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# TEST REPORT

## ST1089

### GYMNASIUM WALL TESTING FOR MBIE AND MOE

#### CLIENT

Ministry of Business Innovation and Employment  
PO Box 1473  
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And

Ministry of Education  
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# 1. INTRODUCTION

BRANZ was commissioned jointly by the Ministry of Education and the Ministry of Business, Innovation and Employment to conduct laboratory tests on a full-scale wall in order to further understand the lateral load resisting behaviour of a particular type of school gymnasium under fully reversed cyclic loading in the longitudinal direction of the gymnasium.

The results of this testing sequence will assist both Ministries with seismic assessments of an extensive stock of school gymnasium buildings throughout the country as well as other similarly constructed buildings.

# 2. ORIGINAL BUILDING AND DETAILS

The wall test specimen was based on details provided for a gymnasium building located at Naenae College in Lower Hutt, just north of Wellington. An original set of drawings was provided and the details on the drawings were verified through two visits by BRANZ staff to the building in Naenae. The building consists of a series of steel portal frames with timber frame infill walls between the frames. The portal frames are connected using steel channel sections bolted to lugs welded to the frames at two locations up the height of the portal frame legs. In some bays between the steel portal frames there are diagonal steel strap braces bolted to lugs welded to the frames. The building has a total of eight portal frames, resulting in seven bays, and on each side of the building there are four sets of diagonal braces, two upper and two lower, as shown in Figure 1.

Most of the details from the original drawings were apparent in the building and were represented in the test specimen. One notable exception is that plywood had been installed on top of the weatherboard on both sides of the longitudinal walls, as shown in Figure 2 and Figure 3. On the north side of the gymnasium the plywood was installed up to the level of the windows, except in the first bay where it goes all the way to the top of the wall. On the south side of the gymnasium the plywood was installed to approximately 2.4 m up from where the weatherboard started at the bottom of the wall. It appeared that the plywood had been nailed directly to the weatherboard cladding with sparsely placed nails. As described in the next section, the plywood was not included in any of the test configurations as it was not considered original and it is possible that other gymnasiums around New Zealand do not include this extra cladding material.

Another aspect of the built gymnasium that was not included in the test specimen is that the steel columns making up the portal frame legs were encased in concrete in the original building. This was not feasible to include in the test specimen and therefore it only included the steel sections as columns under the assumption that this would conservatively approximate the as-built structure.

The drawings also indicated that the bases of the concrete-encased columns were constrained by a 2'4" high reinforced concrete perimeter foundation wall on top of a 1' thick by 2'6" wide reinforced foundation beam. This foundation system would



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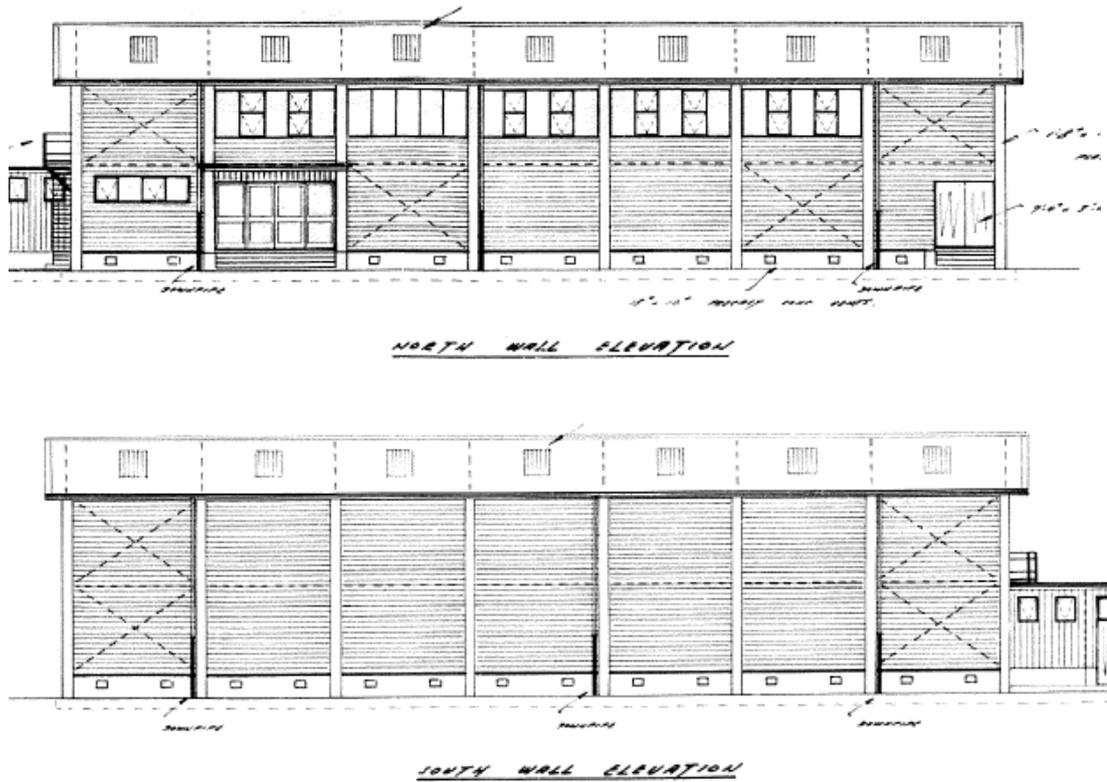
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have offered a reasonable amount of rotational restraint to the column bases which was not able to be replicated in the time available for conducting the tests.



**Figure 1. Exterior Gymnasium Wall Elevations from Original Architect Drawings for Naenae College (Note: Steel Bracing shown as Dotted Lines between Portal Frames)**



**Figure 2. North Side of Naenae College Gymnasium**



**Figure 3. South Side of Naenae College Gymnasium**



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### 3. GYMNASIUM WALL TEST SPECIMENS

In order to determine the contributions of the different materials making up the infill walls between the steel portal frames and possible contributions of these materials to the lateral load resisting behaviour of the walls, a specimen was constructed and tested in the Structures Laboratory at BRANZ. The specimen was tested in several different configurations having different combinations of the steel frames and bracing, interior linings, exterior cladding and openings (representing the large upper windows seen in Figure 2).

The dimensions of the test specimen can be seen in Figure 4, which also shows the steel columns, beams and braces that were included in the first test configuration. The steel columns were intended to model the portions of the portal frame columns that are only encased in concrete but do not have a concrete wall supporting the columns. These short concrete walls can be seen at the bottom of the wall in Figure 3. The columns used in the test were slightly smaller than those in the as-built gymnasium but their weak axis stiffness was very similar to the as-built columns. It was assumed for the test specimen that the columns were essentially pinned at the bottom and were connected to the laboratory strong floor using a 12 mm thick mild steel plate welded to the column bottoms and then bolted to the strong floor using two 16 mm diameter bolts that were in line with the column webs.

A total of nine configurations were tested using the steel columns and beams shown in Figure 4. These configurations are described in Table 1.

**Table 1. Gymnasium Wall Configurations Tested**

Test	Description
A1	Steel frame with steel cross braces – ends bolted with a single grade 4.6 M12 bolt
A2	Steel frame with steel cross braces – ends bolted with a single grade 8.8 M12 bolt
A3	Steel frame with steel cross braces – ends welded to lugs with 6 mm fillet welds (150 mm length)
B	Interior linings with original nailing
C	Interior linings with new nailing
D	Weatherboard exterior cladding only
E	Interior linings (original nailing) and weatherboard with window near top
F	Interior linings (new nailing) and weatherboard with window near top
G	Steel columns and beams and timber frame only (no steel cross bracing, linings or weatherboard cladding)



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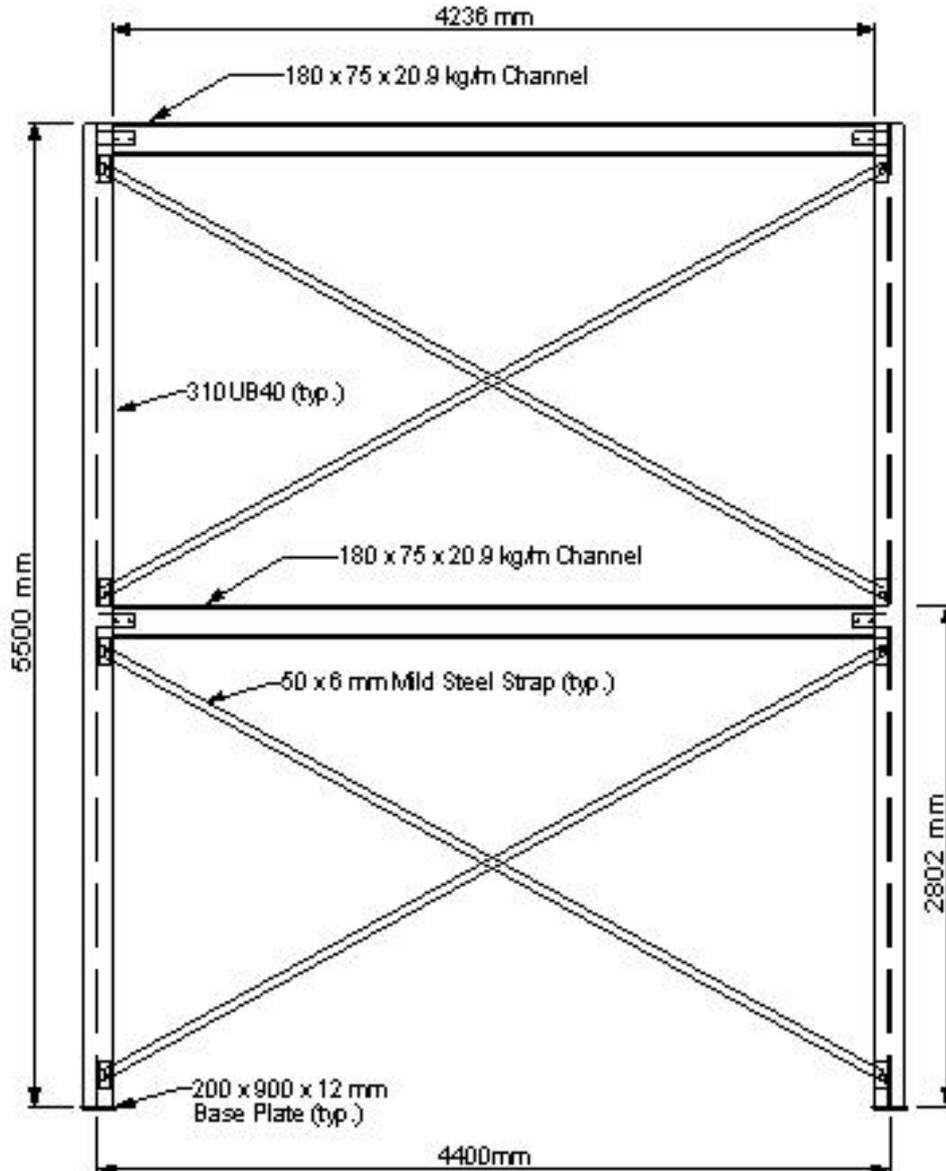


Figure 4. Steel Components of Wall Test Specimen and Overall Dimensions

### 3.1 Tests A1 to A3

Tests A1 to A3 included only the steel columns, beams and braces and were intended to evaluate the response of steel braces and the connections of the braces to the portal frame columns.

### 3.2 Tests B to G

All other tests included the steel columns and beams, but no steel cross braces. In order to minimise the effect of the connections between the beams and columns, only a single 12 mm diameter bolt was used to connect the beams to the columns for all but Tests A1, A2 and A3.

The timber framing was installed from the ground to the bottom of the lower beam and then continued from the top of the lower beam to the bottom of the upper beam and remained in place for Tests B, C and D. All framing timber was 140 mm x 45 mm SG8 material that was supplied from local sources. In order to attach the vertical edge timbers to the steel columns it was necessary to secure lengths of framing timber along the length of each column using shot fired fasteners at approximately 800 mm centres, as seen in Figure 5. The timber bottom plate was secured to the strong floor with 16 mm bolts including 3 mm x 50 mm x 50 mm square washers, placed at 900 mm centres. The other plates were bolted to the channel section beams with M12 bolts and square washers at 900 mm centres.



**Figure 5. Test configuration B under construction**

### **3.2.1 Interior Lining Materials**

The studs were spaced at nominal 600 mm centres to match the size of the lining sheets, which were 12 mm thick sheets of particle board. This thickness of sheet is now only available with 2.4 m x 1.2 m dimensions. In the Naenae gymnasium building, the sheets were 12' x 6' with their long dimension oriented vertically. The stud spacing was 18". However, such a spacing would not have suited the sheet size now available so a decision was made to increase the stud spacing to 600 mm. This was not considered to alter the performance of the test specimens appreciably.

Nogging members were installed at 1200 mm centres between the studs. The sheets were laid up with their long dimension in a vertical orientation and the sheets were laid in a staggered bond pattern to approximate the effect of the larger sheets in service. In the as-built gymnasium, the larger sheets extended over the full 12' height of the particle board section. Particleboard panels for Test B were installed using 1.6 mm x 40 mm jolt head nails at 300 mm centres at panel edges and along intermediate studs. Following Test B the panels were re-nailed using larger nails with flat heads in an effort to assess a possible retrofit solution in Test C. Particleboard panels were re-fixed using 2.5 mm x 50 mm flat head nails at 300 mm centres at panel edges and along intermediate studs.

From 12' (~3.6m) to the top of the test specimen, the lining was 4.5 mm thick hardboard (untempered). These sheets were manufactured to 8' x 4' dimensions and had to be trimmed to 1200 mm wide to suit the test setup. Hardboard panels for Test B were installed using 1.6 mm x 25 mm jolt head nails on 300 mm centres at panel edges and along intermediate studs. The hardboard panels were re-nailed using 2.5 mm x 30 mm flat head nails on 300 mm centres at panel edges and along intermediate studs for Test C.

### 3.2.2 Exterior Cladding Material

Following Tests B and C the interior lining materials were fully removed from the timber framing and nominal 200 mm x 25 mm Rusticated Weatherboards were attached to the opposite side of the specimen from the interior linings for Test D. The weatherboards had a nett vertical coverage of 155 mm per board. While these could have been laid as continuous lengths between the columns, butt joints were created in every fourth row to approximate the arrangement observed on the actual building. The weatherboards were fixed to the studs with 2.8 mm x 60 mm jolt head nails, one per crossing.

### 3.2.3 Large Upper Opening

For Tests E and F, the studs in the upper section of the frame were trimmed down to the level of the sill for the window in the as-built gymnasium, which was taken to be 3.6 m from the bottom of the specimen. A continuous 140 mm x 45 mm SG8 sill trimmer member was placed over the length of the specimen across the top of the trimmed studs and fixed to the top of the studs with two 90 x 3.15 gun driven nails per stud. This was intended to replicate additional framing that would likely have been used around the window opening in the original building and also to avoid any out of plane movement of the framing during testing. Test E was conducted using the original nail size and spacing for the particleboard inner lining and included the exterior cladding material up the bottom of the window opening as shown in Figure 6. Test F was conducted using the same additional nailing for the particleboard as in Test C and included the exterior cladding material up the bottom of the window opening.



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**Figure 6. Test Configurations E and F with Particleboard on the Interior (Left) and Rusticated Weatherboard on the Exterior (Right)**

### 3.2.4 Unbraced Steel Frame with Timber Framing Only

For Test G, the linings and weatherboard cladding were removed entirely and the unbraced steel frame and timber framing were tested to ensure that their contribution to the other test results was not significant.

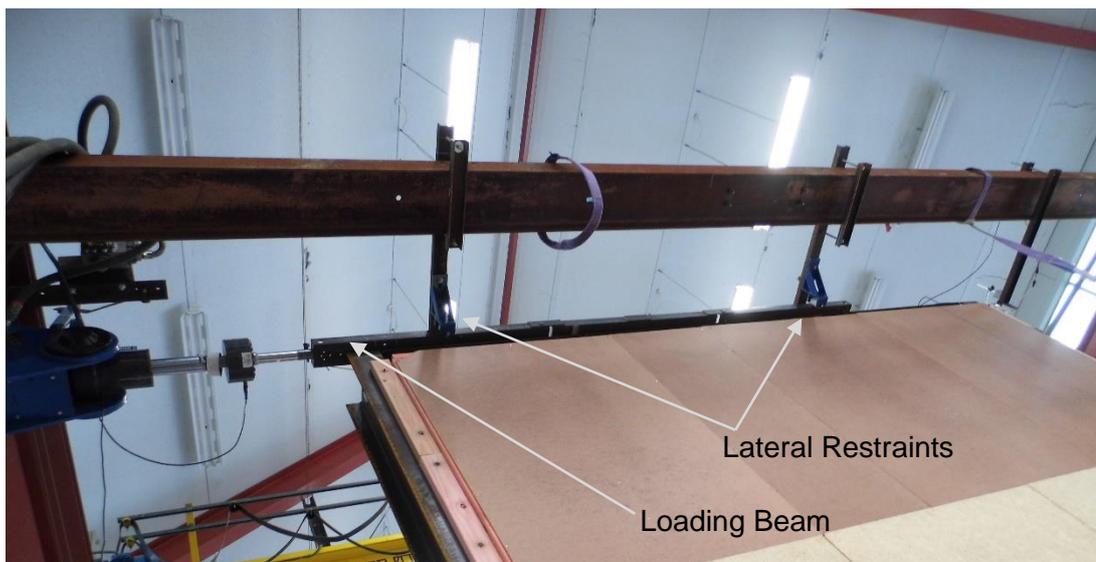
## 4. WALL TESTING METHODS

All wall test configurations were tested using a fully reversed cyclic test protocol derived from the P21 Test Method (2010) typically used to determine bracing ratings for wall constructions for use when designing buildings according to NZS 3604 (2011).

The bottom plate of the test specimen was secured to the strong floor using 16 mm diameter bolts centred on the bottom plate at approximately 900 mm centres. These bolts were installed in holes drilled through the bottom plates and included 50 mm x 50 mm x 3 mm square washers between the plate and nuts. The bolts were secured to the strong floor by threading into specifically designed steel brackets and were tightened to provide a secure connection between the bottom plates and the floor.

Horizontal load was applied to the upper steel beam of the specimen using a 250 kN closed loop electro-hydraulic actuator and loads were measured using a 225 kN load cell. A steel channel was attached directly to the actuator and bolted to the top flange of the upper beam using 16 mm bolts on 450 mm centres. Out-of-plane

movement of test configurations was prevented by roller restraints located at two points along the top edge of the specimens as seen in Figure 7. A rotary potentiometer was used to measure the global horizontal displacement of the top of the specimen. Additional potentiometers were used to measure the horizontal movement of the column furthest from the actuator at mid-height during testing. The test load and displacement measurements were recorded using a computer controlled data acquisition system. The load cell was calibrated to International Standard EN ISO 7500-1 (2004) Grade 1 accuracy and the linear potentiometers were calibrated to an accuracy of 0.2 mm. The rotary potentiometer had a much longer range and was calibrated to an accuracy of 1.0 mm.



**Figure 7. Top of Wall Test Specimen Showing Loading Beam and Lateral Restraints**

The displacement pattern for wall testing used was similar to that required for P21 testing except that it was scaled up from a 2.4 m high wall to a 5.5 m high wall. The scaling factor on the target P21 procedure displacements was therefore  $5.5/2.4 = 2.3$ . Displacements were controlled using a linear variable differential transformer (LVDT) integral to the hydraulic actuator. The loading sequence consisted of three displacement controlled cycles of the specimen top plate to displacements of  $\pm 21$ ,  $\pm 34$ ,  $\pm 50$ ,  $\pm 67$ ,  $\pm 83$  and  $\pm 99$  mm. This displacement pattern was used for Tests A1 and B, but the remaining tests also included a final series of three cycles to  $\pm 130$  mm. Displacements were applied sinusoidally at an average rate of 1 mm per second.

## 5. WALL TEST RESULTS AND OBSERVATIONS

Load and LVDT displacement data obtained during testing provided ample data for the development of hysteresis loops and backbone curves for each test configuration. There was negligible difference between the LVDT displacement and the rotary potentiometer. Observations made during testing also provided qualitative information on the performance of these configurations during lateral loading. A composite series of 1<sup>st</sup> cycle backbone curves were created based on the averages of the positive and negative displacement cycles and these are shown in

Figure 8 for Tests A1 and Test B to Test G. Figure 9 presents the 1<sup>st</sup> cycle backbone curves for Tests A1 to A3.

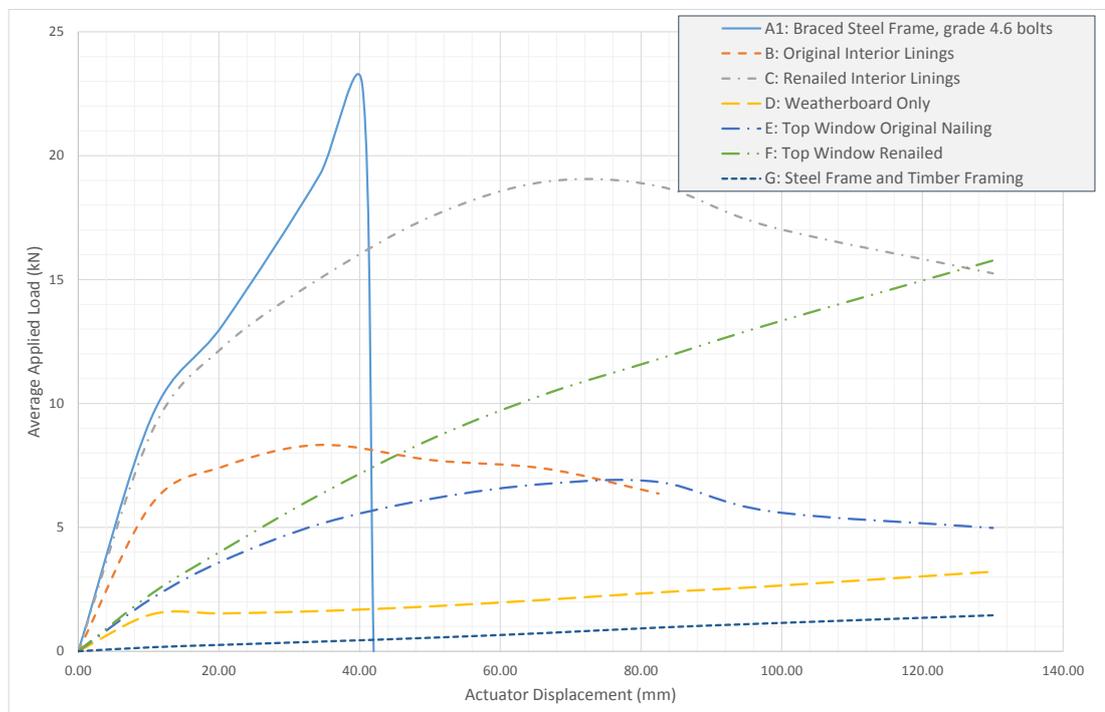
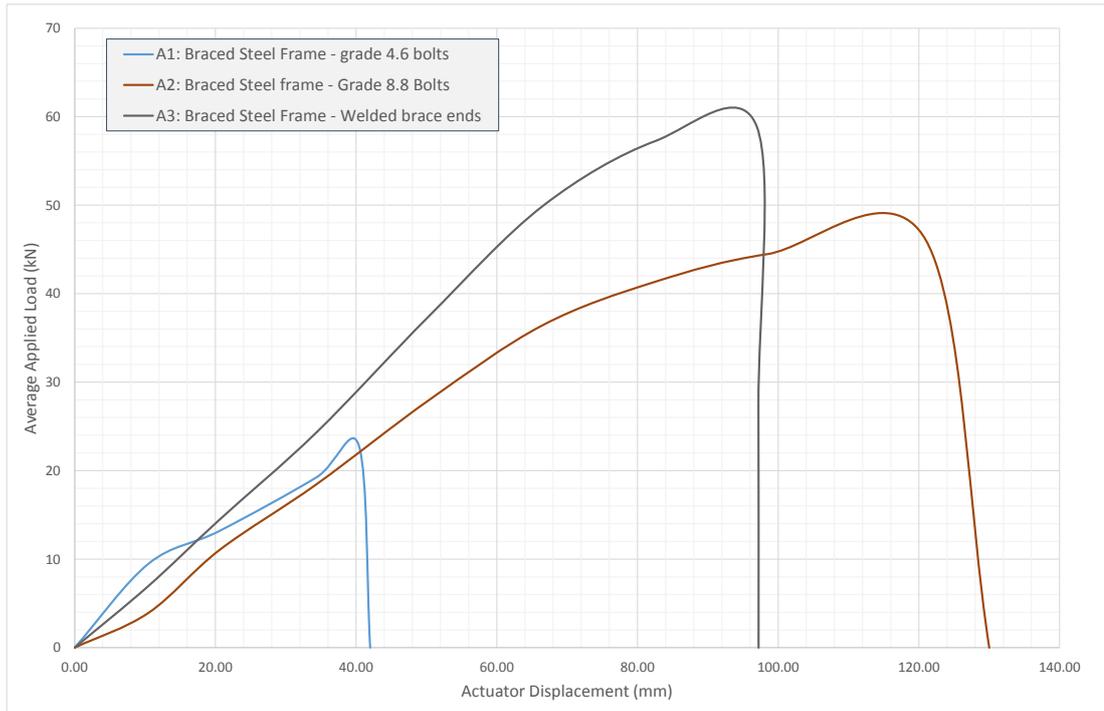
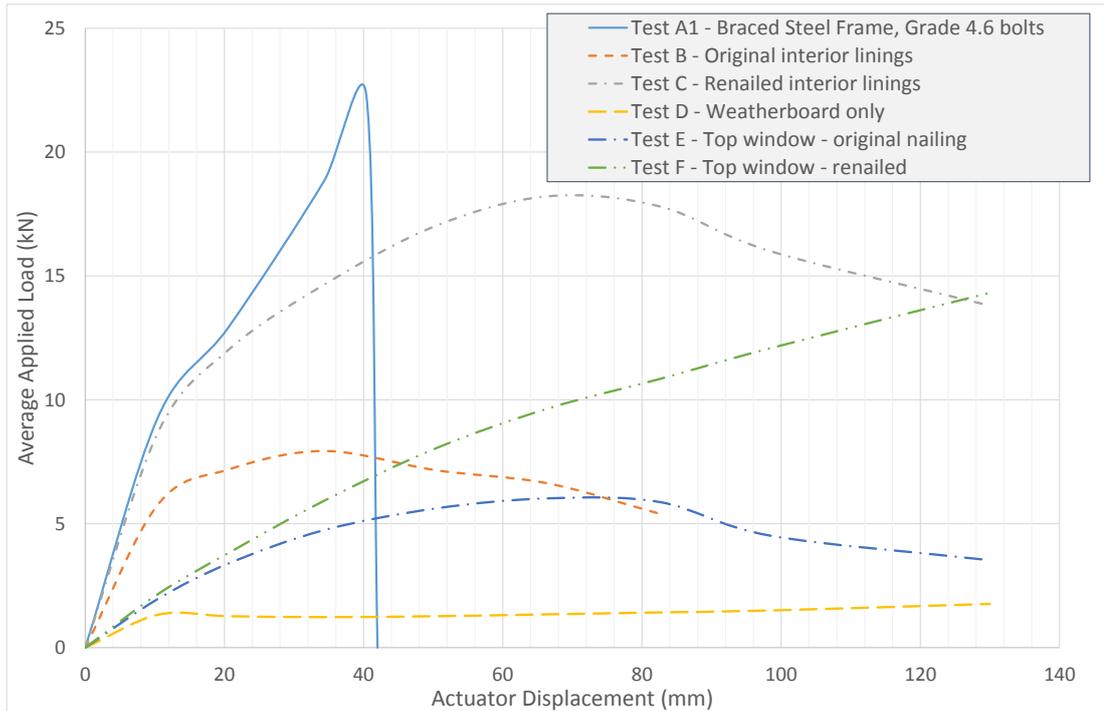


Figure 8. Average Backbone Curves for Tests A1, and Tests B to Test G

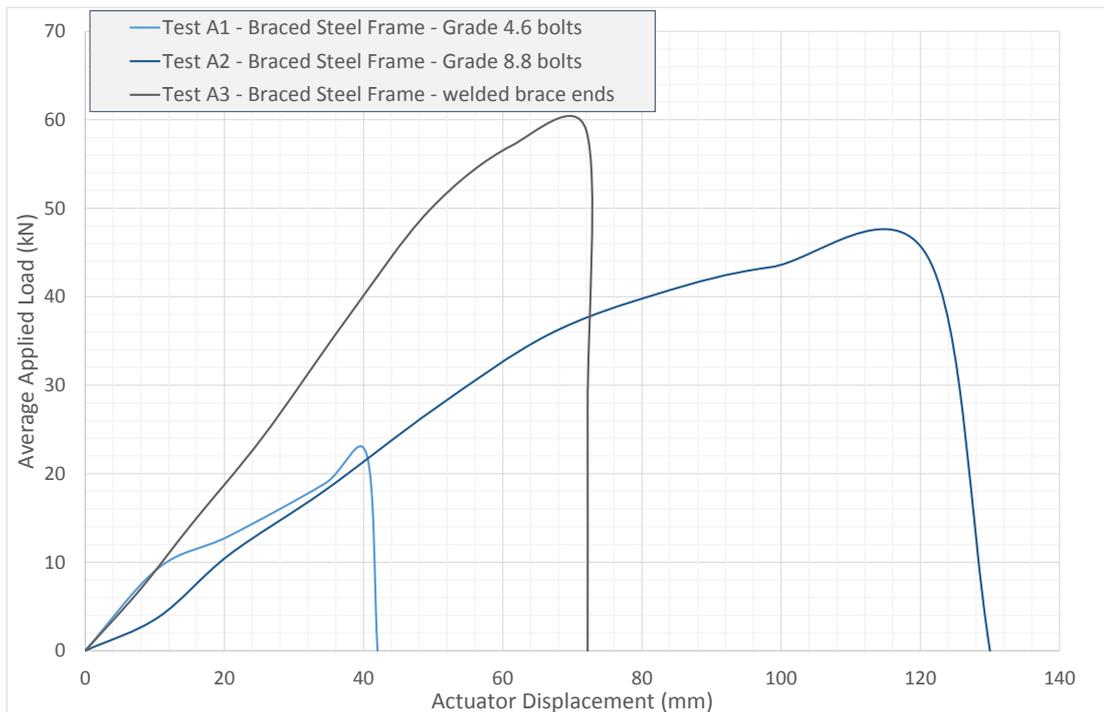


**Figure 9. Average Backbone Curves for Tests A1, A2 and A3**

An adjusted series of averaged 1<sup>st</sup> cycle backbone curves where the contributions of the bare steel frame and timber frame from Test G have been subtracted from the other curves are shown in Figure 10 for Test A1 and Test B to Test F. An adjusted series of averaged 1<sup>st</sup> cycle backbone curves where the contributions of the bare steel frame and timber frame from Test G have been subtracted from the other curves are shown in Figure 11 for tests A1, A2 and A3. An adjustment was also made to the plot for Test A3 to account for flexural uplift of the baseplates. Individual hysteresis plots are presented for all test configurations in the following sections.



**Figure 10. Adjusted Average Backbone Curves with Steel Frame and Timber Frame Contributions Subtracted (Tests A1, B to F)**



**Figure 11. Adjusted Average Backbone Curves with Steel Frame Contributions Subtracted (Tests A1, A2 and A3)**

### 5.1 Test A1: Braced Steel Frame – Grade 4.6 bolts

The full hysteresis for Test A1 is shown in Figure 12. The significant drop in load in each direction was due to shearing of the 12 mm bolts used to connect the braces to the lugs welded to the column webs in the upper bay of the specimen. Testing was stopped soon after the second bolt failure. In this test there was significant “slop” in the system because the diagonal steel straps could not be tensioned and there was a tolerance of 1 mm between the bolts and the hole at the bolted joints.

### 5.2 Test A2: Braced Steel frame – Grade 8.8 bolts

The full hysteresis for Test A2 is shown in Figure 13. As with Test A1, the drop in load in each direction was due to shearing of the 12 mm bolts used to connect the braces to the lugs welded to the column webs in the upper bay of the specimen. Testing was stopped soon after the second bolt failure. In this test there was even more “slop” in the system because the diagonal steel straps could not be tensioned and there was a tolerance of 1 mm between the bolts and the hole at the bolted joints. It was also noted that Test A1 resulted in a small amount of bearing deformation around the holes in the steel straps, resulting in lower frame stiffness in the early stages of the test.

### 5.3 Test A3: Braced Steel Frame – Welded brace ends

The full hysteresis for Test A3 is shown in Figure 14. The drop in load was the result of a failure of one of the columns where it was attached to the baseplate. The test was stopped immediately after this failure as the test specimen had the potential of being unstable following this failure, which was considered a fabrication error in welding the column to the baseplate. Enough data had been recorded up to the point of failure that it was considered a valid test for determining the specimen response over the displacement range that was under consideration for this project.



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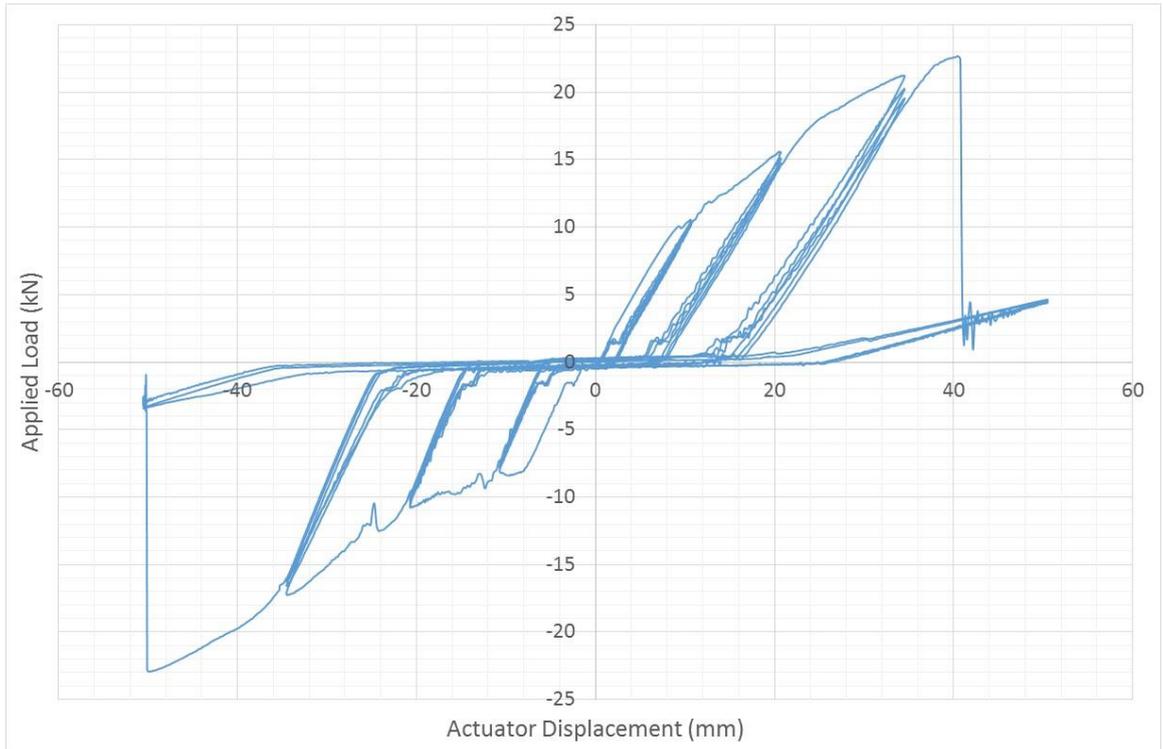
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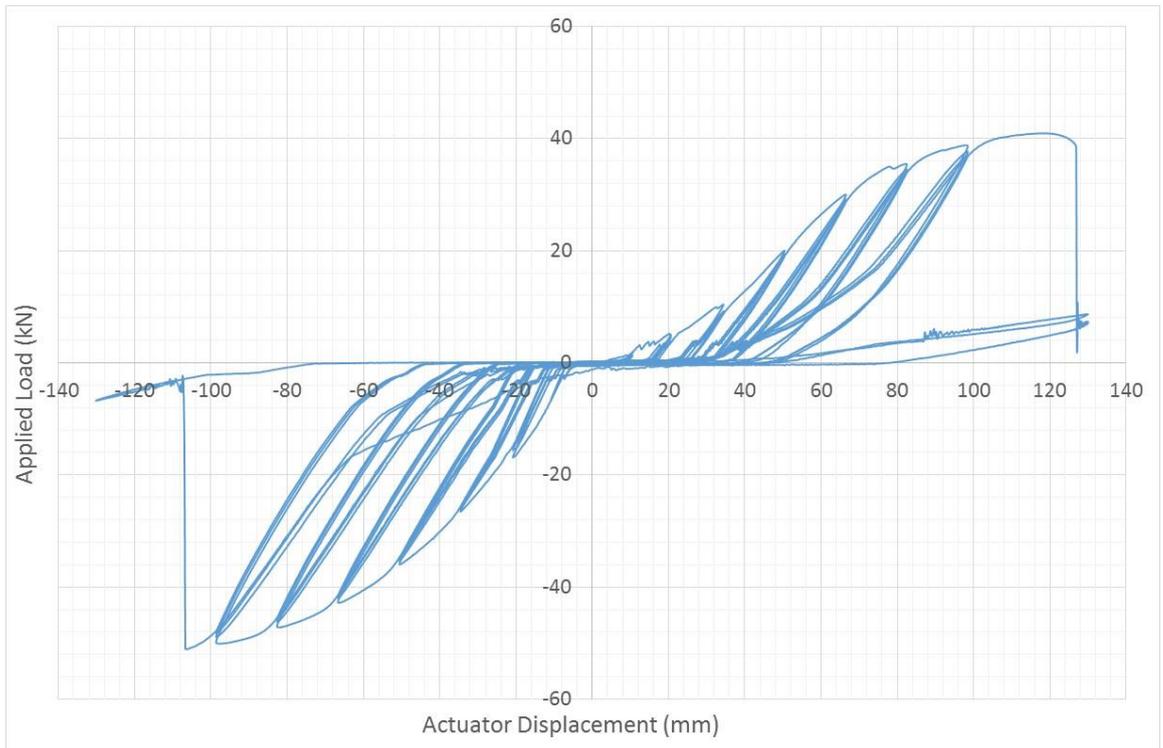
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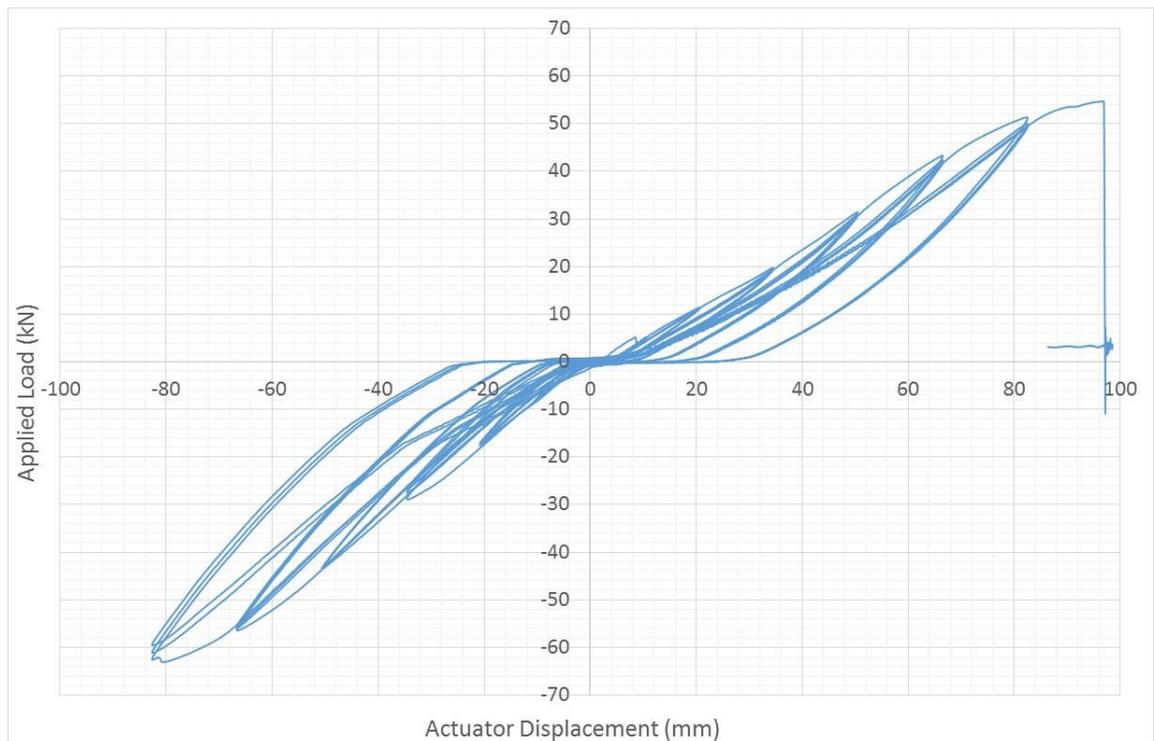
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**Figure 12. Full Hysteresis for Test A1**



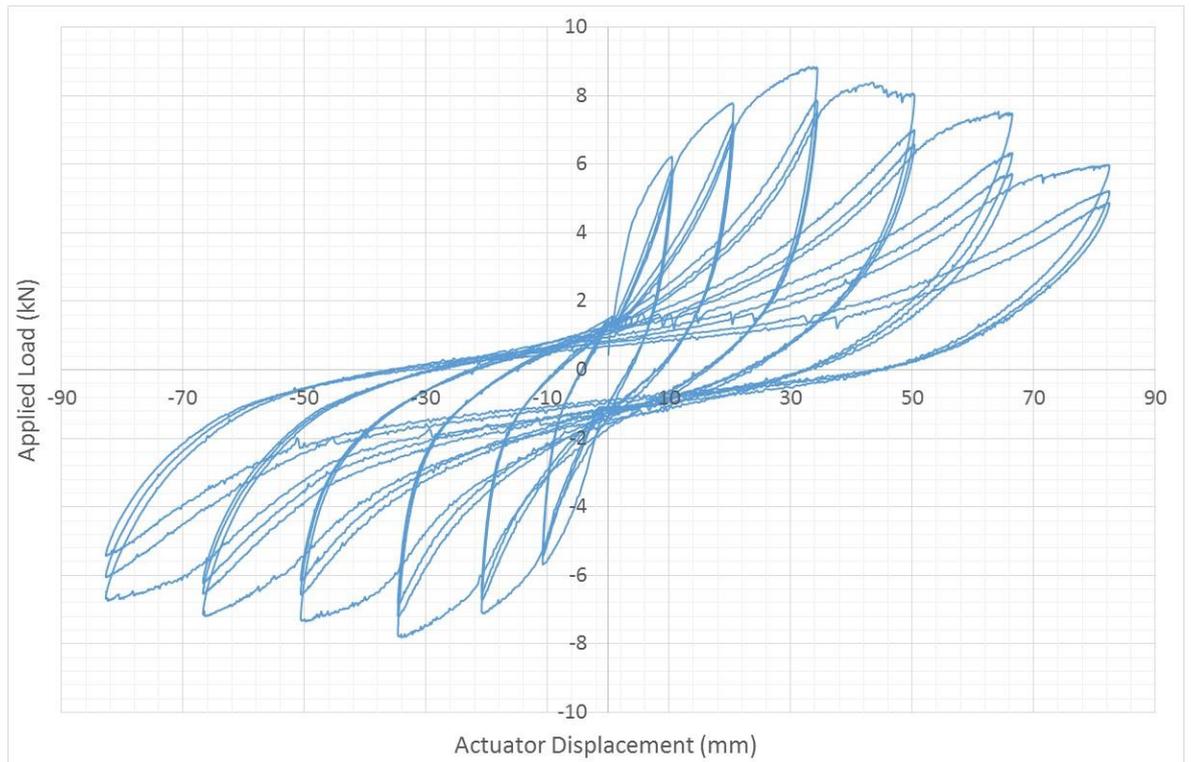
**Figure 13. Full hysteresis for Test A2**



**Figure 14. Full hysteresis for Test A3**

#### **5.4 Test B: Interior Linings with Original Nailing**

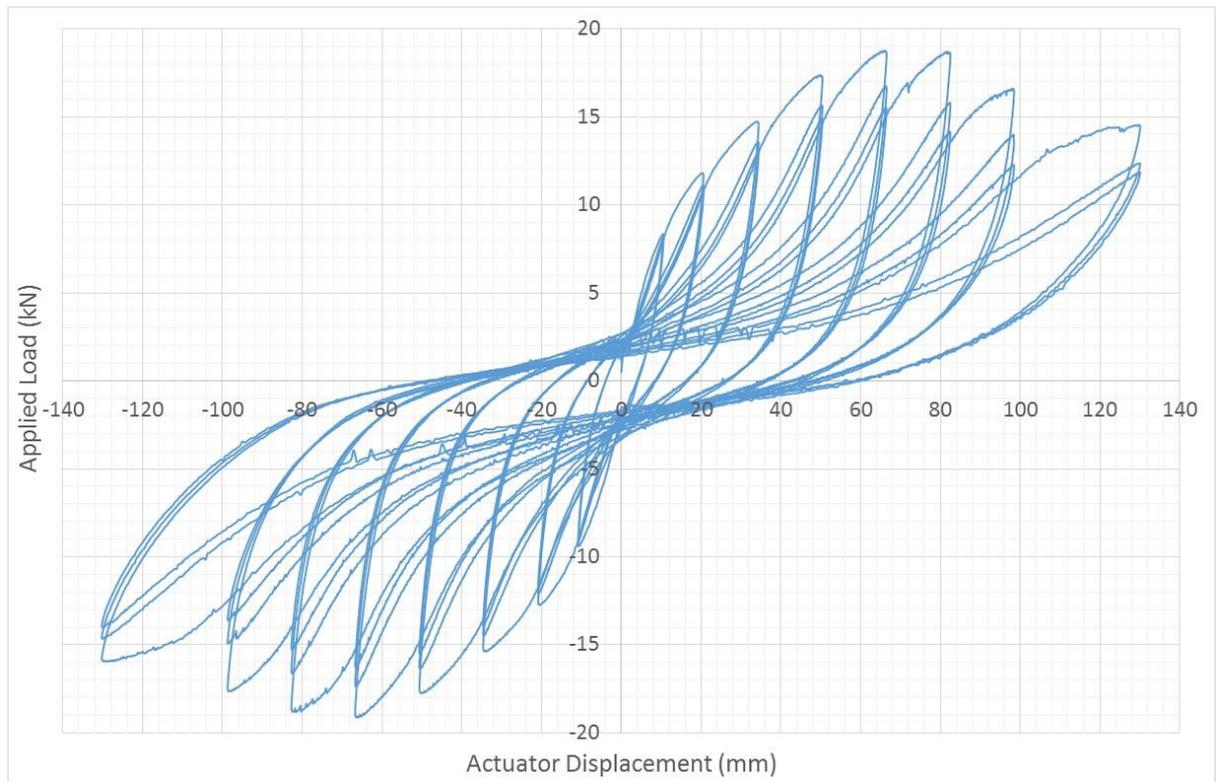
The full hysteresis for Test B is shown in Figure 15. Testing was stopped for this configuration prior to the end of the full set of cycles because it became obvious that the peak load had been achieved and would not be regained. The drop in load resistance was due to the nails holding the particleboard and hardboard linings working out and allowing the panel materials to deform so that they no longer bore against each other along the edges. This was more obvious with the hardboard panels and towards the end of the test one of the hardboard panels became fully dislodged from the framing.



**Figure 15. Full Hysteresis for Test B**

## 5.5 Test C: Interior Linings with New Nailing

The full hysteresis for Test C is shown in Figure 16. The re-nailed interior linings performed much better than the original fastenings and much higher loads were reached before the load resistance began to decline. The nails in the hardboard gouged out the hardboard around the fasteners which allowed for some deformation of the panels at greater displacements. The nails in the particleboard held much better than the original nailing and while some nails fractured at the interface between the particleboard and the timber framing, the remaining nails held the panels in place throughout testing.



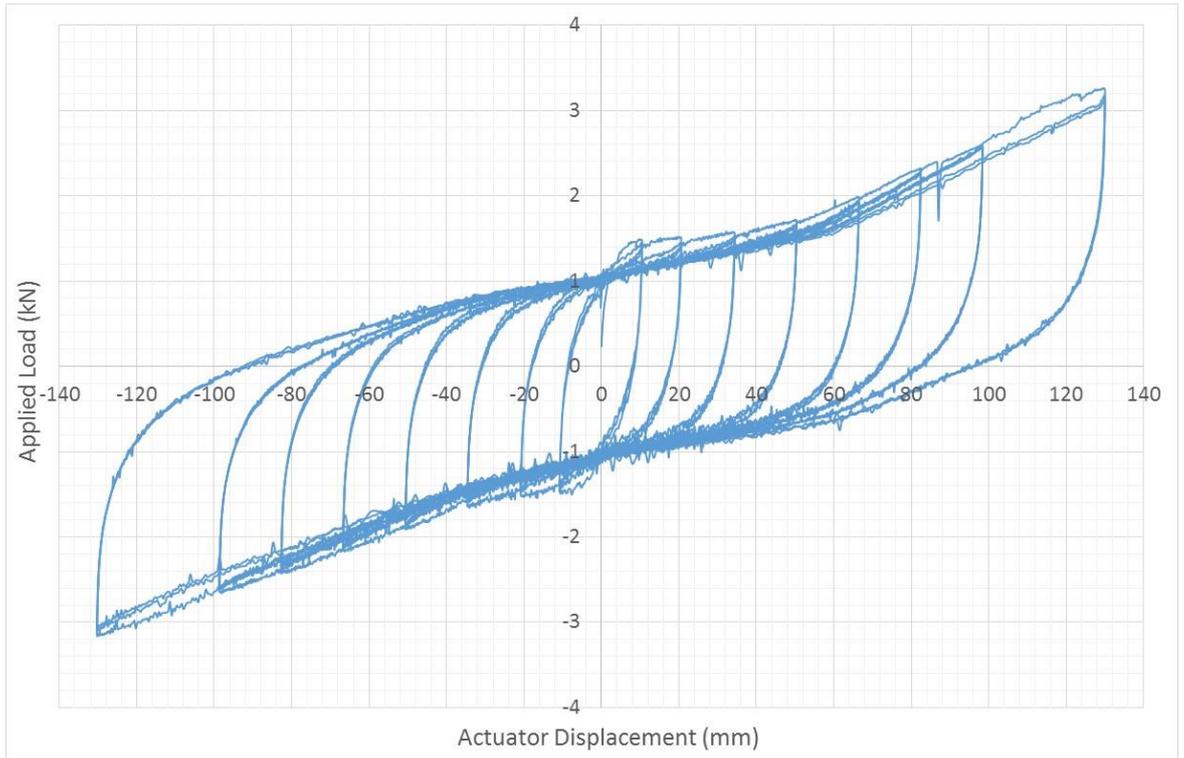
**Figure 16. Full Hysteresis for Test C**

### 5.6 Test D: Weatherboard Exterior Cladding Only

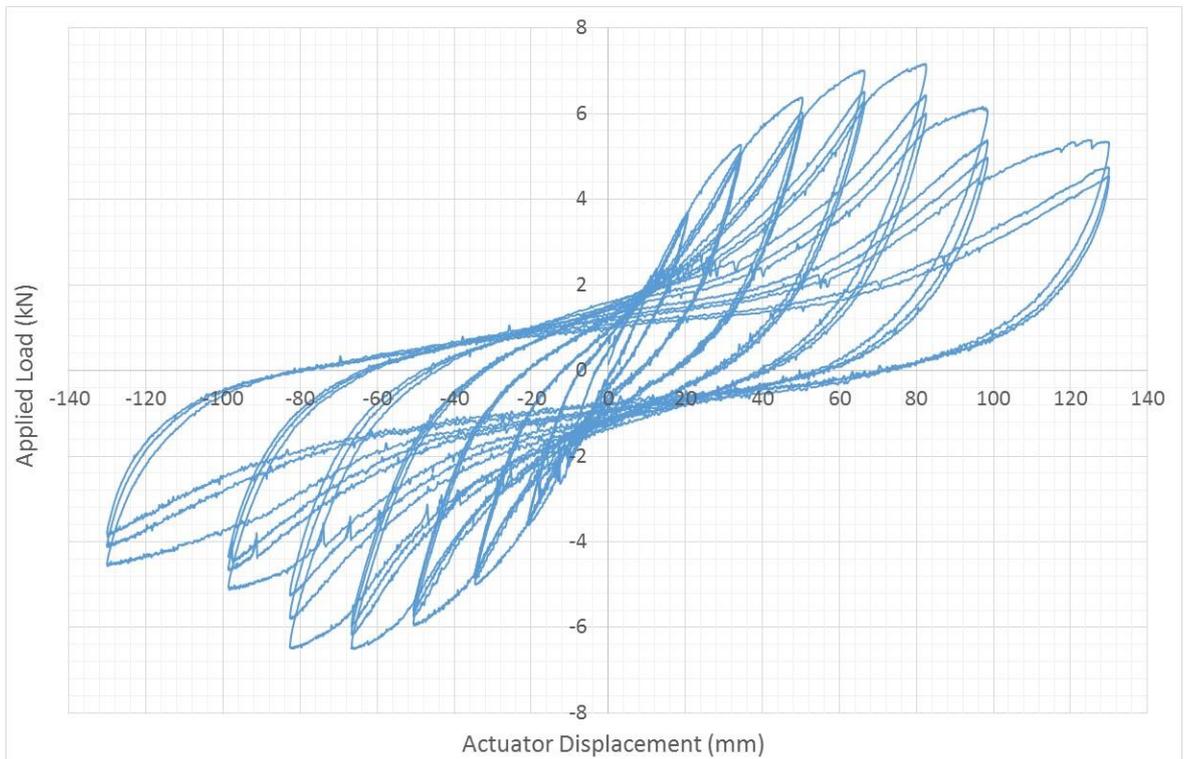
The full hysteresis for Test D is shown in Figure 17. This configuration was dominated by friction of the weatherboards as they slid past each other during lateral loading. No damage was observed to the weatherboards or the connections of the weatherboards to the timber framing.

### 5.7 Test E: Weatherboard Cladding and Interior Linings with Original Nailing and Window Opening

The full hysteresis for Test E is shown in Figure 18. The behaviour of this configuration was similar to Test B except that the additional flexibility of the steel frame above the top of the particleboard lining resulted in lower loads being resisted.



**Figure 17. Full Hysteresis for Test D**



**Figure 18. Full Hysteresis for Test E**

The particleboard panels for Test E were observed to behave similarly to those in Test B in that the panels rotated individually and worked themselves off the timber framing either by pulling out the nails or the nails pulling through the particleboard.

## 5.8 Test F: Weatherboard Cladding and Interior Linings with New Nailing and Window Opening

The full hysteresis for Test F is shown in Figure 19, which shows that even at  $\pm 130$  mm of displacement the specimen is still resisting load and has not had any failure that caused a significant reduction in resistance. The new nails provided a significant increase in load carrying capacity over Test E and the damage to the particleboard was similar to Test C, although with fewer nails broken during the testing.

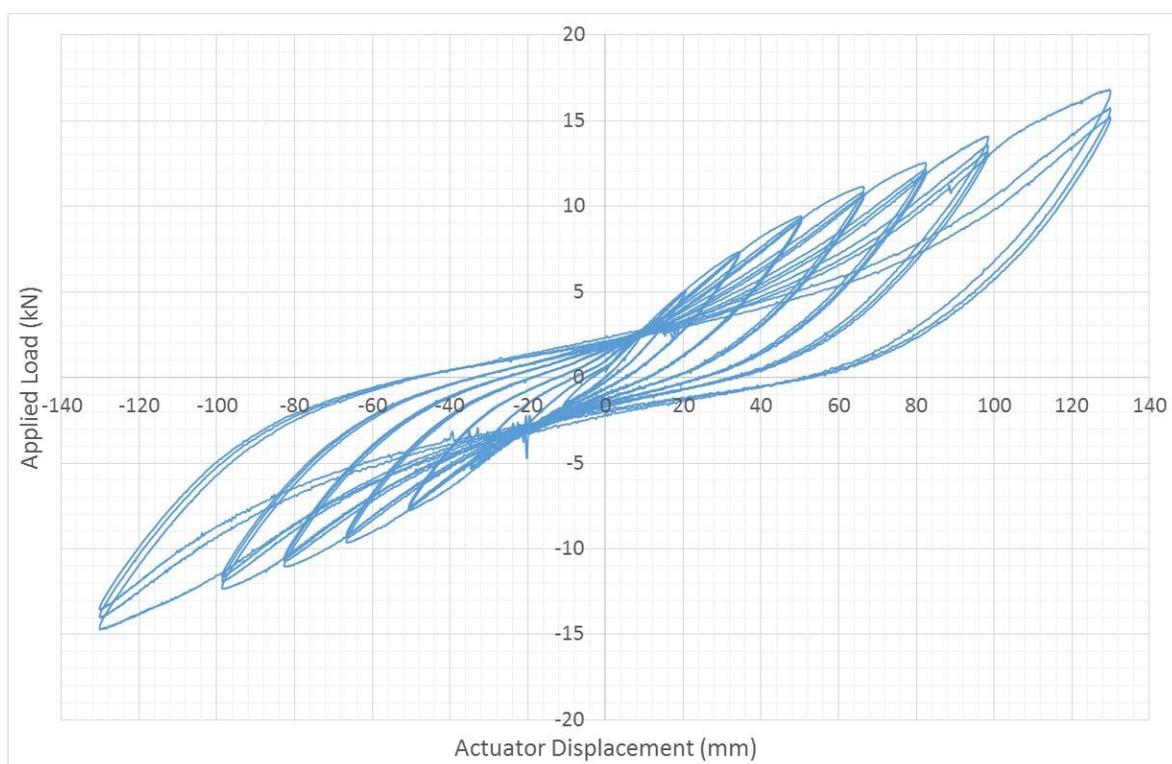
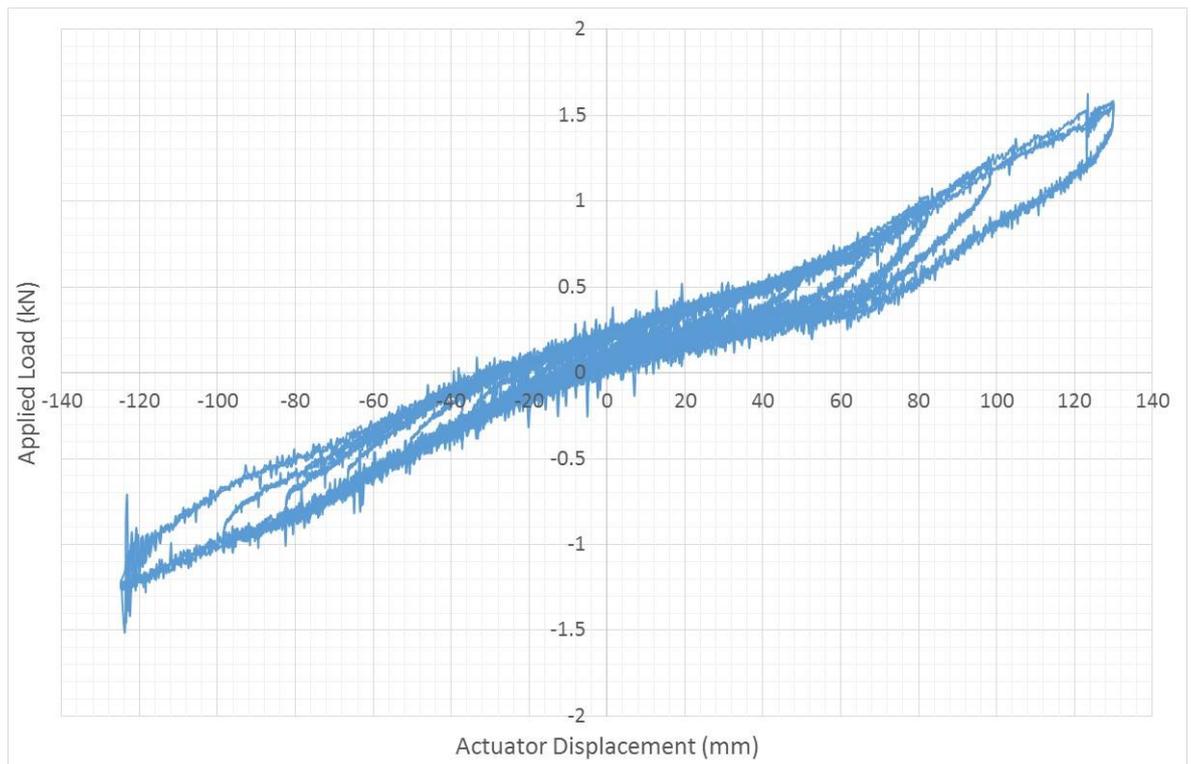


Figure 19. Full Hysteresis for Test F

## 5.9 Test G: Steel Frame with Braces and Timber Frame

The full hysteresis for Test G is shown in Figure 20. No damage was observed during this test to either the steel frame or the timber frame, even at the largest displacement levels. This test confirmed that the contribution from the steel and timber frames by themselves was minimal compared to the contributions of other components of the tested configurations.



**Figure 20. Fully Hysteresis for Test G**

## 6. SUMMARY

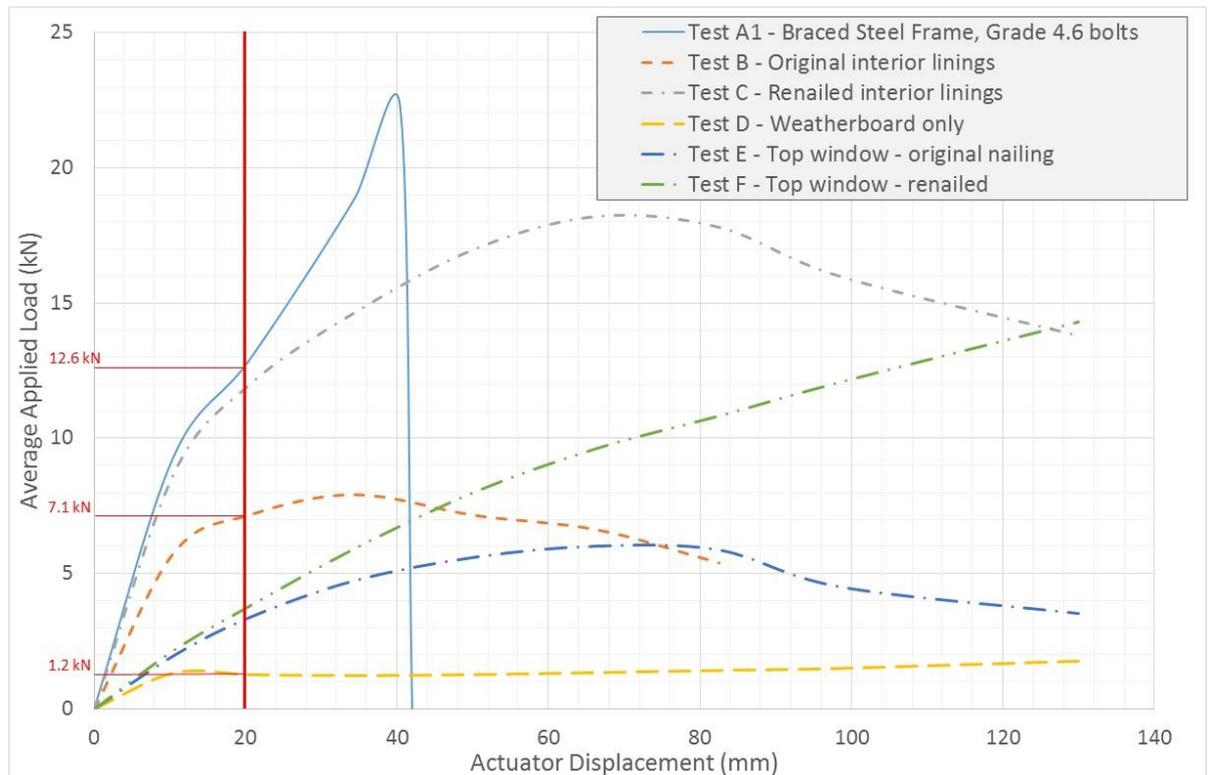
In an effort to assist the Ministry of Education and the Ministry of Business, Innovation and Employment with seismic assessments of an extensive stock of school gymnasium buildings throughout the country, BRANZ conducted full-scale wall tests on a replicate of a typical school gymnasium wall panel between portal frames. Testing was conducted on various configurations of the wall in order to further understand the lateral load resisting behaviour and contributions to lateral load resistance by the various components and systems comprising these walls. Configurations were also tested that utilised additional fasteners that could potentially be used to strengthen existing walls without the necessity for a complete rebuild of these buildings.

### 6.1 Suggested Use of Data

This series of tests has provided load-deflection behaviour for a range of bracing systems that would be expected to be encountered on school gymnasium side walls. It is suggested that engineers assessing the capacity of a school gymnasium side wall can aggregate these responses as required at any particular displacement. As an example, consider the south wall of the gymnasium at Naenae College. This wall consists of seven bays, two of which are cross braced with steel braces with single grade 4.6 M12 end connections. At 20 mm displacement, the resistance expected to be offered by the whole wall would be made up of seven times the resistance achieved for the weatherboards (Test D) at 20 mm (1.2 kN per bay), plus two times the steel bracing resistance (Test A1) at 20 mm (12.6 kN per bay), plus

seven times the resistance from the interior lining result (Test B) at 20 mm (7.1 kN per bay). The values taken from the backbone plots can be seen graphically in Figure 21. The south wall of the gymnasium can therefore be expected to resist approximately 83.3 kN at 20 mm of eave height displacement.

A similar calculation could be done for other displacements in order to determine the full load-displacement backbone curve for the combination of the systems. The same process could also be employed to determine the overall load-displacement backbone curve for the case with strengthened lining connections.



**Figure 21. Load Levels at 20 mm Displacement for Test A1, Test B and Test D**

Individual backbone plots from the tests will be made available to the MBIE/NZSEE Red Book review task group for possible inclusion in the revised document.

## 7. REFERENCES

1. Shelton, R. 2010. Technical Paper P21 (2010) A Wall Bracing Test and Evaluation Procedure. BRANZ Ltd, Judgeford, New Zealand.
2. Standards New Zealand. 2011. NZS 3604:2011. *Timber Framed Buildings*. SNZ, Wellington, New Zealand.
3. International Organisation for Standardisation (ISO). 2004. *ISO 7500:2004 Metallic Materials – Verification of Static Uniaxial Testing Machines, Part 1: Tension/Compression Testing Machines – Verification and Calibration of the Force-Measuring System*. ISO, Geneva, Switzerland.



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