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Performance of the Torre Bosquemar and Olas buildings in San Pedro de la Paz and the Pedro de Valdivia building in Concepción in the 27 February 2010 offshore Maule, Chile earthquake

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SUMMARY

Three of the tall buildings that members of the Los Angeles Tall Buildings Structural Design Council visited after the 27 February 2010 Chilean moment magnitude 8.8 earthquake were reviewed. The three buildings discussed in this paper are located in the Greater Concepción area in the Biobió Region (Region VIII) of Chile. Two of the buildings (Bosquemar and Olas) are located on the west side of the Bio-Bio River in San Pedro de La Paz, while the third building (Pedro de Valdivia) is located on the east side of the river south of the downtown area of the city of Concepción. The tower at Bosquemar was the most thoroughly investigated, although the observations from the damage at all three structures offer important insight that can be applied to current and future earthquake engineering practice. Copyright © 2010 John Wiley & Sons, Ltd.

1. INTRODUCTION

Three of the tall buildings that members of the Los Angeles Tall Buildings Structural Design Council (LATBSDC) visited after the 27 February 2010 Chilean moment magnitude 8.8 earthquake are reviewed. The three buildings discussed in this paper are located in the Greater Concepción area in the Biobió Region (Region VIII) of Chile. Two of the buildings (Bosquemar and Olas) are located on the west side of the Bio-Bio River in San Pedro de La Paz, while the third building (Pedro de Valdivia) is located on the east side of the river south of the downtown area of the city of Concepción as shown in Figure 1.

The soil conditions in Concepción are described in a companion paper by Rojas *et al.* (2010). Rojas *et al.* also presented the record from an accelerometer station located at the high school in San Pedro de La Paz where the peak ground accelerations were 0.65 g in the north–south direction, 0.58 g in the east–west direction and 0.60 g in the vertical direction. It was also reported that a coseismic displacement of the ground in excess of 3 meters to the west–south-west occurred in Concepción as a result of the earthquake.

The tower at Bosquemar was the most thoroughly investigated, although the observations from the damage at all three structures offer important insight that can be applied to current and future earthquake engineering practice.

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Figure 1. Location of the three buildings in San Pedro de La Paz and Concepción.

2. BOSQUEMAR

'Torre Bosquemar' is located along the coastal avenue (formerly interior lane 1) of Road 160 in San Pedro de La Paz, part of the province of Concepción. The structure is composed of 20 stories of living areas as well as a level of subterranean parking. The majority of design was completed by the end of 2006.

The tower under investigation is one of two towers currently constructed. The two buildings, while constructed similarly, have different floor plans, with one having a generally rectangular shape and the other having an angled 'L' shape as shown in Figure 2. The shape and directionality seemed to play an important role, as discussed later, as only the rectangular building saw significant damage. This tower, shown in Figure 3, is the structure that the observation focused upon.

The tower is composed of a large amount of reinforced concrete shear walls throughout the floor plan (see Figure 4). Beams are also arranged around the perimeter of the building along the balconies and window sills. Concrete slabs were used for the diaphragms as well as the foundation. All concrete used had a compressive strength of $f'_c = 3555$ psi. Typical structural element dimensions included 8-in. thick walls, 6-in. thick slabs and 8-in. wide beams with depths of 27, 45 or 61 in. Typical wall horizontal reinforcement ranges from a minimum of 8 mm diameter bars at 20 cm spacing (steel ratio = 0.625%) to 16 mm diameter at 13 cm spacing (steel ratio = 1.93%). Typical vertical reinforcement ranges from 8 mm diameter bars at 20 cm spacing (steel ratio = 0.625%) to 22 mm diameter bars at 20 cm (steel ratio = 2.37%). Boundaries are generally present at the edges of walls, but in the majority of cases, they only confine the last two vertical bars along each face of the wall and typically use eight 12-mm diameter bars at 20-cm spacing (approximately equivalent to no. 3 or no. 4 bars at 8–in. spacing).

Globally, an important observation to make is the difference in damage between the two structures at the site. Although both are located at the same location and presumably upon similar soil conditions with similar design/construction methods, only one of the structures saw significant damage, which

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Figure 2. Satellite image showing the layouts of the two towers from above.

(a) (b)

Figure 3. Front and back of tower. (a) Front of tower (facing inland). (b) Back of tower (facing ocean).

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Figure 4. Typical floor plan (red = walls, blue = beams).

Figure 5. Soil depression along ocean-facing side.

suggests a strong influence of directionality. Future research, once comprehensive ground motions for the relative area have been released, can determine in more detail the magnitude of this effect.

Damage was visible along the exterior of the structure in a variety of locations. The first item of interest is the very noticeable depression in the soil along a large portion of the ground along the ocean-facing side of the tower as shown in Figures 5 and 6. This type of depression occurred at multiple locations around the exterior soil of the structure. These depressions were most likely caused by either settlement of poor backfill compaction against the basement walls or rocking of the building at the foundation level.

There were also quite a few important locations of structural damage along the exterior.

One of the main recurring themes of damage in buildings during this particular seismic event was thin walls with moderate to no boundary confinement steel seeing flexural/compression crushing, oftentimes focused in areas with vertical discontinuities. This type of damage was prevalent at Bosquemar as well, which had 20-cm (7.87 in.) walls, typical throughout the entire structure. Referencing Figures 7 and 8, this type of damage is first noticeable at the ocean-side exterior along gridline 19 that bisects the structure along the transverse direction. The shear wall along this gridline is continuous with no openings from gridline L to the extreme edge of the structure for the entire height of the structure (see Figure 7) other than openings that occur at floor 1 level as shown in the elevation of line 19 as shown in Figure 9(a). One of these openings at floor 1 is very close to the edge of the shear wall, which likely saw very high compressive stresses from the overturning of the shear wall above. This resulted in concrete crushing, shown from afar in Figure 9(b), with a closer view in Figure 9(c). The detail for the boundary confinement at this location, shown in Figure 9(d), was

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Figure 6. Soil depression.

similar to those throughout the structure. The crushing at this location resulted in significant buckling of the longitudinal bars as well as a noticeable out-of-plane offset between the top and bottom of the crushed zone.

Further along this same exterior edge of the building, significant crushing was found along the curved shear wall near gridline O between lines 4 and 5. As shown in Figures 10 and 11, there was significant spalling of concrete at the bottom of the wall. Although the crushed shear wall itself is relatively short in length and most likely did not attract a large amount of in-plane loading, it is used as a flange by transverse walls along both lines 4 and 5. Given that no enlarged boundary elements were used, it appears that the overturning forces from the two perpendicular walls may have had a significant contribution to the crushing stresses in this wall.

Another location of significant damage occurred along the entrance side of the structure, facing inland. Significant cracking was seen along both walls of the entrance lobby as shown in Figures 12 and 13. Referencing Figure 7, these walls span along gridlines 15 and 22 from line K to D above the first floor. However, at the first floor, these walls extend out from the front face of the tower as seen in Figure 8, creating a vertical discontinuity and an outrigger effect.

The elevation along gridline 15 (Figure 14) reveals that an opening exists directly under the diagonal cracking. While the extended wall acted similar to an outrigger and attracted large demands from the walls above, there is not a suitable load path down the through the structure for these large demands. This results in the concrete crushing shown at the edge of the ground level wall near the opening (Figure 15) as this crushing is occurring at the only location where there is another concrete wall underneath that is able to continue the load path down to the foundation.

The elevation along the other wall of the entrance lobby (along line 22) is identical to line 15 with the exception that there is no opening in the floor below. Because of this, the load path does not need to span over an opening in the lower level as it does along line 15, as described earlier. On the other hand, along line 22, it appears that the overturning demands from the floors above create flexural/compressive stresses that are too great, thus creating the large vertical crack that can be seen at the interface of where the elongated wall extends outwards from the typical length walls in Figure 13.

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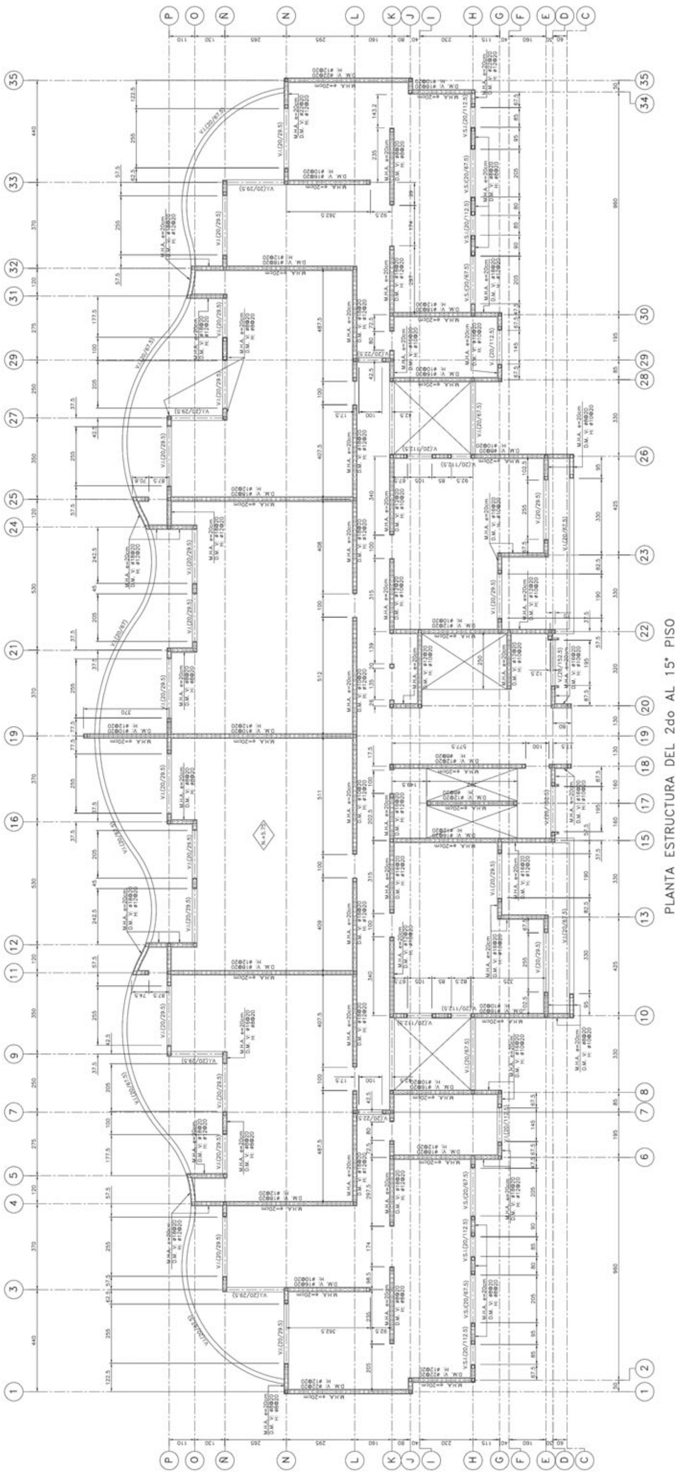


Figure 7. Typical plan for second to 15th floors.

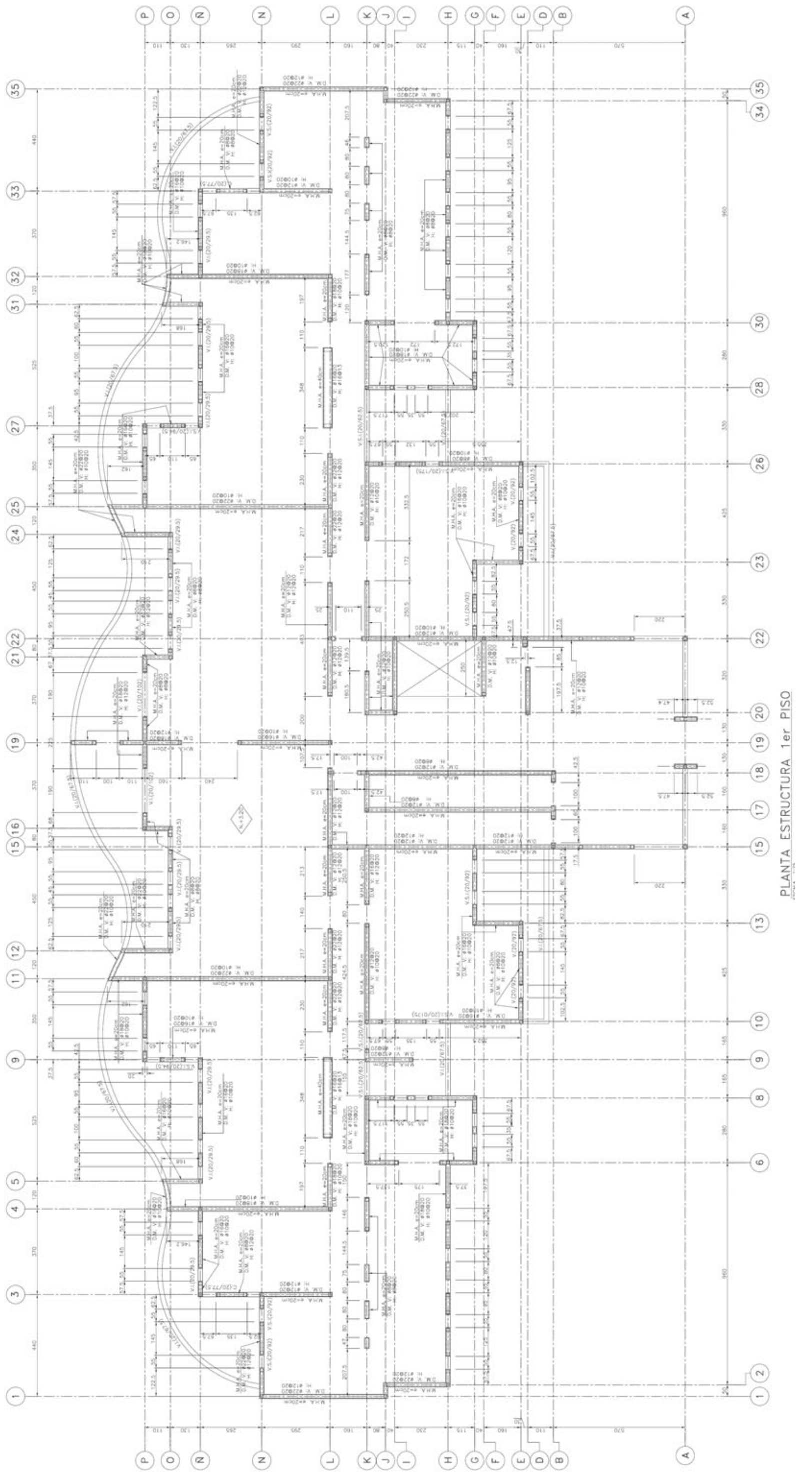
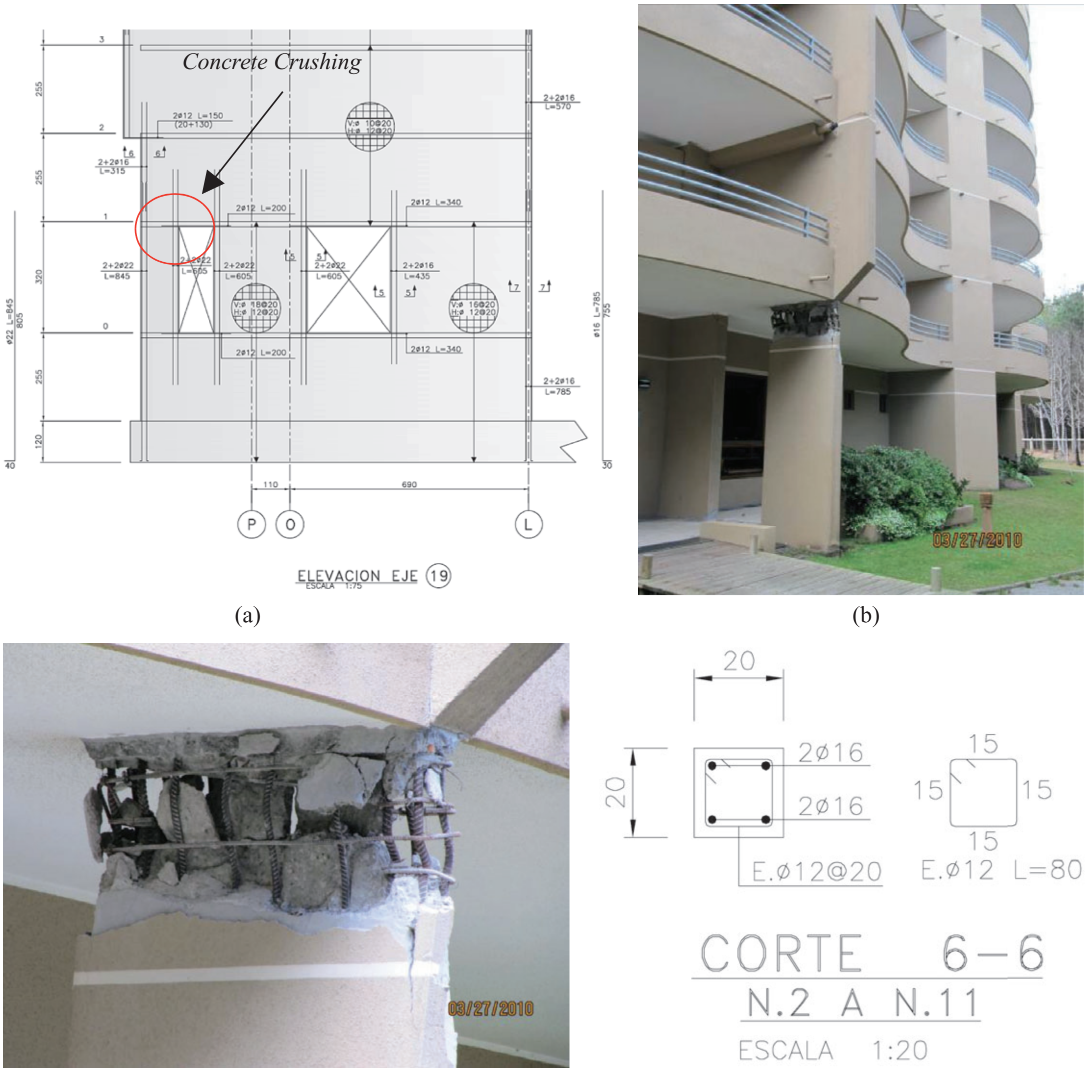


Figure 8. Floor 1 plan.



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

Figure 9. (a) Elevation of lower floors along line 19; (b) concrete crushing along line 19; (c) closer view of crushing at top of wall; (d) boundary confinement details at line 19.

One of the more substantial cases of concrete crushing that resulted in the rupture of vertical bars was within an interior wall shown in Figure 16. This damage occurred at the first floor along line L on the wall centred along line 22 (refer to Figure 8). The large crack turns horizontal, with noticeable vertical bar buckling along its entire length until the edge, where both the vertical bars as well as the transverse bars confining them were ruptured as shown in Figure 16(b).

Reviewing the elevation of this location, this damage seems to be the result of vertical discontinuities within the shear walls combined with the thin wall thickness and lack of confinement. Crushing occurred at the top of the wall where there was likely high compressive stresses caused by the flexural overturning of two different walls from above (See Figure 16(c)). The opening at the first floor between lines 22’ and 23 focuses the majority of the compressive stress from the wall above to travel through the wall centred along line 22. Yet, given the opening at the basement level, between lines 19 and 22, there is another discontinuity in the vertical shear wall, and thus, the cracking ends up horizontal at this end of the wall.

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Figure 10. Crushing at exterior wall between lines 4 and 5.

Figure 11. Close up view of concrete crushing and rebar between Lines 4 and 5.

Figure 12. First floor wall along gridline 15.

Figure 13. First floor wall along gridline 22.

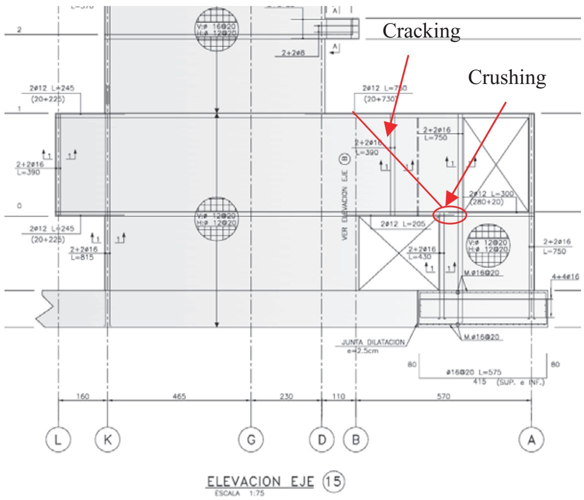

Figure 14. Elevation of gridline 15.

Figure 15. Concrete crushing at edge of 'outrigger wall' along gridline 15.

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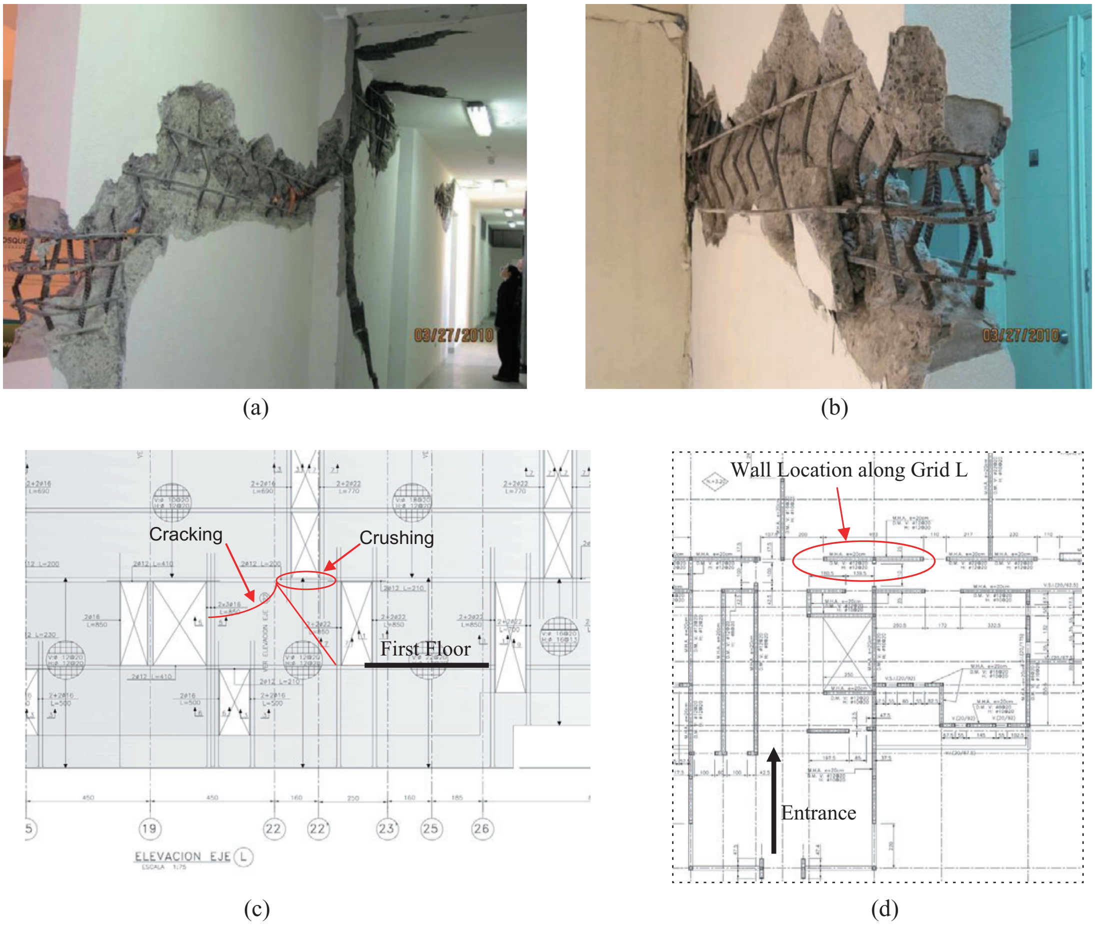

Figure 16. (a) View of cracking/crushing looking down the hallway; (b) close-up view of bar rupturing; (c) elevation of location; (d) cropped plan of location (first floor).

The majority of damage observed in the typical upper floors was in the slab itself. This occurred in slabs acting as coupling beams across door openings as well as in slabs acting as coupling elements between parallel walls. Both of these types of actions were very noticeable along the main corridor of the tower between the closely spaced walls along gridlines K and L for multiple floors along the height of the tower.

Figure 17(a) shows the typical types of slab coupling damage that was observed in the upper floors. The closet cracking in the image along the ceiling slab shows clear coupling between the parallel walls (gridline K on the left side, with gridline L on the right). At the doorway on the right side of the picture, daylight is able to be seen above and below the door where coupling beam-type action within the slab has crushed most of the concrete. In some locations, there was also diagonal cracking that developed between staggered openings within the two walls along the corridor as shown in Figure 17(b).

Although certain locations did see extra reinforcement, typical slab reinforcement through these corridors was 8 cm diameter bars at 20 cm on centre, or approximately no. 3's at 8 in. on centre. This type of damage was noted in other buildings in Chile during the event as well and results not only in substantial required repairs but can also jam doors.

The most substantial wall cracking observed in the upper floors came at the top of the structure in the gymnasium area as shown in Figure 18(a). This was another case of a vertical discontinuity in a shear wall. The location of the damaged wall is shown in plan and elevation in Figure 19. The portion of wall that is damaged does not continue to any of the lower floors below the gymnasium level but

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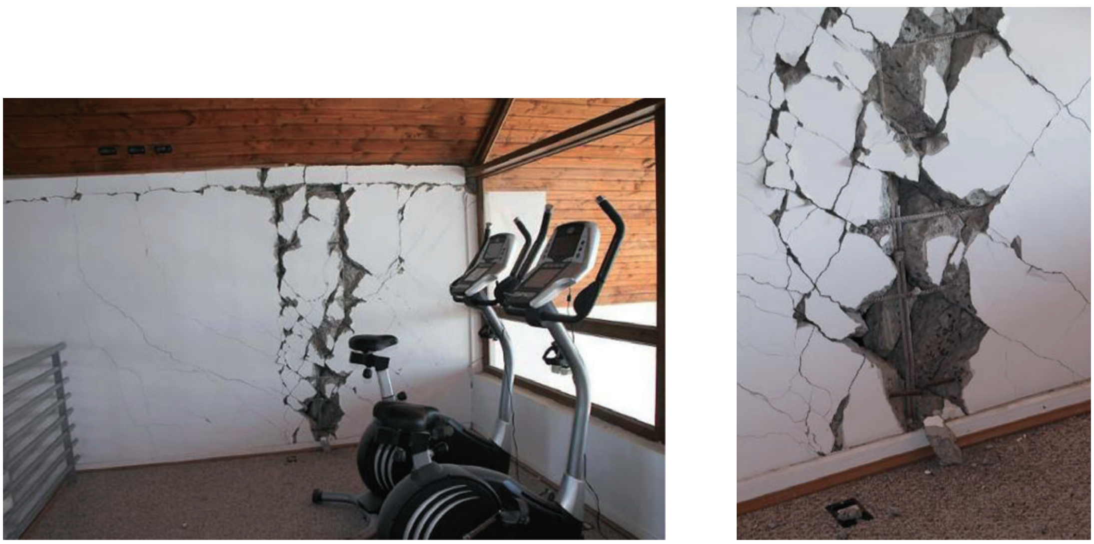
(a) (b)

Figure 17. Slab coupling damage.



(a) (b)

Figure 18. (a) Large cracks at gridline 15; (b) lap splice concrete spalling.

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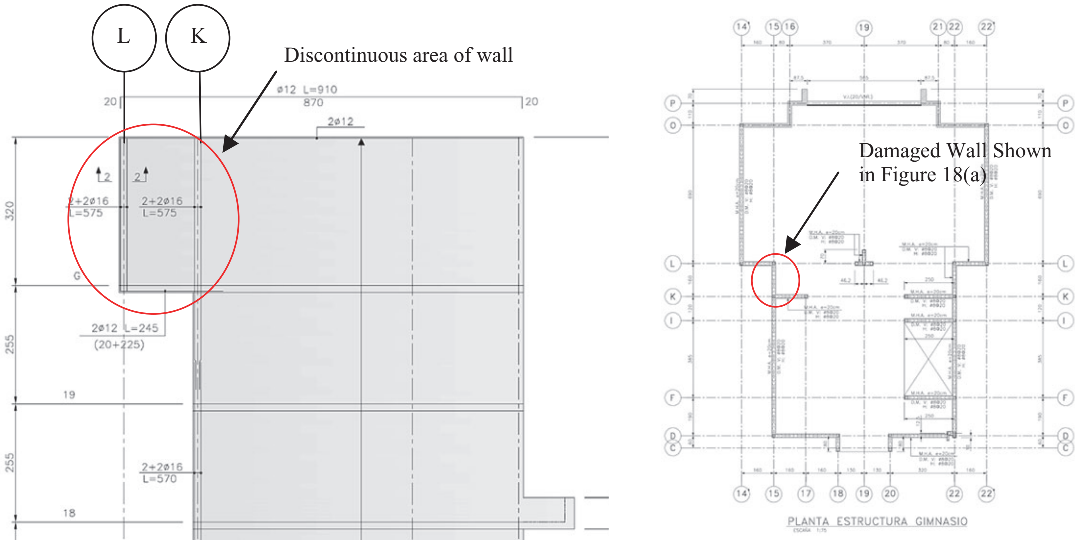

Figure 19. Elevation and plan of damaged wall.

is restrained by the wall along line L, which most likely caused large shearing demands through this short length of wall, resulting in the severe vertical cracking only over a limited section of the wall shown in the image. The bottom of the wall, where the largest amount of concrete spalling occurred, was a lap splice location as shown in Figure 18(b).

3. PEDRO DE VALDIVIA BUILDING

Although less detailed observations are available for the Pedro de Valdivia building in Concepción than were created for Bosquemar, there are still clear and important lessons to be learned from the visible damage. These lessons pertain to concrete stairwells and the surrounding elements acting unintentionally as diagonal braces.

Figure 20 shows the stairwell between two shear walls along the height of the structure. The slanted windows along the stairways resulted in concrete elements that during the seismic event acted as braces between the two walls that they connected to on each end. In addition, areas at about the one-quarter and three-quarters height along the stairway were used for water and power systems. As shown in the image, these locations behaved as short columns and failed in shear. These are clear examples of architectural elements not originally intended to have a structural purpose adopting roles in the lateral system. This therefore results in an alteration of presumed manner in which the lateral system will behave as well as a high probability of damage to the elements in question.

Figure 21 shows the exterior damage that occurred at the ends of the stairwells where the concrete was acting as a sloping non-ductile coupling beam. The interior of the stairwells revealed excessive crushing and spalling of concrete at each edge of the slanted window openings (Figure 22). It also reveals that although the walls on each of the stairwell were detailed with boundary confinement, these boundary zones occur within the span of the stairwell and not within the portion of the wall that is restrained within the structures diaphragm (see plan view in Figure 23).

In addition, these boundary zones are located approximately 6 in. from the edge of the slanted window openings as shown in Figure 24, leaving this amount of concrete to crush and spall. It can also be seen that the slanting concrete sections along the exterior edges of the stairwells were not originally detailed to act as coupling or to behave in a ductile manner. Therefore, these end up behaving as non-ductile coupling beams during the earthquake.

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Figure 20. Diagonal stairwell elements along height.

Figure 21. Damage at ends of stairwell.

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Figure 22. Interior view from stairwell.

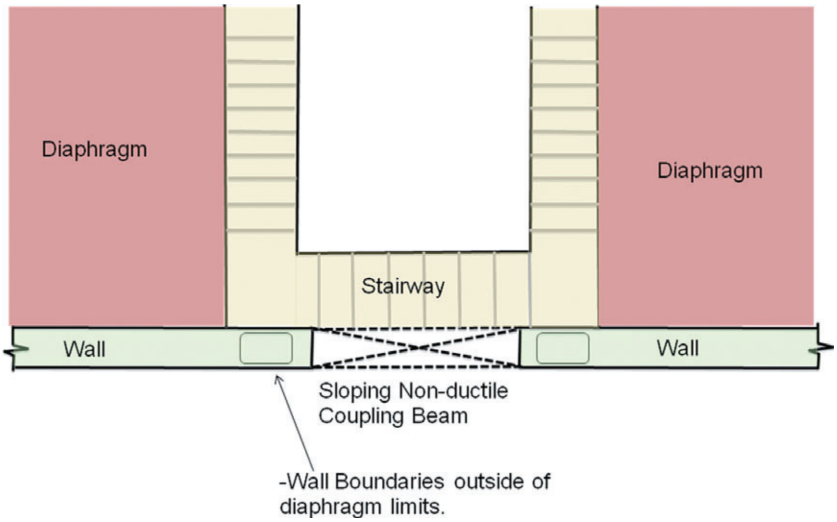

Figure 23. Qualitative plan of stairway and shear walls.

4. OLAS

The Olas building in San Pedro de La Paz was unoccupied during the seismic event. The building is actually two separate structures with a sloping profile along a curving plan. Only limited information could be collected from the observation but two important items were discovered. The first was cracking along the far edge of the structure, which is able to be seen in Figures 25 and 26. It appears as though some cracks may have already been covered or painted over by the time the pictures were taken one month after the earthquake. Figure 27 shows the same end of the building just after the earthquake. In addition, there was evidence of the two closely spaced structures pounding against each other during the earthquake as shown in Figure 28.

Current information dictates that the earthquake caused far greater damage than the exterior images revealed. As of August 2010, a large portion of the taller end of the structure has been under demolition, cited as part of repairs to the structure for the damage caused (see Figure 29). Although the demolition is extensive, it is suggested that the demolished section will be reconstructed, and the building will be completed and opened (per the *Concepción Journal*).

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Figure 24. Close-up of interior damage at an edge of the stairwell.

Figure 25. Olas building in San Pedro de la Paz.

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Figure 26. Close view of cracking at extreme edge with repairs made.

Figure 27. View of damage of end walls after earthquake and before repairs.

5. CONCLUSIONS

Each of the three buildings discussed evoke lessons to be learned. From Bosquemar, the main theme of damage was thin shear walls with minimal boundary confinement seeing localized high axial stresses causing crushing. It shows that even with a great deal of shear walls throughout the floor plan, the ability to confine boundaries (and have a thick enough walls to have an effective boundary area) can be very important to resist crushing, bar rupture and lateral instability, especially where there are vertical irregularities that can disrupt the load path through the shear wall to the foundation or create localizations of high axial stress. In addition, the participation of elements that are not necessarily considered or designed as part of the lateral system need to be considered such as slabs that

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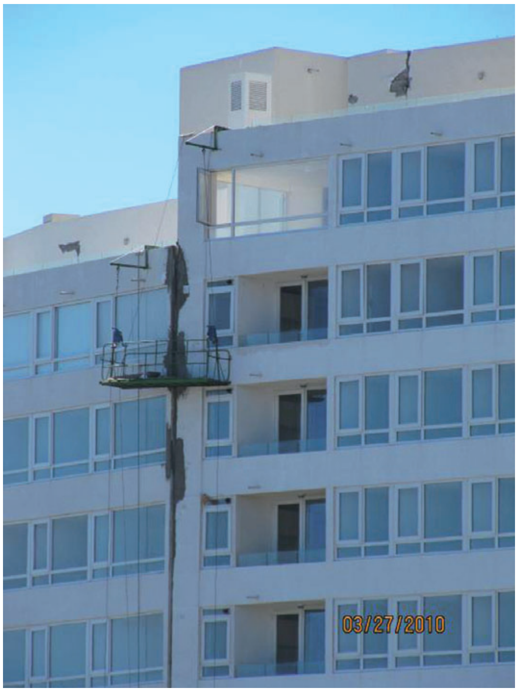

Figure 28. Evidence of pounding.

Figure 29. Demolition work caused by damage from earthquake, as of August 2010 (source: http://i857.photobucket.com/albums/ab134/roogenial2/olasim.jpg).

potentially act as coupling beams over doorways or between closely spaced parallel walls (such as in a hallway) therefore see considerable damage. The site was also an example of the importance of orientation and configuration in relation to the direction of dominant strong ground shaking. Although the other structure at the site had the more irregular shape, the more rectangular tower was oriented in a manner such that it saw much greater effects from the shaking. The Pedro de Valdivia building damage is another example of the value in consideration of the effects upon architectural cast-in-place concrete elements not necessarily designed to act as part of the lateral system. The Olas building showed the clear implications of pounding between closely-spaced structures.

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