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ADDRESSING BOUNDARY DESIGN FOR REINFORCED CONCRETE WALLS, BASED ON STUDIES OF THE 2010 CHILE EARTHQUAKE

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Abstract

The 2010 Maule Chile Earthquake caused damage to several mid-rise and high-rise concrete wall buildings. Based on observations by reconnaissance teams and similarities between U.S. and Chilean seismic design and construction practices, the National Institute of Standards and Technology recognized the unique opportunity to investigate the observed performance of buildings subjected to strong ground shaking, and to identify lessons for improving design and construction of reinforced concrete buildings.

The project was conducted by the Applied Technology Council (ATC-94 project) with participation from engineers and researchers from the United States and Chile. Project objectives are to: (1) evaluate critical issues in the design of reinforced concrete walls; and (2) develop recommendations for improving wall design requirements.

Findings address:

- Unintended coupling of walls (such as through slabs and beams)
- Buckling of longitudinal (vertical) reinforcement and the extent of "buckling-restraint" ties needed over the length of the wall
- Area of concrete (thickness of wall) and amount of hoops and cross-ties needed in cases with high compression demand
- Overall wall buckling
- Damage to configurations with solid wall below (or above) a series of vertically-aligned openings, and configurations with "flag-shaped" walls where the length of wall is reduced at the lower stories compared to the upper stories
- Pier-spandrel systems and capacity design for strong-pier weak-spandrel behavior

Keywords: concrete; earthquake; wall; buckling; Chile

1. Introduction

In the 2010 Maule Chile earthquake, buildings generally performed well, but some types of buildings, including mid-rise and high-rise concrete wall buildings suffered damage. This paper describes studies undertaken by the Applied Technology Council as part of the ATC-94 project [1] to learn from the observed damage.

The earthquake provided a unique opportunity for studying damage to modern, engineered buildings subjected to a strong earthquake. At magnitude 8.8, the earthquake was one of the seven largest earthquakes in recorded history, with peak ground accelerations exceeding the elastic design response spectra from the Chilean building code in several cases. Structural design drawings existed for many of the affected buildings, which provided valuable information for understanding and learning from the observed earthquake damage.

Based on the damage observations provided by reconnaissance teams in Chile and known similarities between U.S. and Chilean seismic design and construction practices, the U.S. National Institute of Standards and Technology (NIST) recognized a unique opportunity to investigate the observed performance of buildings subjected to strong ground shaking, and to identify lessons for improving design and construction of reinforced concrete buildings that may be subjected to future earthquakes.

The project was conducted by the Applied Technology Council with participation from engineers and researchers from both the United States and Chile. Project objectives are to: (1) evaluate critical issues in the design of reinforced concrete shear walls; and (2) develop recommendations for improving shear wall design and construction practice in the United States.

2. Types of Buildings Studied

The ATC-94 project studied mid-rise and high-rise concrete wall buildings that were damaged in the subject earthquake. At the time of the earthquake, most mid-rise and high-rise buildings in the affected Chilean cities were constructed with reinforced concrete structural walls as the seismic-force-resisting system. The typical configuration of concrete walls is different for office buildings compared to residential buildings.

Mid- and high-rise office buildings in Chile tend to have thicker concrete walls at the building core, around elevators stairs, and services. Outside of the service core area, offices tend to have open floor plans, with moment resisting frames at the building perimeter.

Mid- and high-rise residential buildings in Chile tend to have thinner concrete walls located as partitions between living units. Walls tend to run longitudinally along the central hallway, and transversely from the hallway to the building perimeter. The longitudinal and transverse walls were generally cast monolithically creating L- and T-shaped walls in the transverse direction. The walls resist gravity and lateral forces. At ground level and below, transverse walls often include setbacks (reduced wall length near the building perimeter) to accommodate drive-aisles for parking [2].

Post-earthquake reconnaissance teams reported that walls observed in apartment buildings were typically 200mm thick for buildings up to 16 stories and 250mm thick for buildings up to 25 stories [3]. In some cases thicker walls were observed, and in other cases walls as thin as 150mm were observed.

The twelve buildings studied in this project generally follow the typologies described above. Most of the studied buildings are residential buildings because of the prevalence of damage to this type of building configuration (Fig. 1). The studies also examine significant damage that occurred in an office building with plan irregularity and a perimeter pier-spandrel system.

Fig. 1 – Damage to concrete walls at the ground level of a residential building. (photo by Patricio Bonnelli)

3. Observed Damage

Observed damage included:

- damage from coupling of walls through slabs and other elements
- buckling of longitudinal (vertical) wall reinforcement
- crushing of concrete walls
- out-of-plane buckling of walls
- damage concentrated at discontinuities
- damage to concrete pier-spandrel systems

Following sections of this paper describe studies and findings related to each of these damage observations.

4. Unintended Coupling of Walls

Structural elements spanning between concrete walls, such as floor slabs and beams, suffered damage and in some cases caused damage to concrete walls.

Earthquake damage can occur in slabs and beams (Fig. 2) as a result of vertical displacement at the ends of nearby walls. When the wall boundary on one side of the doorway is in tension (upward vertical displacement), the wall boundary on the other side of the doorway tends to be in compression. This induces flexural deformations in the slab or beam, which can cause large plastic rotation demands, particularly for slabs or beams that have short span length.

Damage can occur in walls as a result of coupling that increases shear and axial demands on the walls. Fig. 3 shows how shear demands for an individual wall can be greater for the case with beams or slabs, compared to calculated demands if the gravity framing is ignored. Fig. 4 shows a wall with bar buckling from tension and compression axial loads that result from coupling with another wall on the same line.

As discussed in subsequent sections, the specific type of damage to walls from unintended coupling can vary (shear, bar buckling, concrete crushing, overall wall buckling), depending on the particular demands and reinforcement.

Potential improvements to design practice include:

- Account for coupling from gravity beams and slabs in the seismic analysis and design, even though such elements are not customarily designated as part of the seismic force-resisting system. When such coupling is considered, in some cases it may result in greater tension and compression strain demand, greater shear reinforcement, different flexural reinforcing pattern, and/or greater amounts and better detailing of transverse reinforcement ties to resist bar buckling and/or improve confinement.

Fig. 2 – Slab and beam damage from wall coupling. Flexural deformations in the walls cause differential vertical deformations over a short span between ends of nearby walls. (photos by Joe Maffei)

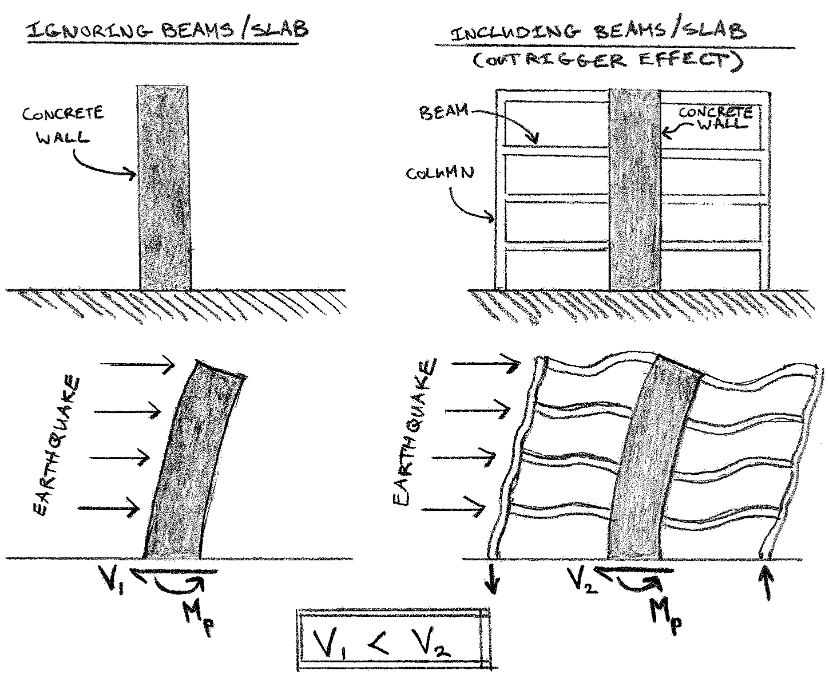

Fig. 3 – Effect of gravity framing on wall shear demand. (image by Karl Telleen)

Fig. 4 – Basement-level wall damage from coupling with another wall on the same line (right side of the photo, not fully visible). Coupling caused increased axial tension and compression in the walls. (photo by Joe Maffei)

5. Buckling of Longitudinal (Vertical) Reinforcement

Vertical reinforcement in walls was observed to buckle, along with spalling of concrete cover over the buckled section of reinforcement. Bar buckling and concrete spalling occurred not only at the ends of walls, but also along much of the length of the wall (Fig. 5). Bar buckling is an undesirable behavior because it can lead to irreparable damage. When vertical bars buckle throughout a wall cross-section, the wall can shorten, putting floors out of level. Shortening of up to 40 cm occurred in some walls in the subject earthquake.

Buckling of a vertical reinforcement bar occurs when the bar has insufficient lateral restraint during a load cycle in which it is subjected to compressive strain. It is more likely to occur in bars where there are no cross-ties (transverse reinforcement perpendicular to the face of the wall) to restrain the bar from buckling, or where the diameter of the vertical bar is small compared to the vertical spacing of cross-ties, or when the cover concrete has spalled off. Bars are more susceptible to buckling if the compression demand follows previous cycles of plastic tension strain.

Bar buckling is also more likely to occur in walls whose configuration causes large strain demand. For example, for a given roof displacement demand, longer walls or coupled walls (Fig. 6) have greater vertical strain demand than walls that have smaller length-to-height ratio. Walls with L-shape or T-shape cross-section (Fig. 7) tend to have greater strain demand at the non-flanged end and along the length of the wall because the neutral axis is located closer to the flanged end [4] in comparison to a rectangular wall of the same length and axial stress demand.

Potential improvements to design practice include:

- Provide cross-ties at each vertical bar where plastic tensile strain is expected.
- Ties should be spaced vertically no further than six times the diameter of the vertical bar being restrained.
- The horizontal extent of ties should consider bar tensile strains throughout the cross-section, rather than just the compression neutral axis depth. This requires that in plastic hinge regions of walls, “buckling-restraint” cross-ties are provided over a greater horizontal extent than the length of “special boundary elements” (which provide confinement for compression). Outside of special boundary elements, buckling-restraint cross-ties are needed only to prevent vertical bars from buckling, so they do not necessarily need to meet the same spacing and area requirements as ties in the special boundary elements.

Fig. 5 – Buckling of vertical reinforcement in a concrete wall. (photo by Joe Maffei)

Fig. 6 – Buckling of vertical reinforcement in a concrete wall. Coupling with another wall on the same line increases axial strain demand. (photo by Joe Maffei)

Fig. 7 – Buckling of reinforcement and crushing of concrete in a concrete wall. T-shaped cross-section increases strain demand at the non-flanged end and along the length of the wall. (photo by Joe Maffei)

6. Crushing of Concrete Initiated at Wall Boundaries

Concrete crushing was observed over the length of walls, initiated at the ends, often in conjunction with buckling of reinforcement (Fig. 1, Fig. 7, Fig. 8). In many cases, it is difficult to distinguish whether damage is initiated by bar buckling (from plastic tension strain followed by compression, as described in the previous section) or by concrete crushing (from compression stress). (See [5] for further discussion.) Each can lead to the other, and in some ways, approaches for mitigation are similar. If the area and/or confinement of concrete is inadequate to resist compression stress demand, bars are likely to buckle when concrete crushes and/or ties fracture. If bars are not restrained against buckling, they provide limited concrete confinement and can reduce concrete area by spalling cover concrete.

Similar to bar buckling, concrete crushing is more likely to occur in walls with large strain demand, such as L-shaped and T-shaped walls and coupled walls, as described in the previous section. Concrete crushing in lightly confined or unconfined concrete is initiated when the compressive strain demand (from combined gravity and seismic actions) exceeds the strain at which concrete strength is maximum. Typically, this strain limit is first exceeded at an end of the wall. When strain demand exceeds this limit, the resistance of concrete at that location reduces, requiring load to be shed to the adjacent portion of the wall. The adjacent portion of the wall then also experiences excessive strain demand, and crushing damage propagates along the length of the wall. The resulting damage is localized in a short height of the wall; it is not distributed vertically as typically expected in a ductile flexural hinge.

The damage observed in the subject earthquake typically occurred in walls with essentially unconfined concrete (having few or no hoops or cross-ties), where special boundary elements would have been required according to ACI 318 [6]. In some cases, the area of vertical reinforcement was also relatively small, and the compression reinforcement would make only a modest contribution to compressive resistance, even if it had not buckled. Providing cross-ties to prevent bar buckling would have led to improved performance in some cases. Providing greater area of concrete (greater wall thickness) and/or hoops and cross-ties meeting the spacing and area requirements of “special boundary elements” would have further improved performance in cases with high compression demand.

Fig. 8 – Crushing of concrete and buckling of reinforcement at the end of a wall. (photo by Joe Maffei)

7. Out-of-Plane Buckling of Walls

In some instances, concrete walls were observed to be buckled out-of-plane over a portion of the wall length and height (Fig. 9). Prior to the subject earthquake, this type of damage had been observed in experimental tests, but had not been reported in an earthquake. Instances of wall buckling were also reported in the 2011 Christchurch Earthquake. Like bar buckling, it is an undesirable behavior because it can lead to irreparable damage.

Like bar buckling, wall buckling is influenced by plastic tension strain, followed by compression. The slenderness of the wall (thickness compared to buckling height) can be related to the maximum tensile strain that the wall can undergo without buckling instability upon load reversal into compression.

While wall buckling was at first thought to be a more prevalent issue in Chile based on post-earthquake reconnaissance, studies of selected buildings suggest that other behaviors, such as bar buckling and concrete crushing, initiated much of the damage initially considered as wall buckling. This assessment is based on studies of several of the buildings in Chile where wall buckling was reported. For these buildings, the tensile strain expected to initiate wall buckling was found to be greater than the strain demand corresponding to the estimated roof drift in the earthquake, whereas compression demand exceeded that which would cause concrete crushing. Walls were found to be much more susceptible to buckling after concrete cover had spalled. Whether initiated by tensile strain or increased slenderness from concrete spalling, neither behavior mode of wall buckling is desirable compared to stable flexural yielding.

Potential improvements to design practice include:

- In regions of potential plastic hinging, provide a lower-bound limit for wall thickness at the ends of walls, as a function of the unsupported wall height (between floors). The Uniform Building Code [7] required minimum wall thickness of 1/16 the unsupported wall height, and similar provisions have been added to ACI 318-14. The minimum wall thickness can be provided as a flange or a thickened section at the end of the wall. The thickness requirement could also depend on variables such as unbraced wall length, axial load, neutral axis depth, or expected strain demand. Provisions for alternative analysis methods, such as based on [8] could justify thinner walls in instances where this requirement may be overly restrictive.

Fig. 9 – Out-of-plane wall buckling. (a) 2010 Chile Earthquake (photo by Jack Moehle). (b) 2011 Christchurch Earthquake [9]. (c) Experiment [10].

8. Discontinuities and Stress Concentrations

Much of the observed damage is concentrated at structural discontinuities. In particular, several instances of damage are associated with the following wall configurations:

- Solid wall below (or above) a series of vertically-aligned openings (Fig. 10).
- "Flag-shaped" walls, where the length of wall is reduced at the lower stories compared to the upper stories, as described in Section 2 for residential towers (Fig. 1).

As shown in Fig. 10, when two wall piers land on a basement wall, lateral earthquake forces can cause a stress concentration at the basement wall because the tension end of one wall pier is close to the compression end of the other pier. The stress concentration is greater if the wall piers are close together, and it is less if the piers are further apart. Similar stress concentrations can occur if wall piers are joined at other levels by an element (such as a solid wall at the penthouse level) that is significantly stiffer than the floor slabs or coupling beams that join the wall piers at typical levels.

In "flag-shaped" walls (Fig. 1), earthquake damage is likely to concentrate in the stories with reduced wall length because flexural, shear, and axial loads must be resisted by a reduced cross-section of wall. Analytical studies (Fig. 11) indicate that for walls with essentially unconfined concrete (having few or no hoops or cross-ties), exceedance of the strain at which concrete stress is maximum can initiate spalling and crushing of concrete at the end of the wall and rapid propagation of strength-loss across the wall section. Such progressive damage would not be captured by analyses that assume plane sections remain plane. Nonlinear models that capture such behavior showed good agreement with the apparent failure mechanisms observed in an example building that collapsed in the earthquake.

Potential improvements to design practice include:

- If possible, avoid configurations with solid wall below (or above) vertically-aligned openings (Fig. 10), and "flag-shaped" walls (Fig. 1).
- For solid walls below openings (Fig. 10), extend vertical reinforcement from the wall piers above one or more stories into the solid wall below. Design the solid wall below for panel zone shear forces.
- For flag-shaped walls (and other configurations that have the potential to concentrate compression strain), adjust analysis assumptions or limits on permissible concrete compression strain. Provide adequate area of concrete (thickness of wall) to maintain low axial stress. Provide reinforcement to restrain crack opening at re-entrant corners, and special boundary element detailing at wall boundaries where yielding and/or high compression stress is likely to occur.

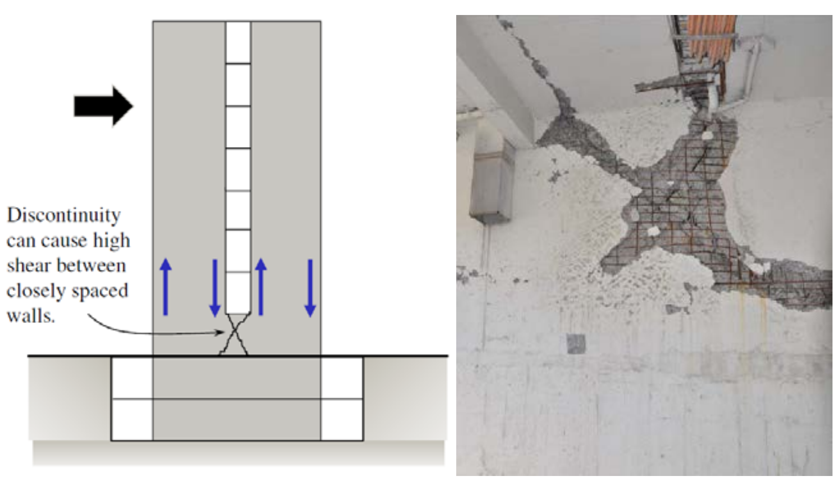

Fig. 10 – Damage from stress concentration at a solid wall below a series of vertically-aligned openings. (image by Karl Telleen, photo by EERI team)

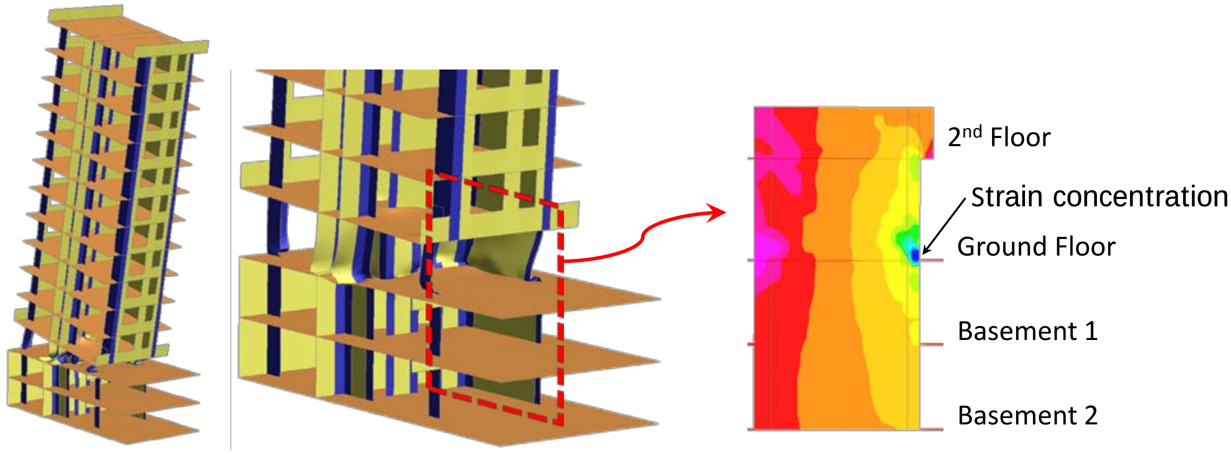

Fig. 11 – Nonlinear finite element analysis of a portion of a residential building that collapsed in the 2010 Chile Eartquake. (images by Michael Willford and Yuli Huang)

9. Pier-Spandrel Systems

The office building shown in Fig. 12 experienced damage related to several structural discontinuities and plan irregularities. In addition, perimeter concrete walls with punched openings exhibited strong-spandrel weak-pier behavior that contributed to a partial story collapse. Current requirements of ACI 318 have provisions to encourage strong-column weak-beam behavior in moment frames, but similar requirements do not exist for pier-spandrel systems. Plastic mechanism analysis of a portion of the damaged building (Fig. 13) indicates that the observed behavior of the pier-spandrel system is predictable using hand calculations.

Potential improvements to design practice include:

- Evaluate the strength hierarchy and expected behavior of pier-spandrel systems, and design to achieve strong-pier weak-spandrel behavior.

Fig. 12 – Office building with shear damage to wall piers, including partial story collapse. (a) photo by Joe Maffei, (b) photo by EERI team.

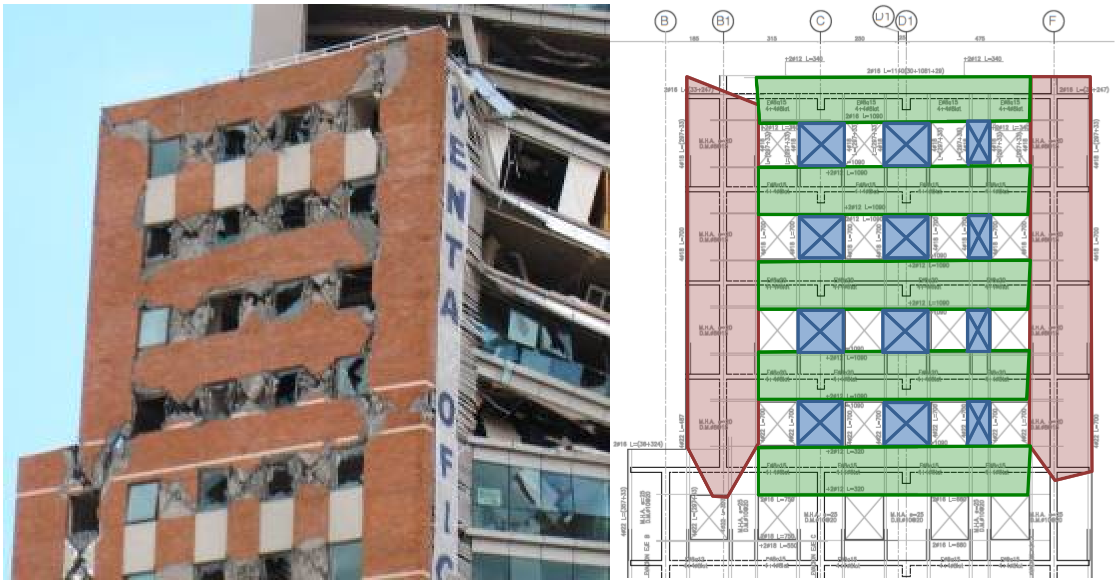

Fig. 13 – Plastic mechanism analysis of the pier-spandrel system indicates that the behavior observed in (a) is predicable using hand calculations to compare pier strength vs. spandrel strength as indicated in (b). (photo by Joe Maffei, image by Ady Aviram)

10. Conclusions

Damage to mid-rise and high-rise concrete wall buildings in the 2010 Chile Earthquake provides valuable lessons for improving design of concrete structures. Characteristic damage described in this paper includes:

- damage from coupling of walls through slabs and other elements
- buckling of longitudinal (vertical) wall reinforcement
- crushing of concrete walls
- out-of-plane buckling of walls
- damage concentrated at discontinuities
- damage to concrete pier-spandrel systems

The "potential improvements to design practice" listed at the end of Sections 4 through 9 are based on observations of damage to residential and office buildings in Chile, but the concepts are also applicable to concrete wall buildings in the United States.

11. Acknowledgements

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12. References

[1] NIST (2014): *Recommendations for Seismic Design of Reinforced Concrete Wall Buildings Based on Studies of the 2010 Maule, Chile Earthquake*, Report NIST GCR 14-917-25, Prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium for Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Maryland.

[2] Lagos R and Kupfer M (2012): Performance of High Rise Buildings Under the February 27th 2010 Chilean Earthquake. *International Symposium on Engineering Lessons Learned from the 2011 Great East Japan Earthquake* 2012, 1: 1754-1765.

[3] Cowan H, Beattie G, Hill K, Evans N, McGhie C, Gibson G, Lawrence G, Hamilton J, Allan P, Bryant M, Davis M, Hyland C, Oyarzo-Vera C, Quintana-Gallo P, and Smith P (2011): The M8.8 Chile Earthquake, 27 February 2010. *New Zealand Society of Earthquake Engineering Bulletin*, 44(3).

[4] Paulay T and Priestley MJN (1992): *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley & Sons, New York.

[5] Telleen K, Maffei J, Heintz J, and Dragovich J (2012): Practical Lessons for Concrete Wall Design, Based on Studies of the 2010 Chile Earthquake. *15th World Conference on Earthquake Engineering*, Lisbon, Portugal.

[6] ACI (2014): *ACI 318-14: Building Code Requirements for Structural Concrete and Commentary*. American Concrete Institute: Farmington Hills, Michigan.

[7] ICBO (1997): *Uniform Building Code*. International Council of Building Officials: Whittier, California.

[8] Paulay T and Priestley MJN (1993): Stability of Ductile Structural Walls. *ACI Structural Journal* 1993; 90(4): 385-392.

[9] Kam WY, Pampanin S, and Elwood K (2011): Seismic Performance of Reinforced Concrete Buildings in the 22 February Christchurch (Lyttelton) Earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering*, 44(4): 239-278.

[10] Goodsir WJ (1985): *The Design of Coupled Frame-Wall Structures for Seismic Actions*, Research Report 85-8. Department of Civil engineering, University of Canterbury: Christchurch, New Zealand.