THE STRUCTURAL DESIGN OF TALL AND SPECIAL BUILDINGS
Struct. Design Tall Spec. Build. **20**, 17–36 (2011)
Published online 9 December 2010 in Wiley Online Library (wileyonlinelibrary.com). DOI: 10.1002/tal.672

Performance of tall buildings in Viña del Mar in the 27 February 2010 offshore Maule, Chile earthquake[‡]

Lauren D. Carpenter[1], Farzad Naeim[2], Marshall Lew[3,*,†], Nabih F. Youssef[4], Fabian Rojas[5], G. Rodolfo Saragoni[6] and Macarena Schachter Adaros[7]

[1] *WHL International Inc., Los Angeles, California, USA*
[2] *John A. Martin & Associates, Los Angeles, California, USA*
[3] *MACTEC Engineering and Consulting, Inc., Los Angeles, California, USA*
[4] *Nabih Youssef & Associates, Los Angeles, California, USA*
[5] *University of Southern California, Los Angeles, California, USA*
[6] *University of Chile, Santiago, Chile*
[7] *Weidlinger Associates Inc., Washington, District of Columbia, USA*

SUMMARY

After the devastating offshore Maule, Chile earthquake of moment magnitude 8.8 on 27 February 2010, the Los Angeles Tall Buildings Structural Design Council (LATBSDC) organized a reconnaissance team to visit the Santiago, Concepción and Viña del Mar areas. This report summarizes the highlights of the damage observed in the Viña del Mar area on the coast 120 km west-north-west of Santiago and much closer to the subduction rupture zone than Santiago. A significant portion of Viña del Mar is uniquely situated on variable ground conditions, particularly in the area with most of the damaged buildings. The Viña del Mar area also included tall buildings constructed before and damaged by the large 3 March 1985 magnitude 7.8 Offshore Valparaiso earthquake, which were damaged again in the 2010 earthquake.

The predominant observation in these buildings was the lack of confinement and ductile detailing of shear walls. The second observation was the high demand in the localized Viña del Mar area.

The apparently high demand on buildings clustered in the Viña del Mar area may be an observation based on the damage since most of the taller structures in the area are somewhat clustered in Viña. But significant damage was not reported in Valparasio, which is largely, situated on higher ground which steps downward to a narrow flat area along the ocean. Similarly, significant damage was not reported in new buildings along the coast just North of Viña del Mar. The authors' conclusion based on these observations is that the ground conditions in Viña del Mar relate to the level of demand and the level of demand is higher.

Demand is variable geoseismically but with the local history of many earthquakes from the same source zone, the characterization should be regularized. However, structural response is much better defined at least for confined elements with tested and predictable analytical characterization. Regardless, the detailing of reinforcing in reinforced concrete must relate to the expected demand and duration. In addition, the structural system and elements need to be designed and detailed to prepare a building to survive a higher unanticipated demand without collapse.

However, despite the apparently large demands due to the large magnitude earthquake with strong aftershocks and the long duration of seismic motions, building collapses did not occur even with limited capability due to non-ductile and non-confined detailing.

1. INTRODUCTION

The 27 February 2010 Offshore Maule, Chile earthquake extended along a subduction zone over 500 km parallel to the coast and extended as far north as the Valparaiso-Viña del Mar area (Figure 1). The Valparaiso-Viña del Mar area is located on the coast about 120 km west-north-west of Santiago, the capital of Chile. The ground motion was greater in the Viña del Mar area since the area is much closer to the offshore fault zone. The coseismic ground movement to the west in Valparaiso-Viña del Mar was reported as about 27.7 cm or about 11 in., which is about twice the coseismic ground movement of Santiago.

*Correspondence to: Marshall Lew, MACTEC Engineering and Consulting, 5628 E. Slauson Ave., Los Angeles, CA 9004, USA
† E-mail: tallbuildings@sbcglobal.net
‡ This article was published online on 09 December 2010. An error was subsequently identified. This notice is included in the online and print versions to indicate that both have been corrected [21 December 2010]

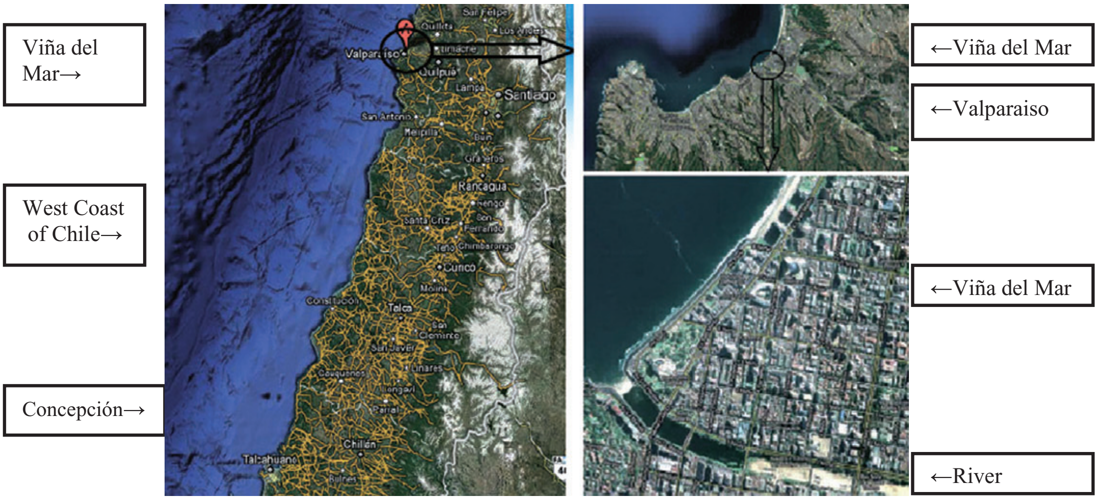


Figure 1. Chile Coast and Viña del Mar Region.

Chilean engineers visited many buildings after the earthquake (on 5 and 6 March). The engineers summarized in a preliminary report (CCE-ACSE, 2010) the apartment quantities and building storeys, and also indicated conclusions as to habitability and collapse status. In Santiago (Region I), 285 buildings were reviewed, and 76 buildings were reviewed in Region V, which includes the Valparaiso-Viña del Mar area. In the much larger Santiago, 37 buildings were 10 to 25 storeys. In the coastal region of the 76 buildings, 54 buildings were single-storey residential buildings in San Antonio, south of Valparaiso. In Valparaiso, three buildings were reviewed, which were 2, 5 and 10 storeys, and the 5-storey building was noted as non-habitable. In Viña de Mar, ten buildings were reviewed, nine buildings were 4-storey and one building was 14-storey. In Viña del Mar, the report noted that two buildings partially collapsed, declared six buildings not habitable and two buildings were concluded to be non-habitable pending further evaluation.

The Valparaiso-Viña del Mar area has highly variable ground conditions. The Valparaiso area is directly on the coast and the majority appears to be located on more sound foundation material, whereas the Viña del Mar area is mostly in a river delta area. The Viña del Mar area foundation conditions are summarized by R.M. Thorson (1999) for the University of Santa Maria (in Santiago) in the 1999 report titled La Falla 'Marga-Marga' Viña del Mar Chile.

The Thorson report indicates that the Viña area is basically over a fault area trending east–west, which formed the deep gorge that was infilled in the river zone. Figure 2a indicates the variations in surficial soils and Figure 3b indicates[§] mostly sandy materials. However, Figure 2b shows a more telling view of the subsoil conditions in two cross sections, which indicates an underlying lagoon, which is covered by the more sandy materials. The plan in Figure 2b shows the river is narrow until reaching an enlarged area near to the shore. Figure 3a indicates more detailed soil profiles and the approximate thickness of 15 m to 20 m of upper soils above the lagoon materials.

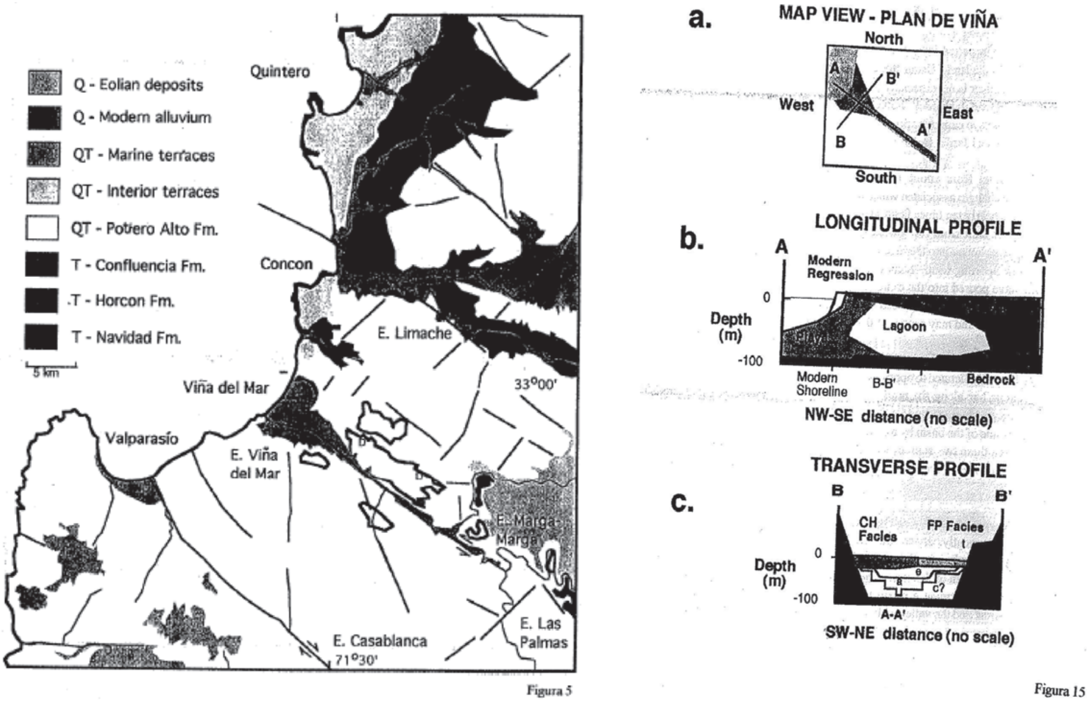
2. PREVIOUS STRUCTURAL RESPONSE

Previous recorded earthquakes in 1730, 1822, 1829, 1851, 1906, 1931, 1945, 1965 and 1971 rocked the coastal area near Viña Del Mar (at approximate latitude −32.5 and longitude −71.5). However, the 1985 earthquake occurred after a number of more modern buildings were constructed. The areas of influence of strong ground motion of some of these earthquakes are shown in Figure 4.

The general street layout of the most significantly impacted portions and locations of the majority of damaged buildings in Viña del Mar is presented in Figure 5. The buildings visited by the Los Angeles Tall Buildings Structural Design Council (LATBSDC) reconnaissance team on 2 April 2010 were generally 'concentrated' in one area. By comparison with Figure 2(b), the primary concentration

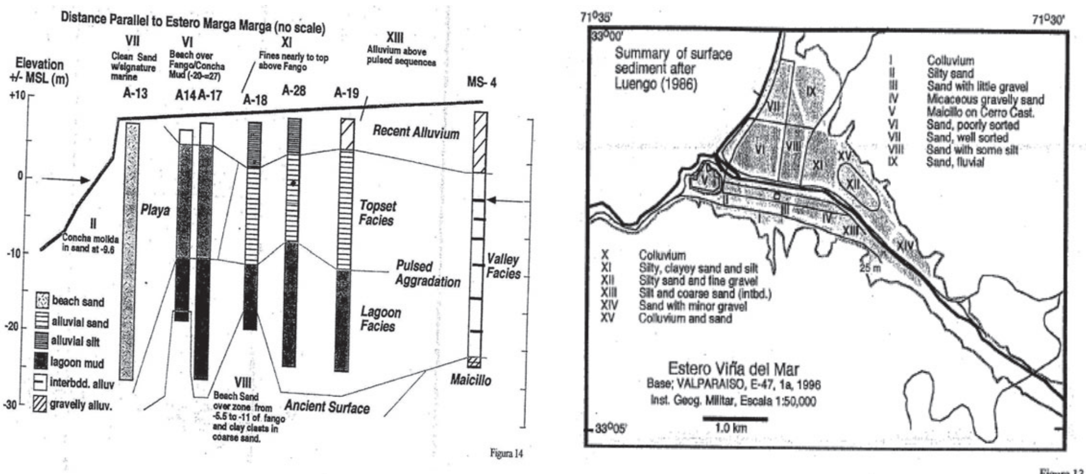
[§]Correction made here after initial online publication.

DOI: 10.1002/tal



(a) (b)

Figure 2. La Falla Marga-Marga Viña del Mar Map and Profiles (Thorson, 1999).



(a) (b)

Figure 3. La Falla Marga-Marga Viña del Mar Soil Distribution (Thorson, 1999).

of damaged buildings is within the roughly triangular area noted in figure 15 of the Thorson report and above the 'Marga-Marga' fault zone. This area includes a large portion of the Viña del Mar area and roughly corresponds to the zone of the soft underlying soils.

A preliminary report on the ground motion recorded during the 2010 earthquake has been prepared by Professor Ruben Boroschek's team at the University of Chile (Boroschek *et al*., 2010). The report provides basic seismic data at nine sites, of which two are located in the Viña del Mar area (Viña del Mar and Marga-Marga). The acceleration time history records are provided for each site and the maximum accelerations of 0.33 *g* and 0.35 *g*, respectively. The corresponding 5% damped response

DOI: 10.1002/tal

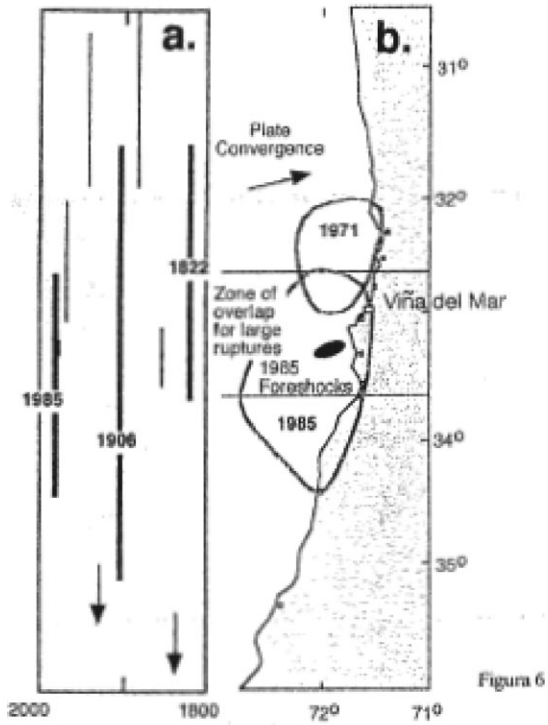


Figure 4. Area of strong ground motion (Thorson, 1999).

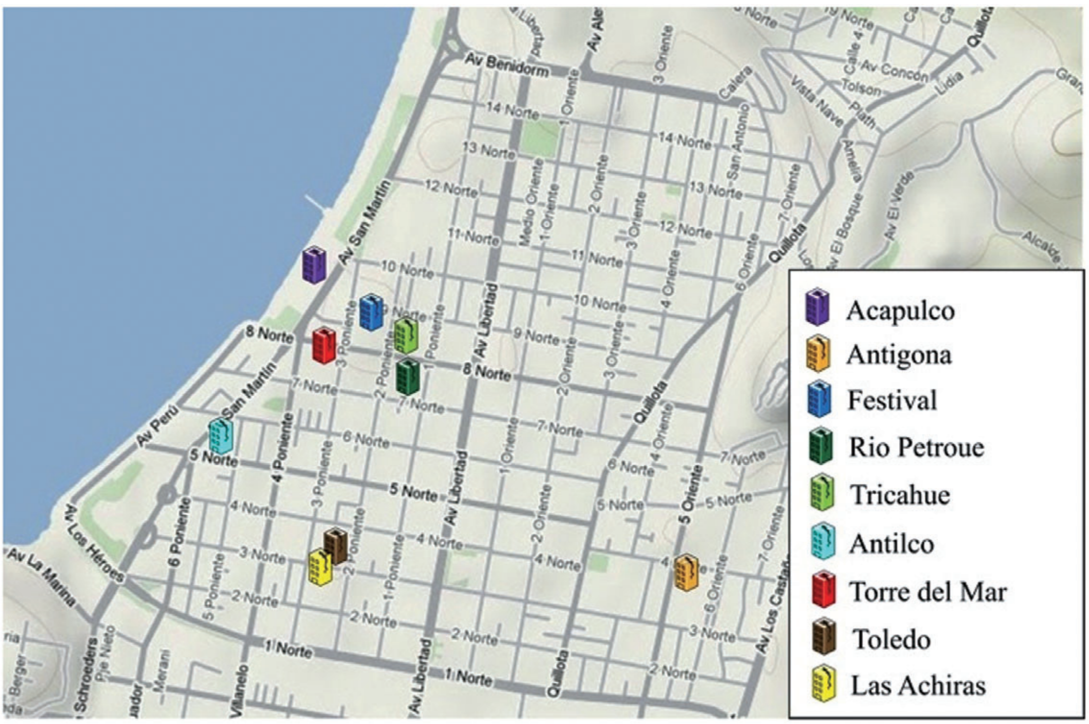


Figure 5. Locations of buildings in Viña del Mar.

DOI: 10.1002/tal

spectra indicate peak spectral responses are indicated up to 1.5 *g* at periods of 0.7 to 1.0 s. Since the initial periods of the buildings were likely near these periods, large demands would be expected. The report notes the record at the Viña del Mar site is the same location as measurements taken in the 1985 earthquake but with a different instrument. The location and characteristics of the site of instruments are not indicated for the Viña del Mar site; however, the Earthquake Engineering Research Institute (EERI) earthquake reconnaissance report for the 1985 earthquake indicates the instrument is in the basement of a 10-storey building (Wyllie *et al*., 1986). The Marga-Marga station is noted to be located at a viaduct.

3. BUILDINGS SUMMARIZED AFTER 1985 EARTHQUAKE

In June 2002, a commission of the Institute of Cement and Concrete of Chile (ICCC, 2002) summarized the basic characteristics of over 50 significant buildings constructed between 1967 and 1998. Some of these buildings were impacted in the 1985 earthquake. Each building is described on two pages with basic data such as name, location, height, floors, construction date, variation of wall thicknesses, etc. A photo or two and a building section amplify the data as well as representative floor plan indicating the locations and thicknesses of the various concrete walls.

This summary report is a very good reference for Chilean building structures. Most of the buildings are multi-storey concrete wall residential buildings; in Viña del Mar, the following buildings are described:

		Page	Wall thickness (cm)	Storeys
1967 Acapulco	Ave. San Martin 821	10 & 11	20, 25, 30, 45, 60	15 + 1
1971 Hanga Roa	Ave. San Martin 925	18 & 19	20, 40	15 + 2
1973 Torres de Miramar	Ave. San Martin 1020	24 & 25	40	20 + 2
1992 Esmeralda	Jackson 900	64, 65	20, 30	20 + 1
1996 Mallen (4C)	Calle El Palto	80, 81	20, 25	19 + 2
1998 Mariana Centro	Arlequi 333	98, 99	N/A	18 + 2
1998 Genova	12 Norte/1 Oriente	100, 101	20	10 + 1

Further correlation of the level of damage in each of these buildings should be encouraged in the local academic and engineering community.

Current buildings visited included the Acapulco building, which was reported to be damaged during the 1985 earthquake.

3.1. Acapulco building

The Acapulco building at 821 Ave. San Martin, was designed to the 1965 Code and built in 1967 (Figure 6). The location is on the ocean side of Avenida San Martin about at 10 Norte and is the farthest away from the river. The 15-storey building with one basement has shear walls varying from 20, 25, 30, 45 and 60 cm (8 to 24 in.) in width. Interestingly, the shear reinforcing was oriented in a diagonal pattern (Figure 6). However, the end of wall vertical reinforcing was not confined with ties and cross ties (Figure 7).

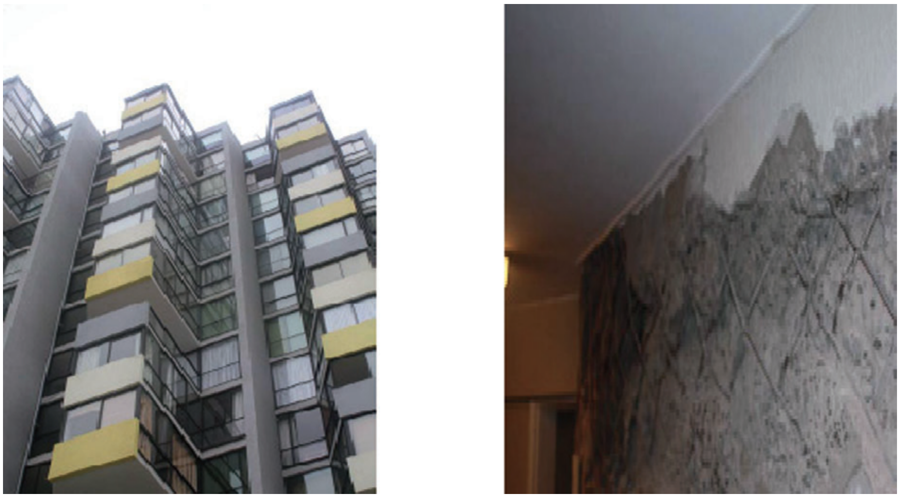
3.2. Antigona building

Antigona at 5 Oriente 260 was 16 storeys with 1 basement (Figure 8). The location is more inland and also three blocks from the river. The shear walls were constructed without ties and cross-ties. The vertical reinforcing at each end of the wall was located inside of the horizontal reinforcing, but the vertical reinforcing was outside of the horizontal reinforcing in the centre portion of the walls (Figure 9). The horizontal reinforcing was terminated at the ends of the wall with 90° hooks (Figures 10 and 11).

3.3. Festival building

The Festival building at 9 Norte 450 was 14 storeys over 1 basement (Figure 12). The location is in the north area near the Acapulco building on the south side of 9 Norte. The building is approximately

DOI: 10.1002/tal



- Exterior portions of concrete walls experienced some minor damage

(a)

- Diagonal shear reinforcing and extensive spalling (on both sides of the wall)

(b)

Figure 6. Acapulco building: (a) exterior perspective (b) diagonal shear reinforcing.

- Diagonal shear reinforcing outside of ”end of wall” zone vertical reinforcing
- End of wall reinforcing not enclosed in ties

(a)

- Cross ties not apparent
- Wall has vertical load along full length
- Aggregate moderately large and rounded

(b)

Figure 7. Acapulco building: (a) end of wall reinforcing (b) interior of wall.

DOI: 10.1002/tal

- Thin “brick” cladding mortar set

(a)

- East face with spandrel distress
- Lower right base of shear wall

(b)

Figure 8. Antigona building: (a) exterior street perspective (b) exterior back.

- Horizontal reinforcing is outside of vertical reinforcing at the end of the wall
- Horizontal reinforcing is inside of vertical reinforcing in the center portions of the shear wall.

(a)

- Shear wall crushing failure
- Horizontal reinforcing 90-degree hooks
- End of wall zone ties and crossties not apparent

(b)

Figure 9. Antigona building: (a) shear wall crushing (b) end of shear wall.

DOI: 10.1002/tal

- Basement shear wall with severe crushing at end of wall

(a)

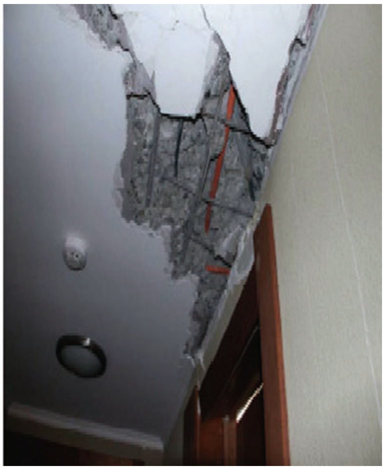
- Basement wall at end; some end of wall enclosing ties are apparent but widely spaced with 90-degree hooks
- Horizontal reinforcing not anchored inside end of wall zone and 90-degree hooks

(b)

Figure 10. Antigona building: (a) basement shear wall (b) end of shear wall.

- End of wall with top of dowels
- Horizontal reinforcing with 90-degree hooks
- End of wall enclosing reinforcing with 90-degree hooks

(a)



- Prevalent slab distress at "coupling" of walls over doorway openings
- Also slab flexural bending along wall face

(b)

Figure 11. Antigona building: (a) end of shear wall (b) slab coupling.

DOI: 10.1002/tal

(a)
- I shaped footprint
- Longitudinal shear walls 10 cm added each side of 30 cm original thickness as remedial/repair after March 1985 earthquake

(b)
- End of wall failure at ground floor
- Horizontal reinforcing 90-degree hooks
- End of wall ties and crossties not apparent
- Short column due to adjacent spandrels.

Figure 12. Festival building: (a) exterior perspective (b) end of typical walls.

(a)
- End of shear wall reinforcing
- Wide spacing of ties
- Ties are horizontal shear reinforcing with 90-degree hooks

(b)
- End of shear wall
- Without ties at end of wall
- Horizontal shear reinforcing with 90-degree hooks

Figure 13. Festival building: (a) end of wall crushing (b) end of retrofit wall.

I-shaped in plan, with the long axis oriented north–south. The shear walls were constructed with 90° hooks on the horizontal reinforcing at the ends of the walls (Figure 13). Ties and cross-ties were not evident. The primary north–south corridor walls and some primary east-west walls were damaged (Figure 14). These primary walls were retrofitted after the 1985 earthquake by adding concrete on each face but this retrofit work was severely damaged and separated from the original walls (Figure 13).

DOI: 10.1002/tal

- Longitudinal shear wall with opening

(a)

- Longitudinal shear wall crushing
- Slab flexure across walls

(b)

Figure 14. Festival building: (a) longitudinal shear wall (b) corridor longitudinal shear wall and slab flexure.

- Thin “brick” façade set in mortar

(a)

- Prevalent behavior of coupling beams
- Closely spaced stirrups
- Complete closed stirrups
- Side bars on beams

(b)

Figure 15. Rio Petrohue building: (a) exterior perspective (b) coupling beam over doorway.

3.4. Rio Petrohue Building

The Rio Petrohue Building at 7 Norte 585 is 17 storeys over 1 basement (Figure 15). The location is about six blocks from the river. The longitudinal axis is oriented east–west. Some ties were evident but alternated with horizontal reinforcing with 90° hooks (Figure 16). Interestingly, fracture of the vertical reinforcing was also observed in severely crushed areas at the ends of some walls (Figure 17). ‘T’ walls in the below grade basement/parking area indicated crushing across the flange and more severe crushing at the tip of the stem (Figure 18).

DOI: 10.1002/tal

- Column ties wider spacing
- Cross ties not apparent

(a)

- End of shear wall crushing
- Some end of wall ties at wide spacing
- Horizontal shear reinforcing 90-degree hooks

(b)

Figure 16. Rio Petrohue building: (a) typical end of wall (b) column ties.

- Prevalent end of basement shear wall at gravity beam
- Fractured vertical reinforcing
- Shear wall reinforcing with 90-degree hooks
- Ties and crossties not apparent

(a)

- Prevalent short shear wall in basement
- Crushing at both edges
- Shear wall horizontal reinforcing with 90-degree hooks
- Ties and crossties not apparent
- Crossties not apparent over the depth of the beam
- Beam reinforcing outside of wall “cage”

(b)

Figure 17. Rio Petrohue building: (a) end of basement shear wall (b) basement short walls.

3.5. Torre del Mar Building

The Torre del Mar Building at 8 Norte 380 at 3 Poniente is 16 storeys tall and built in 1988 (Figure 19). The location is near Avenida San Martin and in the North area. Damaged columns indicated that ties were enclosing the column but hooked 90° at corners (Figure 20). Interestingly, the hooked corners were alternated at consecutive ties.

DOI: 10.1002/tal

- Prevalent basement shear wall with T at end
- Crushing at T end and gravity beam
- Shear wall horizontal reinforcing with 90-degree hooks
- Ties and crossties not apparent.

(a)

- End of basement shear wall at narrow end (opposite end with T)

(b)

Figure 18. Rio Petrohue building: (a) basement T wall (b) basement stem of T wall.

- Change of structure at Level 3

(a)

- Damage at transfer of short upper wall transfer to wider lower wall.
- Column damage also present at some locations

(b)

Figure 19. Torre Del Mar building: (a) exterior perspective with evident change of stiffness at level 3 (b) damage at wall and column transfer.

3.6. Toledo building

The Toledo Building at 3 Norte 487 is 10 storeys over 1 basement (Figure 21). The location is about two blocks from the river. Severe crushing of shorter (north–south) walls (about 30 cm vertical shortening) caused very significant distortions (Figures 22 and 27)[§]. The ends of walls were neither tied nor cross-tied. The horizontal reinforcing was outside of all vertical reinforcing and was 90-degree hooked at ends of the walls (Figures 23, 24, 25, and 26). The longer (north–south) walls exhibited some spalling and tension cracking near the south ends (Figure 24)[§].

[§] Correction made here after initial online publication.

DOI: 10.1002/tal

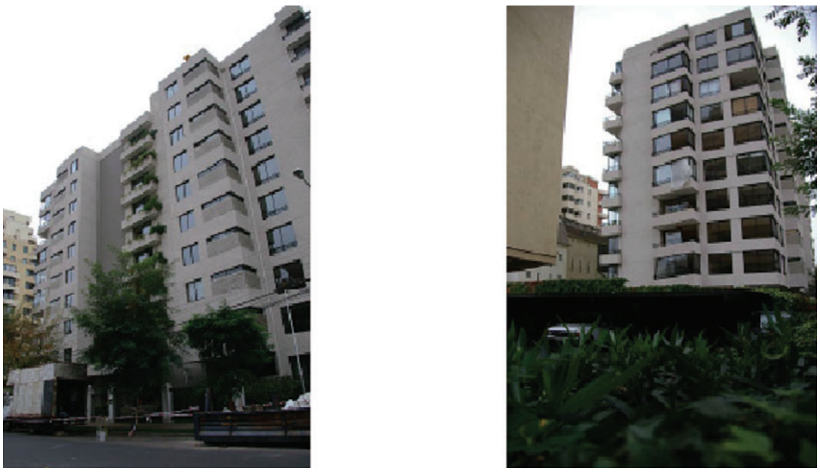
- Wall vertical reinforcing outside of horizontal reinforcing
- Vertical reinforcing not tied
- Cross ties not apparent

(a)

- Column reinforcing tied with 90-degree hooks
- Cross ties not apparent
- Short column effect

(b)

Figure 20. Torre Del Mar building: (a) wall transfer (b) column at transfer.



(a) (b)

Figure 21. Toledo building: (a) exterior perspective (b) transverse shear wall at end of building.

4. REPAIR AND RETROFIT OF EXISTING BUILDINGS

The Viña del Mar buildings damaged in the 2010 earthquake should be able to be repaired and retrofitted. Although the effort may be large in the more heavily damaged buildings, the effort appears to be technically feasible.

After a quick technical assessment and determination that the repairs and/or retrofit are economically feasible, the acceptance of buyers of damaged and repaired buildings as well as the acceptance in principle by the government's peer reviewers are both necessary.

(1) Repair and retrofit may be extensive but possible at high costs for heavily damaged buildings with most shear wall damage localized at one storey[§] such as in the Toledo Building. The feasibility is based on minor damage in the front portion with long transverse and longitudinal shear walls. The primary issue is lifting the back portion or deconstructing the back portion and reconstructing.
(2) For buildings like Festival, the retrofit could again consider the addition of shear wall thickness with the appropriate ties and cross-ties in order to extend the life of the project. The investiga-

[§] Correction made here after initial online publication.

DOI: 10.1002/tal

- Crushing damage at short shear wall at end of building
- Crushing across the full length of the wall of about 30 cm
- Upper shear wall longer than Ground story and Basement walls
- Significant distortion of floor level

(a)

- Crushing damage
- Typical wall vertical reinforcing completely free of the wall

(b)

Figure 22. Toledo building: (a) short shear wall (b) short wall crushing.

- Crushing and shortening of shear wall
- Horizontal wall reinforcing 90-degree hooks
- End of wall ties around end of wall vertical reinforcing not apparent
- Crossties not apparent.

(a)

- Opposite end of crushed and shortened shear wall
- Horizontal wall reinforcing 90-degree hooks
- End of wall ties around end of wall vertical reinforcing not apparent
- Crossties not apparent.

(b)

Figure 23. Toledo building: (a) end of short wall (b) opposite end of short wall.

tion would need to determine the foundation loads and also determine how high up the building the retrofit of walls needs to extend.

(3) For buildings like Torre del Mar that have a transition irregularity with the damage relatively localized at the transition and at the offsets at the stair, the repairs and retrofit can be made. Again, the investigation would determine how far up the building the retrofit aspects are appropriate.

(4) For buildings like Rio Petrohue, the significant damage is located at the base and parking areas. These areas could accommodate thickening and encasement to improve the characteristics of

- Opposite end of crushed and shortened shear wall
- Horizontal reinforcing 90-degree hooks
- End of wall ties around vertical reinforcing not apparent
- Crossties not apparent

(a)

- Tension cracking on street side ends of typical "interior" transverse shear walls

(b)

Figure 24. Toledo building: (a) end of crushed short wall (b) tension in long interior shear walls.

- Cantilevered portions of floors and walls beyond the ends of the transverse shear walls at first story (and end shear walls)

(a)

- Prevalent crushing at end of "interior" transverse shear walls
- Crushing at both top and bottom of the wall

(b)

Figure 25. Toledo building: (a) cantilevered floors and walls (b) end of interior transverse walls.

the non-ductile shear walls and T walls. The coupling beams above performed well and could be repaired.

(5) For buildings similar to Antigona, most of the supporting columns and walls are not heavily damaged. The heavily damaged walls are at the ground level and parking, which should accommodate some retrofit by thickening the concrete walls also.

(6) For buildings similar to Acapulco with the diagonal reinforcing and lack of ties and cross-ties, the walls are probably thick enough to accommodate the loadings. To prepare for the next earthquake, the shear walls would need to be retrofitted with ties and cross-ties at least in the

DOI: 10.1002/tal

- Prevalent crushing at the base of the transverse shear wall at the ground floor
- Horizontal reinforcing 90-degree hooks
- End of wall zones ties not apparent.
- Crossties not apparent

(a)

- Prevalent crushing at the top of the transverse shear wall
- Horizontal reinforcing 90-degree hooks
- End of wall zones ties not apparent

(b)

Figure 26. Toledo building: (a) crushing at base of wall (b) crushing at top of wall.

- Cantilevered back portion of floors
- Set back of upper shear wall at the Ground Floor story effectively shortened the length of the shear wall
- This shear wall crushed and shortened about 30 cm at the base of the wall at the Ground Floor
- Other shear walls along the complete backside of the building had severe crushing
- But the building did not collapse!!!

Figure 27. Toledo building: cantilevered floors and set back of shear wall at the ends of the building and corresponding interior shear walls.

lower damaged areas. The investigation would need to determine the updated design loads and expected capacities of the non-ductile upper walls to determine how far up the building the retrofit needs to extend.

5. CONCLUSIONS AND RECOMMENDATIONS

The overall conclusions, recommendations, and comments are summarized below.

(1) Detailing for confinement was not observed in any shear walls and exposed columns were only observed with perimeter ties hooked 90° at corners. Detailing for confinement is

DOI: 10.1002/tal

necessary for areas that experience high seismicity on relatively frequent intervals. The detailing must include ties and supplementary cross-ties similar to a column and with 135° hooks.

(2) Demand on the building structures needs adjustment or updating based on the recent seismicity and incorporating updated ground motion estimation techniques. Chilean seismicity is predominated by the known sources and distances which appear to be somewhat 'regular'. Therefore, attenuation relationships should be able to be developed or adjusted based on the additional data from the recent seismic activity. In addition, the ability to characterize the fault ruptures are much more developed currently compared to 1985 and earlier.

(3) Shear walls were thin in most instances and too thin to detail for confinement. Thicker shear walls must be encouraged in order to reasonably detail the end of wall zones for confinement. Increasing the 175 mm wall thickness to 225 mm (or 200 mm to 250 mm) tremendously assists the practical detailing particularly in the higher stressed portions of walls. Even a 25% increase in wall thickness increases the confined area 50%. The 25% increase in wall thickness increases the building mass a few percent if the wall is increased over the full height of the building. If the thickness is increased in the lower portions, the percent increase in mass is measurable but essentially nil.

(4) Vertical reinforcing was generally centred in the wall thickness. However, in several instances at damaged locations, the vertical bars were not centred on the wall thickness but closer to one face. Once the walls were damaged and the concrete cover spalled, the loadings were eccentric and the upper wall element 'slid' down past the lower portion of the wall. With a thicker wall, the effects of off centre placement of the reinforcing would be diminished significantly. For a 175 mm wall, the centre-to-centre spacings of vertical bars were about 100 to 125 mm. If the wall thickness was increased to 225 mm the centre-to-centre spacing would increase to 150 to 175 mm. This 25 percent increase in wall thickness is an increase of 50% in the effective width (and area) of the 'confined' concrete. Since placement of reinforcing in thin walls is more critical than thick walls or columns, thicker walls should be encouraged.

(5) The confined thickness in the end of wall zones needs to be recognized as a thin or narrow column. Column research has been corroborated by testing and the ultimate strength determined after the concrete cover has spalled off leaving the confined core. For columns, the cover is a relatively small part of the gross area of concrete and the code equations are adjusted to be based on gross area to simplify design. However, for a thin wall, the cover is a relatively larger percentage of the gross area. Therefore, once spalling occurs, the remaining core area is relatively a much smaller percentage of gross area compared to columns. The 'shock' of the spalling reducing the concrete area so quickly to the confined core is dramatic on the strength and also the stiffness of the wall. If the 'core' is not confined at all, the reductions are even greater and the behaviour is much more unpredictable.

(6) Transfers and discontinuities in the structural system particularly at the ground floor storey were prevalent. Building irregularities largely at the transition areas at the first storey from residential units above to the lobby and also transitions to the parking areas below. Particular detailing of confinement at these unique project specific areas needs much more careful planning and execution.

(7) The detailing to anchor the horizontal wall reinforcing at the 'end of wall' zones with 90-degree hooks on the exterior of the vertical reinforcing is a practice that needs to be revised. The horizontal wall reinforcing needs to be anchored within the confined concrete at the 'end of wall' zones.

(8) Splices in 'end of wall' vertical reinforcing were most often located in the ground floor storey and staggered splices were not observed in any building. The crucial ground floor storey should be detailed to avoid splices and place splices in the first basement level and in the second storey above ground. Confinement in the zone of splicing is necessary. Staggering of vertical reinforcing is also beneficial. Splicing at mid-storey height is necessary as well.

(9) Splices in typical shear wall vertical reinforcing in all exposed locations were extending just above the ground floor level. After concrete failure, the wall vertical reinforcing was no longer engaging any concrete. The typical shear wall vertical reinforcing needs to follow

recommendations noted above for end of wall vertical reinforcing with staggered splices at mid-storey height, etc.

(10) Cross-ties were not apparent in any shear walls in the end of wall zones, in shear walls in the interior zone (field area) nor in any column. Without cross-ties, the vertical bars will buckle or bend outward due to the side pressure as a result of the axial load on the wall or column. Particularly in the end of wall zones, a long rectangular tie is not sufficient without intermediate cross-ties. Cross-ties in the shear wall field areas are also appropriate particularly in the storeys with the potential hinging zone of the wall and at least a storey or two above and below as well.

(11) Shear in shear walls did not appear to be the primary contributor or even a significant parameter in the damage of these buildings.

(12) Construction joints in the shear walls also did not appear to be an issue for shear transfer in the shear walls.

(13) Typical coupling beams performed very well even though damaged and spalled. Deeper exterior spandrels did exhibit the diagonal shear-cracking pattern.

(14) The slabs performed well to transfer shears between adjacent walls even after spalling due to flexure from coupling effects between ends of walls and coupling between parallel walls.

(15) Viña del Mar and regions along the coast are in local Chilean code seismic zone 3 (which has a higher factor than inland areas such as Santiago which is seismic zone 2). Local soil conditions in Viña del Mar may not have been considered in the design of existing buildings. The structural design of the structures may have utilized a stiffer soil condition based on the upper sandy soils. Probably Chilean Code Soil Zone III[§] design was based on presumed stiffer sandy soils rather than utilizing Chilean Soil Zone IV[§] in order to recognize some amplification due to the underlying soft soil conditions. Microzonations should be developed to include the apparently soft soil effects in Viña del Mar. The apparently unanticipated demands should be the first evaluation by local academic and engineering teams prior to new construction in Viña del Mar.

(16) The detailing of reinforced concrete for good performance has significantly advanced over the years following the 1971 San Fernando Earthquake. Buildings designed and constructed to codes in place before about 1973 or so are likely to have well confined columns but the shear walls are detailed with minimal ties. The SEAOC Blue Book, NEHRP, Uniform Building Code, the addition of Chapter 21 into the ACI Code, Park and Pauley's extensive research in New Zealand in the 1970s, etc. have all lead to the need for detailing confinement and ductility into structural systems and structural elements.

(17) In order to better prepare for future large magnitude earthquakes, existing buildings that were damaged need to be evaluated. Initially, the subsurface conditions and the seismicity at severely damaged building sites should be quickly determined or developed. Site-specific response spectra should be developed from time history records and geotechnical conditions. Selected buildings should be evaluated; first modelled and analysed as originally envisioned, and then remodelled and reanalysed with currently available simple models as well as more involved models with time histories. In parallel with the development of demand, the capacity of non-confined reinforcing in walls needs to be determined in order to develop guidelines to evaluate the many other existing buildings. As a third step, the behaviour of the structure with reinforcing confinement needs to be determined for the same structure and compared.

(18) In one building (Festival), shear walls damaged in the 1985 earthquake were repaired with the addition of 100 mm layers of reinforced concrete on each face of the wall. The damage during the 2010 earthquake separated these retrofit layers from the initial wall. The retrofit layers did not appear to be tied through the original wall thickness in order to engage all three elements into one thicker wall. Apparently, roughening of the initial wall was the primary load transfer anticipated by the retrofit designer. Short dowels may have been incorporated to engage the retrofit layers to the walls. However, epoxy grouted reinforcing into unconfined concrete is not expected to be successful.

[§]Correction made here after initial online publication.

However, repair and retrofit of the damaged buildings appear to be technically feasible. After structural engineering and construction methodology evaluations, the final feasibility would determined as well as the cost of the work, etc.

(19) Construction related observations:
- (a) Consolidation appeared good for the most part in the areas of damage, i.e. minimal honey combing, rock pockets, etc.
- (b) Aggregate size and character were typically crushed rock and sand. However, at many damaged locations in the walls and beams over doorways, large aggregate were prevalent and the aggregates were not crushed aggregate but rounded and 'egg' sized aggregates approximately 40 mm in diameter and 70 mm in length.
- (c) Conduits were often cast into floor slabs and were visual in the damaged slab areas.
- (d) Conduits were also noted in concrete walls, but typically conduits were not observed in the end of wall regions.

(20) Lengthening of the building period due to the concentrated damage contributed to maintaining general stability and avoiding collapse of most of the buildings. Even with severe crushing failures of up to about 300 mm shortening, the buildings remained standing. Duration of seismic motion was long and representative so the buildings met the primary criteria of collapse prevention.

(21) Similar to the LATBSDC's Reconnaissance Team's observations after the Chi-Chi, Taiwan earthquake in 1999 (Lew *et al*., 2000), the local Chilean code was widely patterned after the ACI code but allowing exceptions for confinement and ductile detailing.

The Chilean Code process appears to have similar characteristics during development of the code. Based on the evaluation of the buildings after the 1985 Chile earthquake (centred offshore along the coast, south and west of Viña del Mar), the performance of buildings was deemed 'good'. Although the Chilean code closely resembles the ACI code, the ACI confinement provisions and 'end of wall' boundary provisions were permitted to be excluded or exempted by the 1996 Chilean code; i.e. design of walls were 'not necessary to meet the provisions' of sections 21.6.6.1 through 21.6.6.4 of ACI 318-95. Apparently, about 2 years ago, the ACI code was rigorously translated. So now the ACI code detailing provisions are included in the new Chilean code, which was formally approved after the recent earthquake.

The inclusion of the detailing provisions will trend to improve significantly the ability of the structural walls (and columns) to survive a large earthquake and repairability is expected to be improved as well. Duration relates to survivability also since at two cycles at high load, the structural capacity with ductile detailing may reduce to 80% but non-ductile detailing may reduce structural capacity to 60%. At six cycles, structural capacity with ductile detailing may reduce structural capacity to 60% and non-ductile detailed may reduce structural capacity to 20%.

However, the detailing needs to be shown very deliberately throughout the building on the engineering documents. The constructor's detailed reinforcing drawings need to be reviewed by the structural engineer for conformance, etc. Observations by the structural engineer at the jobsite are necessary and fully independent site inspection of reinforcing prior to casting concrete is necessary, too.

In summary, almost all items observed and conclusions reached have been identified previously and many are included in EERI Earthquake Spectra volumes. Particularly for Chile, the 1985 Earthquake is summarized in a special volume (Wyllie *et al*., 1986). This volume describes the damage observed in several buildings in the Viña del Mar area, including Acapulco (15-storey), Ranga Roa (15-storey), Tahiti (15 storey), two Triangular Towers (20-storey), three Pacific Towers (17-storey), Plaza del Mar (23-storey), Don Jose (13-storey) and other projects. Evaluations would be appropriate if original drawings could be located for these buildings and the buildings observed in order to assist development of future codes and guidelines.

Establishing the anticipated strong motion demand through further characterizations of seismicity, seismic zones, microzones, soil effects, etc. is necessary. Establishing detailing and guidelines for shear walls, shear wall thickness, confinement, anchorage, etc. are also necessary.

DOI: 10.1002/tal

REFERENCES

Boroschek R, Soto P, Leon R, Comte D. 2010. Preliminary Information November 4, Accelerograms, for the Chile Earthquake 27 February 2010. Department of Civil Engineering, Department of Geophysics, University of Chile.

Chilean College of Engineers and the Association of Civil Structural Engineers. 2010. Preliminary Report on the Effects of the Earthquake.

Institute of Cement and Concrete of Chile. 2002. Chilean Buildings of Reinforced Concrete.

Lew M, Naeim F, Huang SC, Lam HK, Carpenter LD. 2000. Tall Building Performance in the 1999 Taiwan Earthquake. In *Proceedings of Fifth Conference on Tall Buildings in Seismic Regions*, Los Angeles, California.

Thorson RM. 1999. La Falla 'Marga-Marga' Viña del Mar. University of Santa Maria.

Wyllie LA, Abrahamson N, Bolt B, Castro G, Durkin ME, Escanlante L, Gates JH, Luft R, McCormick D, Olson RS, Smith PD, Vallenes J. 1986. The Chile Earthquake of March 3, 1985. *Earthquake Spectra* **2**(2): 293–371.

DOI: 10.1002/tal