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STRENGTH ASSESSMENT OF TYPICAL WALL-DIAPHRAGM CONNECTIONS IN NEW ZEALAND UNREINFORCED MASONRY BUILDINGS

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ABSTRACT

Most unreinforced masonry (URM) buildings in New Zealand consist of solid URM bearing walls and flexible timber diaphragms (floor and roof), and insufficient or absent positive anchorage between URM walls and diaphragms has previously been identified to be the most common reason for out-of-plane wall and gable failures in URM buildings during earthquakes. A series of case studies was performed to determine typical details for wall-diaphragm connections, to ensure that realistic specimens were reproduced in laboratory testing to accurately assess their strength. It was found that typical wall-diaphragm connections in New Zealand were through-bolt anchors, where one end of a threaded steel rod was bolted with a steel bearing plate at the exterior face of the URM wall and the other end was welded to a rectangular steel plate that bolted to the timber joist. This type of anchor was believed to be applied as a retrofit technique to most New Zealand URM buildings following the 1931 Hawke's Bay earthquake. In order to assess the strength and possible modes of failure of the wall-diaphragm connections, two types of testing were conducted as follows: (1) pull-out tests on URM wall with typical through-bolt anchor type; and (2) bolted timber connection tests loaded parallel to the timber grain.

KEYWORDS: wall-diaphragm connection, bolted timber connection, pull-out test, steel-wood-steel bolted timber connection test, unreinforced masonry structure

INTRODUCTION

Unreinforced masonry (URM) buildings are typically the structures with the highest risk of failure during an earthquake, and the requirement to seismically upgrade these earthquake-risk buildings (ERB) in New Zealand was mandated by The Building Act 2004 [1]. Importantly, these URM buildings form a significant percentage of New Zealand's building stock and represent the predominant national architectural heritage [2, 3].

Frequently, no positive anchorage is observed between URM walls and timber diaphragms, and consequently no lateral support is provided to the URM walls due to their separation from the timber diaphragms under earthquake loadings. This separation causes the out-of-plane walls to act as cantilevers over the total building height as shown in Figure 1a. Due to excessive flexural stresses at the base of the walls, the risk of complete wall collapse due to bending (out-of-plane failure) increases with their height. Consequently, URM walls without connections to the timber diaphragms are mainly vulnerable in the flexural out-of-plane direction, as the seismic lateral loads experienced by the walls cannot be effectively transferred to the diaphragms and then to the parallel walls acting as shear walls. Thus, the wall-diaphragm anchorages are obviously required because of their capability to transform the out-of-plane behaviour of the URM walls from tall unrestrained cantilevers to shorter one storey high panels, which are seismically excited at each end of floor levels. The behaviour of URM walls with adequate wall-diaphragm connections is illustrated in Figure 1b.

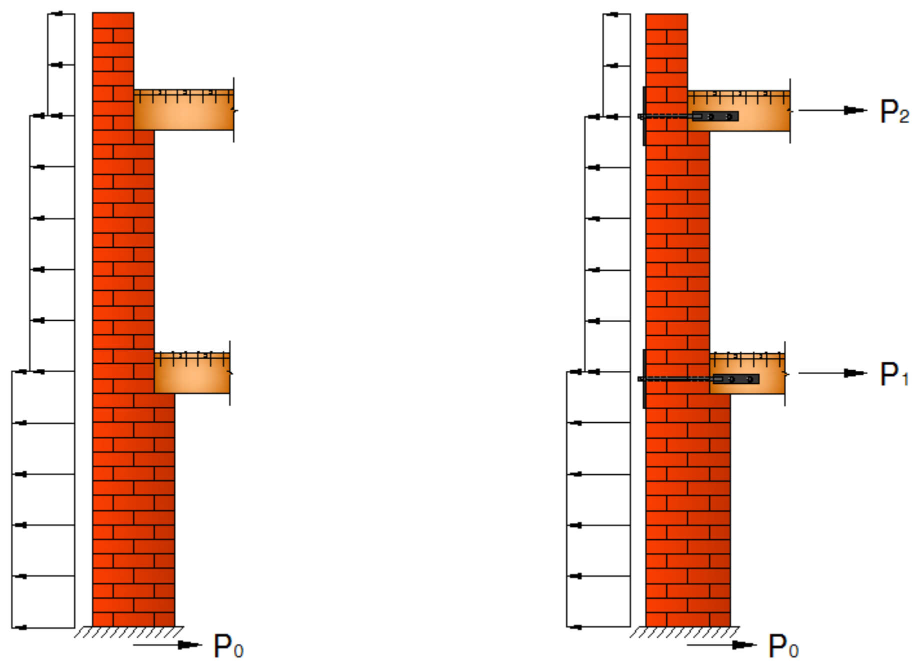

Figure 1: Out-of-plane behaviour of URM walls: a) without wall-diaphragm connections; b) with wall-diaphragm connections

From review of published literature, most URM wall failures in past earthquakes were related to the lack of anchorage between the walls and diaphragms [4-6]. Even if wall-diaphragm anchors are present in URM buildings, they are often not designed to resist seismic loading and anchorage failure is likely to occur. Inadequacy of positive connections in the configuration of URM walls and timber diaphragms was identified to be the most significant problematic detail in New Zealand's URM buildings [7, 8]. From a review of published literature, only Jacks and Beattie [9] have reported laboratory testing of wall-diaphragm anchors to evaluate their performance under dynamic loading. The effects of variations in the size of bearing plates, diameter of anchor rods, thickness and geometry of URM walls, and spacing between anchors on the levels of strength corresponding to the failure modes were not investigated. These are very important parameters to examine as the failure modes in URM buildings vary with the type of anchors used [4, 5]. As connection failure by tearing out part of the diaphragm was observed in past earthquakes [9], the strength of the bolted connection in existing indigenous New Zealand timber joists needs to be assessed.

In an effort to evaluate the strength of wall-diaphragm connections, an experimental study was initiated. The initial objective of the study was to develop a set of design equations to estimate the strength of wall-diaphragm connections corresponding to the possible failure modes. The wall-diaphragm connection strength would then be given by:

$$\text{strength} = \min \begin{cases} \text{bending failure of URM wall} \\ \text{pull-off brickwork failure of URM wall} \\ \text{cone failure of URM wall} \\ \text{steel hardware (rod, plates, weld)} \\ \text{bolted connection failure of timber} \end{cases} \quad (1)$$

However, in this paper, only the bending failure of the URM wall and bolted connection failure of the timber are presented and analysed.

TYPICAL WALL-DIAPHRAGM CONNECTION DETAILS

To ensure that realistic wall-diaphragm connection details are reproduced in laboratory test specimens, an investigation on typical wall-diaphragm connection details was performed by reviewing data from demolished URM buildings in Auckland, participating in meetings with consultant structural engineers in Auckland and Wellington, and by self-observations of URM buildings mainly in the Auckland CBD.

Table 1: Details of wall-diaphragm connections at roof and floor level

Level	Details of connections
Roof	• 16 mm (⅝″) diameter of threaded steel rods • Steel bearing plates at the exterior face of URM walls - 150 mm (6″) × 150 mm (6″) × 6 mm (¼″) thick - 150 mm (6″) diameter with 6 mm (¼″) thick • Bolted connection at the timber joists - 12 mm (½″) diameter bolts - 250 mm (10″) × 50 mm (2″) × 6 mm (¼″) thick steel plate
Floor	• 20 mm (¾″) diameter of threaded steel rods • Steel bearing plates at the exterior face of URM walls - 250 mm (10″) diameter with 12 mm (½″) thick - 250 mm (10″)× 250 mm (10″)× 12 mm (½″) thick • Bolted connection at the timber diaphragm - 12 mm (½″) diameter bolts - 250 mm (10″) × 50 mm (2″) × 6 mm (¼″) thick steel plate

From the case study, it was determined that floor-to-wall connection only occurred at every 5th to 6th joist in Auckland (a comparatively low seismic zone), while they occur at every 3rd to 5th joist in Wellington (a comparatively high seismic zone). Typical wall-diaphragm connections in New

Zealand were through-bolt anchors, where one end of a threaded steel rod was bolted with a steel bearing plate at the exterior face of the URM wall and the other end was welded to a rectangular steel plate bolted to the timber joist. This type of anchors was believed to be incorporated into most New Zealand URM building construction following the 1931 Hawke's Bay earthquake [6]. Bearing plates can be in different shapes such as rosette (circular), spike (two rectangular plates crossed) or square. Details of typical connections used at floor and roof level are described in Table 1.

Typical details of diaphragm framing arrangements were also determined. In New Zealand, typical timber floor diaphragms were constructed using indigenous New Zealand hardwoods such as Kauri, Rimu or Matai, with floor planks nailed to joists that were supported on either timber or steel transverse beams as shown in Figure 2. 25 mm × 150 mm tongue and groove (T&G) floor planks were typically used to eliminate floor gaps and to avoid deflection problems. Joists measuring 50 mm × 300 mm were typically bridged using 50 mm × 100 mm cross-bracing at every one-third of their span to prevent lateral buckling and warping. The span of the joists was commonly not more than 6 m long. It was identified that the URM wall thickness configuration over the height of the building was typically reduced by a single leaf at each storey height to support the diaphragm. Thus, the most common diaphragm seating method was to bear the joists and transverse beams on a single brick width without embedment as illustrated in Figure 2.

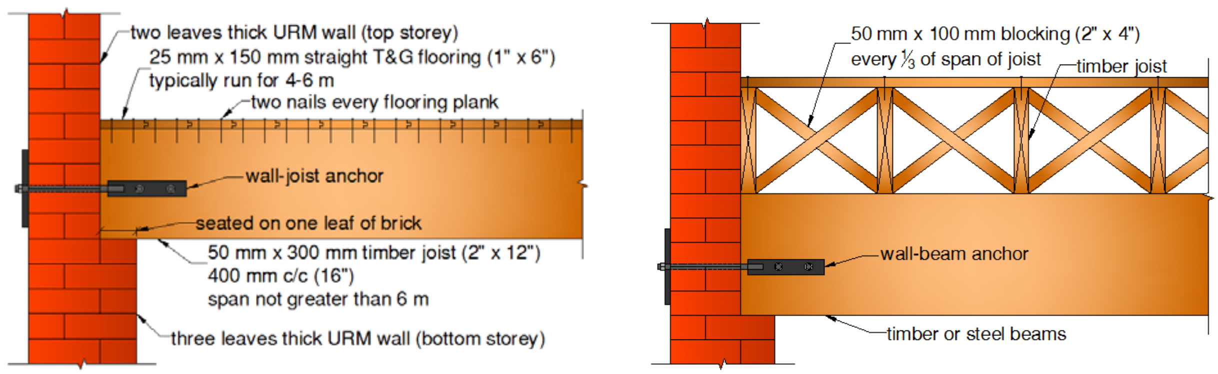

Figure 2: Typical diaphragm details: a) Joist perpendicular to URM wall; b) Joist parallel to URM wall

PULL-OUT TESTS, RESULTS AND DISCUSSION

In order to assess the levels of strength corresponding to the various failure modes in URM walls, preliminary connection pull-out tests were conducted to study the characteristics of wall-diaphragm connections at the roof level of URM buildings. Four URM walls samples were constructed using recycled bricks and a mortar mix proportion of 1:2:9 cement: lime: sand. The first wall specimen, with dimensions of 800 mm height, 700 mm width, and 2 leaf (230 mm) thick was designated as T1. The second, third and fourth walls had dimensions of 600 mm × 700 mm × 2 leaf and were designated as S1, S2 and S3, respectively. A through-bolt anchor type consisting of 16 mm diameter steel rebar with 150 mm square × 12 mm thick steel bearing plate was applied in all URM wall specimens. The 700 mm width of each specimen represents the approximate spacing between connections, which was equivalent to every 2nd joist connection as illustrated in Figure 3.

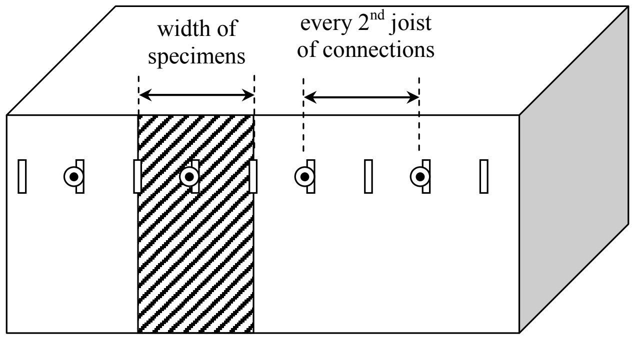

Figure 3: Effective connection spacing

The pull-out connection testing setup is shown in Figure 4. By assuming the density of masonry to be 19 kg/m^3, two springs were attached on the top of each specimen to apply an additional vertical stress of 13 kN/m^2 and 15 kN/m^2 to S and T wall specimens, respectively. Thus, a total of 19 kN/m^2 vertical stress in all S specimens was produced to replicate the dead load (overburden) of a 1 m height of parapet, whereas 23 kN/m^2 was generated for the T specimen. The spring was tested in compression to determine the relationship between its load (kN) and displacement (mm). A displacement gauge was attached to the front face of the wall to monitor the crack opening of the wall and wall deflection was measured using five LVDTs that were attached to the back face of the wall specimen as shown in Figure 5. Two LVDTs were also placed on the top of the wall specimen to monitor changes in the applied vertical stress.

Figure 4: Pull-out connection test setup

Figure 5: LVDTs to measure deflection

From the pull-out tests, the ultimate force (F_u) and failure mode of URM wall were determined. The use of a displacement gauge was found to be very useful for validating the ultimate force at failure of the wall. The ultimate forces and modes of failure for all specimens are summarised in Table 2. The maximum upward movement of 0.83 mm of the wall specimen was also identified, which caused an increment of 0.21 kN/m^2 vertical stress at the top of wall specimen due to the attached spring. The vertical stress increase should in future be avoided as the self weight of the wall remains constant in reality. However, this increase did not significantly affect the results as it was only equivalent to the self weight of an additional 0.011 m 2 leaf thick wall height.

Table 2: Results of pull-out tests on URM walls with anchors

Specimen	Vertical stress	Mode of failure	F_u
T1	23 kN/m^2	Bending	36 kN
S1	19 kN/m^2	Bending	26 kN
S2			24 kN
S3			28 kN

From the experiments conducted, all URM wall specimens failed in bending and the crack occurred at the centre line of the anchor. This indicated that by applying the wall-diaphragm connections at every 2nd joist, the URM walls could be failed in bending. From the results, the ultimate force at failure was significantly increased from 26 kN (average of the S specimens) to 36 kN even if a small increment of vertical stress of 4 kN/m^2 is applied to T specimen. This demonstrates that the 'overburden' produced by the parapets is significantly important and affects the maximum failure load. Thus, the tying of the parapets to the roof diaphragms is a first requirement to prevent the progressive collapse of URM walls at the lower floor level due to the loss of the overburden following the collapse of the parapets.

BOLTED TIMBER CONNECTION TESTS, RESULTS AND DISCUSSION

Due to the possibility for the anchors to fail by tearing out the timber joists parallel to the grain (brittle behaviour), the bolted connection strength in timber was evaluated. Native New Zealand Matai hardwood was used in the experimental investigation as the timber diaphragm in URM buildings are typically constructed using such wood species and due to their availability.

All specimens consisted of three member connections with two steel side plates sandwiching a timber centre piece [10]. The cross section of the timber specimens was 50 mm (thickness) × 100 mm (width). 12 mm diameter (d) bolts of 4.8 grade were used in all specimens. The steel side plates were 10 mm thick. Three groups of specimens were tested, where each group consists of five replicates. All groups had a single number of rows (n_r), but varied with number of bolts (N) and end distance (e). The number of bolts varied from 1 to 2, where the connection with 2 bolts had a 100 mm bolt spacing (s_b). The timber specimens had a moisture content of 13% at the time of testing. All bolts were finger tight to allow self-alignment [11]. The specimens were loaded in tension parallel-to-grain and were fabricated with an identical connection configuration at each end [10]. A universal testing machine was used to apply the load to the specimens. A monotonic tension load was applied through the side steel plates [11]. Both ends were monitored for load and slip, and the ultimate loads recorded were for the extremity that failed. Two displacement gauges were used to measure the slip of the connection at each extremity. Each load-slip data was collected by a data acquisition system and recorded on a personal computer. Figure 6 shows a typical specimen in the testing frame. All specimens were loaded in tension up to the ultimate capacity of one of the two extremities of the connections.

Figure 6: Typical specimen in testing apparatus: a) one bolt; b) two bolts

The load-slip curve of each specimen was plotted and the ultimate load and the type of failure were recorded. The experimental results of the three groups tested are listed in Table 3. The 5th percentile strength of the experimental values $R_{5th\%}$ was calculated assuming a normal distribution. From the connection tests, all of the specimens were observed to fail primarily in bearing (ductile behaviour) until a secondary brittle failure such as splitting or row shear-out causes the load to drop suddenly as illustrated in Figure 7. The dominant final mode of failure in most specimens for groups 2 and 3 was row shear-out, where few specimens failed in splitting. However, it is vice versa for group 1. Typical load-slip curves for all specimens in group 2 that exhibited ductile behaviour are shown in Figure 7. In general, the ultimate strength was considerably affected by increasing the end distance from 100 mm to 200 mm. Specimens in group 3 with an end distance of 100 mm had a lower 5th percentile value compared to those in group 2 with an end distance of 200 mm by a factor of 0.77. This is consistent with other experimental data available in the literature [10].

Table 3: Specimen configuration and summary of results

										Experimental					
Group	d (mm)	e (mm)	s_b (mm)	n_r	N	Species	Cross section (mm)	R_{EYM} (kN)	R_{NZS} (kN)	R_{avg} (kN)	COV (%)	$R_{5th\%}$ (kN)	$\frac{R_{EYM}}{R_{avg}}$	$\frac{R_{NZS}}{R_{5th\%}}$	Failure mode
1	12	200	100	1	2	Matai	50 x 100	67	42	73	2.7	70	0.91	0.59	B
2	12	200	-	1	1	Matai	50 x 100	33	21	40	5.8	36	0.83	0.58	B
3	12	100	-	1	1	Matai	50 x 100	33	21	39	16.9	28	0.85	0.74	B

Note: COV, coefficient of variation; B, bearing.

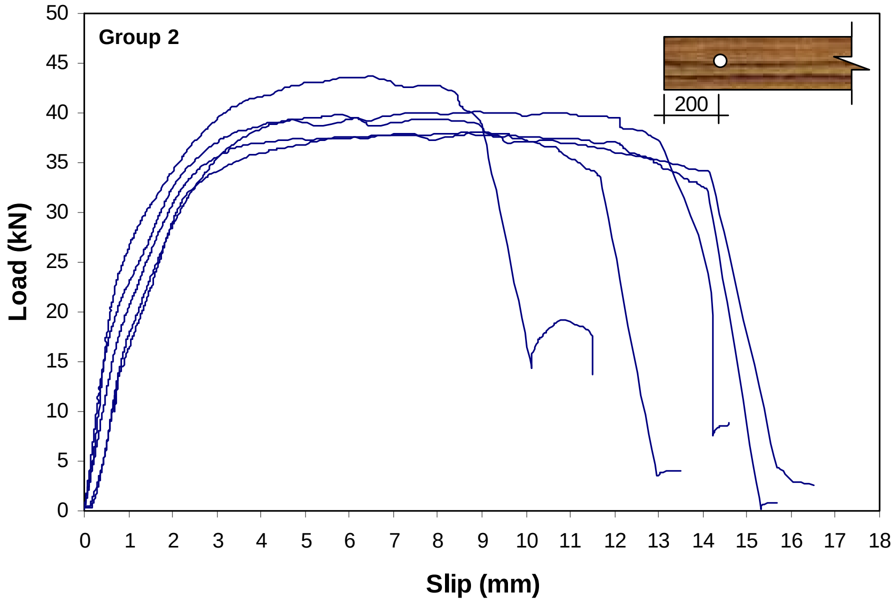

Figure 7: Typical load-slip curves that exhibit ductile behaviour

In order to compare the experimental results, load-carrying capacities *R* of connections were also calculated using the European Yield Model (EYM) [12] and New Zealand timber design standard [13] equations. The EYM equations used were based on Johansen's equations to determine the load-carrying capacity per fastener per shear plane for double shear three member joints. Four equations with respect to four possible failure modes were considered. The equation giving the lowest value of *R* identifies the failure mode of the connection. The effectiveness of the EYM and NZS 3603:1993 predictions versus experimental values is plotted in Figure 8. One can see that the current New Zealand code is far too conservative with ratio of the code values to the 5th percentile of the experimental results as low as 0.58. The EYM equations provide better estimation with ratio of 0.91 to the average of experimental results.

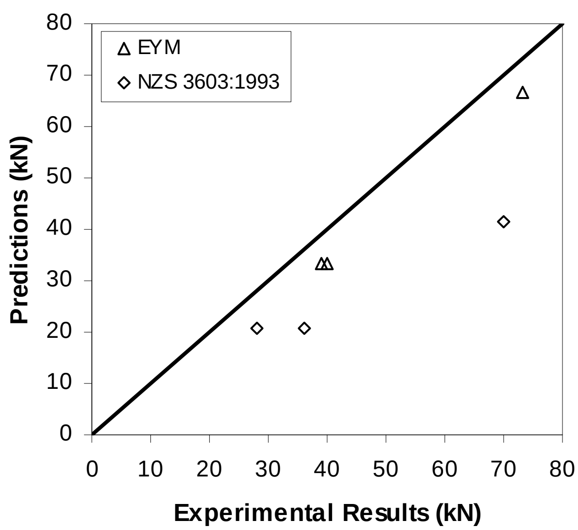

Figure 8: Predictions vs. experimental results

CONCLUSIONS

Based on the experiments conducted in this study, the following conclusions can be reached:

1. By performing the pull-out tests on wall specimens that represent the spacing at every 2^{nd} joist of connection, the ultimate force and mode of failure in URM walls were determined. More URM wall specimens with different widths will be constructed and tested to investigate the effect of spacing of anchors. The dowel type of anchors will also be considered in future.
2. From bolted timber connection test results, the New Zealand timber design standard is far too conservative compared to the actual capacity. The design values provided by the timber standard would make the choice of bolted timber connections impractical. The use of the EYM based design equations for bolted connections is recommended.

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