom plate fixings experienced 'nail head pull through'.

Comments

Both tested panels produce symmetrical hysteresis loops. Peak load for W21 (7.5 kN @ 35 mm) was significantly higher than for W20 (5.5 kN @ 35 mm) indicating that the full tie- down situation is certainly not a lower bound condition.

Maximum reliable displacements were determined to be 35 mm for both panels, (cf 25 mm for panels W18 and W19). Initial stiffnesses for the two panels was 0.5 kN/mm and 0.4 kN/mm respectively.

2.4.6 Panel W22 - PB + BL with end Straps. P21 Restraint

@ Displacement mm	Observations
20	Bottom corner fixings to PB lining began 'working'.
25	Bottom corner fixings to BL lining began 'working'.
30	All of the bottom fixings and 300 mm up each end stud were 'working'. The foundation beam was observed to be lifting from the test rig at end B. This was due to insufficient hold down of the foundation beam to the rig at this point. The uplift was measured as 7 mm on the 30 mm cycle.
35	Bottom corner fixings on both sides were 'working hard'.
40	All bottom corner fixings and bottom fixings to the PB lining experienced 'nail head pull through'. The remainder of the bottom fixings to the BL were 'working very hard'.
45	Damage to the bottom plate fixings was severe, with the first three fixings away from each corner having pulled through on both sides with the remainder free to move within 'slots' formed in both linings, stud uplift was approx 6 mm.
55	The bottom plate split at the junction with the hold down straps and the nails to straps began to withdraw. Hold down straps also began to buckle under compression load.

Comments

The maximum reliable displacement of panel W22 was the greatest recorded for any of the tests undertaken, at 50 mm. This amply demonstrates the greater displacement capacity of shorter length panels when adequately tied to the foundations.

There was a highly asymmetric result due in part to uplift of the foundation beam at end B as noted in the observations. However this would have had little effect on the hysteresis loops produced for loads applied in the push direction.

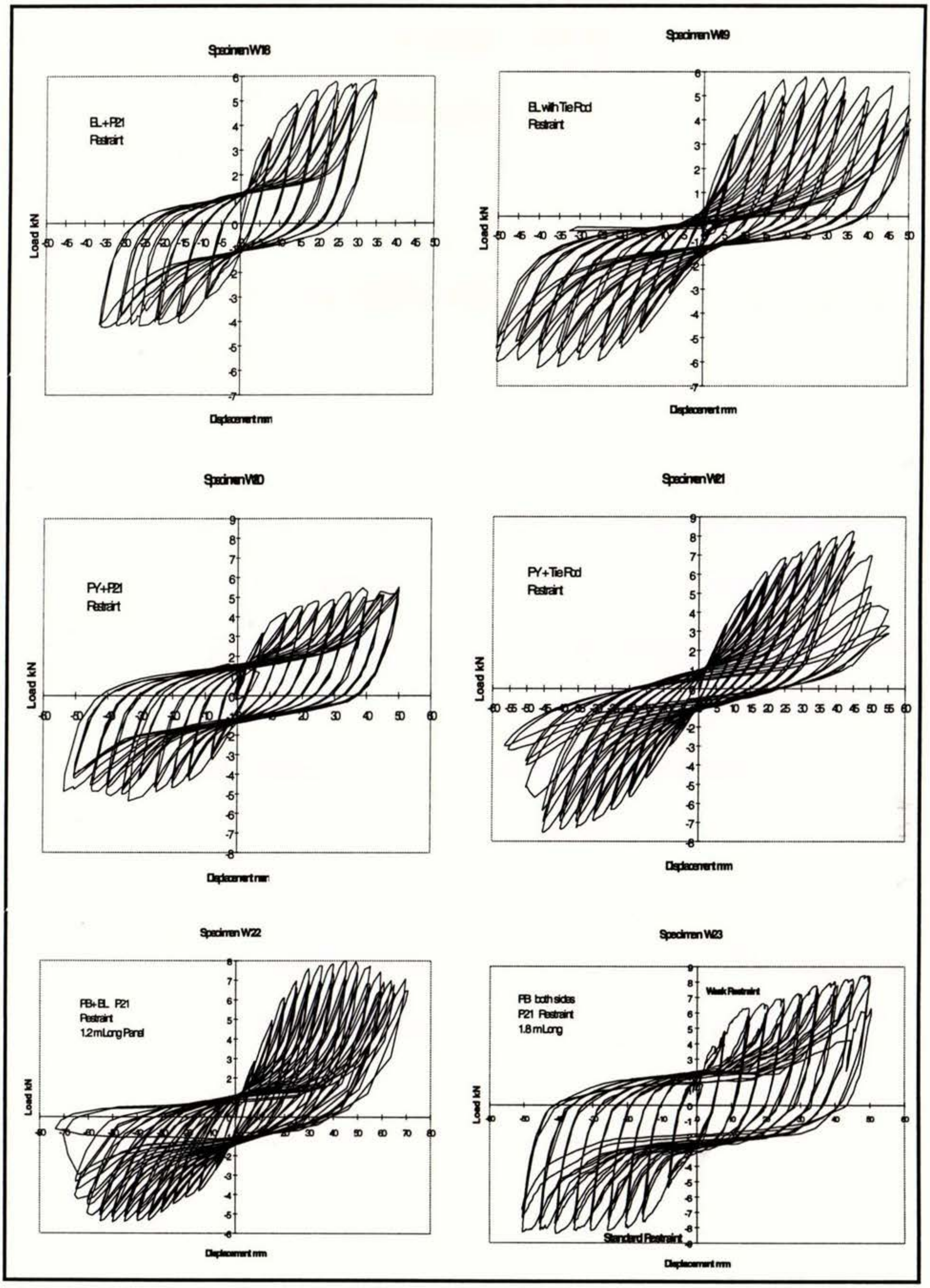


Figure 13 : Hysteresis plots for Specimens W18-W23

2.4.7 Panel W23 - PB Both Sides with P21 Restraint One End and Minimal Restraint the Other

@ Displacement mm	Observations
8	Bottom corner fixings began 'working'.
15	Bottom corner fixings experienced 'nail head pull through'. A split in the bottom plate was observed at the junction with the diagonal strap.
20	The remainder of the bottom fixings were 'working hard'. The end studs were observed to be lifting approx 12 mm from the bottom plate. Racking loads were lower than anticipated and a second 100 x 3.75 FH nail was added to the restraint at end A - making a total of two nails at this end.
25	Bottom plate fixings began to experience 'nail head pull through'. Stud uplift was observed to be 25 mm at end A and 12 mm at end B.
40	Fixings along each end stud began to 'work hard'.
50	All bottom plate fixing experienced 'nail head pull through'. No damage to the top plate fixings were observed.

Comments

The panel exhibited an asymmetric performance due to the different end restraints used. Peak load was not as high as predicted due to splitting of the bottom plate at 8 mm cycles. The peak load on the push cycle, 4.2 kN @ 25 mm (with 1 No. nail restraint) was considerably lower than for the pull cycle, 8 kN @ 40 mm.

An additional nail was added for the 30 mm and subsequent cycles. The effect of this can be seen in the 'step up' in the parent curve at those cycles. The peak loads reached on the push cycle were still less than the pull after the addition of the nail.

2.4.8 Panel W24 - PB Both Sides with P21 Restraint One End and 12 kN Restraint the Other

@ Displacement mm	Observations
8	Bottom corner fixings began 'working hard'.
15	All fixings along the bottom plate were 'working hard'. There was approximately 8 mm uplift at end A (additional restrained end) and 12 mm at end B.
25	Top plate fixings began 'working'. End stud uplift at end A was measured as 6 mm between bottom plate and foundation and 6 mm between bottom plate and stud. The corresponding uplift at end B being 16 mm and 1 mm.
30	The top plate was observed to be moving independently of the frame, with the top plate displacing approx 10 mm more than the top of the end studs. ie the top plate/stud nail shear resistance markedly diminished.
35	The top plate split at the junction with the diagonal brace.

Comments

The results showed a symmetric performance although end A was restrained by six nails.

From section 0 it can be seen that the uplift restraint provided by the standard P21 end restraint on a 2.4 m long panel together with bottom plate nailing was 14.1 kN. This is somewhat lower than the uplift attained in the testing of panel W24 which was approximately 17 kN on a similar boundary restraint. The initial stiffness, k of the panel in the push direction was 3 kN/mm.

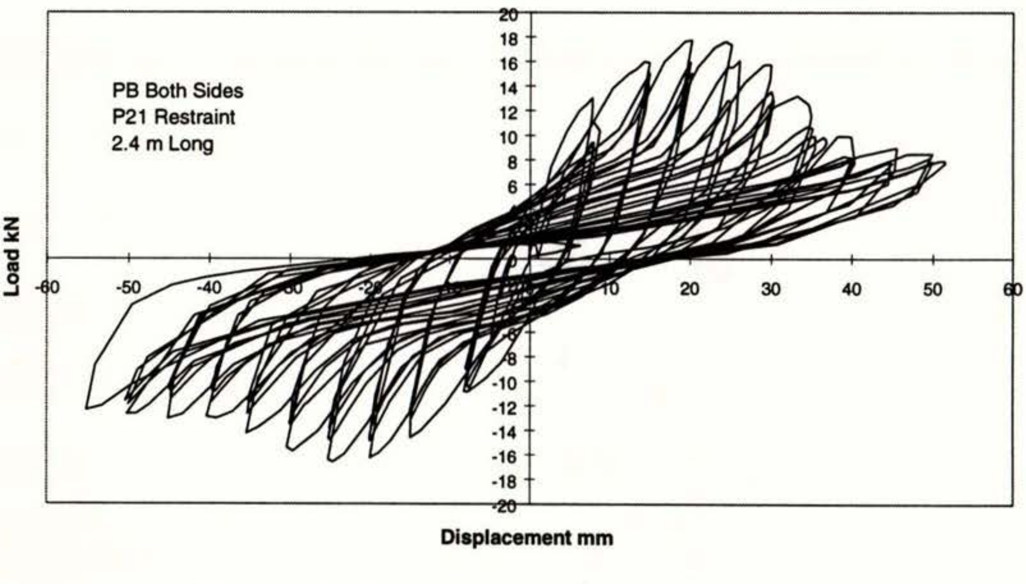


Figure 14 : Hysteresis Loop for Specimen W24

2.5 Summary of Performance Evaluation of Phase II Systems

The results from each panel were analysed using the draft evaluation procedure outlined in Volume 1 Appendix A of this report to ascertain the mass in each case which could be dependably supported by each respective wall bracing panel.

The seismic mass rating procedure outlined in section 3.2 was completed for Test Programme II specimens and the results are shown in Table 7, using the NZA artificial earthquake record at 5% damping.

Panel	Sp. Disp. mm	Stiffness K kN/mm	Period T secs.	Restrained Seismic Mass kg/Panel
W18	10	0.8	0.20	770
BL +	20		0.26	1410
	30		0.30	1770
P21 Restraint	40		0.33	2200
W19	10	0.6	0.20	610
BL +	20		0.28	1190
	30		0.32	1540
Tie Rod	40		0.35	1830
W20	10	0.5	0.21	530
PY +	20		0.29	1090
	30		0.34	1450
P21 Restraint	40		0.38	1850
W21	10	0.4	0.22	490
PY +	20		0.32	1000
	30		0.39	1540
Tie Rod	40		0.45	2000
W22	10	0.5	0.20	510
PB+BL	20		0.30	1140
	30		0.35	1550
P21 Restraint	40		0.42	2230
W23	10	1.4	0.18	1150
PB+PB	20		0.23	1870
	30		0.27	2580
P21 Restraint	40		0.30	3190
W24	10	3	0.18	2460
Push Cycle	20		0.24	4190
	30		0.27	5530
	40		0.31	7100
	50		0.34	8160

Table 7 : Restrained Masses for Specimens W18-W24

2.6 Comparison of Individual Test Panel Results with Previous Work

Thurston (1993) carried out a series of racking tests on long walls with various window and door openings. He concluded that the actual resistance provided by a bracing panel which terminates at a door opening may be significantly less than those determined by tests when the current P21 end restraint is used (King & Lim 1991). Conversely panels which terminate at a return wall actually experience full hold down restraint.

It is shown in section four that panels which are fully taped and stopped at a return wall have greater restraint than the P21 end restraint and that for timber framed walls on timber foundations this can be replicated by using a 12 kN restraint.

A 2.4 m panel (W24) was tested which had an equivalent 12 kN hold down applied to one of the end studs and the results together with the results of the Lintel Panel (W17) were combined and compared to one of the composite long walls tested by Thurston. (The individual panels W24 and W17 constitute the individual panels which when combined made up Long Wall No. 5 as tested by Thurston).

The Parent curves are shown in Figure 15. When the parent curves of panels W17 and 24 were added together on the push cycle [5] they compared well with the parent curve of the long wall [1]. Parent curves on the pull cycle however grossly over estimated the actual long wall response observed by Thurston (curves 1 versus curve 6). While the lintel response (W17) was symmetric, the degree of restraint provided by the standard P21 restraint (W24) appears considerably in excess of that actually provided by a free ended panel adjacent to the door.

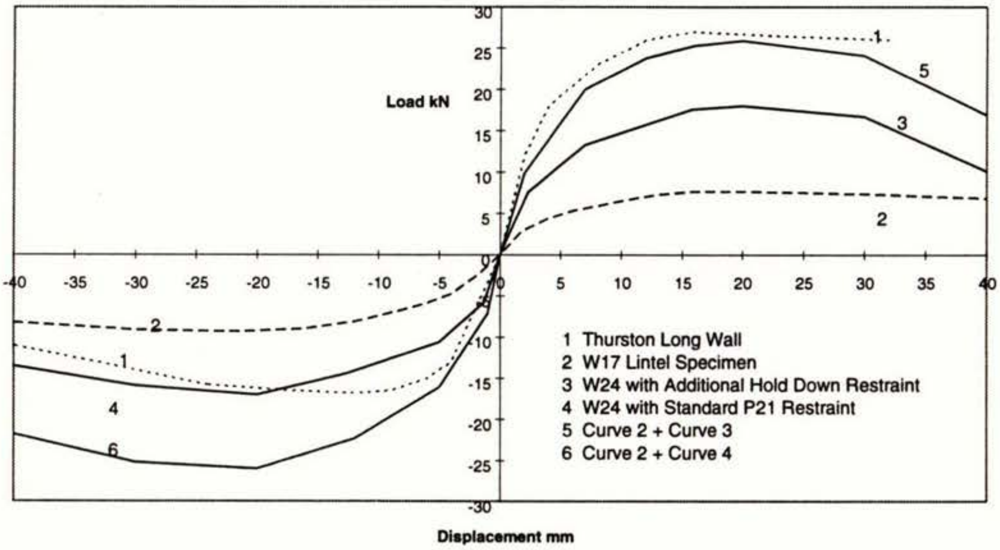


Figure 15 : Comparison of Parent Curves for Lintel and Long Walls

A further comparison was made between the long wall and individual wall components using the draft evaluation procedure (Volume 1 Appendix A). The long wall test hysteresis loops (push cycle only) were matched using the draft analysis procedure prescribed by Phylmas and the restrainable mass at various displacement demands calculated. The results were compared to these obtained on the two individual panels using the same procedure. Again the earthquake record used was the NZA artificial record and damping was assumed at 5%. The results are given in Table 8. The maximum reliable displacement for the long wall and for Panel W24 were both assessed as 20 mm.

The restrainable mass at displacements greater than the maximum reliable displacement are shown in italics for comparison only.

Thurston Long Wall				Mass restrained by Individual Components		
Sp. Disp. min.	**Period T sec.**	**Stiffness K kN/mm**	**Mass kg**	**W24 kg**	**W17 kg**	**Total kg**
10	.17	5.2	**3625**	2460	1150	**3610**
20	.21		**6024**	4190	1880	**6070**
30	*.24*		***7700***	*5533*	*2400*	***7930***
40	*.27*		***9240***	*7100*	*2770*	***9870***
50	*.29*		***11370***	*8160*	*3400*	***11565***

Table 8 : Comparison of Restrained Mass for Long Wall Panel

2.7 Conclusions From the Phase II Experimental Programme

- Panel W24 and Panel W17 (which panels combined to match the composite long wall previously tested by Thurston) were tested in isolation. The addition of the parent curves of these two panels showed good agreement with the parent curve of the long wall. The seismic mass evaluated for Panels W17 and W24 when combined was similar to that of the long wall.

- The restraint afforded to a bracing panel by a standard door lintel, and without vertical load, is approximately 3 kN, ie half of that used in the current P21 test procedure.

- The reliable maximum displacement of short bracing panels was greater than for similar panels of longer length provided reliable end restraints can be assured.

- The reliable maximum displacement of short panels having the same lining were similar for both the standard P21 restraint and the tie down rod restraint. The specimen with plywood lining had the greatest displacement capacity.

3. EVALUATION PROCEDURE USED TO DETERMINE THE DEPENDABLE SEISMIC MASS RESTRAINED

3.1 Introduction

The determination of earthquake design actions is covered by Part 4 of NZS 4203 "General Structural Design Requirements and Design Loadings for Buildings" (SNZ.1992). Equivalent-static, modal and integrated time history analysis techniques are all permitted. The equivalent-static design procedure is the most common and easily applied. It is the permitted default for structures less than 15 m in height or when the fundamental response period is less than 0.45 seconds. All structures within the scope of NZS 3604:1990 (SANZ 1990) are therefore eligible to be designed using this technique. The approach requires a lateral force coefficient to be derived for the appropriate ground conditions (three response spectra being published for Rock or very stiff soils, for Intermediate soils and for Soft soils). Each spectra is truncated at 0.45 seconds, although the unmodified elastic response is provided in each case to permit matching for higher mode response effects to be considered. The elastic spectra were derived for a single mass oscillator with 5% critical damping. The inelastic response spectra were derived assuming an elasto-plastic post elastic responding element with equal energy principles being applied over the short period range and equal displacement principle over the long period range (>0.7 seconds).

While the assumed bilinear elasto-plastic post elastic response may be appropriate for many structural materials (eg. well designed reinforced concrete and steel), they are manifestly inappropriate for systems which experience significant degradation during post-elastic excitation. Wall bracing systems used in timber framed buildings are one such degrading system. An alternative model is thus required to represent the inelastic response of these and other systems, which develop slackness. Although the option remains to design such systems to remain elastic at all load levels, this is unrealistic both with regard the cost and real field response. Dowrick (1977) proposed that, provided such systems are designed to ensure they have sufficient inelastic deformation capacity, then collapse mechanisms can be avoided and satisfactory ultimate limit state performance assured. This is consistent with the generally good performance of such systems in the field. (Moss 1992; King 1990; Pender 1987; Cooney 1979).

The essential feature of the evaluation procedure proposed is that the response observed during the experimental phase is matched electronically. The electronic equivalent element then becomes the core of a non-linear time history analysis (Clough and Penzien 1975) using the design spectra from NZS 4203 as the input record. Each period on the resulting response spectra for that element is related to the mass restrained and the elastic spring stiffness of the system. The mass varies with the square of the period. Thus by limiting the mass supported by the structure such that the system displacement does not exceed that which was shown (during the experimental phase) not to induce unacceptable strength loss, then reliable system performance can be assured.

Several researchers have attempted such simulations in the past (Foschi 1977; Stewart 1987; Dolan 1991; Foliente 1993; Dean 1996). Each developed or adopted models which replicated the observed degrading characteristics of the system under consideration. The problem with each was the inability to easily input the elemental parameters to get an adequate response match. Dean (1994) proposed a spring-bar model which provided an encouraging match but suffer the same cumbersome user interface problems previously experienced. This was further refined by Deam (1994) utilising the Microsoft Windows environment to develop a purpose made computer simulation procedure, Phylmas (Pinched Hysteretic Loop Matching and Analysis System) which enables visual matching of the electronic model to that observed experimentally. This approach forms the engine used to undertake the time-history analysis used to generate the displacement response spectra and thence the determination of the maximum lateral mass which can be sustained by the system without exceeding the maximum reliable displacement.

3.2 Seismic Mass Rating Procedure

The following procedure was carried out using the Phylmas programme in determining the seismic mass rating for each of the panels tested.

(i) The hysteresis loops produced by the test specimen were matched by adjusting the ten generating parameters within the programme.

(ii) A time history analysis was carried out using each of the following earthquake records (all with 5% damping) and the acceleration and displacement spectra were produced for each using the BRANZ PhylMas software. The analysis methodology is fully described in BRANZ Study Report SR73 - Seismic ratings for Residential Timber Buildings (Deam 1996).

   (a) An artificial earthquake record NZA was derived by conversion of the design acceleration spectra published in NZS 4203:1992 (SNZ 1992) from the frequency domain into the time domain.

   (b) The NS component of the 1940 El-Centro earthquake - representative of the acceleration levels found in many of the worlds seismic codes.

   (c) The NS component of the 1977 Bucharest earthquake - representative of a 'soft soil' with relatively large spectral accelerations and displacements in the one to two second period range.

(iii) The maximum reliable displacement at which reliable performance can be assured by observation of the test was plotted on the displacement spectra and the natural period T corresponding to this value found.

(iv) The maximum reliable displacement is the displacement to which the specimen can be cyclically loaded prior to the successive fourth cycle dropping to less than 80 % of the maximum recorded cycle.

(v) The mass M, able to be restrained by the panel was calculated from the natural period T using the stiffness parameter used in matching the hysteresis loops.

The maximum force and displacement for linear elements subjected to dynamic excitation can be shown (Clough and Penzien 1975) to be a function of the natural period T, which for an elastic element is related to the stiffness k of the element and the restrained mass m by

$$T = 2\pi\sqrt{\frac{m}{k}} \quad \text{.........................} \quad (1)$$

The relationship is normally applied to non-linear elements for convenience even though the period of oscillation varies when there is degradation of the element. The initial stiffness is normally used to estimate k with non-linear elements.

Re-arranging equation (1)

$$\text{Mass M} = \frac{kT^2}{4\pi^2}$$

If Mass M is in Kg, T in secs and k in kN/mm

then $M = 25300\,kT^2$ (2)

3.3 Results of Phylmas Analysis for Test Programme I

3.3.1 P21 Restraint

Displacement spectra for tests carried out using the standard P21 end restraint are shown in Figure 16 for specimen W12. The spectra were typical.

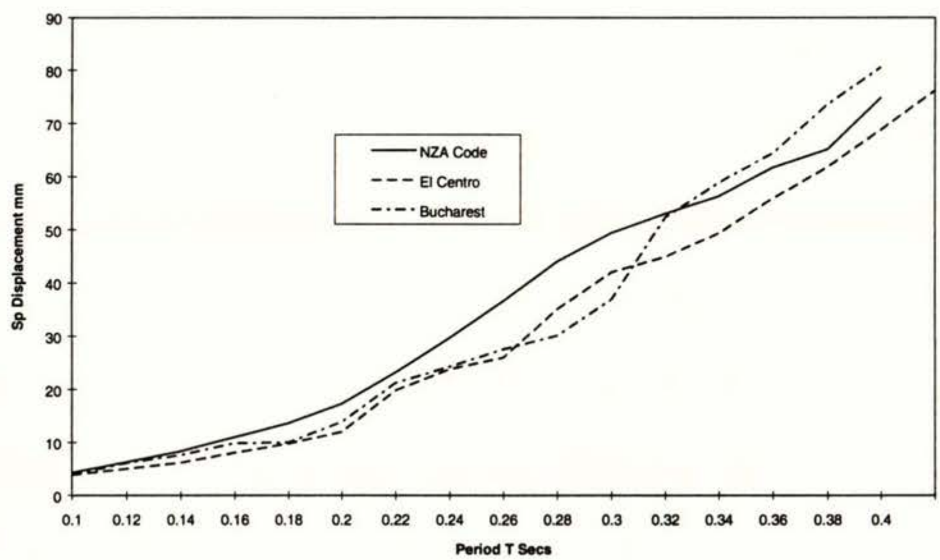


Figure 16 : Displacement Spectra. Specimen W12

Generally the lower bound earthquake record (ie. the record which resulted in the lowest Period T and hence the lowest mass able to be restrained) was the artificial NZA record. This was true for all cases from Period T = 0 secs to approximately T = 0.35 secs when the lower bound earthquake record became the Bucharest 1977 earthquake record. This is not unexpected due to the latter earthquake record being representative of a soft soil site with large spectral accelerations and displacements within the one to two second range.

There was little difference between spectral displacements for earthquake records of Bucharest 1977 and El Centro 1940 up to periods of 0.2 secs.

The validity of results of the time history analysis is dependent upon the characteristics of the earthquake record used. NZS 4203 requires scaling over the period range of interest such that the records match the response spectra of individual events to the uniform risk design spectra. At least three different earthquake records are to be used to provide a representative result.

Spectral accelerations for each of the earthquake records were similar with periods greater than 0.2 secs. Shorter periods showed typically an increase in spectral accelerations for the NZA earthquake record.

3.3.2 NZA Earthquake Record - A Comparison of Spectra

The control earthquake record for all the cyclic/load control tests undertaken using the P21 end restraint was the NZA artificial record.

A comparison of the spectral displacements is given in Figure 16 for specimen W12, and a comparison of mass able to be restrained for each lining using equation (2) at varying reliable displacements is given in Table 9.

The corresponding spectral acceleration and (hence) force is shown for completeness. The initial stiffness k is taken from the appropriate Phylmas generating Parameter.

Panel	Reliable Disp.	Stiffness k, kN/mm	Period T secs.	Mass Kg	Sp. Accl^n	Force kN
	10		0.15	1140	0.58	6.5
W3	20	2.0	0.19	1830	0.40	7.2
PB	30		0.22	2450	0.33	7.9
	40		0.24	2910	0.29	8.3
	10		0.15	1400	0.60	8.0
W12	20	2.4	0.21	2650	0.36	9.5
BL	30		0.24	3500	0.32	11.3
	40		0.27	4390	0.28	12.2
	10		0.16	1540	0.60	9.0
W8	20	1.6	0.22	2730	0.37	9.9
PY	30		0.26	3400	0.33	11.0
	40		0.29	3940	0.29	11.2
	10		0.18	1720	0.52	8.8
W5	20	2.1	0.24	3060	0.32	9.6
FC	30		0.27	3870	0.28	10.6
	40		0.30	4780	0.27	12.6

Table 9 : Comparison of Mass Restrained using P21 End Restraint

Note that the masses are those that will cause a panel to displace to the given spectral displacements when subjected to the NZA earthquake. The ability of one lining to undergo larger deformations whilst still resisting load will be reflected in its reliable maximum displacement.

3.3.3 Displacement and Acceleration spectra using various restraints

Displacement and acceleration spectra were produced for Test Panels W12-W15 using the NZA earthquake record and the mass able to be restrained by the panels determined. The results are shown in Table 10. The stiffness k shown in Table 10 is the initial stiffness parameter taken from the Phylmas generating parameter derived at the response matching phase of the Phylmas process.

Panel	Reliable Disp.	Stiffness k, kN/mm	Period T secs.	Mass Kg	Sp. Accl^n	Force kN
W12 (P21)	10	2.4	0.15	1400	0.60	8.1
	20		0.21	2650	0.36	9.4
	30		0.24	3500	0.32	11.3
	40		0.27	4390	0.28	12.2
W13 (Tie Rod)	10	2.0	0.16	1300	0.6	7.6
	20		0.21	2230	0.40	8.8
	30		0.24	2910	0.35	10.0
	40		0.26	3420	0.31	10.4
W14 (Vert Load)	10	1.6	0.22	1960	0.48	9.2
	20		0.3	3640	0.28	10.0
	30		0.32	4150	0.24	9.8
	40		0.33	4410	0.22	9.5
W15 (unre-strained)	10	1.6	0.18	1310	0.49	6.3
	20		0.23	2140	0.31	6.5
	30		0.25	2530	0.27	6.7
	40		0.27	2950	0.24	6.9

Table 10 : Comparison Of Mass Restrained using Various End Restraints

It can be seen from Table 10 that the mass laterally restrained by the panels using the P21 end restraint is greater than for the corresponding spectral displacements for the tie down rod restraint system. The case which used a vertical load applied to the top plate restrained the greater lateral mass at the lower spectral displacement but was most closely represented by the P21 end restraint condition for displacements up to 40mm.

The unrestrained condition results were always significantly lower than for the other tested panels.

Figure 17 shows the relationship between mass restrained and displacements for each of the end restraint conditions at displacements of 10, 20, 30, 40 and 50 mm.

Note that the masses restrained by the specimen, when subjected to an earthquake record (in this instance the NZA artificial Earthquake record) will displace the specimen to the appropriate spectral displacement. For this purpose the maximum reliable displacements (MRD) have been ignored, hence displacements are shown which are in excess of the MRD.

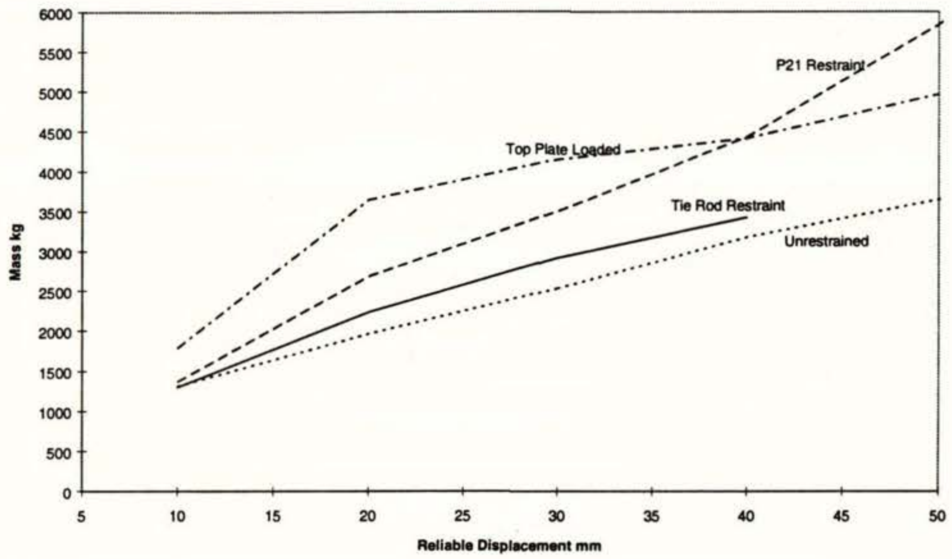


Figure 17 : Mass Restrained using Various End Restraints

3.3.4 Panel W16 - BL Lined Both Sides, Tie Down Rod Restraint

Panel W16 was tested with full restraint of both end studs using the tie down rod system. The test was carried out to investigate the effects of lining a similar panel with the same material on both sides. A comparison could therefore be made with Panel W13. A plot of mass restrained against reliable displacement is shown in Figure 18. The initial stiffness k, of Panel W13 was 2 kN/mm whilst that of Panel W16 was 3.8 kN/mm. Also plotted in Figure 18 is the mass restrained by Panel W13 factored by 1.4.

As part of the background to this study report the effect of initial stiffness on mass restrained using the Phylmas programme has been investigated. One key finding, as might be expected, is that the dependably restrained mass increases with panel stiffness. However, since the panel stiffness is not directly proportional to the panel length, so the dependable restrained lateral mass departs slightly from being linear with panel length. (e.g. Ratio of panels lengths = 2.0 resulting in ratio of restrained mass of 1.9.)

3.3.5 Panel W17 - Lintel Specimen

Displacement and acceleration spectra were produced for the three earthquake records and the lower bound earthquake was found to be the NZA artificial record.

The mass which is able to be restrained by the panel under various reliable maximum displacements was determined and used in the comparison of a long wall tested by Thurston (1993). Refer to section 2.7 for further details.

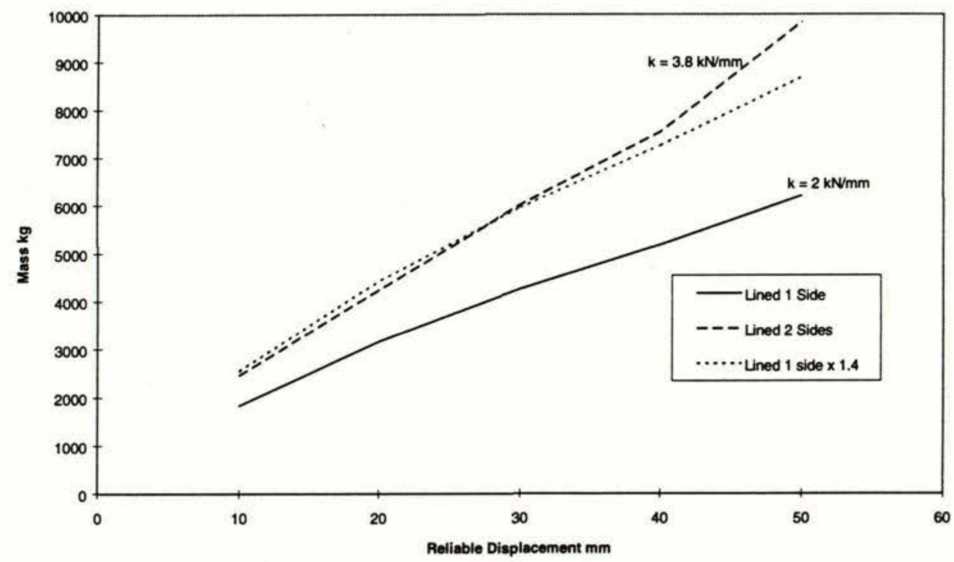


Figure 18 : Mass Restrained by Specimens W13 and W16

4. DISCUSSION

4.1 Boundary Conditions

The seismic mass which a test panel is able to restrain is dependent upon (amongst other things), the initial panel stiffness and the maximum reliable displacement. Both of these are directly affected by the degree of end restraint used to simulate in-service boundary conditions.

The end restraint adopted for the test specimen therefore has a major significance in determining this mass.

The current BRANZ P21 test method specifies that 'appropriate panel end restraint is to be applied'. It gives as an example 'for framed timber, the "P21 end restraint" may be used'. The 'P21 end restraint' referred to is shown in Figure 3 (i.e. an end block nailed to the outer stud with 3 No. 100 x 3.75 flat head nails acting in shear.) This is deemed to be equivalent to the restraint provided to an independent bracing panel when in service (ie as provided by a return wall with cross wall, or by a door or window lintel). Implicit in the method adopted is that the return cross walls are restrained from lifting at loads less than the capacity of the three nails in shear.

In most instances however this method either under or over restrains the specimen for typical cases as described below. As well it does not identify the possible brittle failure of a panel which has sufficient strength to overcome the three nail shear strength by corner gusset action. In this instance the 'ductility' of the specimen could be all attributable to the end restraint rather than the lining material.

4.1.1 Uplift Restraint Attributable to Current P21 (1991) End Restraint

Thurston (1993) evaluated the performance of several test results of P21 tests with P21 type end restraints to establish a relationship between wall racking load, Pw, and wall rocking displacement, Δ_w. The equations he derived for the curves which most closely fitted the results were:

1.2 m long panel	$P_w = (5.6 \times \Delta_w) / (3 + \Delta_w^{0.95})$	(3)
1.8 m long panel	$P_w = (8.7 \times \Delta_w) / (1.3 + \Delta_w^{0.85})$	(4)
2.4 m long panel	$P_w = (12.5\ \Delta_w) / (1.2 + \Delta_w^{0.95})$	(5)
3.0 m long panel	$P_w = (20 \times \Delta_w) / (0.4 + \Delta_w^{0.95})$	(6)

At 30 mm lateral deflection the wall racking loads are Pw = 5.9 kN, 9.8 kN, 14.1 kN and 23.3 kN for equations (3) to (6) respectively which corresponds to end stud uplift forces of 11.9 kN, 13 kN, 14.1 kN and 18.6 kN.

The increase in uplift force with wall length can be attributed to the effect of nailing of the bottom plate to the foundation beam.

Small scale tests were carried out to determine the uplift capacities of the 'P21 end restraint', the bottom plate to foundation beam and end stud to bottom plate connections. These are described in Appendix A. The results showed an average uplift resistance of 6.5 kN for the P21 end restraint.

Taking this test result of 6.5 kN and subtracting from the end stud uplift force derived from equations (3) to (6) gives the effect of uplift restraint by the nailing of the bottom plate to foundation for each panel length:

1.2 m panel = 11.9 - 6.5 = 5.4 kN
1.8 m panel = 13.0 - 6.5 = 6.5 kN
2.4 m panel = 14.1 - 6.5 = 7.6 kN
3.0 m panel = 18.6 - 6.5 = 12.1 kN

The uplift restraint of the 2.4 m panel compared well to the maximum uplift load of 7 kN attained in specimen W15 which had no end restraints. This showed that the uplift restraint afforded by the P21 end restraint became less dominant with panel lengths greater than 1.8 m.

The lateral shear load transferred across the bottom plate/foundation beam connection was determined analytically as being the top plate lateral load, P_w divided by the number of nails. However several of the nails in the tension zone either reached or approached their withdrawal capacity. If the lateral load carrying capacity of these was zero or reduced then the lateral force/nail is approximately as shown in Table 11. There will be some contribution to lateral resistance by the frictional component between the bottom plate and the foundation at the compression end, however this has been ignored in this simplified analysis.

Panel length m	Max lateral load P_w kN	No. of effective nails	lateral load/nail kN
1.2	5.9	4	1.5
1.8	9.8	6	1.6
2.4	14.1	7	2.0
3.0	23.3	9	2.6

Table 11 : Lateral Force per Bottom Plate Fixing

The ultimate strength of a 3.75 mm diameter nail in single shear, from tests described earlier is 2.17 kN, which suggests that panels of lengths in excess of 2.4 m are governed by the lateral load capacity of the bottom plate/foundation nail connection rather than the P21 end restraint. This is confirmed by Thurston (1993) who plotted the predicted parent curves using full and partial (P21 end) restraint for various wall lengths. He concluded that single lined walls 3 m in length behave in a similar manner for both a fully restrained condition and the P21 end restraint. For walls less than 3 m in length the 'P21 end restraint' effectively governs the wall racking resistance.

Nail slip curves conducted over several years show the maximum lateral force/nail in gypsum plasterboard under cyclic load to be approximately 0.35 kN. If this is translated to a typical 2.4 m x 2.4 m test panel with nail spacing at 150 mm then maximum uplift shear transferred to the end stud can be calculated as the load per nail times the number of nails present (i.e. 0.35 kN $\left(\frac{2400}{150}+1\right)$ x 2 sides ≈ 12 kN). This is equivalent to the maximum uplift assessed from the P21 end restraint, with the bottom plate nail fixings taken into consideration and as obtained in equation (5). Thus the current P21 (1991) test procedure is optimum for predicting the strength of plasterboard panels whilst down rating panels lined with stiffer, stronger material.

4.2 In-service Boundary Conditions

In order to obtain a realistic bracing rating resistance of a panel experimentally it is important that the boundary conditions used to restrain the panel in the laboratory replicate those that will be found in-service as closely as practical.

4.2.1 Internal Bracing Panel / External Wall Junction

The current (1991) method of end restraint represents the minimum restraint provided by nailing the end stud to the return wall framing. However it does not represent the majority of wall junctions which are, for internal braced walls, usually lined and stopped at the corners, and it makes no allowance for the more direct load path between the lining of the bracing panel and the return wall. The typical wall junction is shown in Figure 19 : Typical Wall Junction

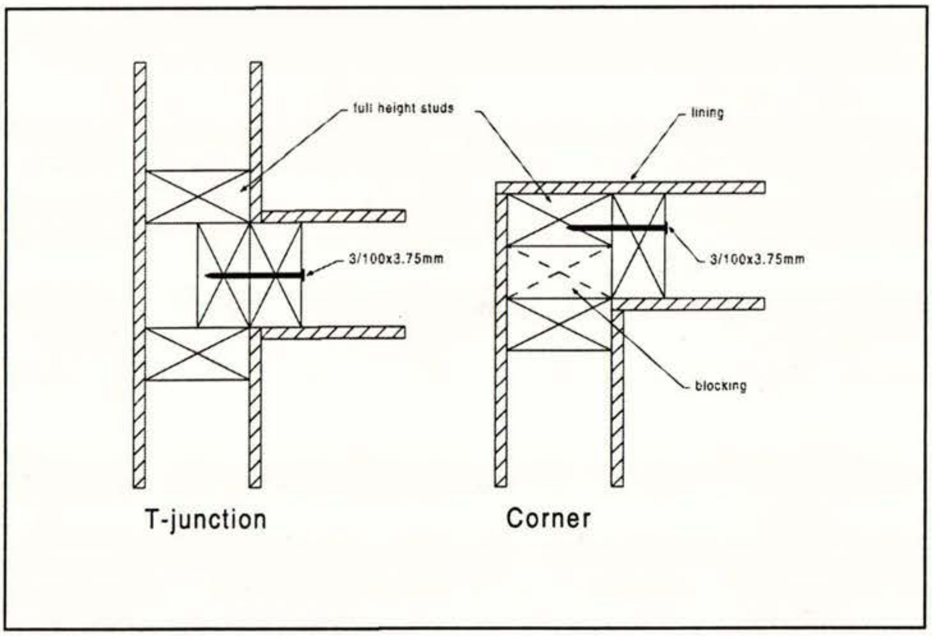


Figure 19 : Typical Wall Junction

4.2.1.1 Stopped Corner Joints

The shear transfer capacity of plasterboard joints which have been taped and stopped has been measured and found to be in the order of 8 kN/m (Thurston 1993). The uplift loads associated with typical bracing panels therefore can easily be transferred to the return wall linings. The uplift force in the panel lining is transferred into the timber studs by a combination of the end brace panel stud nail fasteners and the return wall fasteners.

Nail fasteners in the end stud of the brace wall are subject to load both perpendicular and parallel to the end stud framing.

Referring to Figure 19 it has been found that resistance to uplift of the end stud depends upon:

(1) The load carrying capacity of the nail fasteners connecting the bottom plate to the foundation, and the return wall length.

(2) The nail slip characteristics of the lining in the bracing panel, both perpendicular and parallel to the stud.

(3) The nail slip characteristics of the lining in the return wall parallel to the stud only. This can be determined by small scale testing. The minimum strength joint (ie standard plasterboard lining fixed to the studs using clouts at 300 mm centres) has been found to have a typical nail shear capacity of 0.38 kN for shear applied parallel to the paper bound edge. With nails at 300 mm centres, this results in an uplift capacity contribution of 0.38 kN x (2400/300 + 1) = 3.4 kN. In reality this may be doubled (or more) when the return joint is plastered with uplift forces being passed to the adjacent return panel (refer (5) below).

(4) The stud to bottom plate connection assumed in section 0 to be 5.4 kN for a 1.2 m long panel. Small scale testing, described in Appendix A, showed the uplift capacity of a pair of 4 mm by 100 mm nails to be 3.8 kN with an additional 50% resulting from uplift of adjacent interconnected panels. (Note that additional restraint afforded by any vertical load on the external wall has been ignored.)

(5) The shear capacity of the lining joint shear capacity found to be approximately 8 kN/m or 19 kN for a 2.4 m high wall. Such shear transfer is usually subservient to the shear capacity between the sheathing of the adjacent wall and its framing (refer (2) above).

(6) The extent that the uplifting end stud is connected to the framing of the adjacent return wall (represented in the experimental specimen by the supplementary P21 (1991) end stud uplift restraint shown in figure 3). Small scale testing indicates this restraint to be approximately 6.5 kN (refer Appendix A).

NZS 3604 1990 places no limit on the uplift force which may be imposed on the foundation/floor connections at bracing panels. Similarly the current practice when testing is to assume full foundation beam hold-down is achieved. This is a deficiency of the current system that this revision is attempting to remedy. Appendix B shows that this is excessive for timber framed foundations and a more suitable limit would be 12 kN.

Ideally tests undertaken to determine bracing rating should include the lining joints and return wall to best simulate the in-service condition. However this is not practical. In the case of light timber framed construction, the restraint afforded by return walls can conservatively be taken as being equivalent to 12 kN.

4.2.1.2 Unstopped Corner Joints

In instances where the internal bracing panel is not taped and stopped to the return wall the load path is somewhat different. Uplift forces in such bracing panels are transferred from the sheathing to the end stud of the panel, and then from that end stud into the stud of the adjacent wall by shear transfer through at least three nails. Thence the three nail requirement of the P21 end restraint.

4.2.2 Internal Bracing Panel / Door Opening

Results of the experimental test simulating a panel between door openings (panel W17) is given in section 1.6.4. They show that the uplift restraint afforded by the door lintel is less than that of the P21 end restraint. Hence the use of this particular restraint to simulate the boundary condition over estimates the panel performance. Further testing was carried out in Phase II of this experimental programme to identify the degree of restraint afforded by a door lintel.

4.3 Load Regime

It has been questioned (Dean 1987) whether the P21 test procedure (1991), using a displacement specified regime is adequate to indicate the reserve strength of a test panel or in identifying severely degrading sheets. Dean suggested that a cyclic test to specified loads would be more appropriate.

The current P21 test procedure (1991) nominates a peak displacement and cycles for four excursions to this displacement. No consideration is given to the widely disparate racking performance of different systems beyond that displacement. Some lining material can sustain displacements only slightly higher that that nominated without severe loss of resistance while others can continue to sustain their resistant capacity with only insignificant strength loss. Systems which exhibit this latter response possess both greater ductility and damping and, while being much preferred, are currently given no recognition of this superior performance.

Two means of degrading bracing panels were identified and investigated. The first subjected the test panel to increasing cyclic loads, with a set number four of excursions to each load. Cycling continued at incremental loads until the lateral resistance of the system fell below 80% of the load applied during the previous cycle (i.e. the one proceeding the cycle in which significant strength loss occurs).

The alternative subjected the test specimen to increasing cyclic displacements with a set number of excursions to each displacement. The test continued until a maximum dependable displacement was established. The maximum dependable displacement was considered to be the displacement at the cycle immediately preceding that in which the fourth cycle load resistance fell below 20% of the maximum fourth cycle resistant envelope.

In each case the test panel was subjected to cycles beyond the peak load. The onset of significant strength loss is identified.

Phase I of the experimental programme investigated the significance of load regime by subjecting different panels, first to the load regime and then replicate panels to the displacement controlled regime, as shown in Figures 7 and 10 respectively. The on-set of damage in the system is clearly identified in both.

4.3.1 Load or Displacement Control

One objective of this first phase of the experimental programme was to ascertain whether the means of imposing the cyclic deformation influenced the assessed system capacity.

Figure 20 shows the parent curves for both the load controlled and displacement controlled regimes for three types of lining, on similar panels. The close proximity of each envelope indicates that the basis of loading is irrelevant to the assessed performance of the panel. The experimental results have shown that there is no significant difference in the peak load or peak displacements determined using either control regime.

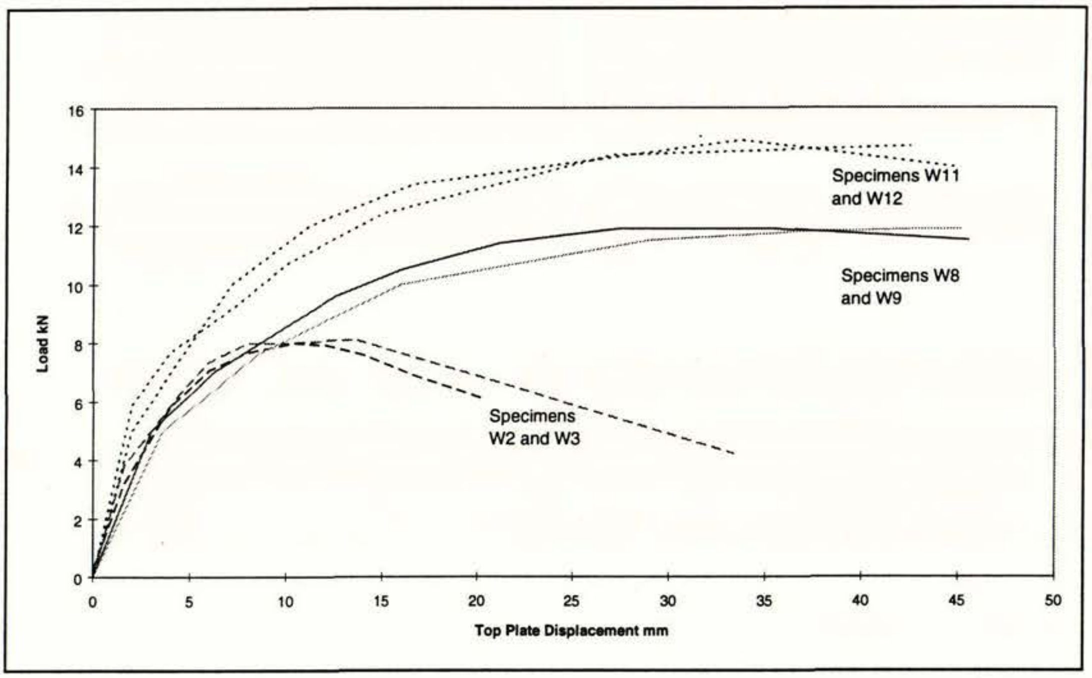


Figure 20 : Parent Curves Under Load or Displacement Control

There are however advantages in using a displacement control method.

- The degree of control provided under displacement control is greater and the system thus safer to operate.
- Displacement cycles are readily matched to the hysteresis loops generated in the Phylmas programme.
- On reaching a particular displacement on the first cycle it is certain that the same displacement can be reached on subsequent cycles. Degradation in the lining may preclude this happening when the test is load controlled.
- Past P21 racking tests and results are based upon displacement control. This may be advantageous when matching test data to the Phylmas programme.

5. CONCLUSION

This report provided the details of the experimental programme undertaken as part of the review process which aimed to rationalise the basis for determining dependable earthquake and wind resistance to bracing panels used in houses.

The necessity of including some allowance for additional bracing panel uplift restraint to reflect actual in-service behaviour was clearly demonstrated and justified. The quantification of the capacity of such experimentally artificial restraint is usually unimportant once a minimum threshold is attained since it will normally be the capacity of the sheathing material itself which controls the inservice performance.

The means or control by which the deformation is imposed has little, if any, relevance to the panel performance since it is the degrading characteristics that need to be matched as input to a suitable time-history analysis technique. The displacement control cycles are thus preferred for experimental simplicity.

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APPENDIX A : SMALL SCALE TESTING OF BRACING PANEL COMPONENTS

Nail pull-out tests were carried out on 10 samples to determine the pull out failure load of the typical bottom plate to timber foundation connection and the failure load of the end stud connection to the bottom plate. Six tests were also carried out on the standard P21 (1991) end restraint to determine the shear capacity.

1. Bottom Plate to end Stud

Two 90 x 3.15 mm gun nails were driven through a 300 mm long section of a bottom plate into the end grain of a 300 mm long 90 x 35 mm timber stud as used in the light timber frames evaluated in experimental phases I and II. The specimen was then assembled into the universal testing machine, with the bottom plate being firmly fixed to the bottom platen and the stud section grasped by the jaws of a vice attached to the machine. The stud was then pulled from the bottom plate at a load rate of 2 kN per minute. Load was measured using a 10 kN load cell. The peak load recorded is shown in Table A.1.

2. Bottom Plate to Foundation

Two 100 x 4 mm bright flat head nails were driven through a section of 90 x 45 mm timber bottom plate, through a 20 mm particleboard strip and into a 150 x 100 mm timber foundation beam. The specimen was then assembled into the universal testing machine with the foundation beam firmly fixed. The simulated bottom plate was firmly gripped and tension load (withdrawal) applied at a rate of 2 kN per minute. Load being measured with a 10 kN load cell. The peak load recorded is given in Table A.2.

3. P21 (1991) End Restraint

Three 100 x 4 mm bright flat head nails were driven through two 90 x 35 mm timber end studs 300 mm long, and were a simulation of the standard P21 (1991) end restraint. The specimens were tested in the universal testing machine with shear load being applied to one of the studs while the other was firmly fixed. The peak load recorded is given in Table A.3.

Specimen	Peak Load kN
1	3.92
2	4.26
3	4.19
4	4.26
5	3.98
Average	**4.1**

Table : A1

Specimen	Peak Load kN
1	3.45
2	3.94
3	3.62
4	3.63
5	4.22
Average	**3.8**

Table : A2

Specimen	Peak Load kN
1	6.5
2	6.3
3	6.7
Average	**6.5**

Table : A3

APPENDIX B : BRACING PANEL UPLIFT ON TYPICAL TIMBER FLOORING

Tests were carried out on a typical house floor to investigate the uplift capabilities at various locations. This may have a direct bearing on bracing ratings, as presently NZS 3604 (SNZ 1990) does not limit the axial load induced in the end studs of bracing panels. There is the possibility of a potential break down in load path from the panel to the foundation.

A 4.4 x 4.4 m square timber flooring system was constructed using 140 x 45 machine stress grade timber joists at 400 mm centres. The ends of the joists were attached to 140 x 45 mm boundary joists. The joists were overlain with 20 mm thick particleboard flooring and all fixings were as specified in NZS 3604. The joists were supported at two locations to provide a joist span of 2.4 m.

Steel straps were firmly coach bolted to the joists at the locations marked 'X' in Figure B1 and load was applied to the straps via an overhead crane. Load was applied until localised failure of the floor occurred. Load was measured using a 100 kN load cell.

Results of the peak load resisted at the various locations are shown in Table B1.

Location	Load At Failure kN	Failure Mode
A	23	Joist fracture
B	20	Joist fracture
C	12	Joist to boundary joist fixing withdrawal
D	14	Joist fracture

Table B1 : Peak failure uplift loads applied to individual floor joists

The results in Table B1 indicate that there is sufficient load sharing within the body of the floor to accommodate the uplift forces generated in the end studs of bracing panel. However, problems may occur at the junction of floor joist and boundary joist where the failure load of 12 kN correlates to a panel bracing rating of 100 Bracing Units for a 2.4 m high wall. Further work is required to verify the significance of this potential failure mechanism.

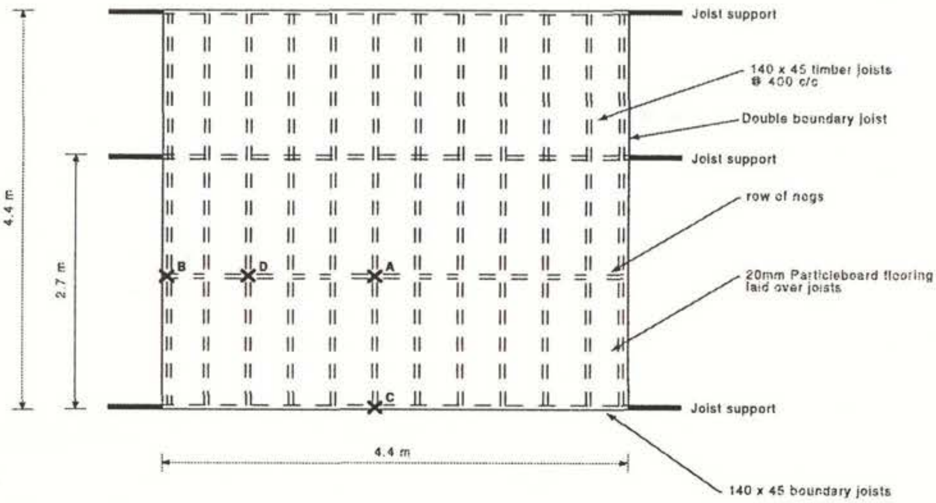

Figure B1 : Experimental procedure used to ascertain load sharing between joists

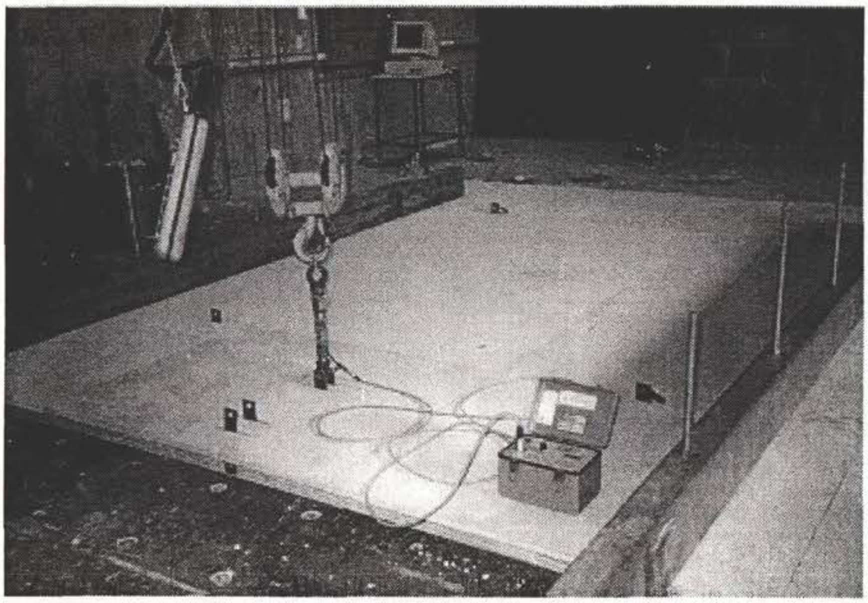

Figure B2 : Vertical load being applied to second joist with load sharing measured

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