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Influence of Loading Protocol on the Structural Performance of Timber-Framed Shear Walls

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ABSTRACT: Timber-framed shear walls are designed to resist the lateral loads on a building from wind and earthquake. Many regions around the world have developed standard test methods for evaluating the performance of timber-framed shear walls. Currently, Australia has no such standard test method for timber-framed shear walls. The aim of this study is to develop a standard loading protocol for evaluating the performance of timber-framed shear walls in Australia that is informed by the concerns of both earthquake engineers and wind engineers. To achieve this aim, the key objective of this study is to compare the performance of a standard timber-framed shear wall under three different monotonic (groups M1, M2 and M3) and four different cyclic (groups C1, C2, C3 and C4) loading protocols according to their respective standards. The number of test panels in each group was three (3) for a total number of 21 individual tests. Structural performance characteristics of the standard test panel, such as ultimate and yield strength and global stiffness, were found to be strongly dependent on the loading protocol. For example, ultimate strength was lower for test panels subject to monotonic loading (*i.e.*, 5.99, 6.41 and $6.34\ kN/m$ for groups M1, M2 and M3 respectively) compared to test panels subject to cyclic loading (*i.e.*, 6.70, 6.73, 8.03 and $8.11\ kN/m$ for groups C1, C2, C3 and C4 respectively). Internal stiffness was found to be relatively consistent regardless of loading protocol. The higher ultimate and yield performance of group C3 (CUREE protocol) and C4 (Cyclone Testing Station protocol) is statistically significant, at the 5% significance level, compared to results from all other test methods (*i.e.*, AS1720.1, EN 594, ASTM E564, BRANZ P21 and ISO 16670). Differences in boundary conditions between the loading protocols might explain some of the differences in results; however, this hypothesis is not strongly supported by the evidence. These results are used to inform our recommendations for developing an Australian standard test method for evaluating the structural performance of timber-framed shear walls.

KEYWORDS: Shear Wall, Monotonic, Cyclic, Comparison Study, Loading Protocols, Standard Test Method, Racking

1 INTRODUCTION

It stands to reason that any standardised test method for evaluating the performance of timber-framed shear wall systems ought to simulate the expected loading profile as much as practicable. While this principle is explicitly mentioned in clause D3.2 of Australian Standard AS1720.1 [1], the test procedure described in clause D5.5, for testing of prototypes, uses a monotonically increasing load which arguably does not properly simulate the expected loading profile. High wind events, such as hurricanes and cyclones, buffet buildings and earthquakes shake them. The cyclic nature of these forces can weaken connections between the framing and the sheathing of shear walls which increases slackness and increases the incidence of fatigue failures of the connectors [2]. We have previously argued that Australia needs to adopt a standard test method for evaluating the performance of timber-framed shear walls [3]. Here we report on our experimental test plan to compare the performance of timber-framed shear walls under three different monotonic loading protocols and four different cyclic loading protocols. The test methods for each of the loading protocols are described and results are presented. Analysis and discussion of the results lead to recommendations for a standard Australian test method for evaluating the structural performance of timber-framed shear walls.

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Parts of this paper appeared in several unpublished reports to the Engineered Wood Products Association of Australasia and BORG Manufacturing. A short summary of this paper, focusing on the Australian loading protocols, was presented in one section of a paper presented at the Australasian Structural Engineering Conference 2020 [4].

2 CURRENT STANDARD TEST METHODS

There are various test methods in use around the world for determining the racking performance of timber-framed shear walls. Given the aim of this study is to develop a standard loading protocol for Australia, we have limited the scope of our review to existing loading protocols in use around the world. This study focuses on a comparison of only seven different test methods, including three monotonic and four cyclic loading protocols. The three monotonic loading protocols include a typical Australian test method based on AS1720.1 Appendix D [1] (test panels M1.1, M1.2 and M1.3), the European EN 594 method [5] (test panels M2.1, M2.2 and M2.3) and the American ASTM E564 method [6] (test panels M3.1, M3.2 and M3.3). The four cyclic loading protocols examined in this study include the New Zealand BRANZ P21 method [7] (test panels C1.1, C1.2 and C1.3), method B of the American ASTM E2126 [8] based on ISO 16670 (test panels C2.1, C2.2 and C2.3), method C of the American ASTM E2126 [8] based on the CUREE method (test panels C3.1, C3.2 and C3.3) and a method based on the TR5 protocol developed by Reardon [9] at Australia's Cyclone Testing Station (CTS) and modified by C.G. McDowall in his unpublished 1995 report titled, "*A Testing Protocol for Determination of Racking Strength of Plywood Sheathed Steel Framed Wall Panels for PAA, Lysaght & Buildex*" (test panels C4.1, C4.2 and C4.3).

Each of the seven test methods differs not only in the profile of the applied load, but also in details of anchoring to foundations and connecting of the loading ram to the test panel. The differences in boundary conditions are described in Table 1. See references [1, 5, 6, 7, 8 & 9] for full details.

The different loading profiles can be seen in Figures 1 to 7. Of note, the M1 and M3 protocols have the longest test duration and the C4 protocol has the most loading cycles. The M1 and C4 protocols were load-controlled and the other protocols were displacement-controlled. The load rates for each of the protocols are shown in Table 1.

Table 1: Loading Rates and Boundary Conditions by Loading Protocol.

<table>
<tr><th>Loading Protocol</th><th>Grp</th><th>Load Rate</th><th>Foundation Beam</th><th>Anchors</th><th>Header Beam</th></tr>
<tr><td>AS1720.1 [1]</td><td>M1</td><td>0.05 kN/s *</td><td>nil</td><td>3M12 bolts with 50 × 50 × 3mm square washers through the bottom plate into the steel floor beam</td><td>nil</td></tr>
<tr><td>EN 594 [5]</td><td>M2</td><td>0.167 mm/s</td><td>90 × 45 MGP10</td><td>4M16 bolts with 75 × 75 × 5mm square washers through the bottom plate into the steel floor beam</td><td>nil</td></tr>
<tr><td>ASTM E564 [6]</td><td>M3</td><td>0.1 mm/s</td><td>nil</td><td>As per Group C2 and C3 below</td><td>nil</td></tr>
<tr><td>BRANZ P21 [7]</td><td>C1</td><td>4 mm/s</td><td>90 × 90 hardwood beam fixed to steel floor beam with M12 cuphead bolts and 19mm thick particleboard fixed to hardwood beam with 2 / 2.8 (ϕ) × 65mm (l) nails at 200mm spacings</td><td>2 / 4.0 (ϕ) × 100mm (l) nails at 600mm spacings through the bottom plate into the particleboard and a supplementary uplift restraint at each end of the test panel to detail in [7]</td><td>Universal joint at ram end with two lengths of 125 × 65 × 4mm Duragal channel connected to each other by a horizontal 24mm (ϕ) pin which is bolted at the middle of the test panel through a 90 × 45mm hardwood packer with 2M16 bolts with 65 × 65 × 6mm square washers</td></tr>
<tr><td>ISO 16670 [8]</td><td>C2</td><td>20 mm/s</td><td>nil</td><td rowspan="2">4M16 bolts with 65 × 65 × 6mm square washers through the bottom plate into the steel floor beam and a custom steel hold-down bracket at each end with 2M16 bolts into end stud and 1M20 bolt through the bottom plate into the steel floor beam</td><td rowspan="3">24mm (ϕ) pin at ram end connected to 125 × 65 × 4mm Duragal channel header beam which is bolted to the top plate of the test panel with 4M16 bolts at 600mm spacings with 65 × 65 × 6mm square washers</td></tr>
<tr><td>CUREE [8]</td><td>C3</td><td>20 mm/s</td><td>nil</td></tr>
<tr><td>CTS TR5 [9]</td><td>C4</td><td>5 kN/s *</td><td>90 × 45 hardwood joists under each stud bolted to the steel floor beam with 2M12 bolts with 50 × 50 × 3mm square washers</td><td>Steel bracket at each end to restrain slip and 2 / No. 14 Type 17 × 100mm (l) screws through the bottom plate into each joist</td></tr>
</table>

Note: * Final push phase was 0.167 mm/s

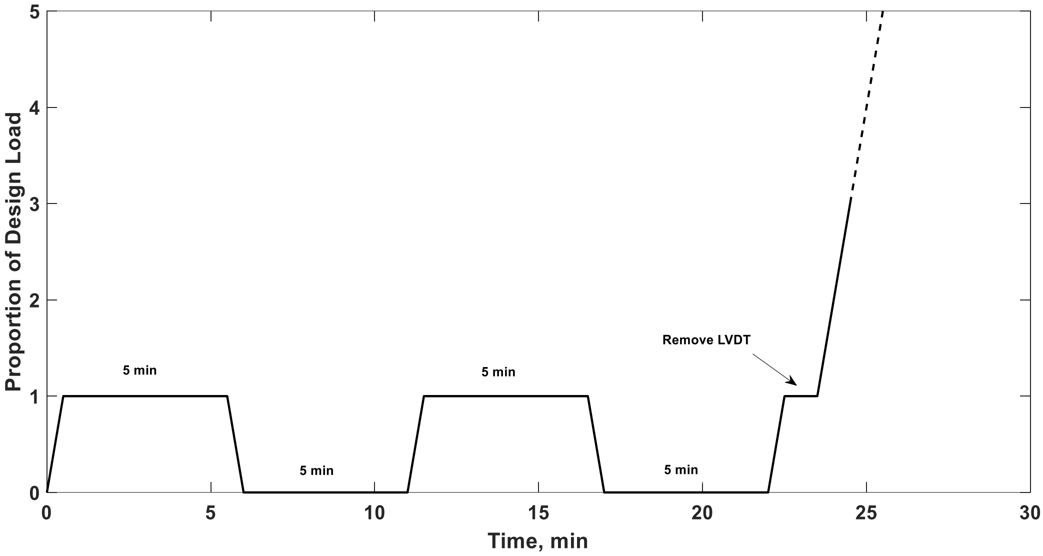

Figure 1: AS1720.1 Loading Protocol for M1 Test Panels [1].

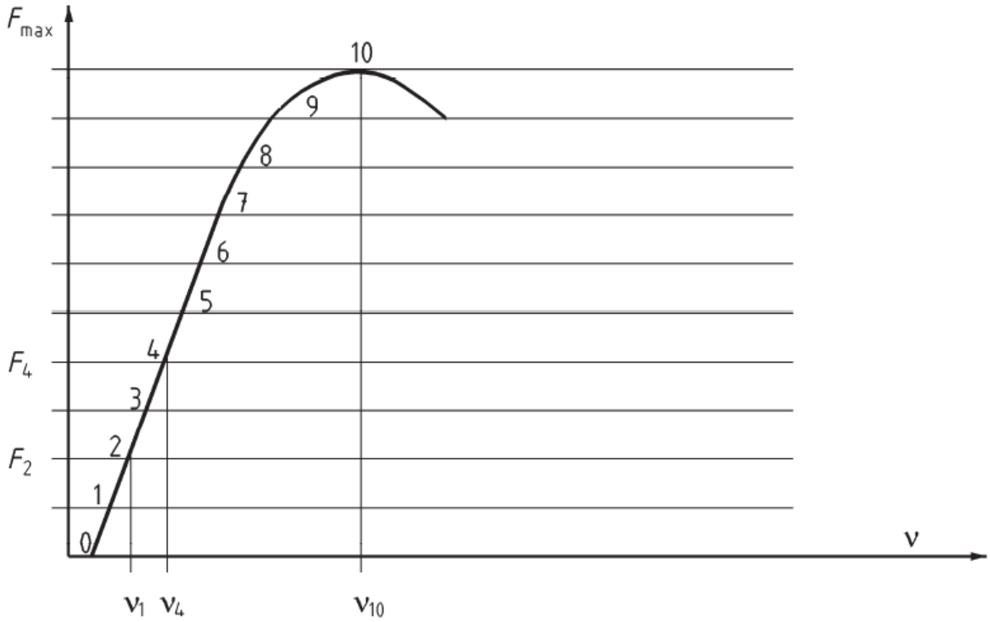

Figure 2: EN 594 Loading Protocol for M2 Test Panels [5].

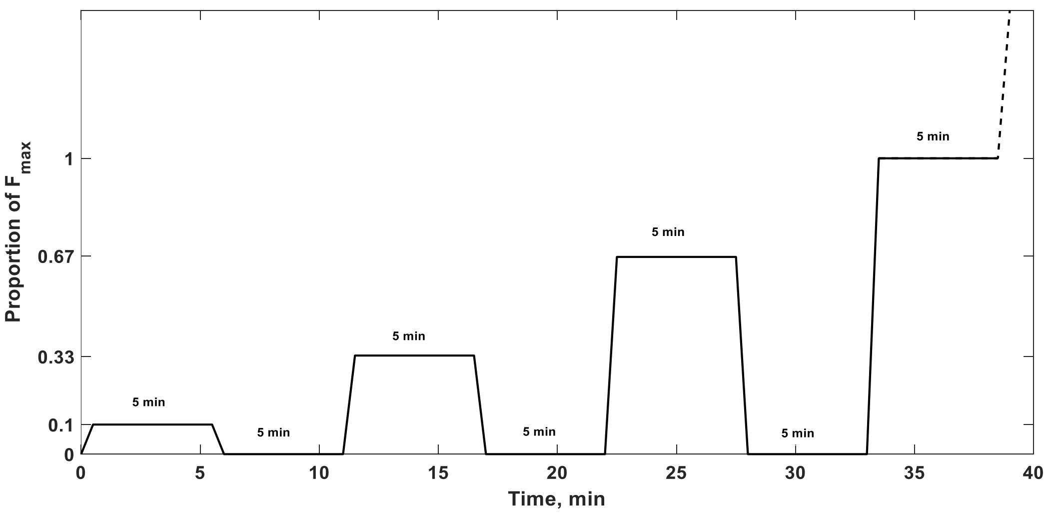

Figure 3: E564 Loading Protocol for M3 Test Panels [6].

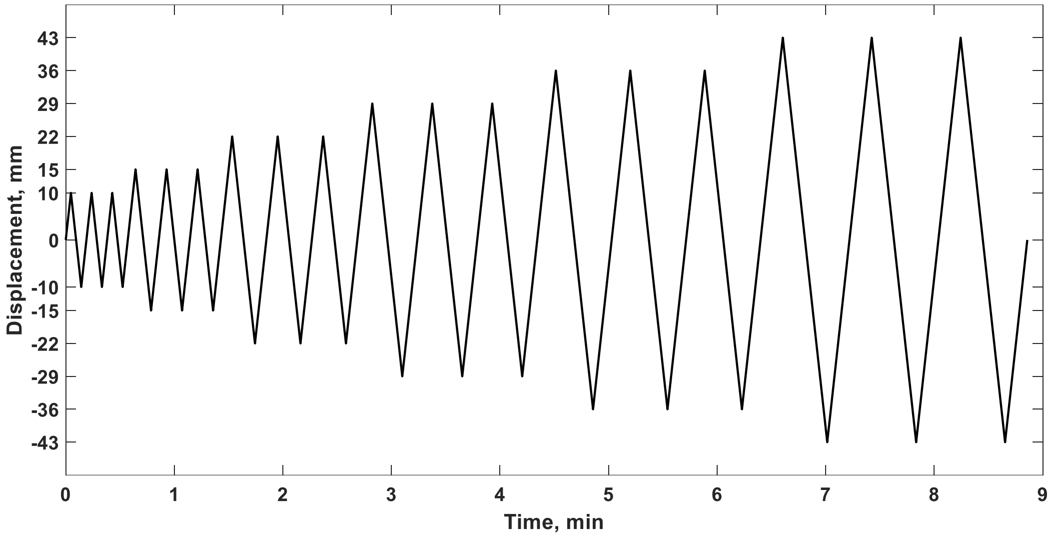

Figure 4: P21 Loading Protocol for C1 Test Panels [7].

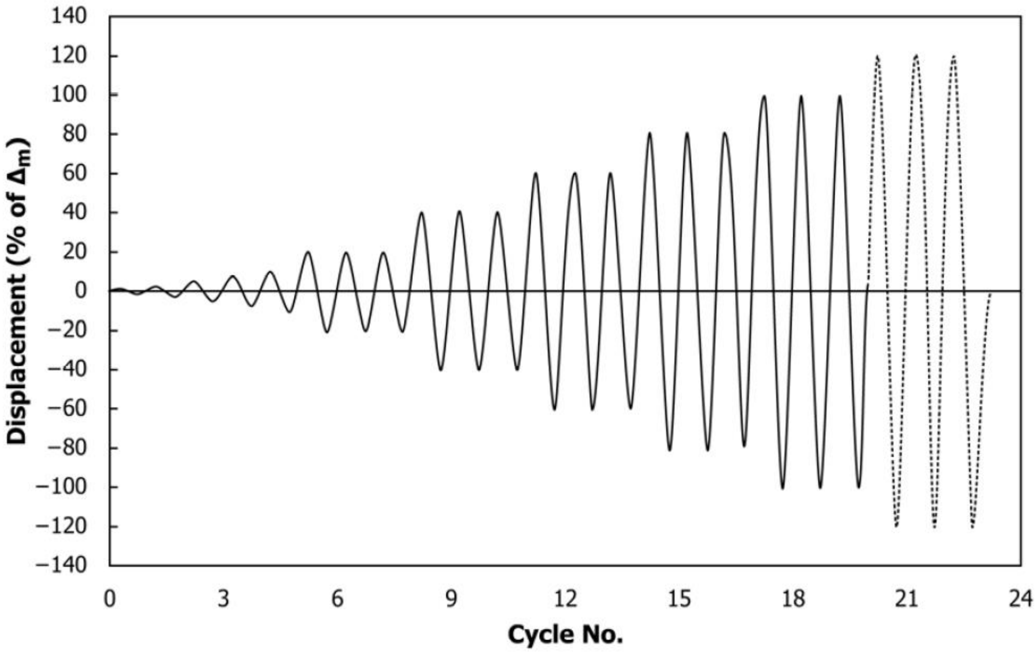

Figure 5: ISO16670 Loading Protocol for C2 Test Panels [8].

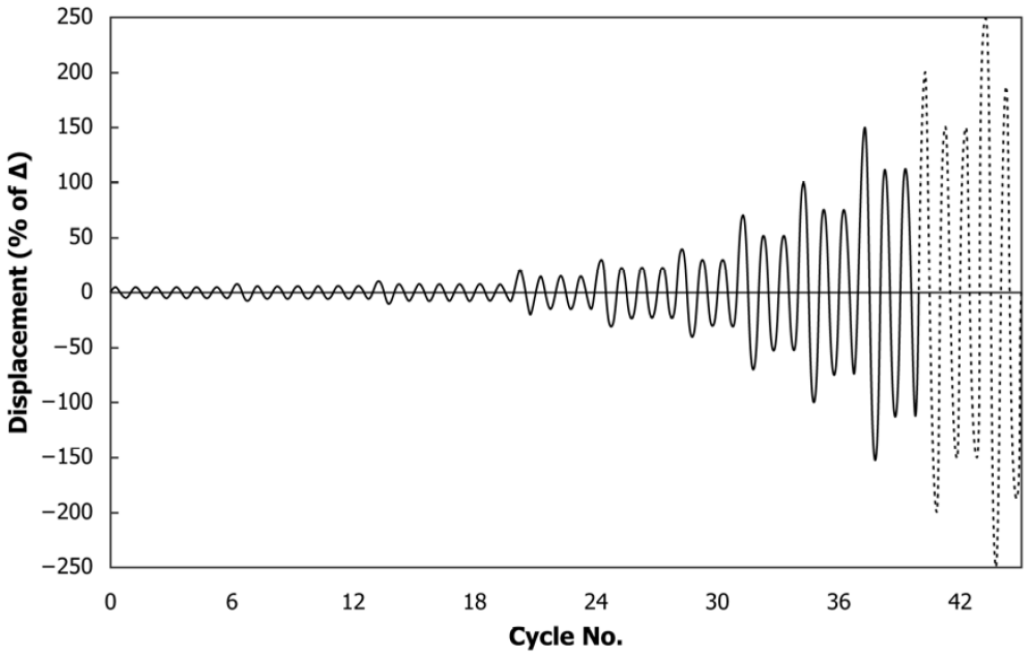

Figure 6: CUREE Loading Protocol for C3 Test Panels [8].

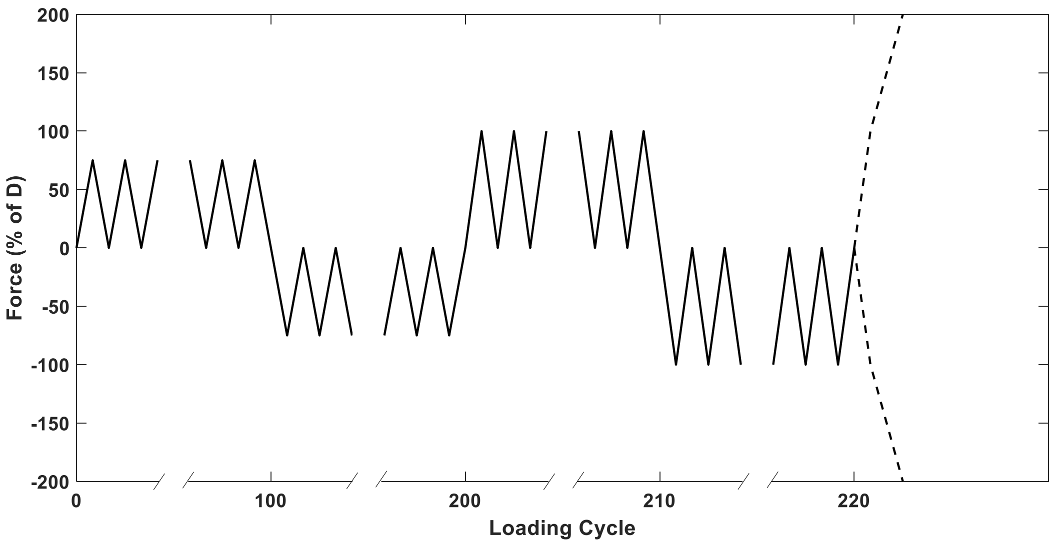

Figure 7: TR5 Loading Protocol for C4 Test Panels [9].

3 STANDARD TEST PANEL

Our standard test panel (see Figure 8) is $2700\ (h) \times 2400mm\ (l)$ with two sheets of structural plywood ($2700 \times 1200mm$) attached to one side of the panel and fixed in accordance with Method A, Detail (h), Table 8.18, AS1684, Part 2 [10]).

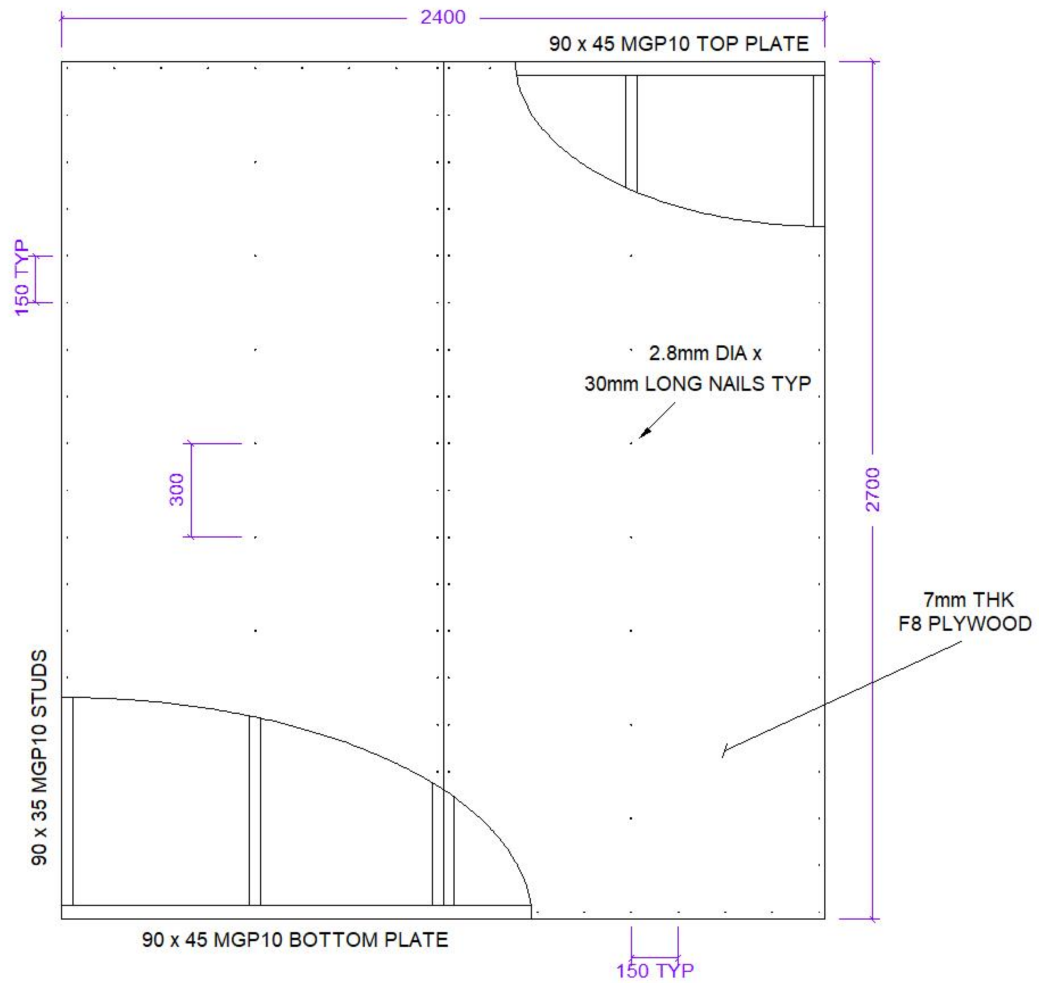

Figure 8: Standard Test Panel.

The timber framing for all test panels, sourced from the Southern Queensland Pine resource, is $90 \times 45mm$ MGP10 top and bottom plate and $90 \times 35mm$ MGP10 studs at $600mm$ spacings. The $7mm$ thick F8 plywood sheathing consists of three (3) veneers of radiata pine with a C grade face and D grade back glued together with a phenolic A bond resin. The sheathing is fastened to the timber framing with $2.8\ (\phi) \times 30mm\ (l)$ galvanised clouts at $150mm$ spacings around the edges of the plywood sheet and $300mm$ spacings at intermediate studs. Overturning forces are resisted by $M12$ tie-down rods at both ends of all 21 test panels with $50 \times 50 \times 3mm$ square washers.

4 INSTRUMENTATION

A $500\ kN$ MOOG actuator was used to apply the load. The actuator records both displacement and applied load. One laser sensor (Omron $600 \pm 400mm$), denoted LVDT1 in Figure 9, was mounted to a steel column separate to the mounting rig to measure horizontal displacement at the top of the specimen. Seven $50mm$ linear variable transducers (LVDTs) were positioned as shown in Figure 9 to record deformations (*i.e.*, LVDT2 to LVDT8). Two laser displacement sensors (Omron $100 \pm 35mm$) were mounted on aluminium frames fixed to the end studs of the wall to measure shear buckling of the sheathing. The data acquisition system was custom-made for the laboratory and was set to record readings at a sampling rate of $10Hz$.

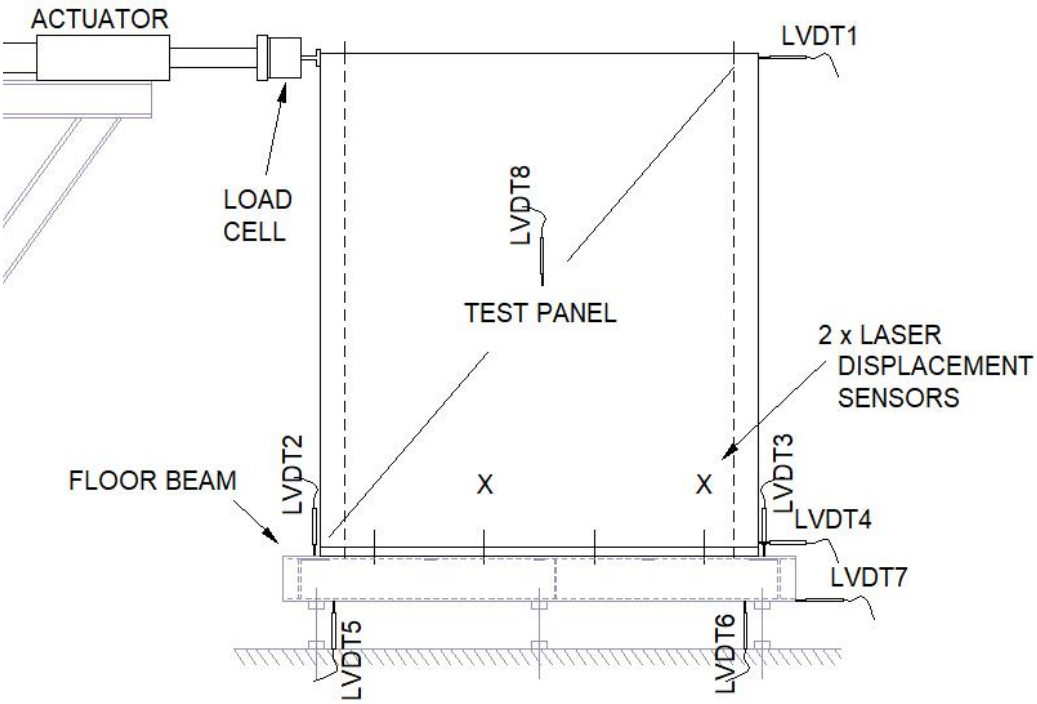

Figure 9: Sensor Layout.

5 DATA CLEANING

We noted and addressed several issues when reviewing the test data:

- Data from the $500\ kN$ MOOG actuator was captured on a different computer than data from the laser sensors and LVDTs;
- Data from the $600mm$ Omron laser sensor at the top of the test panel (LVDT1), was observed to be erratic during some of the tests;
- Occasionally, the $50mm$ LVDTs would record a null reading; and
- Some of the $50mm$ LVDTs would switch off during the test. If the 'sleeping' sensor was noticed during the test, it would be turned back on.

Cleaning of the data has been managed as follows:

- Asynchronous data:
  - Align both sets of data with the point in time when the actuator stops pushing and returns to its zero position.
- $600mm$ Omron laser sensor:
  - Use the data from the laser sensor to calibrate 'proxy' displacement data from the actuator which is only used for plotting load – displacement curves and not for determining structural performance measures such as stiffness and serviceability limits; and
  - Acquire a new sensor (in use from test panel M3.2 and all subsequent test panels).
- Occasional null reading from the $50mm$ LVDTs:
  - Interpolate between adjacent data points.
- Sleeping $50mm$ LVDTs:
  - If the missing section of data occurs in a repeated part of the loading cycle, copy data from the repeated part of the loading cycle of the same sensor and adjust the data up or down to ensure continuity; and
  - If the missing section of data occurs elsewhere in the loading cycle, determine a correlation between the existing data from that sensor and data from one or two other sensors. When a good fit is demonstrated, use the correlation to extrapolate data and adjust the data up or down to ensure continuity.

The MOOG displacement data has been adjusted by a multiplier to fit, as closely as possible, the data from the laser sensors. The multiplier is determined using a nonlinear least squares optimization algorithm. Values for the multiplier range from 0.82 to 0.93 ($\mu = 0.87$, $\sigma = 0.03$). The modified MOOG displacement data has been used to produce load – displacement curves in the Results section; however, the laser sensor data has been used when calculating structural performance characteristics. The maximum magnitude of error introduced because of data cleansing is estimated to be less than $0.1mm$ for all specimens. We consider this error is acceptable since it is not statistically significant with respect to the results.

6 RESULTS

6.1 Determining Limit States

Shear displacement is corrected using the following formula (adapted from [6]):

$$\Delta_s = \Delta_1 - (\Delta_4 + \Delta_7) - (\Delta_2 + \Delta_5 - \Delta_3 - \Delta_6) \cdot \frac{h}{l} \tag{1}$$

where h is the height of the test panel, l is the length of the test panel and the numbering of the displacement subscripts matches the numbering of the LVDTs in Figure 9.

A design load of 6.4 kN/m was expected since this is the rated design load for this system in Australia (see Method A, Detail (h), Table 8.18, AS1684, Part 2 [10]). The design load for the first test panel in this study (M1.1) was set at 6.4 kN/m; however, it achieved an ultimate load of only 5.89 kN/m (see Figure 10 below). We note that this result is substantially lower than the result of 8.21 kN/m achieved by C.G. McDowall in his unpublished 2004 study titled, "*Combined Racking and Uplift Test Results for Panels of DD and Less than DD Grade Plywood & Some Thoughts on Choosing Limit States*," which used F8 timber and plywood from the same mill that was used in this study. The most likely explanation for this difference is that the density and joint strength of the MGP10 timber framing in our study was lower than that in the McDowall study.

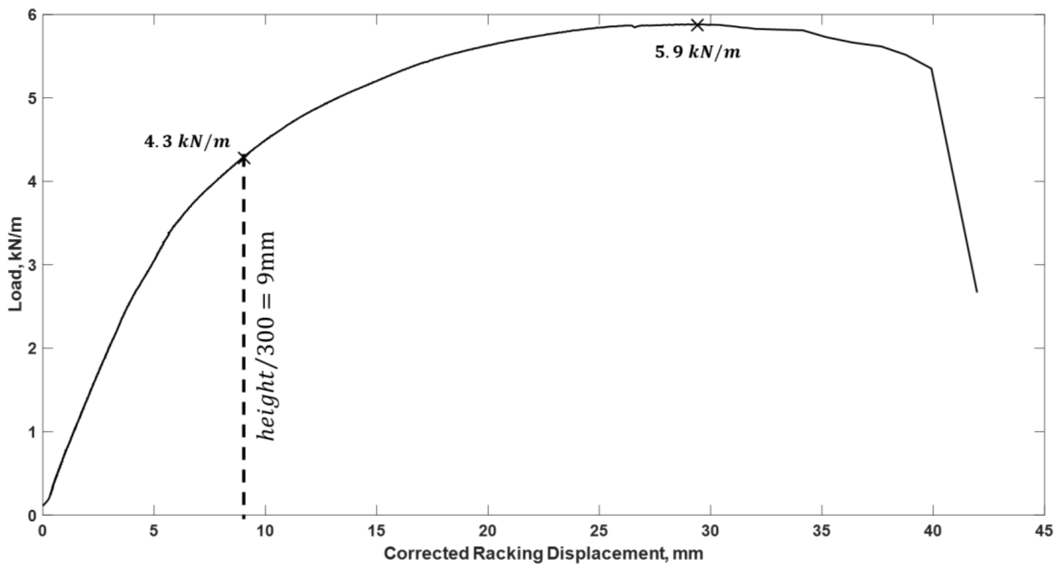

Figure 10: Load – Shear Displacement for Test Panel M1.1.

Since test panel M1.1 failed to achieve the design load of 6.4 kN/m, it seemed appropriate to select a more realistic design load for subsequent test panels. The design load of 5 kN/m was selected as it is higher than the serviceability load of 4.3 kN/m and corresponds with a displacement of $\left({}^{height}/_{150}\right)$.

A suitable design load can be determined by reference to clause D5.4(b) of [1] where the equivalent test action (Q_E) for this system is related to the design load (Q^*) by:

$$Q_E = \frac{k_2 k_{26} k_{27} k_{28}}{k_1} Q^* \qquad (2)$$

Where $k_2 = 0.8$ for domestic construction where failure occurs at the connectors, $k_{26} = 1.0$ for failure of metal fasteners in timber that is initially dry, $k_{27} = 1.0$ for metal fasteners where the duration of load is $15min$, $k_{28} = 1.7$ where the number of specimens is 2 and the coefficient of variation is less than or equal to 10%, and $k_1 = 1.0$ for strength of joints using laterally loaded fasteners and a duration of peak action of $5min$.

From Equation (2) then:

$$Q_E = \frac{0.8 \times 1.0 \times 1.0 \times 1.7}{1.0} \times 5.0 = 6.8\, kN/m$$

The load – displacement curves for the remaining M1 test panels are shown in Figure 11. Evidently, none of the panels achieved the Q_E of $6.8\, kN/m$; however, the panels did hold the design load of $5.0\, kN/m$ twice without showing any signs of failures. To meet the requirement for strength according to [1], the design load of the system should be revised down to:

$$Q^* = \frac{1.0}{0.8 \times 1.0 \times 1.0 \times 1.7} \times 5.98 = 4.4\, kN/m$$

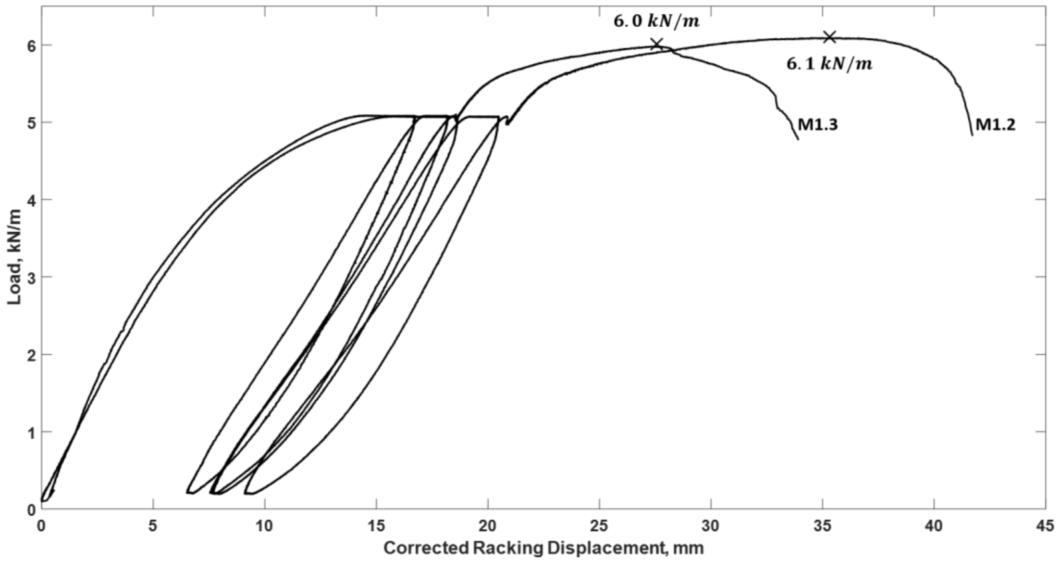

Figure 11: Load – Shear Displacement for Test Panels M1.2 and M1.3.

Clause D4.4 of [1] requires that the residual deflection of the assembly be no more than 30% of the deflection at Q_E. Based on readings from LVDT1, corrected for displacement of the floor beam, the residual deflection of M1.2 and M1.3, after unloading from the design load of $5.0\, kN/m$, passes this criterion with 25% of their respective deflection at the ultimate load. Clause D5.6.2 of [1] requires that the residual deflection from the second preloading phase must not exceed 5% of the deflection under short duration loading. On this criterion, both test panels fail with a residual deflection of 6% and 7% respectively of the deflection under short duration loading.

C.G. McDowall conducted many tests on timber-framed shear walls in Australia from the 1980s to the 2010s and argued in his 2004 report that the method in the Australian Standard AS1720.1:1997 [11] for determining limit state design values was not appropriate for shear walls. He proposed three criteria for determining an appropriate design racking load for walls, which have formed the basis for the racking strength of panels currently specified by producers in Australia (emphasis in original):

- Test panels "*are **stiff** enough to ensure the **serviceability limit state** is satisfied. To this end a **deflection limit** of panel height / 300 … is imposed.*"

- Test panels "*are **strong enough** to satisfy the **strength limit state**. This is interpreted herein to mean, although some **connector and material distress** may be evident, the panel can take further load*."
- Test panels "*remain **stable**, i.e., show no significant signs of buckling at the serviceability limit state*."

Further guidance is provided by McDowall on how to ensure these three criteria are satisfied. The serviceability limit, in kN/m, is determined by examining the load – deflection curves of test data and taking the average value at the deflection limit of $\left({}^{height}/_{300}\right)$. The strength limit state (*i.e.*, design racking load) is then determined by satisfying two criteria:

- Strength Limit State ≥ 1.5 × Serviceability Limit State; and
- Strength Limit State ≤ 0.8 × Average Ultimate Racking Load.

Since the Strength Limit State has an upper bound of 80% of the average ultimate racking load, this means that the Serviceability Limit State design value must be reduced if the first criterion is not met. Adopting McDowall's 2004 methodology and applying it to the results of all three M1 test panels, the upper limit for the factored design strength of this wall system is:

$$Q^* \leq 0.8 \cdot \frac{\sum P_{ULT}}{n} \tag{3}$$

$$\therefore Q^* \leq 0.8 \times \frac{5.89 + 6.09 + 5.98}{3} \leq 4.79\ kN/m$$

and the upper limit for the serviceability limit state is:

$$Q_{serv} \leq \frac{Q^*}{1.5} \tag{4}$$

$$\therefore Q_{serv} \leq \frac{4.79}{1.5} \leq 3.19\ kN/m$$

6.2 Load – Displacement Curves

The load – displacement curves for the other loading protocols are shown in Figures 12 to 17.

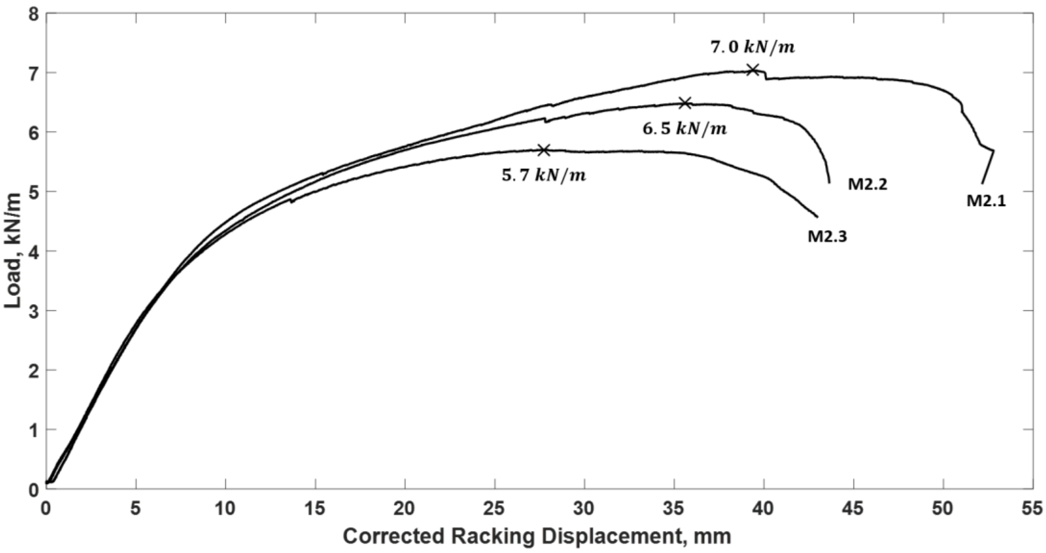

Figure 12: Load – Shear Displacement for M2 Test Panels.

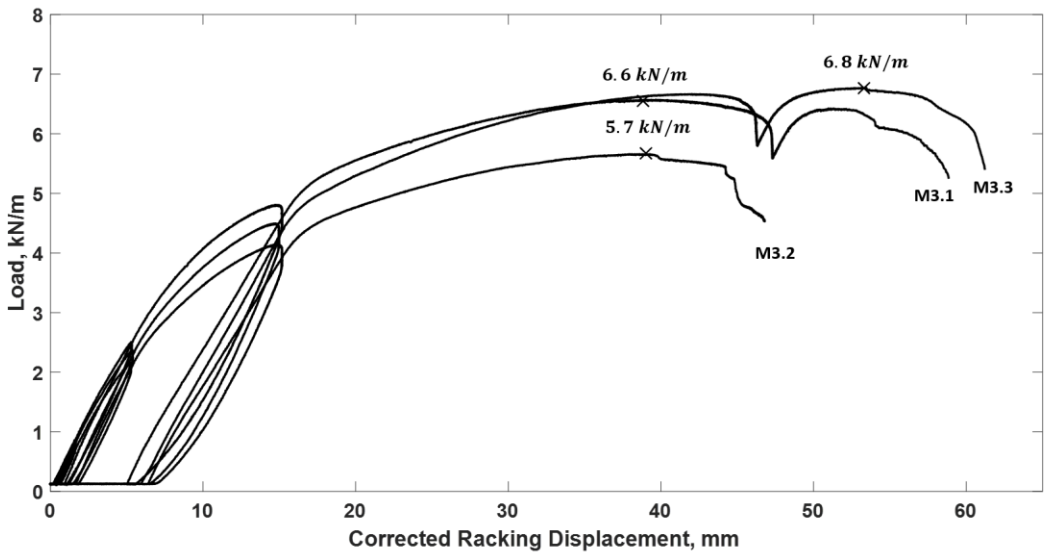

Figure 13: Load – Shear Displacement for M3 Test Panels.

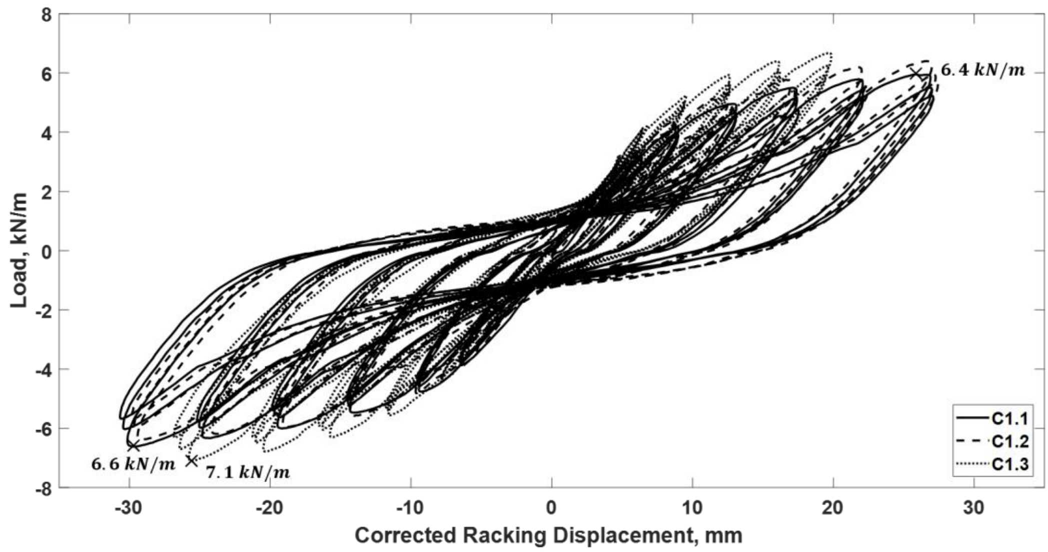

Figure 14: Load – Shear Displacement for C1 Test Panels.

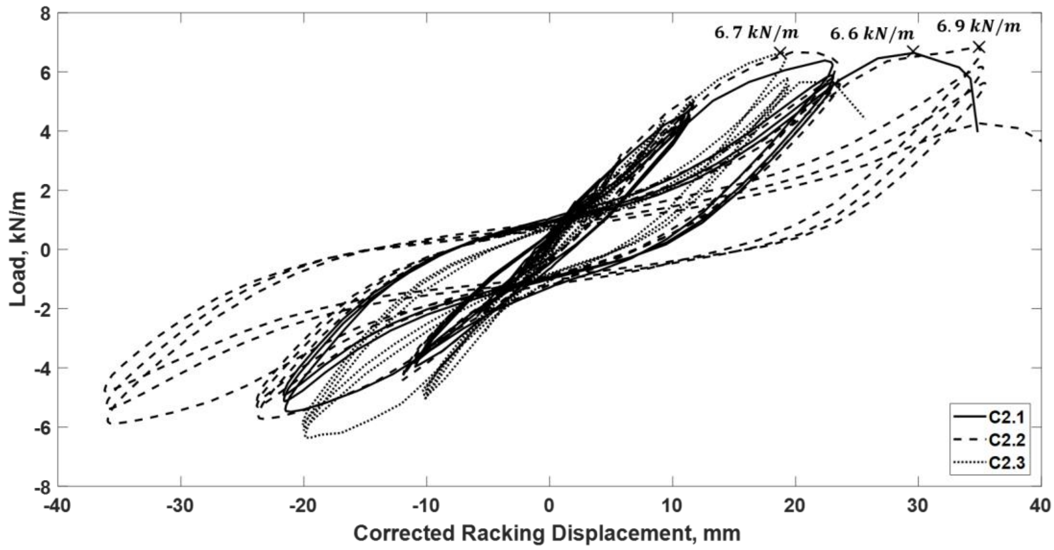

Figure 15: Load – Shear Displacement for C2 Test Panels.

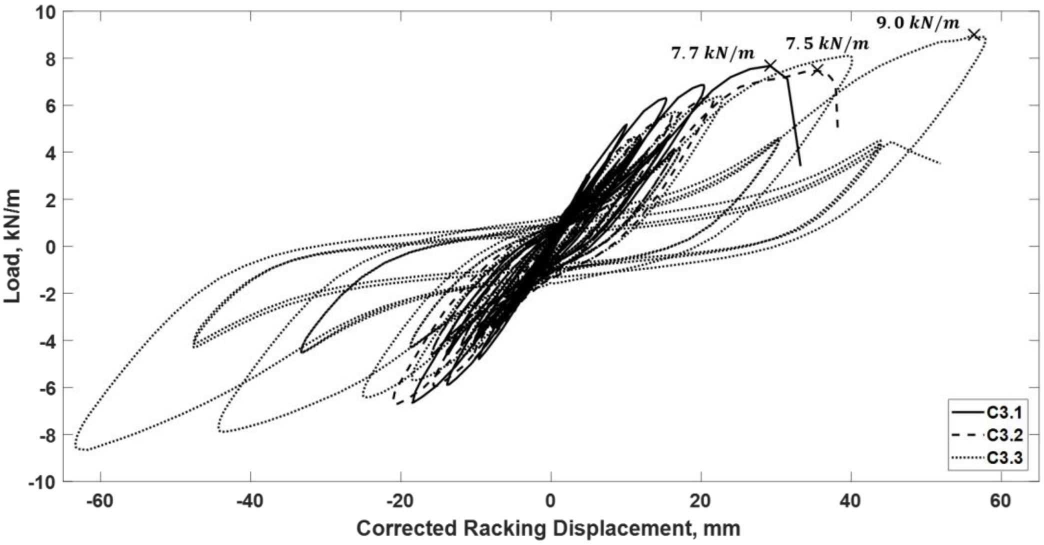

Figure 16: Load – Shear Displacement for C3 Test Panels.

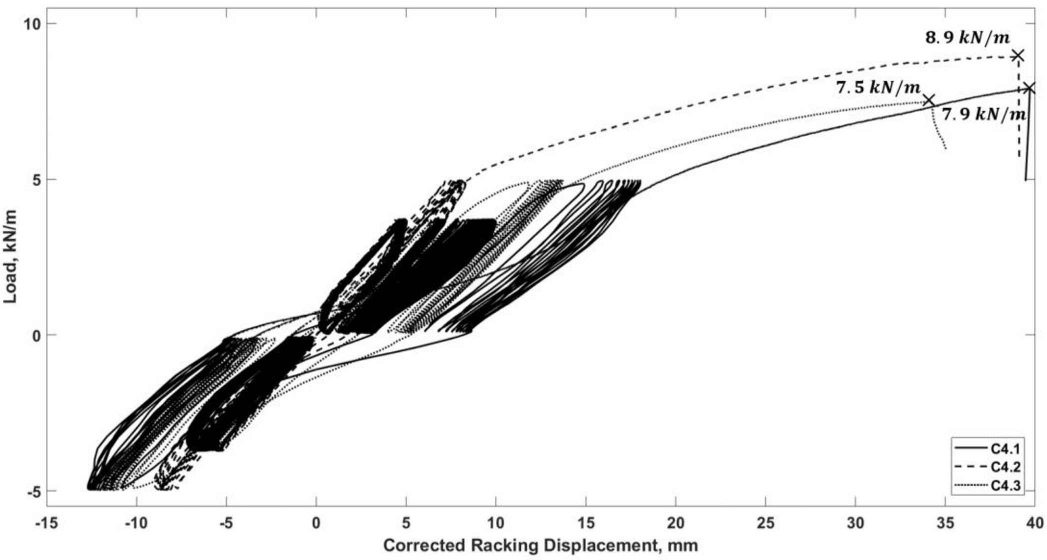

Figure 17: Load – Shear Displacement for C4 Test Panels.

6.3 Ultimate Capacity by Group

All the test methods in this study determine strength from the ultimate load achieved in each test. Ultimate load results are summarised in Table 2.

Table 2: Ultimate Load by Group.

Group	P_{ULT} (kN/m)	$\mu(P_{ULT})$	$\sigma(P_{ULT})$	% diff*
M1	5.89, 6.09, 5.98	5.99	*0.10*	-
M2	7.04, 6.48, 5.70	6.41	*0.67*	+7.0%
M3	6.57, 5.66, 6.78	6.34	*0.59*	+5.8%
C1	6.62, 6.40, 7.10	6.70	*0.36*	+12.0%
C2	6.64, 6.88, 6.66	6.73	*0.13*	+12.4%
C3	7.67, 7.46, 8.95	8.03	*0.81*	+34.1%
C4	7.91, 8.93, 7.49	8.11	*0.74*	+35.5%

Note: * Last column is percent change compared to M1.

Ultimate loads of each group are compared in a boxplot in Figure 18. The tight variation in group M1 and C2 is worth noting (σ of 0.10 and 0.13 respectively).

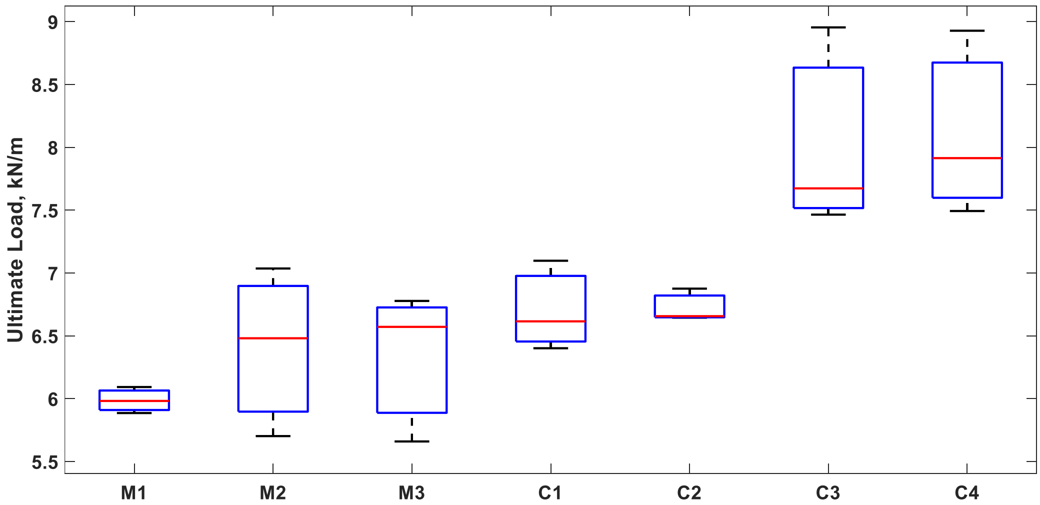

Figure 18: Boxplot of Ultimate Load by Group.

Adopting Equation (3) and applying it to the results of all three C4 test panels, the upper limit for the factored design strength of this wall system is:

$$Q^* \leq 0.8 \times \frac{7.91 + 8.93 + 7.49}{3} \leq 6.49\ kN/m$$

and the upper limit for the serviceability limit state, using Equation (4), is:

$$Q_{serv} \leq \frac{6.49}{1.5} \leq 4.33\ kN/m$$

6.4 Significance of the Results

These results confirm that the decision as to which loading protocol and test method one adopts has an influence on the ultimate capacity. The monotonic test methods all produced ultimate loads that were lower than the lowest results of the cyclic test methods. All the test panels under the CUREE and CTS TR5 protocols (group C3 and C4) produced higher ultimate loads than any other group. These results are consistent with previous work by Gatto and Uang [2] in which test panels recorded higher ultimate capacities under the CUREE loading protocol compared with the ISO 16670 protocol (method B of ASTM E2126 [8]) and the monotonic protocol of ASTM E564 [6]. A possible explanation for this result is work hardening of the connectors under cyclic loading as suggested by Dolan, Gutshall and McLain [12] who encountered a similar result in a monotonic and cyclic study of nailed connections into timber through three different materials (timber, plywood and steel) where 'cycled' specimens achieved an ultimate capacity $1 - 7\%$ higher than those specimens which had not been cycled. Reardon's [9] comment that "*there is evidence to show that for timber framed walls the wall strength is unaffected by a previous history of cyclic loading*" understates the reality. Evidently, wall strength is improved by a previous history of cyclic loading using these test protocols. The higher ultimate capacity of test panels under cyclic loading with many load cycles was not anticipated at the outset of this study. Previous unpublished work by the first author with small scale specimens of sheathing-to-timber connections showed, with one interesting exception, a consistent reduction in ultimate capacity of about 5% for cycled specimens compared with those that had not been cycled. Also, He, Lam and Prion [13] found that cyclic test methods with an excessive number of cycles, applied to full-scale timber-framed shear walls, correlated with lower ultimate capacities.

A statistical analysis of the ultimate capacities is needed to determine whether the choice of loading protocol produces a statistically significant effect. Parametric statistical analyses rely on relatively large sample sizes and a normally distributed population from which samples are gathered [14]. A nonparametric statistical method, such as the Wilcoxon rank-sum test, is preferred since it relies on fewer assumptions, does not assume a normally distributed population and is less sensitive to outliers [15]. The null hypothesis being tested by the rank-sum statistical test, summarised in Table 3, is a right tailed test in which the median of the first group (first column in Table 3) has to be less than or equal to the median of the second group (first row in Table 3). If the null hypothesis is rejected with a p-value less than or equal to 0.05 the result is statistically significant and we can say with reasonable confidence that the loading protocol had a statistically significant influence on the ultimate loading capacity of the standard timber-framed shear wall in this study.

Table 3: Results of One-Sided Wilcoxon Rank-Sum Test of Ultimate Load Data (from Table 2 above) by Group.

Group	M1	M2	M3	C1	C2	C3	C4
M1	-	0.80	0.80	1.0	1.0	1.0	1.0
M2	0.35	-	0.50	0.80	0.80	1.0	1.0
M3	0.35	0.65	-	0.80	0.90	1.0	1.0
C1	***0.05***	0.35	0.35	-	0.80	1.0	1.0
C2	***0.05***	0.35	0.20	0.35	-	1.0	1.0
C3	***0.05***	***0.05***	***0.05***	***0.05***	***0.05***	-	0.65
C4	***0.05***	***0.05***	***0.05***	***0.05***	***0.05***	0.50	-

Note: Statistically significant results in ***bold italics***.

These results in Table 3 show that the higher ultimate load of group C3 (CUREE loading protocol [8]) and C4 (CTS TR5 loading protocol [9]) is statistically significant with respect to all other groups. The higher ultimate load of all cycled test panels (group C1 to C4) is also statistically significant with respect to group M1 (loading protocol based on AS1720.1 Appendix D [1]).

6.5 Failure Modes

Most of the test panels in this study failed at the connectors between the sheathing and the timber framing. The most common failure mode was nail pullout (see Figure 19).

Figure 19: Test Panel M1.3 After Failure showing Nail Pullout Failures.

Nail pull-through failures were observed in some instances (*e.g.*, three connectors in C3.2, two connectors each in test panels M1.1 and C4.2 and one connector in test panel C2.2). Fatigue fractures of connectors were observed in test panels C2.2 (two connectors) and C3.3 (thirteen connectors).

Failure of the sheathing by tearing was noted in test panels C2.1 (one tear), C2.2 (tears in both sheets), C3.2 (two small tears in one sheet and two large tears in the other sheet) and C4.2 (one tear, see Figure 20). Tearing of the sheathing only occurred on the bottom corners of the sheathing. Of note, the panels tested under the BRANZ P21 loading protocol [7] (C1.1 to C1.3) did not fail completely. This is because the maximum displacement under the BRANZ P21 loading protocol was much lower than the maximum displacement under the other protocols. Even so, localised nail pullout failures were observed with test panels C1.1 to C1.3 at the middle top of the test panels where the load was applied.

Figure 20: Test Panel C4.2 After Failure.

Test panel C4.3 is shown in Figure 21 after failure.

Figure 21: Test Panel C4.3 After Failure.

6.6 Structural Performance Characteristics by Group

Many structural performance characteristics, such as stiffness and ductility are not explicitly defined in some of the test methods and, where they are defined, they are defined in different ways. For example, racking stiffness is defined in EN 594 [5] as the secant modulus of the load – displacement curve taken between a racking load of 20% and 40% of the ultimate load, whereas stiffness is defined in ASTM E564 [6] as the secant modulus of the load – displacement curve between a racking load of 0% and 33% of the ultimate load. Curiously, the ASTM E2126 method [8] defines stiffness as the secant modulus of the averaged envelope load – displacement curve between a racking load of 0% and 40% of the ultimate load.

The New Zealand BRANZ P21 method [7] does not provide guidance on calculating structural performance characteristics but does provide a detailed methodology for separately determining the earthquake and wind rating of the tested shear wall system.

Neither the CTS TR5 protocol [9] nor C.G. McDowall's unpublished 2004 report, which modifies the protocol, provide guidance on calculating structural performance characteristics; however, there is some guidance on selecting appropriate serviceability and design loads. McDowall's 2004 guidance for determining serviceability and strength limit states, outlined above, is to be preferred.

For the sake of comparison in this paper, structural performance characteristics will be calculated in accordance with the methods outlined in [8] using data from LVDTs. In the case of cyclic test methods, the averaged envelope curve will be used. The global secant shear modulus is calculated at $0.4 \times P_{ULT}$:

$$G' = \frac{P}{\Delta} \times \frac{h}{l} \quad (5)$$

where P is the load at $0.4 \times P_{ULT}$ in kN, Δ is the displacement at a load of $0.4 \times P_{ULT}$ measured by LVDT1 and corrected for displacement of the floor beam, which includes components of shear, rotation and translation.

Using corrected shear displacement Δ_s from Equation (1), calculated at $0.4 \times P_{ULT}$, the internal secant shear modulus is:

$$G_{int} = \frac{P}{\Delta_s} \times \frac{h}{l} \quad (6)$$

The yield load is determined by deriving an equivalent energy elastic-plastic (EEEP) curve from the global load – displacement or envelope curve:

$$P_y = \left(\Delta_u - \sqrt{\Delta_u^2 - \frac{2A}{K_e}} \right) K_e \quad (7)$$

where A is the area under the envelope (cyclic) or load – displacement (monotonic) curve, Δ_u is the ultimate displacement corresponding to the last data point on the envelope or load – displacement curve where the absolute load is equal to or greater than $0.8 \times P_{ULT}$, K_e is the elastic shear stiffness equal to $(0.4 \times P_{ULT})/\Delta_e$ and Δ_e is the displacement measured by LVDT1 and corrected for displacement of the floor beam at $0.4 \times P_{ULT}$.

The ductility ratio is:

$$D = \frac{\Delta_u}{\Delta_y} \quad (8)$$

where Δ_y is the displacement taken from the EEEP curve at P_y.

Total dissipated energy is calculated as the accumulation of area under the load – displacement curve as measured at the MOOG actuator.

The test durations and mean structural performance characteristics of the standard test panel under seven different loading protocols are summarised in Table 4 with standard deviations italicised in parentheses under. It is worth noting that Δ_u is not properly determined under the BRANZ P21 protocol. We understand this is because the New Zealand method is primarily used to assess timber shear walls braced by gypsum plasterboard, which has a lower deformation capacity than typical plywood or OSB braced shear walls.

Table 4: Test Duration, Mean Stiffness, Yield Load, Ductility and Total Dissipated Energy by Group.

Grp	Test Duration (min)	G' (kN/mm)	G_{int} (kN/mm)	P_y (kN/m)	D (mm/mm)	Total Dissipated Energy (N.m/m)
M1	42.98†	0.95	2.27	5.41	4.32	307.9
	(0.23)	*(0.15)*	*(0.81)*	*(0.06)*	*(1.54)*	*(14.8)*
M2	6.38	1.57	3.16	5.65	6.88	379.5
	(0.42)	*(0.41)*	*(0.49)*	*(0.49)*	*(1.41)*	*(68.9)*
M3	52.65	1.48	1.65	5.66	6.13	363.8
	(6.39)	*(0.23)*	*(0.30)*	*(0.51)*	*(1.02)*	*(84.1)*
C1	7.75	1.61	2.34	5.64*	3.38*	1 794.4
	(0.00)	*(0.21)*	*(0.24)*	*(0.35)*	*(0.61)*	*(118.3)*

C2	0.89 *(0.31)*	2.23 *(0.45)*	2.37 *(0.45)*	5.55 *(0.17)*	4.89 *(1.44)*	814.9 *(294.4)*
C3	1.46 *(0.58)*	2.16 *(0.46)*	2.38 *(0.72)*	6.71 *(0.54)*	4.99 *(0.92)*	1 251.2 *(777.0)*
C4	19.65 *(0.30)*	1.16 *(0.21)*	2.34 *(0.74)*	6.51 *(0.70)*	3.60 *(0.48)*	1 385.1 *(94.2)*

Note: Standard deviation *italicised* in parentheses.

† Based on M1.2 and M1.3 only – M1.1 had a test duration of 3.06 min.

* Δ_u not properly determined in group C1.

While global stiffness, G', shows some variation across the different groups, internal shear stiffness, G_{int}, is quite similar. Duration of load (DOL) effects, such as creep, may have influenced the results for stiffness. Notice that the three test methods with the highest test duration also have the lowest results for internal shear stiffness. DOL is certainly a known effect in timber members (*e.g.*, [16] and [17]); however, the influence of DOL effects on timber connections is not that well established (see for example [18] and [19]). These results neither support the hypothesis that DOL effects had an influence nor that DOL effects had no influence.

Since the yield load here is a function of the EEEP curve and not the ultimate load, the difference between the groups is less pronounced ($5.41\ kN/m - 6.71\ kN/m$) than is noticed in ultimate loads in Table 2 ($5.99\ kN/m - 8.11\ kN/m$). As noted above with ultimate load, however, the yield load of test panels in groups C3 and C4 (CUREE [8] and CTS TR5 [9] loading protocols) is noticeably higher than the yield load of test panels in the other groups. A one-sided rank-sum test demonstrates that this result is statistically significant at the 5% significance level.

The in-group variance of ductility is high enough to make the between-group variance of ductility seem unremarkable, meaning that the ductility of the standard timber-framed shear wall cannot confidently be predicted by one's choice in loading protocol.

More energy is dissipated by test panels under cyclic loading compared with panels under monotonic loading.

7 DISCUSSION

We were surprised by the results of this comparative experimental study. Earlier unpublished work by the first author on small-scale sheathing-to-timber connections found that cycled specimens achieved, on average, a 5% lower ultimate load compared with specimens tested under a monotonic load. Tests on full-scale timber-framed shear walls [13] found that cyclic test methods with an excessive number of cycles correlated with lower ultimate capacities. Shear capacities for timber-framed walls published by the American Wood Council (AWC) allow higher capacities for walls under wind load than under seismic load [20]. The commentary in [20] makes it clear that seismic capacities of shear walls are based on cyclic testing to the CUREE protocol in [8] and wind capacities are based on monotonic testing to [6]. Another study [21], using small specimens of sheathing-to-timber nailed connections, found that cycled specimens achieved 15% lower ultimate load than specimens tested to a monotonic loading protocol. A recent full-scale study [22] found that cycled specimens achieved lower ultimate loads in all instances except one. With this information in advance, we expected to find that, as the number of cycles increased, test panels would fail at lower loads. Instead, we found that hysteresis on full-scale walls seems to improve ultimate capacity. It seems, however, that our finding is not without precedent (see for example [2] and [12]). Our findings have implications for the wind and seismic rating of timber-framed shear walls in other parts of the world. We have shown that the monotonic test methods in our study produced more conservative results than either the cyclic CUREE protocol (C3) or the modified CTS protocol (C4). This finding is consistent with the work of Gatto and Uang [2] and leads us to ask why the AWC published ratings for walls under wind load, in Table 4.3A of [20], are higher than the ratings for walls under seismic load.

It is likely that differences in boundary conditions influenced the results of this study to some extent. While the tie-down rods at both ends of the test panels were identical in all loading protocols, there were some differences in the manner of fixing to the floor beam and loading ram. Three of the loading protocols, M3, C2 and C3, had an identical method of fixing to the floor beam and yet the ultimate loads from these three groups range from the lowest in the study (test panel M3.2 $= 5.66\ kN/m$) to the highest in the study (test panel C3.3 $= 8.95\ kN/m$). If the method of fixing the test panel to the floor beam had any effect at all, it is not clear from these results. The three monotonic loading protocols had different methods of fixing to the floor beam and identical methods of applying the load and yet the results from these three groups are quite similar. Three of the cyclic loading protocols, C2, C3 and C4, had an identical method of fixing to the loading ram and yet the C2 group had a statistically significant lower ultimate load than the other two. This analysis supports the view that the loading protocol itself accounts for most of the difference in results and that the influence of different boundary conditions, which no doubt exists to some extent, is not well supported by the evidence. A separate study would be needed to examine the influence of boundary conditions on the structural performance of timber-framed shear walls.

If a conservative design rating is preferred, it seems clear that the ideal loading protocol is a monotonic test method. In particular, the Australian practice, based on [1], produces a tight cluster of results with the lowest ultimate and yield strength. On the other hand, it could be argued that the cyclic test methods are more accurate at replicating real world loading profiles and, for that reason, are to be preferred even though they produce less conservative results. We have discussed our findings with other academics ([23] and [24]) and practising engineers ([25], [26] and [27]) to assist our interpretation of the results in determining whether to adopt a cyclic test method as standard in Australia. There was no consensus arising from these discussions. Our peers valued conservative results (*i.e.*, monotonic test results) but some also valued test methods that better replicated expected load profiles (*i.e.*, cyclic test results). The prospect that load ratings could be higher when tested under cyclic loading was viewed as a potential benefit to the industry. Indeed, if the results from group C4 are used, there is no need to reduce the design load of $6.4\ kN/m$ already published in AS1684 [10]. Reflecting on the CTS TR5 [9] results (group C4) we have since questioned our original decision to eliminate 800 cycles from the test method by adopting the modified 1995 methodology of C.G. McDowall. The test duration of group C4 was about $20min$ compared to a test duration of approximately $43min$ for test panels M1.2 and M1.3. We recognise that longer test durations would more accurately represent the expected load profile of a high wind event such as a cyclone, which can last hours. It would be instructive to conduct an additional test of three test panels under the CTS TR5 protocol with the extra 800 cycles at 62.5% of the design load. It is not clear whether such a test method would lead to higher or lower ultimate capacities. For the present, we recommend adopting a monotonic test method, like the method based on [1] applied to group M1 of this study, as the standard test method for timber-framed shear walls in Australia. The method is simple to perform and produces conservative results.

Practical considerations also need to be considered in choosing a test method as the standard approach for Australia. Some methods are easier to implement in the laboratory compared with others.

By far, the most difficult test method to execute was the New Zealand BRANZ P21 test method [7] (group C1). The BRANZ P21 detail for attaching the header beam to the test panel seemed to contribute to a concentration of load at the centre of the test panel where failures were first noticed during testing. Whether this detail is a better representation of how load is transferred from the horizontal diaphragm to the vertical diaphragm in real structures, in comparison to the simpler ASTM methods [8], is questionable. After all, the ISO 16670 method (group C2) produced similar results to the BRANZ P21 method. Several details in AS1684 [10] illustrate how the roof structure is to be connected to the top of the shear wall. At a minimum, two connections would be required over a $2.4m$ long wall to ensure load transfer between the roof and the wall. We believe the BRANZ P21 practice of connecting the loading apparatus to the wall at just one location is not representative of construction practice in Australia.

The test setup for the CTS TR5 [9] protocol was more difficult than most other setups due to the use of 'dummy' joists under each stud. While this detail is certainly representative of many houses built in Australia, it seems unnecessary given the results here. We recommend against using dummy joists or foundation beams in the test setup.

Load-controlled methods such as the Australian practice based on [1] and the CTS TR5 [9] protocol are quite tricky to implement because the actuator must be tuned to the stiffness of the test panel. Load-controlled methods also have the potential to be dangerous and costly in the laboratory if the test panel fails under load control because the actuator 'chases' the load, which can lead to dramatic failures, potentially damaging instrumentation and equipment. Test panel M1.1 failed in this manner. While a strong argument can be made in support of adopting a displacement-controlled test method, we also recognise that there is a wide variance in stiffness between individual test panels. This means that, for a given displacement, the load will vary considerably between test panels. Notice in Figure 17 that test panel C4.1 experienced more than double the displacement of test panel C4.2 at the design load of $5\ kN/m$. As we showed in [3], wind load governs the design of lateral force resisting systems in Australia and, if we ignore possible dynamic interactions between wind and structure, the wind load *is what it is* regardless of the stiffness of the structure. We recommend adopting, as standard, a load-controlled test method. Safety concerns can be managed by first conducting a displacement-controlled monotonic test to obtain a suitable 'design load' for a given shear wall system.

8 CONCLUSIONS & RECOMMENDATIONS

We have presented here some of the findings from our experimental study comparing the structural performance characteristics of a standard timber-framed shear wall under three different monotonic and four different cyclic loading protocols. Test panels were $2700\ (h) \times 2400mm\ (l)$ using MGP10 framing and $7mm$ thick F8 radiata pine plywood sheathing fixed with $2.8\ (\phi) \times 30mm\ (l)$ galvanised clouts at $150mm$ spacings around the edges of each panel and $300mm$ spacings along the intermediate stud. Most notably, the choice of loading protocol influenced the measured strength of the test panels. Ultimate strength was between 6% and 36% higher in those test panels that were tested to a cyclic loading protocol compared to those tested under a monotonic loading protocol. Surprisingly, the group with the highest average ultimate strength (group C4) was also the group with the most cycles (220). We acknowledge that differences in boundary conditions may have some influence on the results; however, our analysis of the data does not provide any support for this hypothesis. We speculate that work hardening of the connectors and duration of load effects may have contributed to this result. Internal shear stiffness was similar regardless of loading protocol.

Based on these results, we recommend that a standard test method be developed for use in Australia with the following characteristics:

- First test a panel with a monotonic loading protocol under displacement control, similar to EN 594 [5];
- Let Q_E equal the ultimate load obtained and determine an initial estimate for design load Q^* using clause D5.4(b) of [1] – Equation (2) above;
- Conduct testing on a minimum of three (3) panels with a monotonic loading protocol, under load control, in keeping with the principles of [1]; and
- Use C.G. McDowall's 2004 methodology to determine a final strength limit state design load, Q^*, and serviceability limit state load, Q_{serv}.

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