LESSONS FROM 2011 CHRISTCHURCH EARTHQUAKE FOR IMPROVING SEISMIC PERFORMANCE OF CONCRETE WALLS

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ABSTRACT

The 2011, 6.3 magnitude Christchurch earthquake in New Zealand caused considerable structural damage. It is believed that this event has now resulted in demolition of about 65-70% of the building stock in the Central Business District (CBD), significantly crippling economic activities in the city of Christchurch. A major concern raised from this event was adequacy of the current seismic design practice adopted for reinforced concrete walls due to their poor performance in modern buildings. The relatively short-duration earthquake motion implied that the observed wall damage occurred in a brittle manner despite adopting a ductile design philosophy. This paper presents the lessons learned from the observed wall damage in the context of current state of knowledge in the following areas: concentrating longitudinal reinforcement in wall end regions; determining wall thickness to prevent out-of-plane wall buckling; avoiding lap splices in plastic hinge zones; and quantifying minimum vertical reinforcement.

INTRODUCTION

Field observations of structural performance in previous earthquakes have significantly contributed to research advancements, prompting improved design procedures and reinforcement detailing for concrete walls. As with the 2011 Christchurch earthquake in New Zealand, field observations have confirmed the improved seismic performance of structures resulting from improved design approaches. For example, due to the stringent application of the capacity design approach, classical shear failures of reinforced concrete walls were rare. However, new or previously uncommon failure modes were observed to reinforced concrete walls in this event. This paper focuses on the lessons learned by evaluating performance of reinforced concrete walls in the 2011 Christchurch earthquake. A more detailed treatment of specific issues discussed in this paper is presented in Sritharan et al. (2014).

With emphasis on achieving ductile behavior for reinforced concrete walls, this paper highlights: (a) Impact of concentrating the main longitudinal (i.e., vertical) reinforcement in wall boundary elements instead of distributing it along the wall length; (b) Influence of large tensile strain demand on the longitudinal reinforcement causing local buckling of the wall due to compression zone

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instability upon subsequent load reversal; (c) Effect of using lap splices at the wall base; and (d) Consequence of not providing adequate minimum vertical reinforcement in walls.

DESIGN PRACTICE

Current Approach

In modern seismic design, reinforced concrete walls are designed with the intention of providing sufficient strength and adequate flexural ductility while preventing brittle failure modes such as those from insufficient shear capacity, inadequate anchorage of reinforcement, inadequate lap splice length, and sliding at the wall-to-foundation interface. While some design standards aim to achieve ductile wall response by adopting the capacity design philosophy (e.g., NZ 3101:2006; CEN 2004), others attempt to achieve the same behavior without explicitly implementing this design philosophy (e.g., ACI 318-11).

A common feature of seismic force-resisting walls subjected to large moments and shears is that they are designed with boundary elements, which are regions located at the wall ends with additional reinforcement requirements, increased thickness or both. Comparable to highly reinforced ductile columns, these regions may use a combination of high concentration of longitudinal and transverse reinforcement to ensure that high compressive strains needed for ductile wall response can be developed in these regions. This is why ACI 318 (2011) and Eurocode 8 (CEN 2004) require the use of boundary elements in walls when compression in the wall’s end regions exceeds certain stress or strain limits. Although confinement reinforcement is required in the compression zone, the NZS 3101 (2006) standards do not require the use of boundary elements, but encourages it. In this case, there is no specific requirement to use a concentration of longitudinal reinforcement, because it aims at minimizing the likelihood of the wall experiencing out-of-plane buckling as subsequently discussed. One advantage of using larger diameter longitudinal bars in the end regions is to increase the minimum required spacing of transverse reinforcement, reducing the steel congestion. Another benefit of the highly reinforced boundary elements is that it increases the moment resistance of the walls by 5-15% compared to walls with the same total area of longitudinal reinforcement distributed evenly along the wall length (Dai 2012). In these situations, the wall regions between the boundary elements are typically designed with minimum amounts of vertical reinforcement in two parallel layers.

To limit premature out-of-plane buckling of walls in the potential plastic hinge region, NZS 3101 controls the minimum wall thickness as a function of wall length and aspect ratio. While the commentary section of NZS 3101 acknowledges that the maximum tensile strain developed in the longitudinal reinforcement influences this wall instability by acknowledging that the original equations proposed by Paulay and Priestley (1992) were used in deriving a simple design equation, the wall thickness is not determined as a function of an expected tensile strain. Eurocode and ACI have requirements for minimum thickness for boundary elements, but they are not based on minimizing potential wall buckling resulting from large tensile strain in the longitudinal reinforcement.

Other notable current wall design practices include permitting the use of conventional lap splices in the potential plastic hinge region. While this practice is strongly discouraged or eliminated in Eurocode 8 and NZS 3101, ACI 318 does not preclude the use of lap splices in the plastic hinge zone. In fact, use of lap splices at the wall base is common industry practice in the U.S. ACI 318, however, requires an increased lap splice length but no increase in confinement reinforcement if the lap splice is situated in the plastic zone. If lap splices cannot be avoided within the plastic hinge zones, NZS3101 requires them to be staggered, with an allowance for only 1/3 of the longitudinal bars to be spliced at any one section along with a requirement for lateral ties when the longitudinal bar diameter exceeds 16 mm.

Historically, the minimum vertical reinforcement in concrete walls was based solely on requirements for temperature and shrinkage. Current versions of NZS 3101 and ACI 318 include more rigorous minimum vertical reinforcement limits to ensure that a minimum level of ductility is achieved. NZS 3101 (2006) adopted the same equation for walls as that previously developed for minimum longitudinal reinforcement in beams to ensure that the yield moment is greater than the
probable cracking moment. The NZS 3101 (2006) procedure results in vertical reinforcement contents of 0.25% or greater depending on the concrete and reinforcement strength. In special structural walls, ACI 318 requires a minimum reinforcement content of 0.25% that is not dependent on the concrete or reinforcement strength; this can be reduced to 0.12-0.15% when the shear demand is below certain limits. Eurocode 8 requires a minimum vertical reinforcement of 0.25% across cold joints, which is to minimize shear sliding at the crack interface and 0.5% in the boundary elements.

Undesirable Failure Modes
Assuming concrete walls are designed with adequate confinement and shear reinforcement, the discussion of this section focusses on less obvious design deficiencies and associated failure.

Distributed vs. Concentrated Longitudinal Reinforcement

Despite the expected benefits and the code recommendations to use heavy longitudinal reinforcement in boundary elements, seismic testing on concrete walls with such reinforcement layouts has often produced unsatisfactory overall performance at moderate to large ductilities. While the boundary elements exhibit satisfactory response, the web region between the boundary elements experiences significant damage. Figure 1 shows a rectangular wall, RWN, tested by Aaleti et al. (2013) and a U-shaped concrete wall, TUB, tested by Beyer et al. (2008). The unsymmetrical damage pattern seen on RWN is a reflection of the use of different amounts of vertical reinforcement in the two boundary elements and asymmetric loading to achieve specific research objectives, while TUB was subjected to a bidirectional loading pattern comprising cycles in the web, flange and diagonal directions.

An important observation from the two tests is that the extent of damage to walls in the boundary elements is relatively less compared to the web regions. Formation of cracks with larger width and wider spacing, crushing and spalling of concrete that began within the cover and penetrated well into the core region, and subsequent reduction of wall thickness beyond that experienced by the boundary elements are direct consequences of using light reinforcement in the web regions. As evidenced from the tests, potential failure modes of walls with heavy longitudinal reinforcement in the boundary elements and light longitudinal reinforcement in the web regions are: (1) Crushing of concrete in the web regions, which can be exacerbated if the axial load in the walls increases due to vertical acceleration and/or framing action resulting from interaction between walls and floors; and (2) Large shear deformation and potential for shear sliding due to the development of wider cracks in the web; and (3) Buckling of boundary elements due to the web experiencing significant damage. It is emphasized that the aforementioned web crushing occurs under in-plane loading. This is different from possible web crushing that may occur especially when low amounts of reinforcement presents in the web region combined with out-of-plane response (Paulay and Priestley 1992).

Potential improvements to wall performance have been recently investigated by Brueggen (2009) and Dai (2012). In a T-wall tested by Brueggen, the web of the tee wall was designed following the current ACI practice, including a boundary element. A longer length for the confinement region was used because this was found to be necessary based on a section analysis and noticeable damage observed to this region in NTW1—a reference wall tested by Brueggen (2009) following the
code approach. However, the flange of the second wall, NTW2, was designed with distributed reinforcement. This resulted in a longitudinal steel ratio, $\rho_l$, of 2.16% along the entire length of the flange in NTW2, whereas $\rho_l$ of 3.78% and 0.59% were used, respectively, within and outside of the boundary elements in the flange of NTW1. While the performance of the web in NTW2 improved due to the use of a longer confinement region, the drastic difference to the damage between the boundary element and the region in between the boundary is seen in Figure 2a. On the other hand, a significantly improved performance was obtained for the flange with distributed reinforcement (see Figure 2b). The distribution of reinforcement in NTW2 resulted in a 13% reduction in lateral force resistance and an increased displacement of 22% at the maximum lateral load resistance.

A systematic analytical study by Dai (2012) examined the ductility capacity and failure strains of rectangular concrete walls. In comparison to the ACI 318 (2011) recommendations, the study concluded that overall seismic performance of the walls could be enhanced by increasing the confinement reinforcement quantity by 30% and providing it along the length of the compression regions experiencing strain beyond 0.0015.

![Figure 2](image)

**Figure 2** A T-wall test completed by Brueggen with distributed reinforcement in the flange

**Wall Instability**

Structural walls designed to current practice can experience significant ductility demand with large tensile strains being imposed on the longitudinal reinforcement in the plastic hinge region. This strain magnitude will depend on the axial load, and importantly, the wall geometry. Concrete and steel properties also impact tensile strain demand, but to a lesser extent. For planar rectangular walls, equilibrium of internal forces dictates the distance to the neutral axis, as measured from the extreme compression fiber, to be greater than that of T, L or U-shaped walls, indicating that for the same curvature the tensile strain in the end region of the wall is higher in the case of non-planar walls. The large tensile strain is of importance as it affects the lateral stability of the wall depending on its magnitude. Cracks, developed as a result of a large inelastic excursion, must close in order to provide the local compressive force needed for developing the in-plane lateral strength in the reversed direction. Referred to as local wall instability, this phenomenon was first investigated by Goodsrin et al. (1983), and a set of expressions to control wall buckling was proposed by Paulay and Priestley (1992).

Chai and Elayer (1999) demonstrated the mechanism of wall instability using cyclic tests of axially loaded reinforced concrete columns, which essentially represented the end tension/compression region of walls. Despite the lack of strain gradient effects, such idealization was useful in identifying the critical parameters governing the buckling mechanism. Photographs in Figure 3 show the condition of a reinforced concrete column under large tension/compression cycles. The test column was rectangular in cross-section (102 mm × 204 mm) and longitudinally reinforced with 6#3 bars ($d_b = 9.5$ mm, where $d_b$ is the bar diameter) giving a reinforcement ratio of 2.1%. The length of the column was 1498 mm giving a length-to-width ratio of 14.75. Transverse ties were provided at a close spacing of $6d_b$ to represent a well-confined end region of the wall and to prevent local buckling of the longitudinal reinforcement as typically used in design of ductile walls.
The loading protocol for the test column imposed first a tensile half-cycle followed by a compression half-cycle with a compressive strain targeting about 1/7 of the tensile strain amplitude. In Figure 3a and b, amplitudes of the axial strain in the tensile half-cycle were 0.0078, 0.0108, 0.0133 and 0.0161. For axial tensile strains less than or equal to 0.0133, the test column was stable and it was able to fully develop the compressive force associated with the target compressive strain and the out-of-plane displacement was small. For a large axial tensile strain of 0.0161, however, significant out-of-plane displacement developed in the compression half-cycle, leading to column buckling. The stable column response following a tensile strain of 0.0133 can be seen in Figure 3a, while the buckled column after a tensile strain of 0.0161 is shown in Figure 3b. Thus, the tensile strain amplitude must be recognized as an important parameter governing the cyclic stability of reinforced concrete structural walls.

Guided by experimental observations, Chai and Elayer (1999) proposed a phenomenological model for limiting the axial tensile strain in the wall end region to prevent buckling when subjected to reverse cyclic loading. This approach is less conservative than that proposed by Paulay and Priestley (1993). Neither equation is given consideration in any design code except for NZS 3101, which uses a simplified version of that proposed by Paulay and Priestley but it is not based on expected tensile demand.

### Lap Splices in Plastic Hinge Zones

In almost all concrete walls tested under laboratory conditions, the research community has preferred to use no lap splices near the wall base where the plastic hinge is expected to form. This omission is to minimize the influence of lap splices on the wall’s behavior when attempting to study other modes of failure. Aaleti et al. (2013) investigated the use of no splices, mechanical couplers, and conventional lap splices at the base in a series of concrete wall tests as permitted by design codes and found that: (1) A capacity-designed wall with couplers produced comparable lateral load response to a wall with no splices while an equivalent wall with lap splices noticeably underperformed; and (2) The response of the wall with lap splices was marred by strain concentration in the longitudinal reinforcement at the wall-foundation interface as well as by bond deterioration, slippage of reinforcement in the splice during repeated load cycles, and strength degradation (the same observation was also made in Bimschas, 2010; Hannewald et al., 2013).

### Minimum Reinforcement Requirement

As required for other flexural members, structural walls must also be designed with a minimum longitudinal reinforcement. When the minimum reinforcement governs the design, walls are detailed with distributed longitudinal reinforcement along the length and without boundary elements. This
issue becomes critical in regions of low to moderate seismicity, such as Christchurch, where the abundance of load-bearing concrete walls in certain building types can result in sufficient lateral resistance being achieved through a combination of axial load effects and minimum vertical reinforcement. As highlighted by Paulay and Priestley (1992), in addition to satisfying the temperature and shrinkage requirements, this minimum vertical reinforcement should ensure a ductile response for the walls. During seismic loading, lightly reinforced concrete walls are vulnerable to sudden failure resulting from fracture of vertical tension reinforcement following the initiation of the first flexural crack and the concentration of inelastic demand largely at this crack as opposed to distributed cracks. Insufficient vertical reinforcement was attributed to failure of several walls during the 1985 Chilean earthquake (Wood et al. 1991). After analyzing the results of 37 wall tests, Wood (1989) concluded that walls with less than 1% longitudinal reinforcement ratio were susceptible to fracture of reinforcing steel. To date the majority of lightly reinforced walls that have been tested are squat walls (Greifenhagen and Lestuzzi 2005; Hidalgo et al. 2002; Wood 1989), which provide limited knowledge for understanding the behavior of flexural dominant walls with minimum vertical reinforcement.

The minimum required vertical reinforcement in walls has historically been less than the equivalent minimum longitudinal reinforcement in beams. Prior to 2006, the New Zealand Concrete Structures Standard required that walls contain a vertical reinforcement ratio greater than or equal to \( \frac{0.7}{f_y} \), where \( f_y \) is the yield strength of the longitudinal reinforcement in MPa. This equates to \( \rho_v \) in the range of 0.14 – 0.23% depending on the \( f_y \) value. In the 2006 version of the Concrete Standard (NZS 3101), \( 0.25(f'_c)^{0.5}/f_y \) defines the minimum vertical reinforcement in walls, where \( f'_c \) is the specified concrete compressive strength in MPa, and \( f_y \) is the yield strength of the reinforcing steel in MPa. For a 30 MPa concrete strength, this corresponds to \( \rho_v \) between 0.27 and 0.46% depending on the reinforcing steel grade.

The results of moment-curvature analysis conducted by Henry (2013) have highlighted several deficiencies of this approach. This equation was developed for beams with top and bottom layers of reinforcement only and fails to account for the distributed reinforcement in walls, slenderness of wall sections, size effects, aspect ratio and axial loads. Walls designed with minimum vertical reinforcement using the same approach may be vulnerable to sudden failure unless a significant axial load exists.

FIELD OBSERVATIONS

Prior to the 2010/2011 earthquakes, seismic hazard in Christchurch was considered to be moderate; the peak ground acceleration (PGA) of the 500-year elastic design spectrum corresponding to deep soil sites was 0.22g. The first of the Canterbury event, the Darfield earthquake, occurred on September 4, 2010, with a moment magnitude, \( M_w \), of 7.1. With an epicenter approximately 35 km west of Christchurch, this event caused damage primarily to unreinforced masonry buildings in the CBD. A typical sequence of aftershocks followed this event although it was later found that some of these events occurred in smaller faults closer to the city (Hare et al. 2012). The most damaging event of this sequence was the 2011, \( M_w \) 6.3, Christchurch earthquake that occurred on February 22\textsuperscript{nd}, 2011, at a depth of 5 km and a distance of about 10 km from CBD. The duration of strong shaking for the September event was estimated to be 15 seconds and the corresponding value for the February event was about 7 seconds. The spectra from the September event were noted to be comparable to the design spectra, while the February event produced considerably higher spectral accelerations than those expected for a design level earthquake.

The maximum recorded peak accelerations in the Christchurch earthquake were 2.2g and 1.7g in the vertical and horizontal directions, respectively, with horizontal PGAs exceeding 0.7g around the CBD based on four recorded motions within 1.5 km of CBD (McVerry et al. 2012). The elastic spectra corresponding to the recorded ground motions from these sites were about twice the 500-year return period design spectrum and were stronger than the spectra for return periods of 2500 years. Therefore, the buildings in CBD were subjected to high intensity, short-duration horizontal ground motions during the Christchurch earthquake. What is also apparent is that the vertical motion was strong even during the strong horizontal shaking due to the close proximity of the CBD to the earthquake source.
A general field observation in the Christchurch earthquake is that a variety of buildings with concrete walls achieved their life-safety design objectives. Although many buildings showed only minor damage, severe damage and undesirable failures were identified for a number of concrete walls. Given the short duration and relatively small number of excursions with large accelerations, the observed wall damage is likely to have occurred rapidly.

An overview of wall damage in the Christchurch earthquake may be realized from Figure 4, which shows the result of rapid building safety evaluations that was conducted during the national state of emergency immediately following the Christchurch earthquake (Kam et al. 2011). This figure shows three categories of damage distribution as a function of design era. Accordingly, red indicated unsafe to enter, yellow corresponded to restricted entry and green indicated unrestricted entry though a detailed evaluation was still needed. An underlying assumption here is that access to building reflects the extent of damage to concrete walls. In this context what is important to realize is that the modern wall buildings, designed after the 1990s, show approximately two times the red category as the pre-1980 buildings and three times as many as the 1980s. It is also worth noting that as of the writing of this paper about 60% of the multi-story buildings with reinforced concrete walls in the CBD have been demolished, which is likely to have included most of the red and yellow placarded buildings.

Anecdotally, the 1980s walls were designed with two layers each for the vertical and horizontal reinforcement and their typical thickness ranged from 300 to 500 mm. They were often designed with boundary elements with large vertical and confining reinforcement ratios to increase the robustness of the walls for resisting earthquakes. From 1990 onwards, an increasing number of relatively thin (less than 200 mm thick) load bearing walls with one layer of both vertical and horizontal reinforcement have been built without boundary elements. A number of such modern walls performed poorly in the earthquake. Pre-1980, the necessary detailing of the 1980s (in the boundary elements) was not employed and probably accounts for the increasing need for red placards. Typically, these walls used 200-250 mm or greater wall thickness.

Limited Damage

The concrete wall shown in Figure 5 experienced noticeable distress due to the earthquake motion. Three observations from this figure are: (a) The intensity of ground motion at this site was significant enough to cause cracking but not spalling of cover concrete in the plastic hinge region near the wall base; (b) Use of increased longitudinal reinforcement in the end regions of wall seems to have controlled the crack width in these regions; and (c) Relatively wider cracks with large spacing apparent on the wall surface in the web regions confirm the use and possible consequence of lightly reinforced concrete in that region. Figure 6 shows a concrete wall in a 14 story hotel building that had well distributed flexural cracking. As described by Wilson and Lewis (2011), the plastic hinge region in this case was located above level 4 where the building footprint was reduced. The 8 m long and 0.3 m thick wall was designed in accordance with current NZS 3101 standards (2006) and had 1 m long boundary elements with a $\rho_l$ of 2.7% and the web region contained well-distributed longitudinal...
reinforcement with a $\rho_l$ of 1.0%. The resulting crack widths were between 0.5-0.8 mm and the wall was easily repaired.

![Figure 5](image1.jpg) Observed distress to a concrete wall with minimal damage (Photos: Courtesy of Elwood)

![Figure 6](image2.jpg) Well-distributed flexural cracks on a wall (Wilson and Lewis 2011)

Hidden Damage

In ductile concrete walls, a plastic hinge is expected to form at the wall base when subjected to seismic loading. While this is typical of what has been seen in many tests in laboratories around the world (e.g., Figure 1 and Figure 2), formation of significantly fewer cracks in the plastic hinge zones occurred in many walls in Christchurch. Based on the proximity of buildings to the epicenter, the plastic strains in some walls were expected to be large. However, fracture of several longitudinal bars in the wall end regions—as observed in the field—was unexpected. This observation is attributed to the formation of fewer cracks in the plastic hinge than those expected from during typical laboratory tests.

Two examples of this type of wall behavior were observed in multi-story Gallery Apartment building built in 2006 (see Figures 7 and 8). Damage to both walls was characterized by formation of only few flexural cracks in the plastic zone. In the first example (Figure 7b), it appears that the cover concrete spalled off first followed by the buckling of the longitudinal reinforcement. Figure 8 shows the second example, in which a wall with a single flexural crack and fracture of multiple longitudinal bars is seen. This type of damage was particularly concerning due to the fact that the fractured bars were hidden behind what appeared to be relatively minor damage. Subsequent reports prepared by CERC (2012) and Smith and England (2012) highlighted several deficiencies in the Gallery Apartment building including a mismatch in the assumed ductility and wall detailing. Additionally, the walls in the building were designed prior to the introduction of more stringent minimum vertical reinforcement limits in NZS 3101 (2006). The grid-F wall shown in Figure 7 had a total vertical reinforcement ratio of 0.16%; only 55% of the vertical reinforcement required by NZS 3101 (2006). The low vertical reinforcement content combined with measured concrete strengths that were significantly higher than the specified strength likely contributed to the observed lack of flexural cracking.
Significant Damage

A number of walls, including those that appeared to have good reinforcement details, suffered buckling in the plastic zone. As previously discussed, the cyclic combination of large tension strains and subsequent compression can trigger local wall buckling. This issue is exacerbated when walls are designed with boundary elements and lightly reinforced web regions. Figure 9 shows a 7-story reinforced concrete wall structure built in 1984, which comprised of two L-shaped walls. The wall on the north experienced significant damage to concrete just adjacent to a boundary element and out-of-plane buckling of the boundary element. The longitudinal steel ratios in the boundary element and outside of the boundary element were estimated to be about 2% and 0.12%, respectively. While 0.12% is lower than the current minimum vertical reinforcement ratio, this amount is consistent with the minimum requirement of the era when the building was designed.

The lack of damage to the south L wall may be attributed to the building not being as well connected to that wall due to stair and lift shaft penetrating through the diaphragms, adjacent to the wall. In Figure 9d, the back of the south L wall is shown, which shows virtually no damage to this
A similar observation was made on the inside faces of this wall. Likewise, the interior of the building also showed limited damage, including to the non-structural elements.

As seen in Figure 10, failure of lapped splices in reinforcing bars caused significant wall damage in a 13 story apartment built in 1999. This building also used a combination of a long coupled (L = 10 m) and short (L = 3 m) walls in the building configuration, with significant damage occurring only to the long wall. The vertical reinforcement was spliced over part of the wall length with the damage concentrated about the splice. The severely damaged region also had poorly detailed horizontal (or shear) reinforcement and a lack of ties between the two layers of reinforcement in the web region. As shown in Figure 10c, the horizontal reinforcement was terminated with a 90 degree bend that was not anchored into the confined boundary element, and the shear reinforcement was also lapped in the cover concrete, which pulled out when the wall was damaged. Since the lap splice was not in the plastic hinge region, this issue is not further investigated. However, specific failure of walls as in Figure 10 would be worthwhile studying in detail in the future.

Figure 10 Performance of a concrete wall in one building of the Terrace on the Park

OTHER CONTRIBUTING FACTORS

Besides the specific concerns noted above, there were evidence of other design concerns raised based on the damage patterns. Although full treatment of these issues is beyond the scope of the paper, some of these issues that are basic requirements of design codes are summarized for completeness. Not maintaining stiffness regularity by placing walls asymmetrically in a building is suspected to have contributed to wall damage in a number of buildings. This is true for both the Canterbury Television and Pyne Gould Corporation buildings that collapsed and caused the majority of the earthquake causalities. Both buildings used concrete walls in their configuration including core walls. Although neither building was compliant with modern design standards, the asymmetry of the walls was identified as a contributing factor to the building collapse (Jury 2011; Hyland and Smith 2012). Influence of increase in axial loads in walls could have contributed some failure, where the increase could have been due to vertical acceleration and/or due to the interactions with floors and gravity load resisting elements. Evidence of column damage due to axial compression was also evident during field observations.

CONCLUSIONS

Following a noticeably large number of failures of concrete walls that were designed to behave in a ductile manner in the Christchurch earthquake, this paper was dedicated to summarizing the important lessons from field observations and understanding potential causes of wall failures in the context of current state of knowledge. Based on this effort, the following conclusions have been drawn:

1. Whether it is required by design codes or not, concrete walls in seismic regions are often designed with boundary elements containing heavy longitudinal reinforcement ratios and lightly reinforced middle regions. Even if these walls are not susceptible to out-of-plane stability problems, the use of minimal reinforcement in the web will lead to undesirable consequences. It is recommended that distributing a significant portion of the reinforcement along the wall length with proper confinement in the compression zone is expected to improve seismic performance of walls at large displacements and minimize shear deformations.

2. Wall out-of-plane instability resulting from large tensile strains developing in the longitudinal reinforcement at the wall base can be controlled by appropriately choosing the wall thickness. Only the NZS 3101 (2006) uses this concept in deciding the wall thickness, but its simplified
approach makes the minimum wall thickness independent of the maximum expected longitudinal tensile steel strain. It is suggested that wall design should include a minimum wall thickness calculation directly based the maximum expected longitudinal tensile steel strain.

3. Field observations and laboratory tests suggest that lap splices should be avoided in the plastic hinge regions by relocating them to the second story or higher level, or by use of mechanical couplers at the wall base. If splicing of the reinforcement cannot be avoided in the plastic hinge region, NZS suggests the splices should be staggered, but this detail requires experimental verification.

4. Walls designed with current-code based minimum vertical reinforcement may not behave in a ductile manner. An appropriate amount needs to be established taking into account realistic concrete strengths, dependency of tensile strength on wall length and other influencing parameters.

While the conclusions noted above are based on the field observations and findings from completed research, further research may be required to formulate appropriate guidelines suitable for improving provisions in the seismic design codes.

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