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Performance of tall buildings in Santiago, Chile during the 27 February 2010 offshore Maule, Chile earthquake

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SUMMARY

There are many modern tall buildings in Santiago that were subjected to the 27 February 2010 earthquake in Chile. Although there was not widespread damage in Santiago, there was notable damage to some tall concrete buildings that may have resulted from lack of proper detailing, the absence of 135° seismic hooks and inadequate confinement of walls in the boundary zones. These caused buckling of the main bars and tension compression failure of the walls. Copyright © 2010 John Wiley & Sons, Ltd.

1. INTRODUCTION

This paper presents an overview of observations of the Los Angeles Tall Buildings Structural Design (LATBSDC) reconnaissance team on performance of tall buildings in Santiago during the 27 February 2010 offshore Maule, Chile earthquake. Our team visited many buildings and shot thousands of photographs. We also obtained structural drawings for some of the buildings that were visited. This paper concentrates on providing a general understanding of tall building performance in Santiago by providing examples of buildings that performed well in addition to those that did not.

2. LOCATION AND RECORDED EARTHQUAKE GROUND MOTIONS

Santiago, the capital and the most populous city in Chile is located 335 km (210 mi) to the north-east of the epicentre of this earthquake. Given the large distance to the epicentre, it is common for those unfamiliar with the earthquake fault rupture process to express astonishment at the mere fact that there was some damage in Santiago due to this event. However, during a large magnitude earthquake such as this one, the fault rupture length is several hundred kilometres and the zone of energy release constitutes a very large area (see Figure 1). The proximity to the zone of energy release is much more important than the distance to the epicentre, and Santiago is located much closer to this zone than a cursory review of the epicentral distance would indicate.

The same effect amplified by site soil conditions may be observed by examining the preliminary ground shaking contours released by the United States Geological Survey (2010), as shown in Figure 2, whereas areas near the west coast of Santiago and south of Chillan show significantly larger accelerations than areas within the close proximity to the epicentre. The earthquake accelerograms obtained

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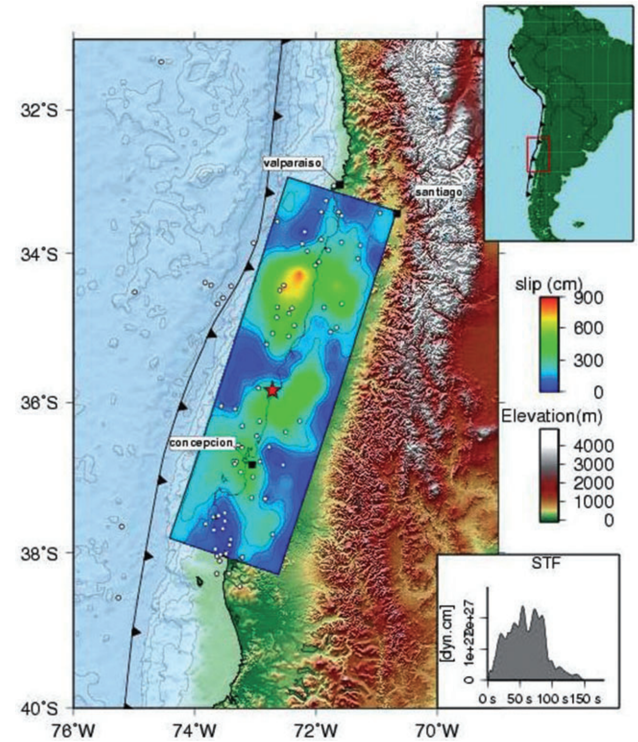

Figure 1. Fault rupture area associated with the 27 February 2010 Earthquake. Note the relative proximity of Santiago to the top corner of this zone compared to its distance to the epicentre, which is marked by a star within the zone (Sladen, 2010).

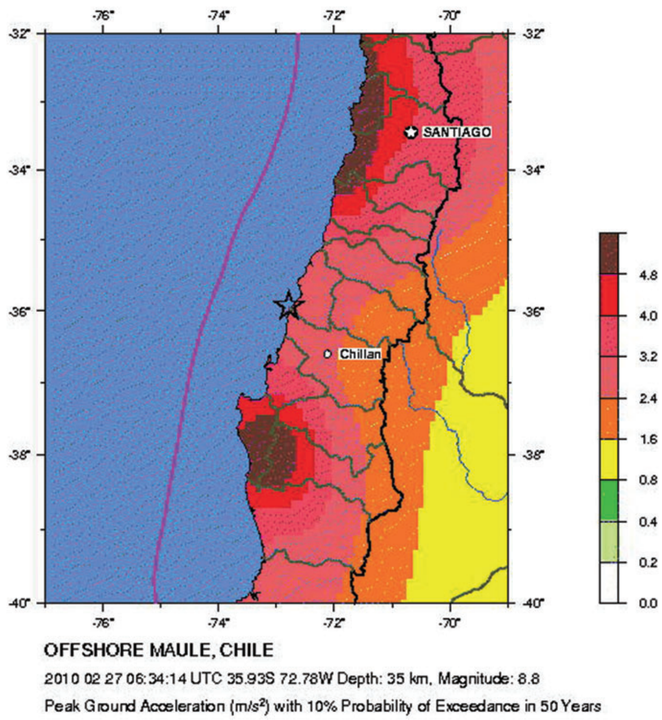

Figure 2. Estimated peak ground accelerations due to the 27 February 2010 Earthquake (United States Geological Survey, 2010). (a) Three components of accelerogram. (b) 5% damped response spectra of the three components shown in (a) compared to Santiago Design Spectrum from NCh433 (shown in black).

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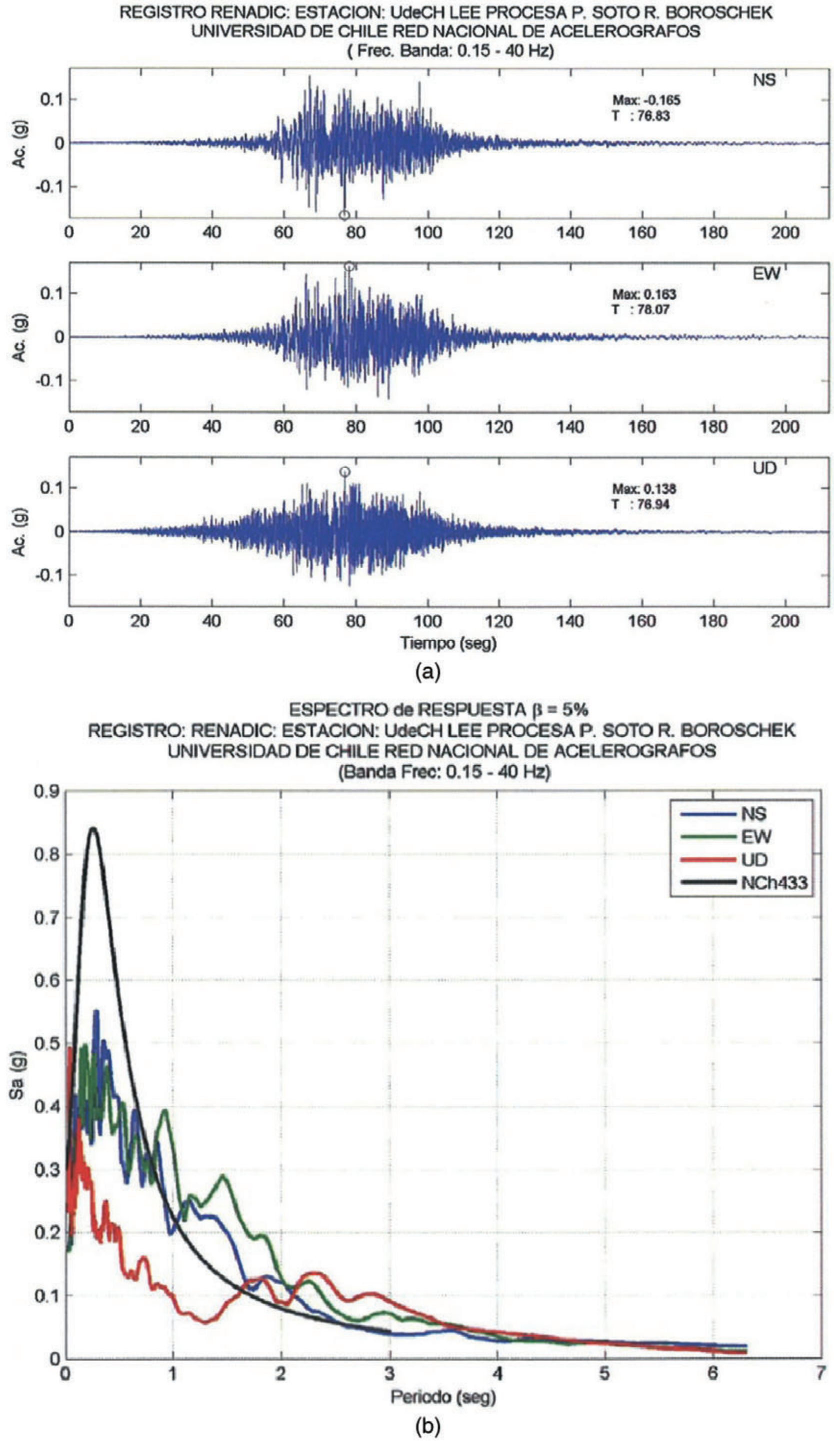

Figure 3. Accelerograms and corresponding response spectra obtained at the University of Chile, Santiago campus (Boroschek *et al*., 2010). (a) Three components of accelerogram. (b) 5% damped response spectra of the three components shown in (a) compared to Santiago Design Spectrum from NCh433 (shown in black).

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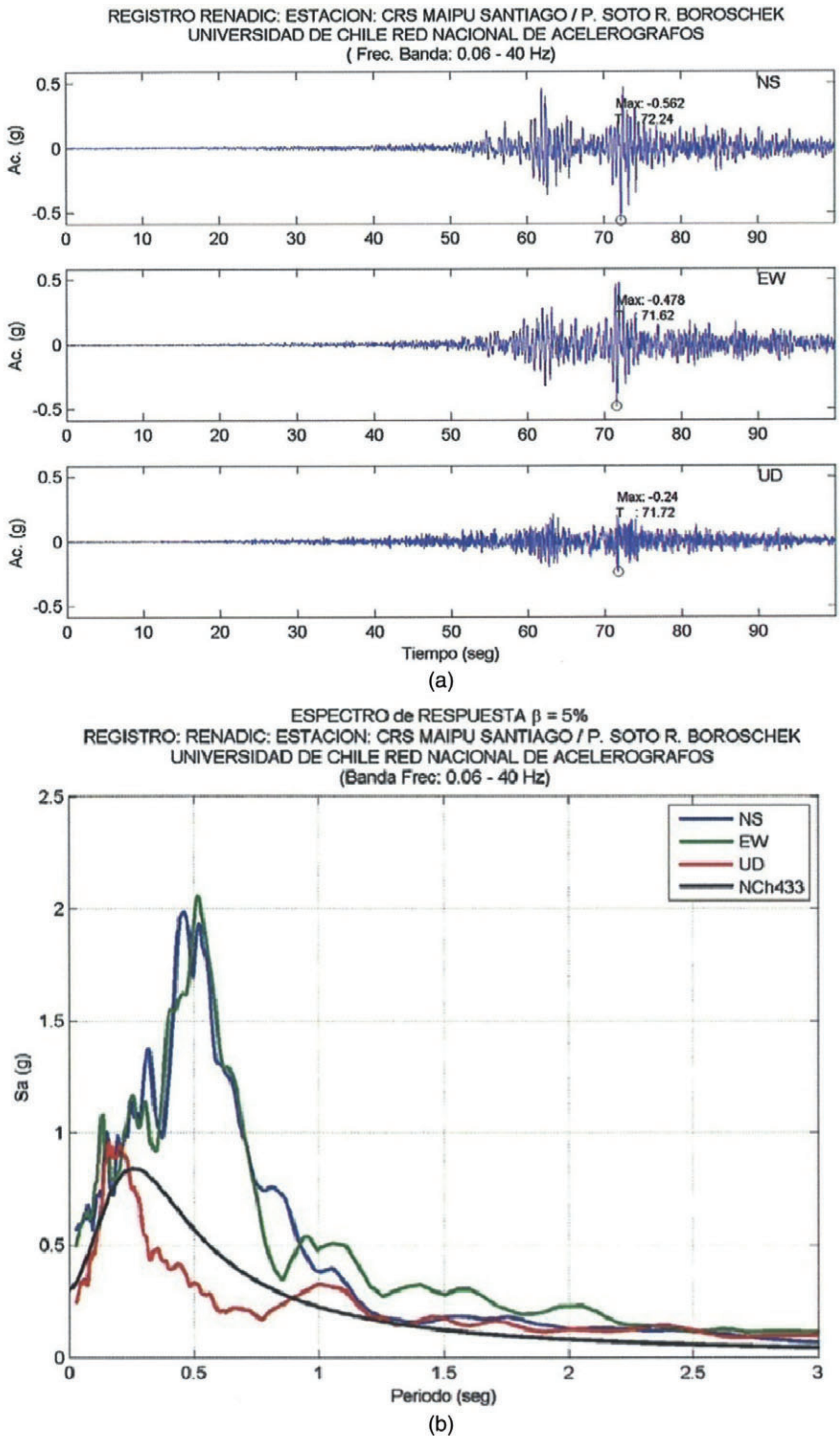

Figure 4. Accelerograms and corresponding response spectra obtained at the Maipú region of Santiago (Boroschek *et al*., 2010).

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from two sites within the city of Santiago exhibit long duration ground shaking and peak ground accelerations that vary from 0.10 *g* to about 0.50 *g* (Figures 3 and 4). What is more significant is that for a relatively wide range of vibration periods (1 to 3 s in the case of the University of Chile site and 0.3 to 3.0 s in the case of the Maipú site), the recorded motions substantially exceed the design spectrum of Santiago from the NCh433 Chilean code (National Institute of Normalization, 1996). This is particularly significant in the case of the Maipú site, as indicated in Figure 4b. Therefore, it is not astonishing that there were pockets of damage in Santiago in the aftermath of this earthquake.

3. PERFORMANCE OF TALL BUILDINGS

3.1. General observations

The Santiago skyline is filled with modern tall buildings the great majority of which performed very well during this earthquake and did not suffer any structural damage (Figure 5). This is noteworthy because this paper and most other papers dedicated to documenting damage due to the earthquake concentrate on the performance of damaged buildings, and the reader may be left with the impression that the damages reported were widespread throughout the city. In the case of Santiago, this was definitely not the case as our team had to hunt for damaged buildings, and an ordinary visitor to the city who was not aware of the location of damaged building may have wrongfully concluded that there were no significantly damaged buildings in the city.

We will begin our review of performance of tall buildings with a look at two buildings that were not damaged, and then we proceed with review of the buildings that did suffer structural damage. Locations of damaged buildings visited by our team are superimposed on the map of Santiago presented in Figure 6.

Two types of damages were observed repeatedly in the damaged buildings inspected by our team in Santiago, as well as other cities. First, the predominant mode of failure appeared to be the tension-compression failure of the shear walls at the ground floor or the floor immediately below the ground level (see Figure 7a), and second and closely related, the tearing of main bars at the extreme ends of walls at the location of failure, particularly when the main bars were bent (see Figure 7b). It is possible that the tearing of these bars is related to low-cycle fatigue caused by the numerous cycles of loading and unloading caused by the long duration of the strong ground motion during this earthquake.

Figure 5. The great majority of tall buildings in Santiago did not suffer any structural damage.

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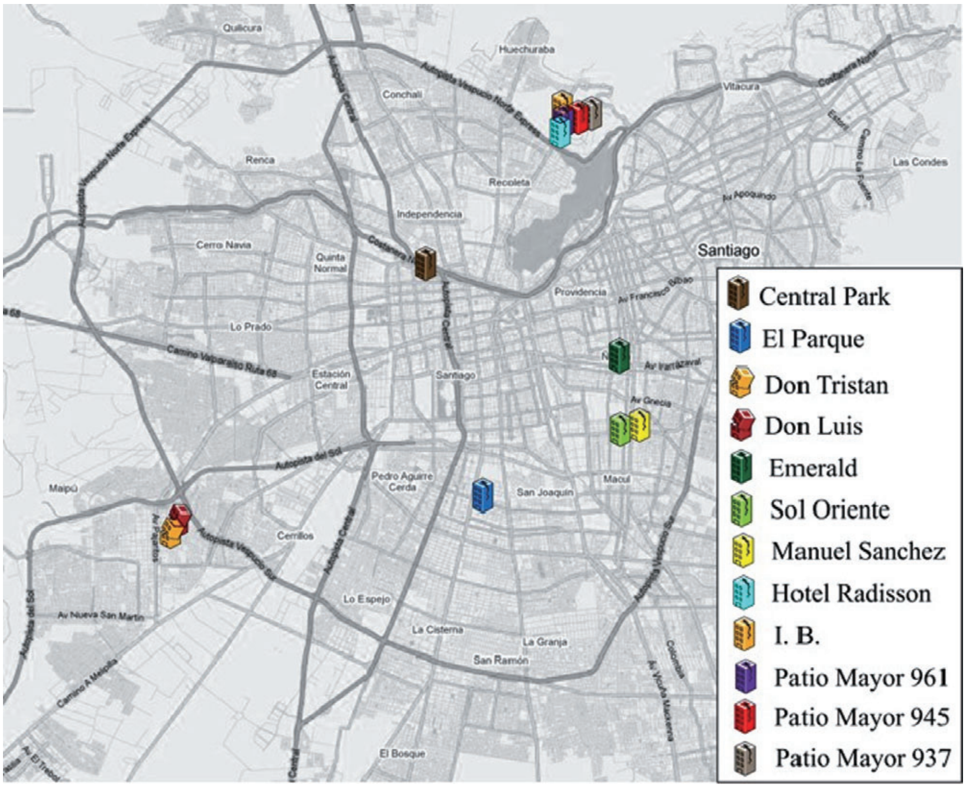

Figure 6. Location of damaged buildings in Santiago visited by our team. (a) Typical tension-compression failure of the wall (El Parque building). (b) Tearing of main bars at the wall failure zone near the end of the wall; transverse ties typically had only 90° hooks (Sol Oriente building). (c) Lack or shortage of transverse reinforcement and/or inadequate confinement (Empresarial Complex).

(a) (b) (c)

Figure 7. Typical failures.

3.2. *The Titanium Tower*

The almost completed Titanium Tower with 52 storeys above the ground and 7 subterranean floors rises 181 m above the ground level and is currently the tallest building on the South American continent. This distinction, however, will be soon claimed by the 70-storey Gran Costanera Tower (300 m) currently under construction a few blocks away in Santiago. The Titanium Tower did not suffer any

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structural damage. The non-structural damage was limited to partial separation of a cladding piece at about 40 floors above the ground (Figure 8). The structural system of the tower consists of precast floor panels with cast-in-place topping and concrete columns and walls (Figure 9). The lateral system is augmented by a passive energy dissipating device located at the two narrow ends of the building and connected to the floors above and below with large pipe section braces (Figure 10). No damage to the basement walls and subterranean floors were observed in any of the seven subterranean floors (Figure 11).

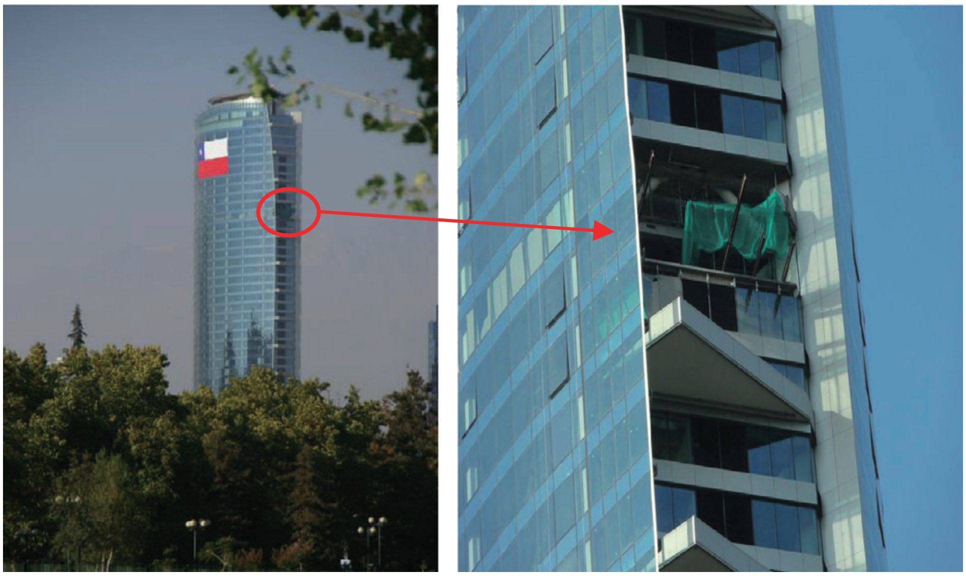

Figure 8. The Titanium Tower, local cladding damage.

Figure 9. The precast columns and floor system implemented at the Titanium Tower. (a) Connection of damping brace to the floor above. (b) Damping system at the middle of braces. (c) Passing of the damping brace through intermediate floor.

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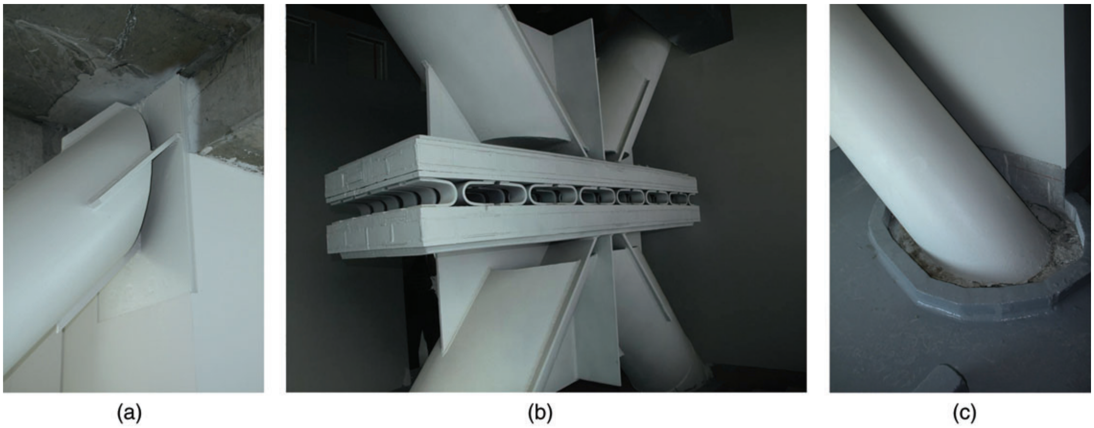

Figure 10. The precast system and passive energy dissipating system implemented in Titanium Tower.

Figure 11. No damage occurred at any of the seven subterranean floors or the basement walls in the Titanium Tower.

3.3. The Parque Araucano building

The Parque Araucano has 22 floors above ground and 6 subterranean floors. The lateral system of the building consists of a core reinforced concrete shear wall system. In order to reduce the lateral displacements caused by earthquakes, two innovative tuned mass dampers (TMDs) were installed on the 21st floor (Figure 12).

The TMD units were constructed as multi-cell reinforced concrete boxes on the 21st floor, jacked up to the desired elevation and hung from the roof above with a series of bolted plates and then filled with metal balls to attain the desired mass. The system performed very well during the earthquake and the building did not suffer any damages above the ground or at any of the six subterranean floors and basement walls.

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(a) (b) (c) (d) (e)

Figure 12. The Parque Araucano building and its TMD systems. (a) Post-earthquake view from outside. (b) Computer model with TMDs shown in red. (c) TMD box during construction. (d) Size of metal balls filling the TMD cavities. (e) Completed and filled TMD hanging from the roof above.

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3.4. The Echeverria Izquierdo building

This 22-storey building having 9 subterranean floor levels was under construction when the earthquake occurred. The building consists of two separate wings that were interconnected by a series of bridges at several floors (Figure 13).

This is a reinforced concrete building with a post-tensioned system deployed for the bridges connecting the two wings. Although a large gap and isolation bearings where placed to protect the bridge from the adverse effects of the relative movement of the two wings (Figure 14), it appears that the likelihood of buildings moving towards each other was not properly contemplated either during design or implementation. As a result, during the earthquake, there was relative movement of the wings and damage was observed at the bridge–wing interfaces (Figure 15).

Inspection of the subterranean levels below grade found no evidence of any damage to the lower floors or the deep basement walls as shown in Figure 16.

Figure 13. The Echeverria Izquierdo Building.

(a) (b)

Figure 14. Bridge Details. (a) Separation gap at the bridge-wing junction. (b) Isolation bearing supporting the bridge truss.

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Figure 15. Damage to bridge due to relative movement of the wings.

Figure 16. A photo of the Echeverria Izquierdo building at the ninth subterranean floor where no damage was observed to basement walls or floors.

3.5. The Central Park building

The Central Park building is a 19-storey reinforced concrete shear wall building. Above the ground level, the building suffered tension-compression failure of one of the exterior walls, a column and a transverse wall, all at the second floor above the ground (see Figure 17). The significant buckling of the transverse wall main bars revealed lack of seismic hoops, small diameter of the transverse reinforcement and the large distances between transverse reinforcement bars provided. The transverse reinforcement typically only had 90° hooks. In combination, this apparently caused the transverse reinforcement incapable of providing adequate bracing for the main bars, resulting in their buckling.

The damage observed in the subterranean parking areas (Figure 18) was very typical of the damage observed in many buildings in Chile. That is tension-compression failure of the wall, which was probably instigated by high axial forces at the extreme ends of the wall, where lack of adequate bracing for the main bars and confinement apparently caused failure in this zone, which then propagated along the length of the wall.

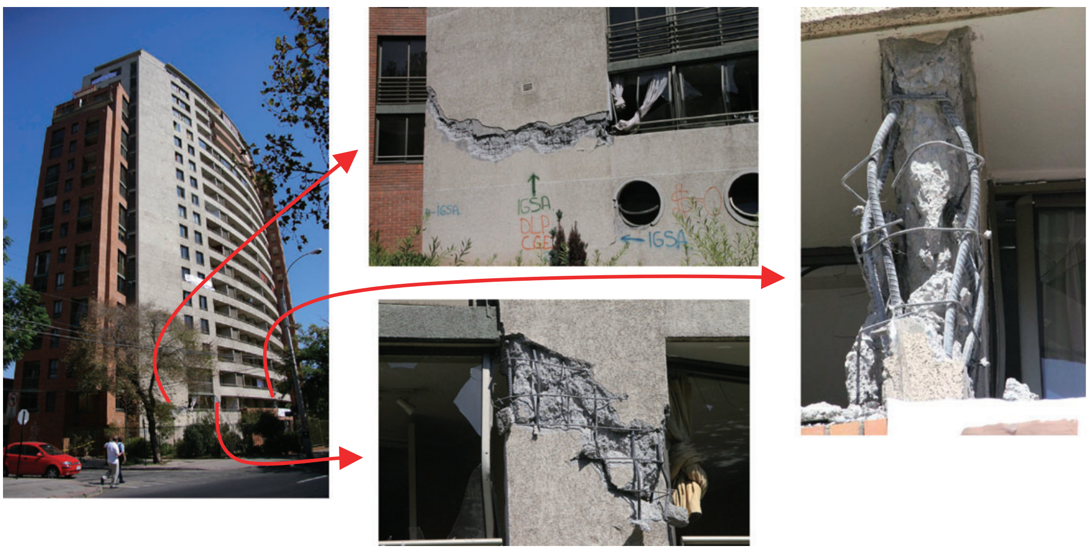

Figure 17. Aboveground damage.

Figure 18. Typical below ground damage at the Central Park building.

3.6. The Emerald Building

There are two 16-storey buildings on this site with similar general geometry. One of the buildings was significantly damaged while the other one was not. The buildings have a narrow front in the transverse direction (plan dimension of about 7 m) that widens towards the back (plan dimension of about 14.5 m). The plan length in the longitudinal direction is about 56.5 m. With a ground-to-roof height of about 52.6 m, the largest aspect ratio of the building is about 8 to 1 which makes it very slender and susceptible to potential overturning issues (Figure 19). Our cursory review of the plans of the two buildings at the site revealed a difference in termination of the walls, where in the damaged building, a long wall was interrupted at the level immediately above the ground, transferring its load to two perpendicular walls of significantly shorter length. These shorter walls were severely damaged (Figure 20).

Figure 19. Views of the Emerald Building.

Figure 20. Severe buckling of the walls at the first subterranean floor level.

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At the time of our visit, the damaged building was out of plumb and the roof was leaning by about 24 cm with respect to the ground. The shortening of the floor-to-floor height was clearly visible on the second floor of the building (Figure 21). The damage similar to what was explained in Section 2.5 at the subterranean floors was observed at other walls, as well for the same probable reasons explained previously (Figure 22). Extensive shoring in the lower levels was implemented to prevent the building from leaning further to the side (Figure 23).

Figure 21. Shortening of floor height causing buckling of non-structural panel.

Figure 22. Typical wall damage (note lack of seismic hooks and buckling of main bars).

Figure 23. Extensive post-earthquake shoring implemented in the subterranean levels to prevent further leaning of the building to the side.

4. CONCLUSIONS

Results of the LATBSDC reconnaissance team observations on performance of tall buildings in Santiago, Chile, during the Moment Magnitude 8.8, 27 February 2010 offshore Maule, Chile earthquake were presented. In general, the performance of most tall buildings located in Santiago was satisfactory. The handful of buildings that were damaged exhibited lack of proper detailing, particularly the absence of 135° hooks, inadequate confinement of walls in the boundary zones resulting in buckling of the main bars and tension-compression failure of the walls that spread across the length of the wall and generally was limited to a narrow height.

Factors that probably contributed to the failure of a number of tall buildings in Santiago and elsewhere in Chile include:

(1) Adoption of two exceptions to the ACI 318-95 code by Chilean authorities because of the perceived satisfactory performance of reinforced concrete shear wall buildings during the March 3, 1985 Offshore Valparaiso, Chile earthquake (see Rojas *et al*., 2010). These exceptions involved elimination of the need for confined boundary elements at the wall ends and requirements for ductile detailing in these areas.
(2) The confidence gained from satisfactory performance of shear wall buildings during the 1985 Chilean earthquake probably caused design and construction of taller buildings with the same thin walls (7 inch and 8 inch typical thickness) used for shorter buildings resulting in increased compression on the walls due to gravity forces and reducing the available wall ductility. It has to be mentioned that even with the new trend the ratio of wall cross-sectional area to floor area provided in a typical Chilean shear-wall building is generally larger than that in the similarly situated United States buildings. Therefore, abrupt changes in the geometry of the walls passing from the typical floors to the lobby area and subterranean parking levels may have contributed to zones of high stress concentration and failure (see Naeim *et al*., 1990).
(3) Lack of cross ties between the horizontal bars in the walls resulting in horizontal bars located outside the main vertical bars to span extended lengths without any lateral support once the wall's concrete cover had spalled. Out of plane bowing of the unsupported horizontal bars reduced or eliminated lateral support for the main vertical bars and contributed to their buckling.
(4) Lack of 135° (seismic) hooks and prevalence of the 90° hooks provided little confinement. Although we saw seismic hooks specified in several of the structural drawings of the buildings, we rarely saw any implemented in the damaged buildings we visited.

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