LATERAL LOAD TESTS ON EARTHQUAKE DAMAGED HOUSES IN CHRISTCHURCH

Hugh Morris[1], Joshua Briscoe[2], Logan Holt[2], David Carradine[3], David Yeoh[3]

ABSTRACT: *Thousands of houses in Christchurch were damaged in the Christchurch earthquakes of 2010 and 2011. Uni-directional lateral load tests were undertaken on two houses in the residential red zone in Christchurch in June 2012. The aim of the tests was to measure the lateral load stiffness of typical moderately damaged houses to identify the change in stiffness due to the seismic load history. The wall bracing systems of the houses are assumed to satisfy the regulations in force at the time, which are outlined. Several relevant tests from the literature are briefly reviewed.*

A diagonal tension load system was designed that provided a near horizontal load of 130kN at the ceiling level. A number of electronic measurements were recorded with dial gauges and simple measurements to provide backup and to check uplift or slip from the foundation. The minimally damaged house tested on Wairoa St included fibre cement weatherboard, internal gypsum plasterboard, with a heavy tile gable roof. It was built in 1983 and had a timber pile foundation. The narrow weatherboard house in Bexley Rd was built in 1947 with a heavy tile hip roof with a pile and ring beam foundation. This house had pre-existing damage due to differential settlement, movement relative to the foundation and damage at the internal fibrous plaster wall to ceiling interface. Under the same maximum 130kN test loading, maximum deflection of the 1983 house was approximately half that of the 1947 house.

Both houses exhibited considerably more stiffness than anticipated, with the 1980's gypsum and diagonal steel brace system approximately twice as stiff as the 1940's. The expected seismic softening was not detected so the earthquake softened deformations are likely to be less than 1mm and of modest serviceability concern. The damage indicated the applied load was in excess of the earthquake loads but the pattern of damage was typical of earthquake damage observed in many houses. The residual stiffness of timber framed houses, even after significant shaking, is considerable and a future test load system needs to be stiffer with a much higher load capacity to determine the residual strength.

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KEYWORDS: Light Timber Framed Houses, Seismic resistance, On-site testing

1 INTRODUCTION

1.1 House damage in Christchurch

Buchanan et al. [1] Surveyed the performance of light timber frame houses which performed very well during the Canterbury earthquake sequence providing life safety even under serious liquefaction and severe horizontal and vertical shaking. Damage to houses varied from minor cracking to complete collapse.

Hazards due to landslide, rockfall and liquefaction caused many land areas of Christchurch to be "red zoned" with no reconstruction permitted and houses required to be removed or demolished.

Alsamarra'I [2] identified the damage patterns in houses after the 2010-2011 Canterbury earthquake sequence. Six distinct damage types were identified from 2,835 rapid house surveys undertaken immediately after the earthquake. These were: chimney collapse, external cladding collapse, internal wall collapse, detaching of roof tiles, foundations cracking, and partial collapse of building. A correlation was established between horizontal peak ground accelerations and internal and external wall damage, and vertical peak ground accelerations were correlated with heavy tile roof damage.

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1.2 New Zealand Light Timber Framed Housing and Regulations

In 2005 work by Thurston and Beattie [3] included a review of regulations for residential structures. Prior to the limited requirements of the model bylaw NZSS 95 1935, structurally sound construction relied on "experience and trade practice". Improvements in standard construction were made with model bylaw NZS 1900 in 1964 which included lateral bracing requirements.

Major improvements were made with the introduction of NZS 3604:1978 [4] which was built on proven engineering principles to determine earthquake bracing demands based on building weight. The theoretical racking strength of a Light Timber Frame (LTF) building can be determined by summing all the bracing elements [3]. Manufacturers of bracing systems use the BRANZ P21 test to assess the performance of individual bracing elements [5]. The standards provide an expectation of structural detailing that exists within a LTF building based on the year it was built [3]. This is important for non-destructive field testing where the frame and bracing elements are hidden and may not be able to be visually inspected.

1.3 Previous House Testing

The bracing capacity is also significantly influenced by non-structural elements. The capacity may be greater than the sum of the individual elements due to the complex nature of many different interconnected wall, ceiling and floor elements. Liew et al. [6] also considered the bracing capacity of LTF with plaster board walls and found the resistance was very variable depending on boundary conditions. Stand-alone plaster board bracing elements were tested with corner studs representing the intersecting walls found in buildings. This allowed the plasterboard to bear on the corner studs as well as the nail fixings. While crushing occurred at the bearing locations the test wall with corner studs sustained approximately 50% greater maximum load that the test wall without.

In 2003 a full scale test undertaken by Thurston at BRANZ [7] to compare the measured racking strength of an actual house with calculated strength based on design provisions of NZS 3604:1999 [8]. A simple, low cost, single storey house with fibre cement weatherboard cladding and plaster board lining was cyclically tested until failure. Strength of the plaster board bracing in the house was found to be 50% greater than that calculated. The natural frequency of the house was also measured and found to be 20.8Hz [7].

1.4 Opportunity and New Zealand Housing

The expected overall cost to repair earthquake damaged houses in Christchurch is approximated to be ten billion dollars. A better understanding of how this damage affects the structural integrity of the houses is expected to aid the repair strategies and give residents greater confidence in the capacity of their homes to survive future quakes. Due to severe liquefaction in the suburb of Bexley most houses were in the "red zone" and were scheduled for demolition due to ground conditions and not necessarily their structural damage. This provided the unique opportunity to conduct semi-destructive testing on earthquake damaged houses.

2 OBJECTIVES AND HOUSE SELECTION

The initial objectives of this research were:

- To determine the residual stiffness of earthquake damaged, light timber frame (LTF) residential buildings in Christchurch and estimate possible future deformations during a large-scale seismic event.
- To determine if the serviceability performance of typical surviving houses is adequate.

Typical minor damage, such as cracking of plaster board joints around doorways and windows appears to be cosmetic, but residents have stated that there was increased movement within the houses in subsequent earthquakes. This indicated that damage to bracing walls has introduced some 'slack' to the system which needs to be quantified. The tests should provide understanding of the relationship between damage observed and loads applied that will contribute to fragility relationships for the less damaged houses in Christchurch and for the wider New Zealand building stock.

The secondary objectives were:

- To validate that a portable test load system could apply equivalent static loads to that of an earthquake.

This experience would provide data to develop the design of future multidirectional field test loads systems for earthquake damaged houses.

A street survey of several hundred red zone houses was undertaken on foot to determine the general context and construction of houses. Via aerial maps the plan suitability of a subgroup of about 30 were submitted to CERA, of this group 12 had the correct ownership and repair or demolition status and detailed site inspection reduced this to 3 or 4 that were suitable for the proposed loading system.

On-site testing was achieved for two moderately damaged timber frame houses in the suburb of Bexley that had minimal site slope, minimal lateral spreading, near rectangular plan, and weatherboard or low stiffness cladding systems that would not seriously dominate the structural frame and lining system.

Wairoa St. was a LTF house on timber piles with a gable end roof structure, built in the 1980's. Construction would have used plasterboard and steel angle bracing systems. The timber frame was Radiata Pine.

Bexley Rd. was a LTF house on a concrete ring beam foundation and timber piles with hip roof structure built in the 1950s. Construction would have used plaster board and 150 x 25 mm diagonal timber braces. The timber frame was Rimu and was generally sound. Some evidence of borer was encountered in bottom plate, where the steel anchor plate was positioned.

3 TEST SYSTEM

3.1 CONCEPT

The concept was to apply a lateral load equivalent to that applied by an earthquake. The load would be anchored to the foundation and applied at the opposite end at ceiling height of a house with moderate earthquake damage. Approximately rectangular single storey houses with access to all four sides were required. Hydraulic load systems were not available so two 7.5T chain blocks were hired, a guarantee was provided that these would be fully loaded and gave a maximum load capability of 130kN.

3.2 TEST SETUP AND LOAD SEQUENCE

3.2.1 Bexley Road House

No house plans were available on the Christchurch City building files. On-site measurements were done and plans were created. The test rig, shown for the Bexley Rd. house in Figure 1, comprised of a 360UB57 (I-section) load beam that was borrowed and placed against one end of the house at ceiling height with its strong axis parallel to the wall. Adjustable steel Acrow-Prop posts supported the load beam at both ends and mid span. The posts were designed to support the vertical component of the applied load. 7.5 ton capacity chain blocks were attached to both ends of the load beam using a 15mm thick attachment plate that slid over the flange closest to the house, as shown in Figure 2. The chain was anchored to a point at the opposite end of the house with an inline load cell to record the tension in the system.

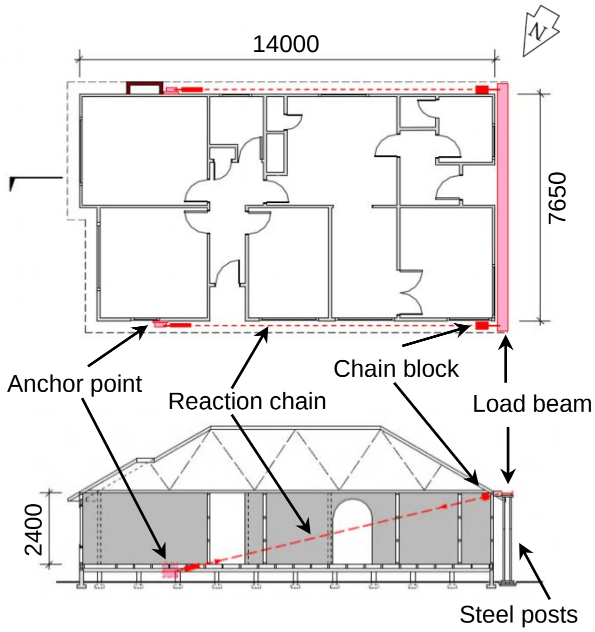

Figure 1: Floor plan and section of Bexley Rd. with test rig in place. All dimensions in mm. By L. Holt.

The test house at Bexley Rd. had a hipped roof structure therefore tiles had to be removed and the load beam was propped off the top plate using 400 mm long timber blocks. Props of 90x45mm timber were located at points coinciding with bracing lines and by using props the roof's timber frame could remain in place during testing without conflicting with the load beam, see Figures 2 and 3. The load beam was placed along the west wall. Due to a step in the east wall the anchor beam from the other test house could not be used. Instead anchor plates, see Figure 4, were fixed at the base of the north and south walls using large timber screws into the bottom plate and studs and dyna-bolted into the concrete ring beam foundation.

Figure 2: Load beam with slide plate with 90x45mm props to transfer load to the top plate, Bexley Rd. Note a string line along the outside edge used to monitor beam bending. Photo: L. Holt

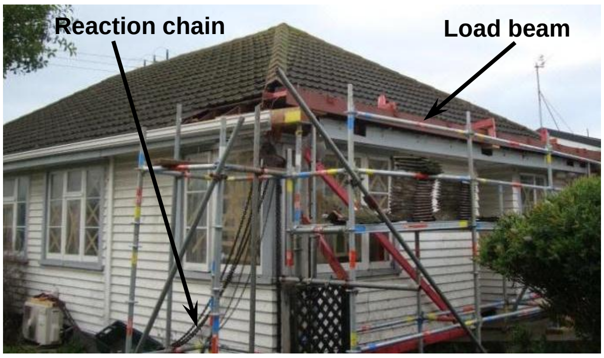

Figure 3: Load beam in place on west wall, reaction chain slack prior to loading, Bexley Rd Photo: L. Holt

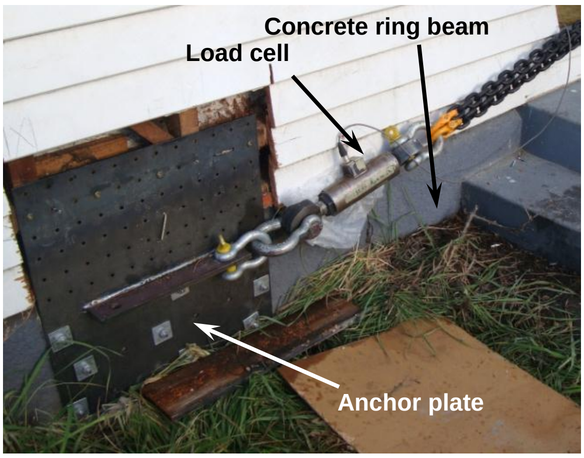

Figure 4: Anchor plate at Bexley Rd screwed to wall and subfloor and dyna-bolted to foundation. Photo: L. Holt

Two people operated the loading chain blocks simultaneously from both sides with real time load monitoring, as shown in Figure 5. Scaffold safety towers were built beneath the ends of the load beam to catch the beam in case of an earthquake or failure of the posts.
A near pinned connection was made at the ground support of the steel posts so the entire horizontal component of the test load was applied to the house.

Figure 5: *Applying load to the house via the chain block, Bexley Rd.* Photo H. Morris

3.2.2 Wairoa Street House

Figure 6: *Wairoa St. House, base anchor beam visible*

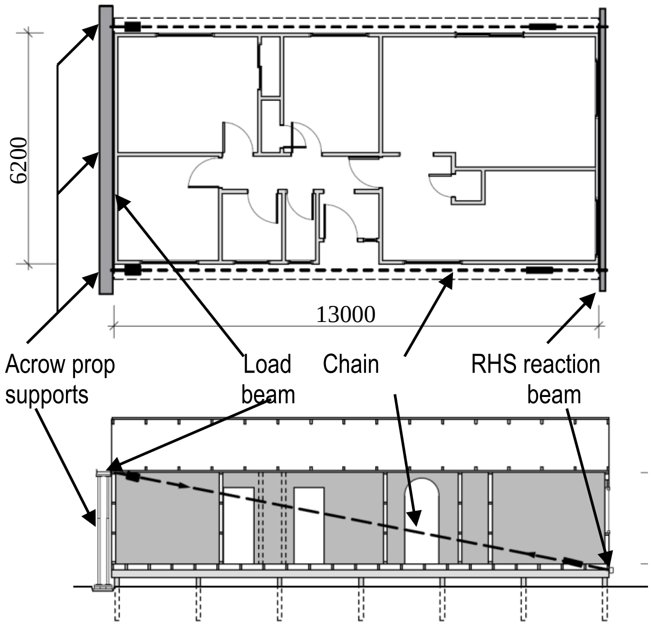

Figure 7: *Floor plan and section view of Wairoa St. house showing load system* By L Holt

The test house at Wairoa St. had a gable end roof (Figure 6). The load system is illustrated in Figure 7. The load beam was able to sit directly against the top plate of the timber frame at the west end, see Figure 8a. Limited exterior cladding was removed to accommodate the load beam. The anchor point shown in Figure 8b consisted of a 150x100x6mm rectangular hollow section (RHS) beam at floor level along the east end of the house.
The RHS was placed with its strong axis parallel to the ground to prevent lateral deflections and 'rolling' from occurring. Hold downs were screwed to the wall studs to prevent the RHS from sliding up the wall. Chains connected to the load cells were looped over the ends of the RHS.

Figure 8 *The 360 UB 57 Load beam positioned at ceiling against the gable end(a left). Load cell and reaction chain looped around RHS at floor level (b right). Wairoa St.*

3.2.3 Measurement system

A University of Canterbury laptop computer and data-logging system was used to collect data from the potentiometer deformation gauges and load cells at one second intervals. Manual dial gauges and rulers were read at each major load step to verify the computer data.

Figure 9: *Potentiometer and manual dial gauge supported by timber props nailed to floor, Wairoa St* Photo: D. Yeoh

Eight potentiometers were used at each site to measure horizontal deflections. Seven were located at ceiling height anchored to the floor inside, see Figure 9, or to the ground outside. A single potentiometer was used to measure slip between the floor and the foundation.

At Wairoa St, five manual gauges were used inside to corroborate the electronic data and four outside to monitor horizontal and vertical movement between the floor structure and ground or foundations. At Bexley Rd only one dial gauge was used inside to corroborate one of the potentiometers. Rulers were used at ceiling and floor height at the northwest corner outside to measure lateral displacement.

3.2.4 Pre-existing damage

Both houses were visually inspected for damage prior to testing. Damage was typical of moderately damaged houses in Christchurch with cracking along joints in plaster board, particularly above openings, shown in Figure 10, and wrinkling of the wall paper layer. Movement between finishing lines and walls was indicated by the reveal of unpainted areas. Minor settlement, less than 20mm, was evident at Wairoa St. in the NE corner. Major settlement, greater than 50mm was seen at Bexley Rd.

At Bexley Rd. the internal chimney had sunk into the ground causing a depression in the kitchen floor. The external chimney was leaning 30mm away from the building and had diagonal shear failure, it had separated from the house with cracks evident at the internal lining interface. The concrete ring foundation was cracked all the way through leaving a 10mm gap.

Figure 10: *Vertical crack above window corner existing prior to testing, Wairoa St.* Photo D. Yeoh

3.2.5 Loading sequence

Loading was applied to both sides evenly, see Figure 5, pausing every 20kN (10kN each side) for approximately 15 minutes to read manual gauges. At 100kN the load was reduced back to 20kN gauges read and then loaded to 120kN within five minutes. This was done to give some cyclic response and also keep tension in the catenary load system. Finally, the max load applied was 130kN which was the capacity of the test rig. The load was then removed in approximately two minutes. The total test period was two hours. The entire loading sequence for Bexley Rd. is shown below in Figure 11 which is similar to the 2 hour sequence for Wairoa St.

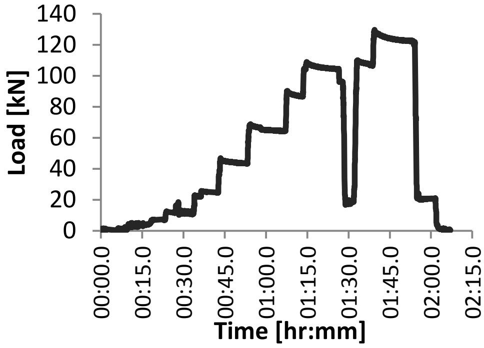

Figure 11: *Load vs Time during testing at Bexley Rd.*

4 ANALYSIS AND RESULTS

4.1.1 Data filtering

Load and deformation data included electrical noise that was manually filtered by removing all rows with load values less that -1 and greater than 150kN, some noise generated extraordinary load steps were also removed.

4.1.2 Deflections

There were variations between the measured maximum deflections at different locations within the houses, as seen in Figure 12, and Figure 14. Differences between the average deflections of the two test houses, is seen in Figure 16. The average deflection values were calculated as the average of the data recorded by the six potentiometers in each house. The locations of these potentiometers at Bexley Rd. are circled in Figure 13 and arrowed for Wairoa St. in figure 15. Deflections were greatest at the loaded end of the house and significantly less at the non-loaded end. Maximum deflections for Wairoa St. and Bexley Rd. were 10.1 and 30.0 mm respectively.

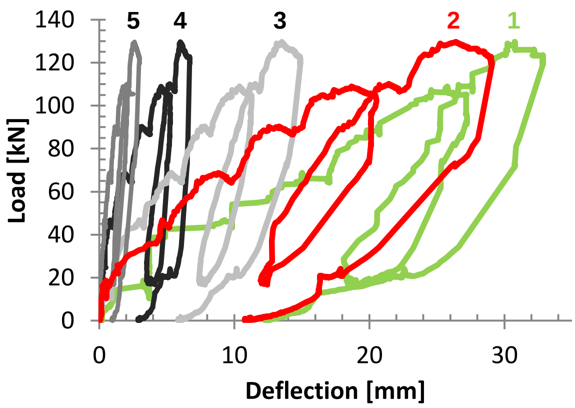

Figure 12: *Load vs deflection plot showing five of the six locations measured during the test at Bexley Rd.*

Significant variation in deflection was observed depending on location within the house. Numbers above the plots in Figures 12 (and 14) refer to the locations of the potentiometers shown in Figure 13 (and 15).

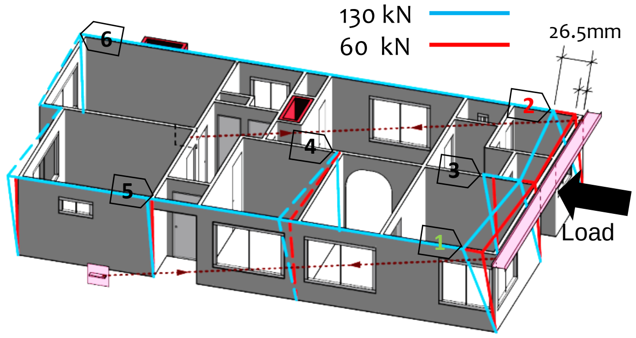

Figure 13: *Isometric of Bexley Rd. The blue line shows the (exaggerated 30:1) deflected shape of the house under maximum test load (130kN). The relative deflections at measured points are circled and all intermediate points are estimated.*

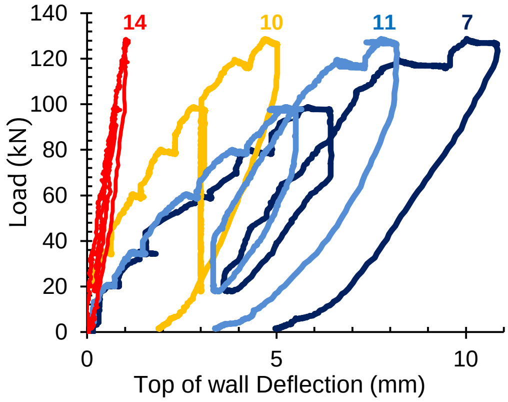

Figure 14*: Deflections at Wairoa St related to the location of potentiometers*

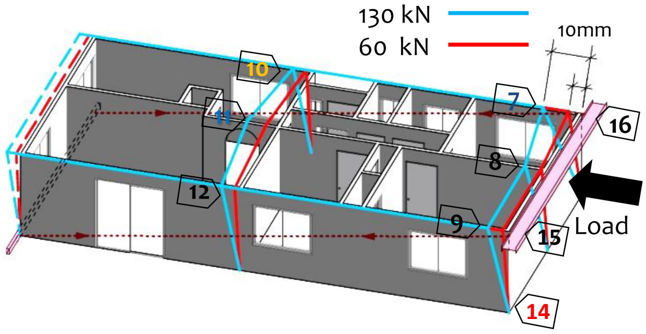

Figure 15: *Deflections at Wairoa St (exaggerated 100:1) and location on potentiometers*

The 360UB load beam available was lower stiffness than ideal and was measured to bend approximately 5mm away from the building at Wairoa St and 10mm away from the building at Bexley Rd. A simplified bending analysis indicates a load distribution in Bexley Rd of around 50% to the central wall area and 25% to each of the side walls.

Average deflection under maximum load and residual deformation were 6.5mm and 2.7mm at Wairoa St and 14.7mm and 6.0mm at Bexley Rd as shown in Figure 16.

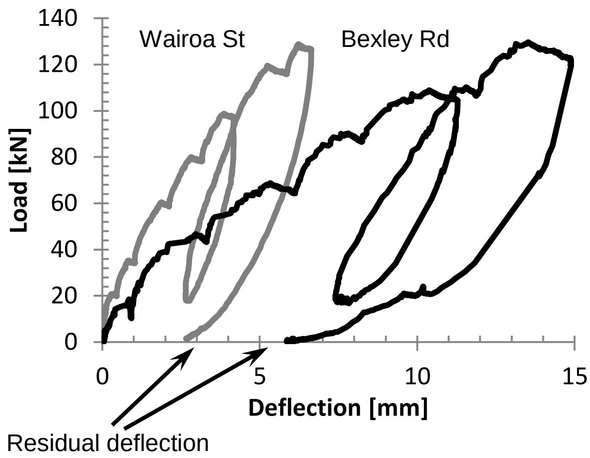

Figure 16: *Load vs average deflection for both test houses showing similar pattern of stiffness.*

Manual deflections were very close to the electronic records. No movement due to testing was recorded between the ground and the floor of either house.

5 DISCUSSION

5.1 DAMAGE OBSERVED

The damage caused during the testing was consistent with that caused by prior earthquakes. Typical evidence of movement similar to that described in the pre-test damage was observed but with increased magnitude. At Wairoa St all windows were open during testing and no breakages occurred. At Bexley Rd small or fixed windows remained closed and one fixed pane cracked diagonally 50mm from the corner of the frame. No other window panes were damaged but the opened windows had distorted and were difficult to close.

Damage caused by testing exacerbated that caused by the earthquakes, as shown in Figure 17. Test loads also created new cracks and additional movement between building components. Movement due to the applied test load was seen throughout both houses but the loaded end of the house exhibited greater local effects.

Figure 17: *Arched opening in central bracing line of Wairoa St while under 100kN test load. This existing damage was enhanced by the application of the test load.*

In addition to the cracking of wall panels, two other failure patterns were considered to be of particular interest. The first pattern was clear evidence of movement between the ceiling diaphragm and the adjacent walls as shown in Figures 18, 19 and 20. This indicated a failure in the connection between the two bracing elements. All deflections were measured off the walls so movement of the ceiling was not recorded. However, the exposed unpainted area, indicated in Figure 20, shows movement of approximately 5mm.

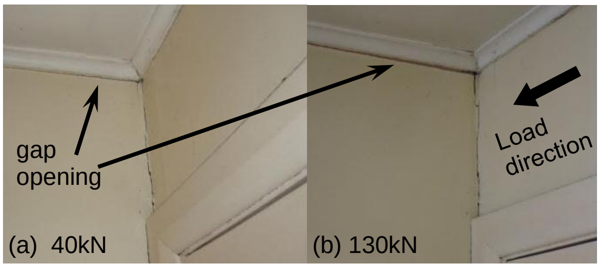

Figure 18: *Increased cracking in corner and opening of a gap between scotia and wall while increasing the applied test load, Bexley Rd.* Photo: L. Holt

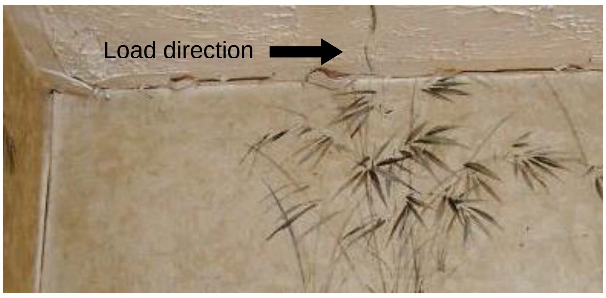

Figure 19: *Cracking showing movement between ceiling and wall at 130kN load, Bexley Rd.* Photo: L. Holt

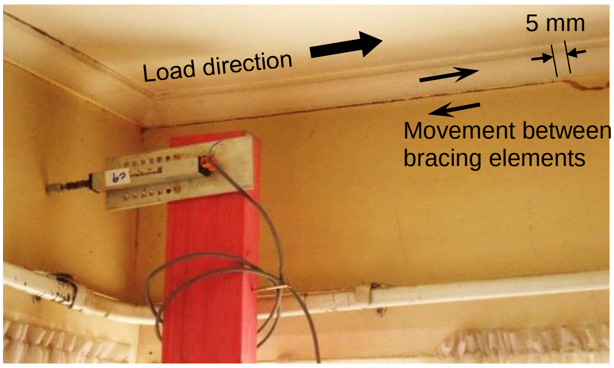

Figure 20: *Movement of ceiling relative to wall during test, 130kN load, Bexley Rd* Photo: L. Holt

The second damage pattern investigated was evidence of shortening of walls parallel to the load direction. This is shown in Figure 21 by the buckling of the wall lining. The evidence of wall shortening in Figure 21 is confirmed by the changes in the recorded deflections along the length of the building shown in Figures 14 and 16 in the Results section. This particular damage pattern was specific to the nature of the test rig. This type of damage was due to the test rig applying a concentrated line load along one end of the building and was not seen in the existing earthquake damage as ground motion distributes loading evenly throughout the structure. The loss in length of the wall was attributed to axial strain, local crushing and closing of gaps along the length of the top plate.

Figure 21: *Buckling of wall lining above window at 130kN load, Bexley Rd.* Photo: L. Holt

5.2 DEFLECTIONS

Many residents commented on increased movement in their houses from minor earthquakes and wind and due to earthquake damage. These claims indicated deflections of 50mm or more. The test result showed significantly less than this in both houses. This indicates that there is still substantial stiffness within both structures.

Because of the numerous reports of flexibility due to the existing damage it was predicted that load deflection plots would show less stiffness under low loads, an increased but uniform stiffness under medium loads and then a reduction in stiffness under maximum load as the bracing elements began to fail. This expectation is conceptually shown by the grey line in Figure 22. This was not observed during testing. All load deformation plots showed a moderately high initial stiffness and an approximately linear relationship.

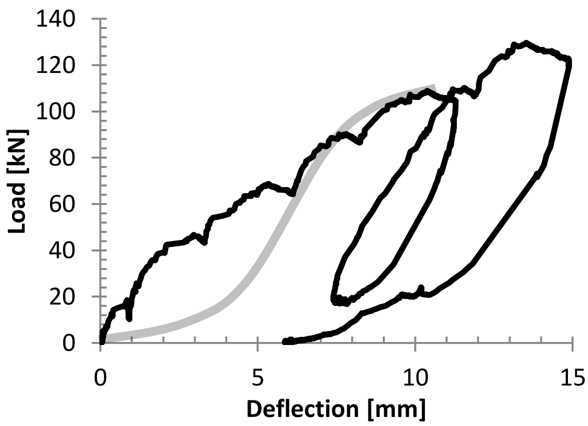

Figure 22: *Stiffness profile comparison. Light grey line represents predicted changes in stiffness compared to the black line representing average deflections at Bexley Rd.*

Loads of less than 10kN are not considered to be precise because of the catenary in the chain. Until the chain was pulled taut the proportion of recorded load that is horizontal onto the house is difficult to assess. Detailed plots of the beginning of the test still showed a linear relationship for loads as low as 10kN and deflections less than one millimetre. While reduced stiffness was not detected due to the precision of our system, any initial 'slack' in the buildings would be at very small amplitudes. A new load system with greater stiffness would be required to determine this. The residual deflections were in the range of 3-6mm but these were only briefly monitored, final residual deformation would have been less.

5.2.1 Short Term Creep

Creep relaxation in the houses and load system was clearly evident in the load deformation plots. Load was increased in approximately 20kN steps over one to two minutes then left for approximately 15 minutes while observations were made. During this time deflections continued to increase and the load reduced as the system relaxed as expected for LTF.

As shown in Figure 16 earlier there was residual deformation once the test load had been removed. However it should be noted that deflection data was only recorded for five minutes after the test load returned to zero. While it is likely that there will be permanent/plastic deformation due to the test load the magnitude may be less than that shown in the results due on-going creep recovery over a greater period of time.

5.3 COMPARISON OF HOUSES AND LOAD LEVELS

The stiffness of both buildings was estimated from the backbone trend lines shown in Figure 23 and was found to be 18kN/mm for Wairoa St. and 9kN/mm for Bexley Rd. Thurston [7] states, during the testing of an undamaged LTF house, a static load of 30kN produced an approximate deflection of 2mm. This suggests a stiffness of around 15kN/mm which is similar to that of the test houses.

The mass of the roof, ceiling structure and upper half of the walls was calculated and assumed to be lumped at ceiling height. The houses were idealized as single degree of freedom, lumped mass models. Fundamental frequency and damped frequency were calculated using a damping ratio of 18% as recommended by Chopra [10], the natural period of each house is determined to be 0.14s for Wairoa St. and 0.23s for Bexley Rd. A summary of these results are shown in Table 1.

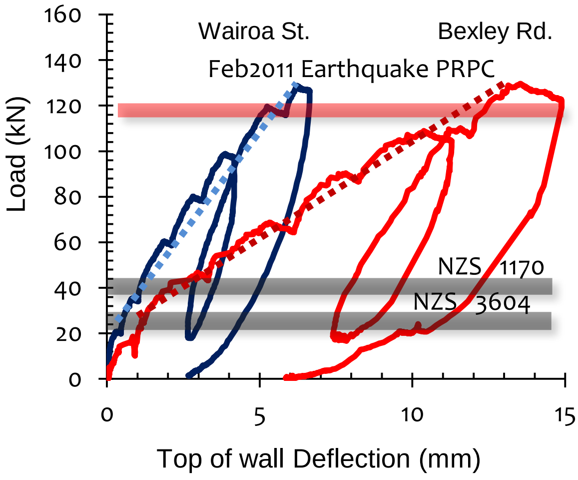

Figure 23: *Load vs average deflection plots for both test houses showing approximate linear stiffness and equivalent load levels from the NZ Standards and the Pages Road Spectra.*

Table 1: *Structural properties of test houses*

Structural Property	Wairoa St.	Bexley Rd.
Mass, m [tonne]	9.2	12.2
Stiffness, k [kN/mm]	18.0	9.0
Natural frequency, ω [rad/s]	44.2	27.2
Period, T [s]	0.14	0.23

These values are significantly different to Thurston's results [7], which stated the BRANZ test house had a natural frequency of 20.8Hz (Period = 0.05s) recorded from a free vibration test. It should be noted that the house tested in Thurston's paper had a light iron roof and was loaded perpendicular to the long dimension of the house where as both test houses in this paper had heavy roofs and were loaded parallel to their long dimension. This being said, this reasoning is insufficient to account for the difference between the periods of the Christchurch test houses and that of Thurston's, considering the stiffness of all three houses is similar. Further investigation is required determine the relationship between stiffness calculated from applied static loads and the natural period found through free vibration.

5.3.1 Design Loads

Using the structural properties of the houses defined in table 1, earthquake design loads were found using New Zealand Standards 3604:2011 [11] and 1170.5:2004 [12]. The amended value of the seismic hazard factor, Z, for Christchurch of 0.3 was used in the calculations and actual earthquakes loads were found using a pseudo spectral acceleration plot. Observed ground motions at the Pages Road Pumping Station strong motion station (PRPC) were used to derive the pseudo-spectral acceleration plot in figure 24 [13]. The PRPC station was initially assumed to be nearby on similar soils but was 680m from the Bexley Rd house and 1070m from Wairoa St house. Using the spectral acceleration plot and the period of each house, shown in table 1, an acceleration coefficient related to the actual base shear of the February quake can be determined. The coefficient multiplied by the estimated mass lumped at ceiling height calculates the actual load applied to the houses during the February quake. These different loads are compared with the test load in table 2. The periods calculated in section 5.3. were used to determine the acceleration experienced by each house during the February earthquake as presented in table 2.

The PPRC record is severe and it is likely that a detailed evaluation of actual site response would be different. It does however indicate a near maximum load scenario.

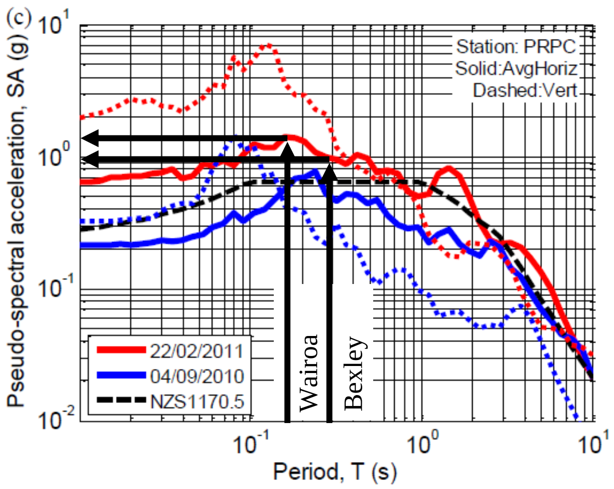

Figure 24: *Pseudo acceleration response spectra from Pages Road Pumping Station (PRPC)* B Bradley [10]

Table 2: *Comparison of Accelerations and Loads*

Source	Accel coeff (g)	Lateral Wairoa (kN)	Load Bexley (kN)
NZS 3604:2011		22.0	-
		-	29.0
NZS 1170.5:2004	0.37	33.3	-
	0.33	-	39.6
Mass, m [tonne]		9.2	12.2
Pseudo-spectral acceleration (T=0.14)	1.30	117.0	-
Pseudo-spectral acceleration (T=0.23)	1.0	-	120.0
Test Load	-	128.50	129.70

It is clear that we loaded near the design spectrum for the two buildings when compared with the earthquake February 22 PRPC earthquake force.

5.4 Recommendations

Recommendations for extending the current study:

- Determine site ground motions with more precision using other stations such as HPSC.

Recommendations for similar testing are:

- Take large numbers of high resolution photos of damage, test equipment and structural detailing to refer to during analysis.
- Install gauges to measure drift and movement between bracing elements including uplift of bracing walls due to overturning.
- Allow for longer term recovery before testing a new load sequence.
- After testing, remove linings to examine and record hidden house structure and damage.
- Test a post 1995 house with modern plaster board bracing systems.

Recommendations for more advanced testing:

- Use a stiffer reaction system with capability to measure initial stiffness with higher precision.
- Load in both directions to determine real system slip and check if stiffness is the same in the opposite direction.
- Apply loadings until there is significant strength drop-off to determine residual strength.
- Undertake snap-back testing to verify the dynamic system response including period and eccentric stiffness torsion effects.
- Support load apparatus off the adjacent ground to account for subfloor stiffness

6 CONCLUSIONS

- The damaged houses exhibited little change in stiffness over the loaded range
- Both the test houses showed significant residual strength and stiffness comparable to that of undamaged houses.
- Lateral deflections during large scale seismic events in the future are likely to be similar to the average deflections observed during testing of 6.0mm for Wairoa St. and 13.3mm for Bexley Rd.
- Damage caused by the test rig was similar to that caused by earthquakes.
- The test load applied was comparable to that of the earthquake loading that the buildings were subjected to during the February 2011 earthquake.
- NZS 3604 EQ Bracing demand was lower than NZS 1170 design loads.
- Further tests should apply significantly larger loads but with reaction stiffness adequate to detect initial softness.
- Lining damage that was caused within serviceability limits (wall height/300) needed repair; this may be a failure of the NZS 1170 serviceability criteria.

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