Recent Research to Improve the Seismic Performance of Lightly Reinforced and Precast Concrete Walls

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ABSTRACT: The Canterbury earthquakes in New Zealand caused significant damage to a number of reinforced concrete (RC) walls and subsequent research that has been conducted to investigate the design provisions for lightly reinforced RC walls and precast concrete wall connection details is presented. A combination of numerical modelling and large-scale tests were conducted to investigate the seismic behaviour of lightly RC walls. The model and test results confirmed the observed behaviour of an RC wall building in Christchurch that exhibited a single flexural crack and also raised questions regarding the ability of current minimum reinforcement requirements to prevent the concentration of inelastic deformation at a small number of flexural cracks. These findings have led to changes to the minimum vertical reinforcement limits for RC walls in in the Concrete Structures Standard (NZS 3101:2006), with increased vertical reinforcement required in the end region of ductile RC walls. An additional series of wall tests were conducted to investigate the seismic behaviour of panel-to-foundation connections in singly reinforced precast concrete panels that often lack robustness. Both in-plane and out-of-plane panel tests were conducted to assess both grouted connections and dowel connections that use shallow embedded inserts. The initial test results have confirmed some of the previously identified vulnerabilities and tests are ongoing to refine the connection designs.

1 INTRODUCTION

The 2010/2011 Canterbury earthquake series in New Zealand tested the built infrastructure to beyond the design level seismic loading and caused significant damage to both traditional and modern reinforced concrete buildings (Kam et al. 2011). In particular, severe damage was observed to reinforced concrete (RC) walls in several modern multi-storey buildings (Sritharan et al. 2014). Undesirable failure modes that were observed included a lack of distributed flexural cracks, premature fracture of vertical reinforcement, global and local wall buckling, bar buckling, shear failure, and evidence of poor detailing. The Canterbury Earthquakes Royal Commission (CERC) highlighted the need for further research to improve the seismic design of RC walls (CERC 2012).

A research program was initiated at the University of Auckland to understand the observed performance of RC walls during the Canterbury earthquakes and to develop verify improved design methods and detailing practice. A major focus of this research program has been lightly reinforced walls in multi-storey buildings and connections in lightly reinforced precast concrete wall panels. A combination of experimental testing and numerical modelling has been used to investigate the seismic behaviour of these types of walls and is summarised here.

2 LIGHTLY REINFORCED CONCRETE WALLS

Assessments of buildings following the Canterbury earthquakes highlighted several examples of RC walls in multi-storey buildings that had formed a limited number of cracks in the plastic hinge region as opposed to the expected larger number of distributed cracks (Kam et al. 2011, Sritharan et al. 2014). After breaking out the surrounding concrete it was found that the vertical reinforcing steel was often fractured due to the inelastic strain demand at the crack location. If too little vertical reinforcement is used in walls, there is insufficient tension generated to replace the tensile resistance provided by the surrounding concrete after a crack forms, resulting in a reduced number of cracks in the critical moment region, large crack widths, and likely fracture of the reinforcing steel during earthquakes. The Canterbury Earthquake Royal Commission recommended that research be conducted to refine design requirements for crack control in RC walls (CERC 2012).

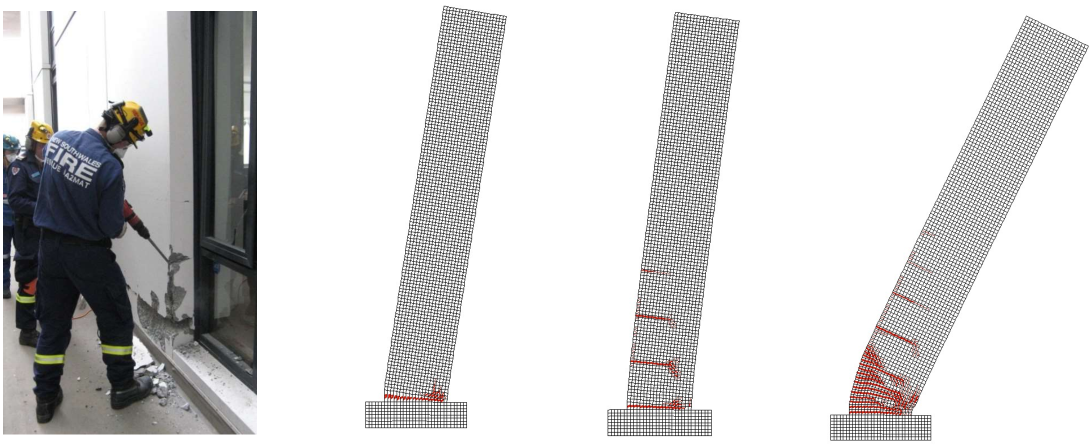
2.1 Minimum Vertical Reinforcement Limits

Historically, minimum requirements for vertical reinforcement in RC walls were governed by shrinkage and temperature effects. More recently, minimum vertical reinforcement limits for RC walls have been increased in design standards worldwide to ensure that ductile behaviour is achieved when yielding of reinforcement is expected. In the 2006 revision of the New Zealand Concrete Structures Standard, NZS 3101:2006, the minimum required vertical reinforcement in RC walls was increased by over 80% with the adoption of a similar equation to that previously used for RC beams. Because of these recent changes, some of the RC walls in Christchurch that were observed to have only a few flexural cracks and fractured vertical reinforcement had vertical reinforcement contents below the current limit in NZS 3101:2006. Additionally, higher than expected concrete strengths may have contributed to the lack of flexural cracks in some lightly reinforced RC walls in Christchurch (CERC 2012).

Moment-curvature analysis was used to provide an initial assessment of the current vertical reinforcement limits for RC walls (Henry 2013). From this analysis it was found that even when the concrete strengths are known, the current minimum vertical reinforcement limits for RC walls in NZS 3101:2006 (A2) may not be appropriate as the margin of separation between cracking and nominal strength would be less than for an equivalent beam unless a significant axial load is applied.

2.2 Finite Element Modelling

A series of numerical analyses were conducted to investigate the lateral load response of RC walls with minimum reinforcement using nonlinear finite element program VecTor2 (Wong & Vecchio 2003). The development of the VecTor2 wall models is described in more detail by Lu et al. (2015a, 2015b). The grid-F wall from the Gallery Apartments building in Christchurch was used as the baseline for the analyses. The grid-F wall had a length of 4300 mm, a thickness of 325 mm with a vertical reinforcement ratio of 0.16%, less than the 0.27% currently required by NZS 3101:2006 (A2). The behaviour of the modelled as-built grid-F wall was similar to the failure mode observed during the 22 Feb 2011 Christchurch earthquake, as show in Figure 1a and b. A single flexural crack was observed at the wall base with the strain in the vertical reinforcement concentrated at the crack and not distributed along a large length of the bar. Because of the reduced spread of the plasticity, the wall demonstrated only limited ductility with fracture of vertical reinforcement occurring at only 0.75% lateral drift.



(a) Crack in grid-F wall (b) As-built grid-F (c) NZS 3101 (A2) (d) Additional reo at ends

Figure 1 Actual grid-F wall damage compared with deformed shape (magnified x5) and crack patterns from wall models

Additional analyses were conducted using the dimensions of the grid-F wall with modified reinforcement detailing in accordance with the current minimum requirements from different concrete standards worldwide (Lu et al. 2015b). The behaviour of the grid-F wall with a distributed vertical reinforcement ratio of 0.274% in accordance with minimum requirements of the amendment 2 version of NZS 3101:2006 (A2) is shown in Figure 1c. A total of four primary flexural cracks were observed, but still insufficient reinforcement to generate well distributed secondary cracks. A third analysis was conducted with increased vertical reinforcement at the ends of the wall (0.5% reinforcement ratio), as shown in Figure 1d. A significant improvement over the as-built and NZS 3101 wall was observed with the concentrated reinforcement in the end regions of the wall sufficient to generate a large number of secondary cracks in the plastic hinge region.

Based on the preliminary numerical modelling results, it was suggested that RC walls designed prior to the introduction of stricter minimum reinforcement limits in NZS 3101:2006 (A2) are highly susceptible to a non-ductile failure characterised by a single crack. Walls with distributed vertical reinforcement in accordance with NZS 3101:2006 (A2) may exhibit some ductility, but that adding additional reinforcement at the wall ends can significantly improve the crack distribution and ductility in the plastic hinge region.

2.3 Experimental tests

A series of experimental tests was conducted to further investigate the existing minimum vertical reinforcement limits in NZS 3101:2006 (A2) and to verify the modelling predictions (Lu et al. 2015a, 2015b). A total of six large-scale RC walls were subjected to pseudo-static cyclic loading. The 1.4 m long, 2.8 m high and 150 mm thick test walls were designed as monolithic walls with distributed vertical reinforcement in accordance with minimum requirement in NZS 3101:2006 (A2). The cross section and reinforcement details are shown in Figure 2 and the test variables are summarised in Table 1. The six walls were subjected to different loading configurations including three shear span ratios (2, 4, and 6) and axial load ranging from 0-6.6% of the wall axial capacity. Because of height limitations in the structural test hall, a test setup was designed to simulate the expected seismic loading on the bottom two storeys of a 40-50% scaled wall from a multi-storey building, as shown in Figure 3. An actuator was attached between the steel loading beam and the strong wall to apply horizontal loads to the wall, and two additional actuators were attached vertically at each end of the wall to achieve the required moment and axial load at the top of the wall.

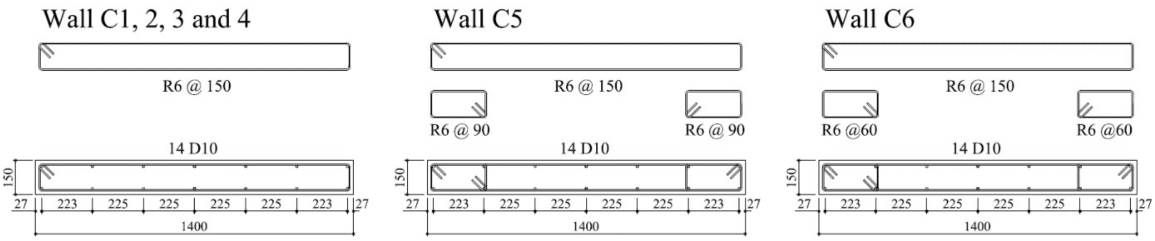


Figure 2 Cross sections of test wall specimens

Table 1 Details of six test walls

Wall	M/VL_w ratio	Axial load ratio	Specified material properties		Vertical reinforcement ratio (%)	End stirrups (mm)
			f_c' (MPa)	f_y (MPa)		
C1	2	3.5%	40	300	0.53	No
C2	4	3.5%	40	300	0.53	No
C3	6	3.5%	40	300	0.53	No
C4	2	0	40	300	0.53	No
C5	2	6.6%	40	300	0.53	D6@90
C6	4	3.5%	40	300	0.53	D6@60

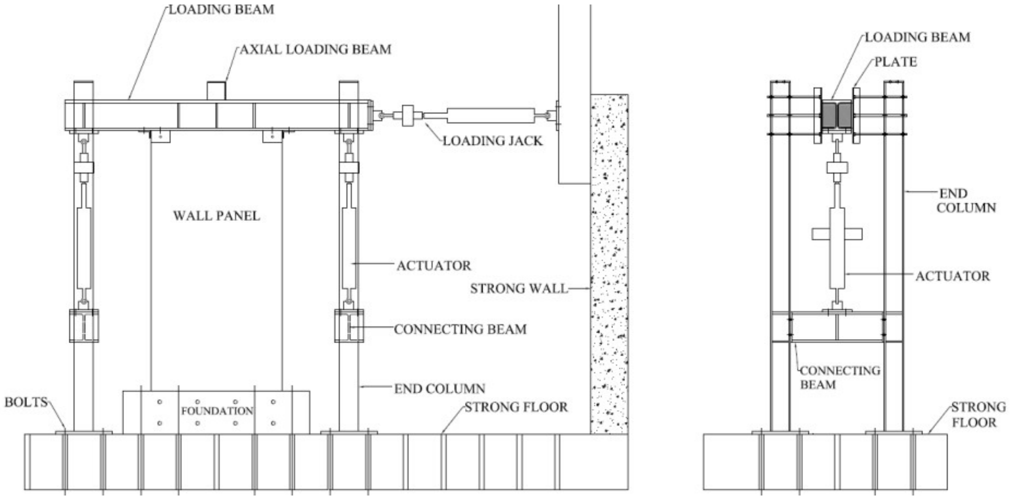
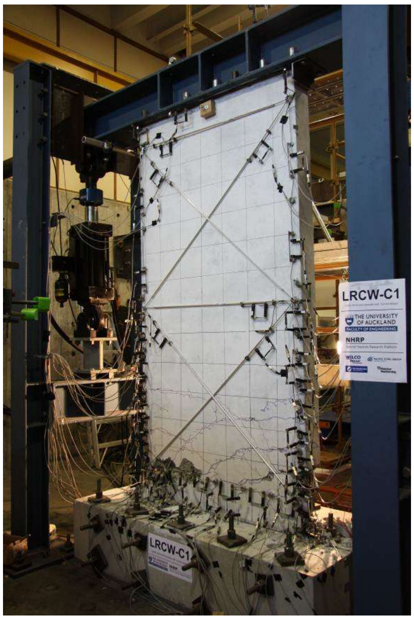
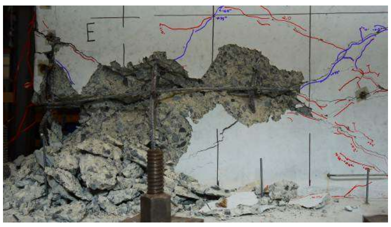


Figure 3 Experimental test setup for RC walls

The response of all six test walls was dominated by flexural behaviour with 3-4 main flexural cracks forming in the lower portion of the wall height, as shown for wall C1 in Figure 4. These 3-4 cracks were triggered before the lateral drift of ±0.25%, after which no significant new flexural cracks occurred. During high lateral drift cycles, the wall deformation was primarily concentrated at a 1-2 main cracks that opened up to 20 mm wide. The concentrated inelastic strains resulted in buckling of the vertical reinforcement during cycles to ±1.5% lateral drift with fracture at 2.5% lateral drift for all test walls. The shear span ratio and axial load had only a minor influence on the local response parameters and did not significantly alter the failure mode. The equivalent plastic hinge length of the test walls was typically less than half the recommended hinge lengths that are used to determine curvature ductility and rotational capacity.



(b) Extent of flexural cracking



(a) Overall condition at 2.5% drift (c) Concrete crushing and bar buckling at east end

Figure 4 Photos of wall C1 at the end of testing

The main outcome from the experimental tests was that the current minimum vertical reinforcement requirements for RC walls in NZS 3101:2006 (A2) are sufficient to prevent a sudden loss in strength after first cracking, however, they are insufficient to ensure that a large number of secondary cracks form in plastic hinge regions. Additionally, the concentration of inelastic strains in lightly RC walls makes the vertical reinforcement highly vulnerable to buckling at modest lateral drifts.

2.4 **Amendments to NZS 3101**

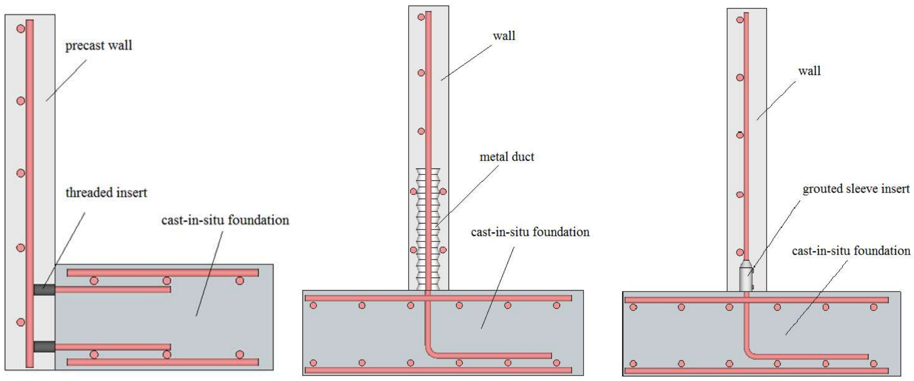
Based on the observations from the Canterbury earthquakes and the initial research findings presented, amendments were proposed to the minimum vertical reinforcement limits in NZS 3101:2006. As described by Russell et al. (2015), these changes published as amendment 3 have resulted in the following requirements:

- Minimum distributed vertical reinforcement content of $\sqrt{f'_c}/4f_y$ in all walls (similar to existing requirements).
- Vertical reinforcement of content of $\sqrt{f'_c}/2f_y$ in the end region of walls with limited ductile or ductile plastic hinge regions. The end region reinforcement content was derived to ensure that secondary cracks would form when considering expected concrete tensile strengths.
- Requirement that the web (central) vertical reinforcement ratio be at least 30% of the reinforcement ratio at the ends of the wall to ensure that the distributed secondary cracks propagate through the web region.

Further experimental tests are currently in progress to verify and refine these proposed changes. In addition, research is required to develop assessment procedures for lightly reinforced concrete walls in existing buildings.

3 **PRECAST WALL CONNECTIONS**

Precast concrete construction is common in New Zealand and walls in both low-rise and multi-storey buildings are typically constructed from precast panels. A review of manufactured precast concrete panels was undertaken in order to develop a comprehensive understanding of the common typologies for connections between precast concrete panels and foundations (Seifi et al. 2015). The database confirmed that buildings in regions of low or moderate seismicity are commonly constructed from thin precast concrete panels (~150 mm) with only a single layer of minimum required horizontal and vertical reinforcement. In addition to the the aforementioned issues with lightly reinforced walls, the seismic behaviour of such wall precast panels is highly dependent on the connections. Examples of three commonly used wall-to-foundation connection details are shown in Figure 5.



(a) Threaded inserts (b) Metal duct (c) Grout sleeve

Figure 5 Cross sections of typical wall-to-foundation connections

3.1 **Out-of-plane tests**

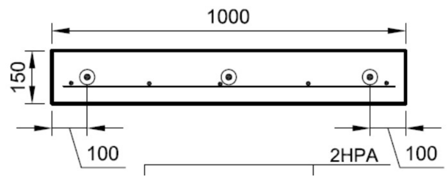
The out-of-plane response of wall-to-foundation connections that used threaded inserts to connect the starter reinforcement (Figure 5a) was questioned following the Canterbury earthquakes due to the shallow embedment of the inserts and lack of robustness to cyclic loading. A series of 12 panel tests were completed to investigate both the monotonic and cyclic response of typically constructed threaded insert connection designs (Burley et al. 2014). As shown in Figure 6, the test panels did not perform well with the flexural cracks in the panel propagating vertically into the joint region behind the back of the inserts and the connection started acting like a pin. In many cases the panel did not reach its full flexural capacity prior to the onset of this failure mode. The tests highlighted the vulnerability of the connection with the inserts not embedded deep enough in the panel to intersect the compression region, as shown by the strut-and-tie model in Figure 6c. Further tests are ongoing to investigate improved detailing of this type of wall-to-foundation connection.

(a) Panels setup for testing (b) Connection failure (c) Strut and tie model

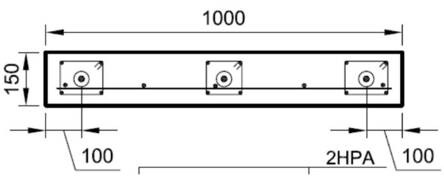
Figure 6 Out-of-plane tests of threaded insert connections

3.2 **In-plane tests**

In multi-storey panels it is common to use grouted connections to splice vertical reinforcement. As shown in Figure 5b and 5c, grouted connections can use either a metal duct cast into the panel or a proprietary grout sleeve reinforcement coupler. An experimental programme was developed to assess the seismic behaviour of precast concrete panels connected to the foundation using grouted connections. The test program is currently in progress with four panels with metal duct connections have been tested to date (Seifi et al. 2015). The test walls were 3000 mm height and a 1000 mm length, with vertical reinforcement of HD12 spaced at 225 mm c/c, and horizontal reinforcement of HD12 spaced at 250 mm c/c that was uniformly distributed and had 90° hooked ends, as shown in Figure 7. Three HD16 bars with a spacing of 400 mm were used to connect the wall to the foundation. The connection bars protruded from the foundation and were embedded inside the wall in 600 mm long corrugated metal ducts that were filled with grout. A 20 mm gap between the foundation and the panel was provided by placing two shims underneath the wall. The area around the gap was dry-packed and a day later was filled with grout. The walls were tested in-plane using a test setup that was similar to that shown previously in Figure 3, but with only the horizontal actuator providing lateral loading at the top of the wall.



(b) Panel details – No stirrups

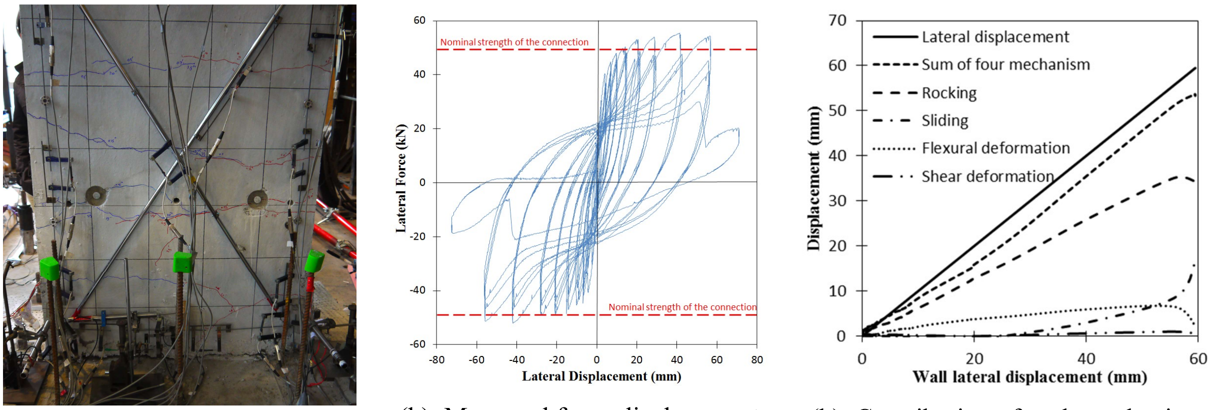


(a) Panel during construction (b) Panel details – Stirrups around connection bars

Figure 7 Details of in-plane precast test walls

The behaviour of the test walls was characterised by the opening of the joint at the wall-to-foundation interface, as shown in Figure 8a. The connection was the weakest section and the joint opened up widely in all tests, with cracks less than 1.2 mm wide occurring in the wall panels. The widest crack in the panel occurred at a height of 600 mm where the connection reinforcement was terminated.

Crushing the grout layer and yielding and elongation of connection bars also allowed the wall to slide on the foundation. The tests were continued until the rupture of the connection reinforcement at approximately 2.5% lateral drift. During the test, no reinforcement pulled out from the metal ducts and no other premature failure was observed. The measured force-displacement response of the panel is shown in Figure 8b. The wall displayed a ductile behaviour with a maximum strength that was approximately 10% larger than the design flexural capacity of the connections.



(a) Wall MD-S-N at -2.5% drift

(b) Measured force-displacement response

(b) Contribution of each mechanism to the overall displacement

Figure 8 Results of in-plane test of grouted metal duct connection with no stirrups

The contributions of wall flexural deformation, wall shear deformation, sliding at the wall-to-foundation interface, and rocking at the wall-to-foundation interface on the total force-displacement behaviour of the wall were obtained by displacement gauges that were installed on the wall, as plotted in Figure 8c. Rocking had the largest effect on the behaviour of the test walls with more than 60% of the lateral displacement of the top of the panel due to rocking. The second and third more dominant mechanisms were sliding and flexural deformation of the panel. Before the yielding of the connection reinforcement, the flexural deformation of the panel formed approximately 15% of the overall displacement. At larger lateral drifts the yielding and elongation of the connection bars allowed the panel to slide and the flexural deformation contribution was reduced. Conversely, the sliding of the panel had more influence on the behaviour of the panel as the panel was subjected to larger displacements.

In-plane tests of precast panels with grouted connections are ongoing. The currently planned tests will be conducted on larger panels and with an appropriate axial load applied. It is expected that the combination of these factors will increase the neutral axis depth and therefore increase the possibility of concrete spalling and crushing that may degrade the connection during cyclic loading. The results of these tests will be used to refine design guidelines for precast panel connections.

4 CONCLUSIONS

A combination of numerical modelling and large-scale tests were conducted to investigate the seismic behaviour of lightly reinforced concrete walls. The numerical modelling confirmed the observed behaviour of an RC wall building in Christchurch that exhibited a single flexural crack. The results of six large-scale walls indicated that the current minimum distributed vertical reinforcement limits in NZS 3101:2006 are not sufficient to prevent the concentration of inelastic deformation at a small number of flexural cracks. These findings have led to changes to the minimum vertical reinforcement limits for RC walls in in the Concrete Structures Standard (NZS 3101:2006), with increased vertical reinforcement required in the end region of ductile RC walls.

Additional wall tests have been conducted to investigate the seismic behaviour of panel-to-foundation connections in singly reinforced precast concrete panels. Out-of-plane tests of panels with dowel connections that use threaded inserts highlighted the lack of robustness of these shallow embedded insets. Furthermore, in-plane panel tests were conducted to assess the cyclic response of grouted connections using metal ducts. The initial test results have confirmed that the panel response is dominated by the connection and tests are ongoing to investigate the performance of these types of connection to more demanding loading conditions.

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