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Behaviour of Inadequately Detailed Reinforced Concrete Walls

Adane Gebreyohaness[1], Charles Clifton[2] and John Butterworth[3]

1. Corresponding Author. PhD Candidate, Department of Civil & Environmental Engineering, The University of Auckland, Auckland, New Zealand.
   Email: ageb004@aucklanduni.ac.nz

2. Associate Professor, Department of Civil & Environmental Engineering, The University of Auckland, Auckland, New Zealand.
   Email: c.clifton@auckland.ac.nz

3. Associate Professor, Department of Civil & Environmental Engineering, The University of Auckland, Auckland, New Zealand.
   Email: j.butterworth@auckland.ac.nz

Abstract

The existing building stock incorporating reinforced concrete walls built prior to the introduction of seismic design requirements poses a significant risk to their occupants and the public in general. These walls are characterized by the use of plain bar flexural reinforcement, deficient and inconveniently located splices, lack of proper bar end anchorages, insufficient transverse reinforcement, and provision of significantly less vertical reinforcement than required by current standards to generate a ductile response. In addition, the concrete is not confined in the compression zone and the vertical reinforcement is not properly supported laterally to prevent buckling in the compression zone.

A series of experimental tests are being conducted at the University of Auckland to determine the behaviour of these walls under inelastic cyclic loading. The properties of the reconstructed full-scale wall components are adapted from an existing building built in 1928 and located in Wellington, New Zealand. The primary test variables considered are vertical bar splices, level of axial load and wall thickness. The lateral load capacity of the walls is dependent on their flexural strength; and this capacity increases with an increase in the level of the axial load. Although the strength of these walls drops significantly and drastically after low-level drift cycles, the walls are able to maintain a near constant reduced strength to considerably large drift cycles.

Keywords: Non-ductile RC walls, deficient lap splices, old buildings

1 INTRODUCTION

Old reinforced concrete (RC) buildings comprise a considerable portion of vulnerable structures that pose a considerable seismic risk in many seismically active parts of the world. Significant damage to these buildings was reported in the earthquakes of Chile, 2010 (EERI, 2010) and Kocaeli, Turkey, 1999 (Sezen et al., 2003) and more severe damage was observed in the 1985 Chile earthquake (Wood et al., 1987). RC buildings built in New Zealand, particularly those built prior to 1975, weren't designed and detailed to undergo a ductile mode of failure. During the recent earthquake series in Christchurch, New Zealand it was observed that these buildings responded undesirably, and in some cases collapsed catastrophically. The majority of the deaths in the 6.3 magnitude Christchurch Earthquake on February 22, 2011 which claimed 181 lives (New Zealand Police, 2011) are attributed to the collapse of two RC buildings built in 1963 and in 1986 (IPENZ, 2011). Therefore seismic assessment and, if deemed necessary, retrofitting of non-ductile walls, preferably without loss of heritage attributes to the buildings, are crucial steps towards ensuring life safety in future earthquakes.

Non-ductile walls are typically provided with inadequate transverse and vertical reinforcement; the single layer of flexural bars are often spliced just above the floor levels in potential plastic hinge regions; the reinforcing bars are plain round bars, have insufficient lap lengths and lack proper end anchorages. There is also a lack of detailing to contain the concrete in the compression zone and to prevent flexural reinforcement buckling. The concrete is often low strength. In the absence of boundary frame elements, lightly reinforced non-ductile walls can suffer a predominantly flexural mode of failure, due to lack of adequate vertical reinforcement (Greifenhagen and Lestuzzi, 2005, Ireland et al., 2007), although walls of this kind are likely not to achieve a desirable level of ductility. The experimental investigations presented in this paper are designed to reveal the extent of influence of these deficiencies and to contribute to a more realistic assessment of the probable as-built performance of buildings containing these walls and their effective retrofitting.

2 EXPERIMENTAL PROGRAM

The principal aim of the experimental program is to investigate and document the performance of inadequately detailed walls under quasi-static cyclic loading. The investigations are based on the Hope-Gibbons building in Wellington, New Zealand. This nine-storey dual wall-frame building was built in 1928 prior to the introduction of seismic resistant design requirements in the New Zealand building code in 1935. The original structural drawings and results of strength tests conducted on concrete cores extracted from this building are available. The walls in this building are lightly reinforced with plain round bars. The walls are provided around the building's perimeter and comprise one of its lateral load resisting systems. They limit the lateral displacement capacity of the building and have been identified through nonlinear inelastic time history (NITH) analyses as being critical to the seismic performance of the building (Gebreyohaness et al., 2010).

Although a number of investigations have been conducted on inadequately detailed columns (Cho and Pincheira, 2006, Melek and Wallace, 2004, Lynn et al., 1996, Aboutaha et al., 1996, Valluvan et al., 1993), studies which assessed the impact of the inadequate detailing technique on the stiffness and strength degradation and the axial load carrying capacity of non-ductile walls when they are subjected to considerable lateral loads are rare. Experimental investigations conducted in the past on RC walls based on relatively old detailing techniques either used deformed bars, e.g. Greifenhagen and Lestuzzi (2005) and Orakcal *et al* (2009); or were doubly reinforced, e.g. Greifenhagen and Lestuzzi (2005) and Ireland *et al.*(2007). Thus, a series of experimental and numerical studies is being undertaken at the University of Auckland to determine the influence of these parameters.

The reconstructed full-scale wall specimens were subjected to a double-curvature loading condition, representative as closely as possible to the fixed-fixed sway support condition of these walls in multi-storey buildings. Four of the specimens with spliced vertical bars are discussed herein.

2.1 Description of specimens

The actual geometries, material properties and reinforcement arrangement of wall components found in the Hope-Gibbons building are being used in the construction of the specimens. A concrete compressive strength of 21MPa was targeted to reflect the strength of the existing walls found from core samples of the concrete. In the absence of other information, Grade 300 plain reinforcing bars are being used as these are close to the strength of the nominal grade 240 or 250 reinforcements used at that time (NZSEE, 2006). As seen in Table 2, some of the boundary bars used in the tests were Grade 500. This is known after the walls were tested; however, due to the amount of bars present in the walls, bar strength makes little difference to the behaviour of the walls.

The vertical bars were spliced and boundary reinforcements were provided (See Figure 1). Although the thicknesses of the walls in the building range from 6″(150mm) to 9″(230mm), a single layer of reinforcement is provided to all wall types at 12″(305mm) centres at the mid-thickness plane of the walls in both the horizontal and vertical directions. In addition, two ½″(12.7mm) diameter bars are carried around all openings, and these make up the boundary reinforcements. The bars are spliced just above the floor levels with lap lengths of 12″(305mm) and 18″(457mm) for the 3/8″(9.5mm) and ½″(12.7mm) bars respectively, with no transverse reinforcing bar enclosing the lap.

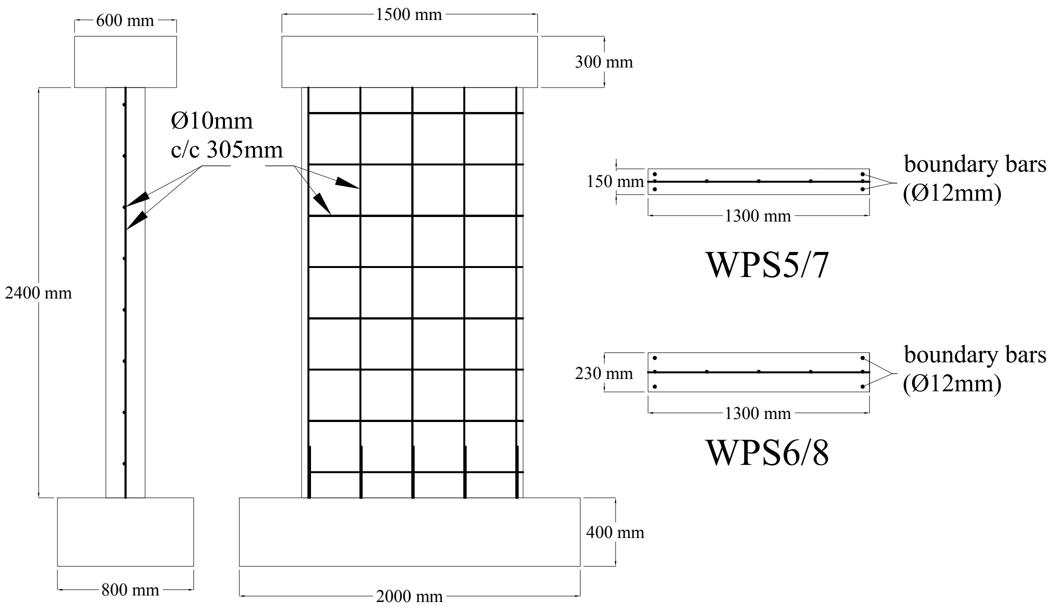

Figure 1: Experimental specimens

The properties of the experimental specimens and the level of applied axial load are presented in Table 1 and Figure 2. In Table 1 L_w represents length of wall specimen, H is height of wall specimen, N is applied axial load on the specimens, A_g is the gross cross-sectional area of the wall and $f_c^{'}$ denotes compressive strength of concrete. Table 2 summarizes the strengths of the reinforcing bars and concrete used in the construction of the specimens. In the table fy is yield strength of the vertical reinforcing bars, f_{yt} is yield strength of the horizontal reinforcing bars, and f_{ult} is the tensile strength of the bars.

Table 1. Geometric properties of the test specimens and level of applied axial load

Specimen	L_w, m	H, m	t, m	N, kN
WPS5	1.3	2.4	0.15	0
WPS6	1.3	2.4	0.23	0
WPS7	1.3	2.4	0.15	$200 \approx 0.05\ A_g f'_c$
WPS8	1.3	2.4	0.23	$300 \approx 0.05\ A_g f'_c$

Table 2. Material properties

	Reinforcement (Plain round bars)						Concrete
Specimen	Vertical/Horizontal	f_y *and* f_{yt}, MPa	f_{ult}, MPa	Boundary	f_y *and* f_{yt}, MPa	f_{ult}, MPa	f'_c, MPa
WPS5	ϕ10 c/c 305mm	348	487	4ϕ12	516	662	29.4
WPS6	ϕ10 c/c 305mm	348	487	4ϕ12	516	662	24.8
WPS7	ϕ10 c/c 305mm	344	456	4ϕ12	305	438	21.3
WPS8	ϕ10 c/c 305mm	344	456	4ϕ12	305	438	22.5

2.2 Experimental setup

The wall components were built on RC foundation blocks which were then anchored to a strong floor, to provide a fixed condition at the base. The foundation blocks also provided anchorage to the flexural reinforcements. An RC block was constructed on the top of the specimens to maintain continuity of reinforcement and to facilitate a smooth transfer of gravity and lateral loads. Out-of-plane movement of the specimens was prevented using channel sections provided on both sides and parallel to the loading beam.

No axial load was applied on two of the specimens, while the remaining two were subjected to axial loads which are approximately 5% of the section capacity. The axial loads were applied using four high strength bars positioned parallel to the centreline and down the sides of the specimens and anchored to the strong floor. Each pair of bar was connected to cross beams, and they were loaded simultaneously using a jack placed underneath the cross beams

(Refer to Figure 2). The jacks were monitored to keep the axial loads relatively constant throughout the tests. Coil springs were employed at the base of each bar underneath the strong floor to prevent the high strength bars from contributing to the stiffness/strength of the walls.

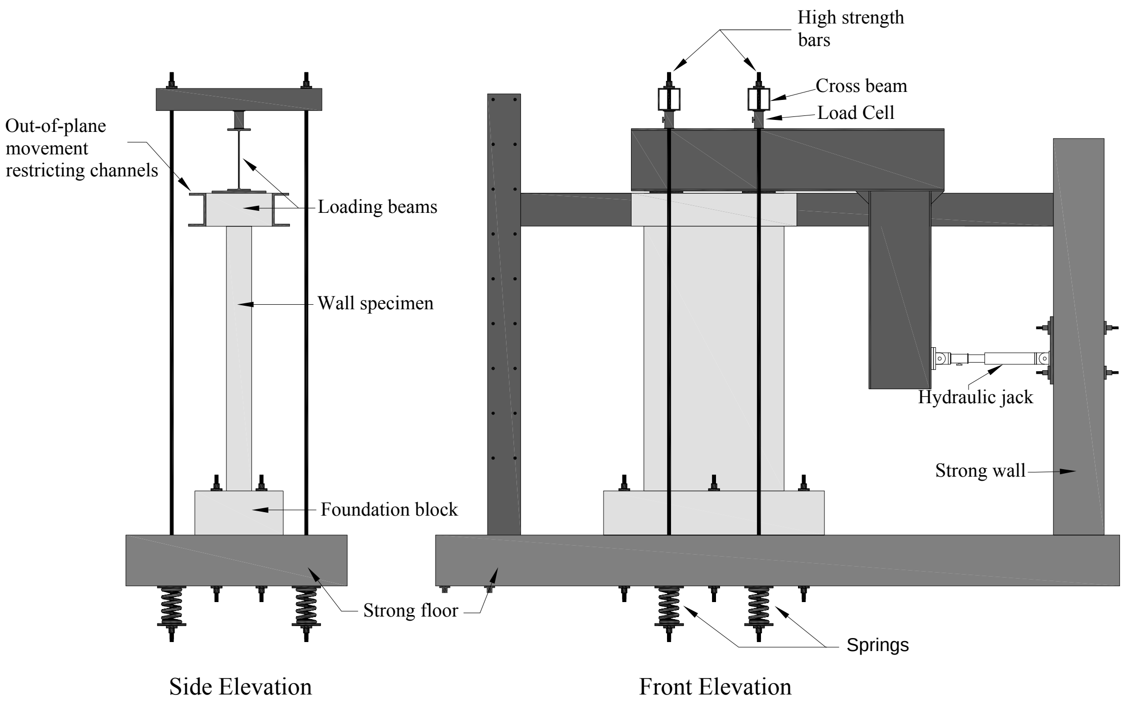

Figure 2: Test setup

2.3 Testing procedure

The walls were subjected to quasi-static cyclic loading. Incrementing sets of three complete displacement-controlled reversed cycles shown in Figure 3 were applied using a hydraulic jack mounted on the strong wall (See Figure 2). The load was applied on the steel loading beam at the mid-height of the walls to create a double bending loading condition.

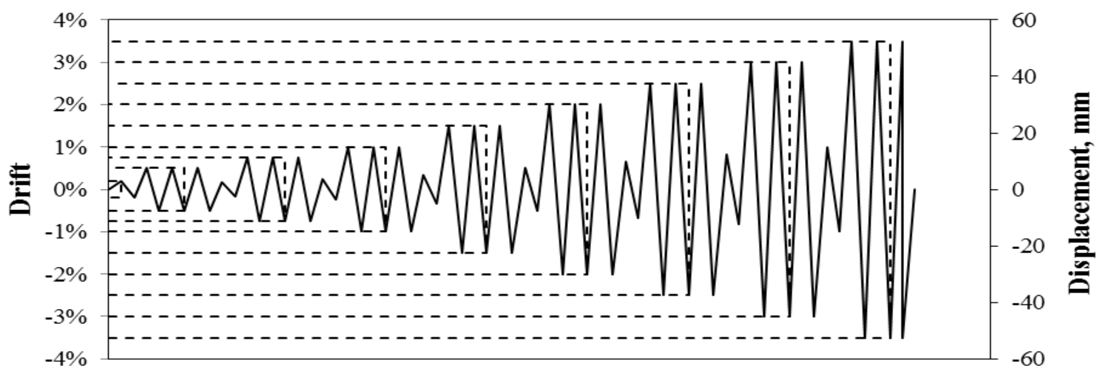

Figure 3: Applied loading regime

2.4 Instrumentation and data acquisition

Load cells were employed to measure the magnitude of lateral and axial loads applied on the specimens. The horizontal displacement at the top and the lateral displacement profiles of the specimens were determined using rotary potentiometers. Portal gauges were used to measure rocking, flexural and shear deformations. Any possible relative sliding displacements that could have occurred during the tests at the strong floor - foundation block, foundation block–specimen, and specimen-loading beam interfaces were also monitored. Ten strain gauges per specimen were attached to the horizontal and vertical reinforcing bars. The strain gauges facilitated determination of the strain profile over the length and height of the specimens.

3 EXPERIMENTAL OBSERVATIONS AND RESULTS

The load-deformation responses of the four specimens are presented in Figure 4. All exhibited a limited amount of energy dissipation capacity. The lateral load capacity of the specimens was dictated by their flexural strength. For these specimens the contribution of shear deformations measured at various locations of the wall appear to have insignificant influence on the responses. No significant flexural deformations were recorded within the body of the panels as well. This is mainly due to the splice location and provision of fewer amounts of flexural reinforcing bars than required by current standards.

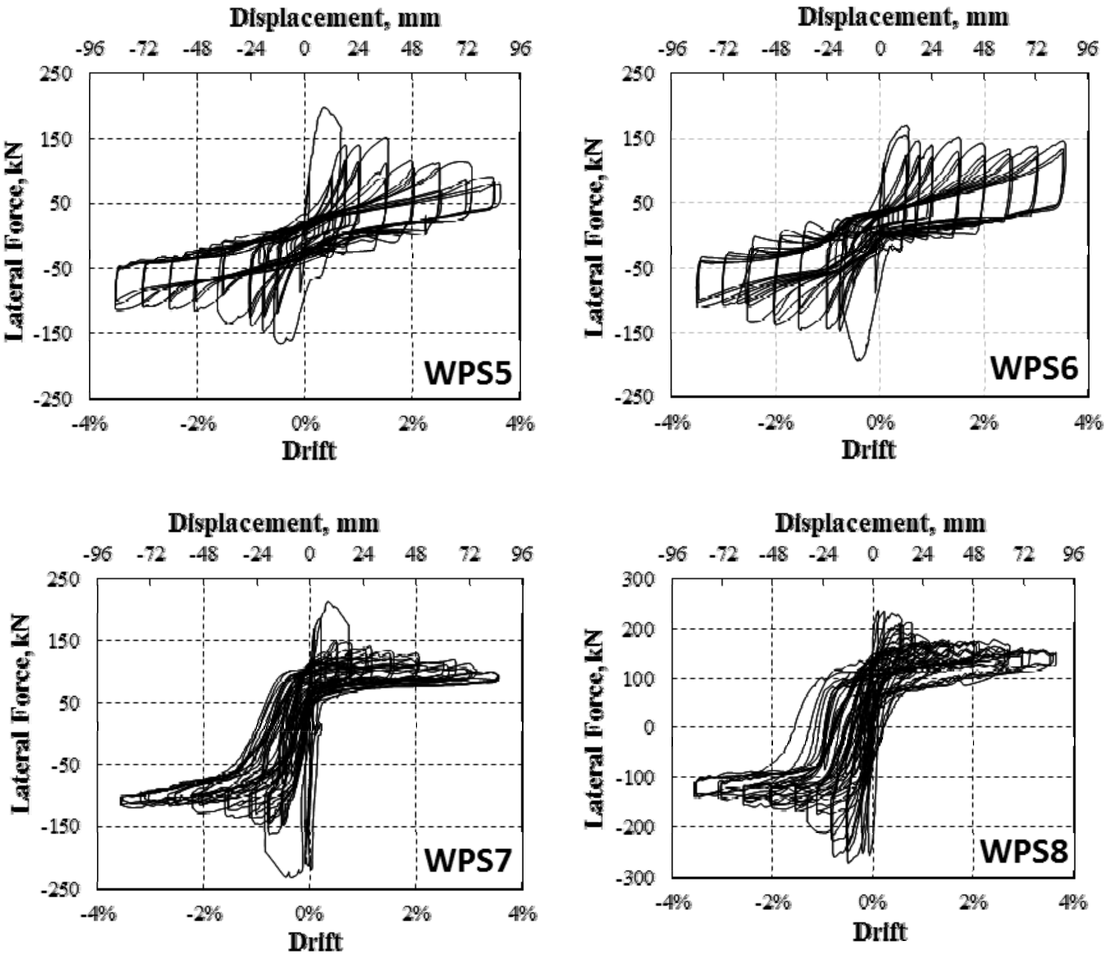

Figure 4: Lateral force vs top displacement of the specimens

The walls responded in double curvature, as intended, until significant damage occurred at their bases. In all the tests the first hairline cracks appeared during the first cycles at the wall/concrete loading beam interfaces. After the initial cycles and at the lower drift levels (<1%), cracks were visible at the top of the near-end (closer to the strong wall) and at the bottom of the far-end during pull cycles, and at the opposite ends during push cycles. The cracks at the top of the walls at these drift levels were wider (up to 10mm gaps were recorded during the testing of WPS7). These cracks kept on growing and extended across the full length at a drift level of 0.75% and 0.50%, for the walls without and with axial load respectively, both at the top and at the bottom of the walls.

At higher drift levels (>1%) the cracks at the top nearly closed back (during testing of WPS5 and WPS6 became almost invisible) and they exhibited a predominantly rocking response about a single crack located at the wall/ foundation interfaces followed by limited sliding at these interfaces and significant slip along the splice lengths. No sliding took place at the loading beam/wall interfaces in all of the tests and a maximum sliding of 0.5mm of a foundation block was recorded. The 150mm thick walls (WPS5 and WPS 7) experienced significant crushing and spalling of concrete at their corners, while the 230mm thick wall with no axial load (WPS6) experienced none and the one with axial load (WPS8) experienced relatively limited crushing and spalling at the base edges only (Refer to Figure 5). Apart from crushing and spalling at the corners, which were unsymmetrical, and the single crack which extended across the full length of the wall, there were no other significant cracks within the main body of the panels.

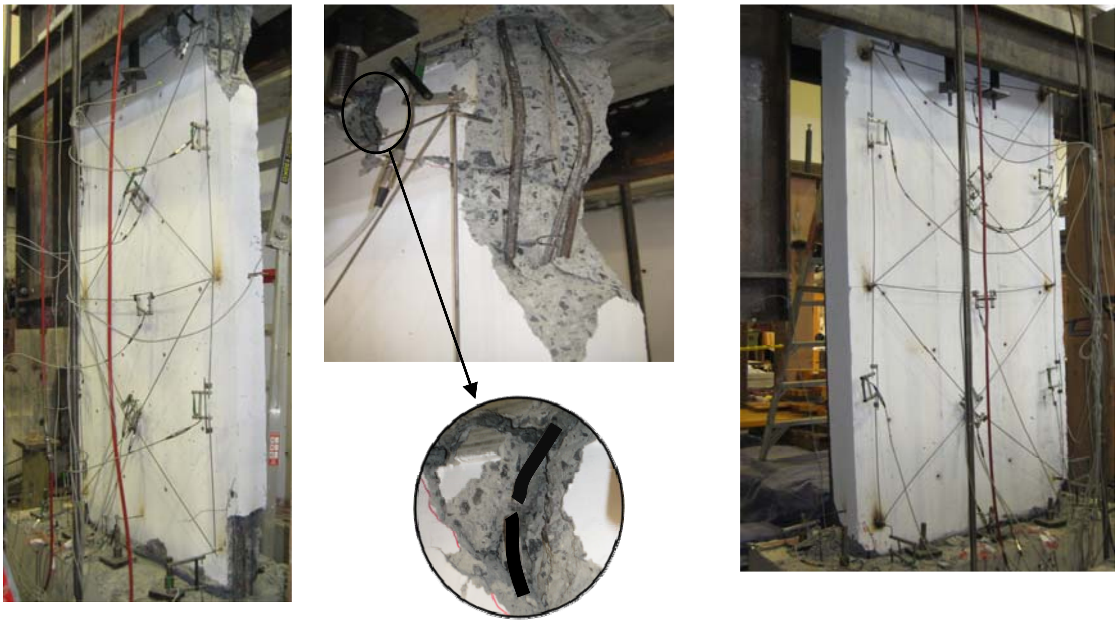

Figure 5: Left: WPS7, significant crushing and spalling of concrete and buckled bars at the corners; middle: buckled and ruptured bars at the top right corner of WPS 7; and right: WPS8, limited crushing and spalling at the base edges only

During testing of WPS7 the concrete at the top right corner cracked under tension at low drift levels (<0.5%), and when subjected to compression in the next cycles, it just gave way and spalled when pushed out by the buckling reinforcing bars at that location. After the concrete has crushed/spalled at that corner, the compression block moved further into the wall and caused buckling and then rupturing of an interior reinforcing bar during successive cycles causing compression at that location (See Figure 5).

The walls were able to develop 97-102% of the computed yield strengths (Refer to Table 3), although strain gauge readings show that slip between starter and vertical bars was observed even during the initial elastic cycles. The readings also indicate that the peak strengths were reached just after or prior to yielding of the starter bars at the edges. These peak strengths dropped significantly after very low drift cycles; however, the walls were able to maintain the reduced strength to relatively higher drift levels of up to 5%.

Table 3. Comparison of predicted wall strengths and test results

	Peak lateral strength		Predicted lateral strength	
Specimen	V_{Test},kN	Drift level, %	V_p ,kN	V_{Test}/V_p
WPS5	199	0.35	195	1.02
WPS6	194	0.44	197	0.98
WPS7	231	0.36	233	0.99
WPS8	271	0.50	278	0.97

4 CONCLUSIONS

The paper presented experimental investigations conducted on reconstructed wall specimens of an existing building. The walls are representative of some pre-1975 RC wall constructions in New Zealand. The vertical reinforcing bars in the test specimens were spliced just above the base of the walls and boundary reinforcements were also provided.

The experimental investigations highlighted that when lightly and singly reinforced non-ductile walls are subjected to seismic loading, buckling of the vertical bars and crushing and spalling of concrete at the corners occurs, since the bars lack adequate lateral support and the concrete within the compression zone is not confined. The walls exhibit a predominantly rocking response accompanied by significant slip along the splice lengths; their lateral load capacity is dictated by their flexural strength.

Given the splices are located just above the floor levels and plain round bars are used, slip along the splice lengths at load levels less than that are required to initiate yielding of the bars is expected. However, the tested walls were able to carry loads close to the computed yield strengths, indicating that the actual bond stresses were significantly higher than those assumed in the past. In addition, bearing in mind the high bond strengths observed in RC structural components after the recent earthquakes in Christchurch, better performances are likely, when these walls are subjected to earthquake-dynamic loadings.

Due to the unfavourable location of the splices and provision of less flexural reinforcement than required by current standards in conjunction with unconfinement of the compression zones and lack of vertical bar lateral support, these walls do not develop distributed flexural cracks and have a limited energy dissipation capacity. In addition, the peak strengths drop significantly after very low drift cycles; although the walls are able to maintain the reduced strengths to a relatively high level drift cycles with no further appreciable strength degradations.

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