

Analysis and Repair of an Earthquake-Damaged High-rise Building in Santiago, Chile



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SUMMARY:

During the February 2010 earthquake in Chile, an 18-story building was badly damaged when three shear walls failed in flexural compression just below grade. The corner of the ground floor slab moved down 75 mm and the corner of the building roof moved horizontally almost 200 mm. All occupants were safely evacuated; however the damaged building was no longer habitable. This paper describes how the building was successfully repaired. Nonlinear finite element analyses of shear walls and floor slabs was used to understand the observed crack patterns and measured residual displacements, and to determine appropriate effective stiffnesses for a three dimensional model of the entire building used to estimate the required jacking forces. Existing cracks in shear walls were injected with epoxy, and FRP fabric was used to strengthen the wall to withstand the jacking forces and control new cracks. Extensive instrumentation was used to monitor the building during jacking.

Keywords: concrete shear walls, earthquake damage, nonlinear analysis, repair, 2010 Chile earthquake

1. INTRODUCTION

More than one hundred high-rise concrete shear wall buildings were badly damaged during the earthquake that occurred in Chile on February 27, 2010. A common form of damage was compression failure of thin walls in the lower levels of the buildings. In one 18-story building in Santiago, three adjacent shear walls at one corner of the building failed in flexural compression immediately below grade (Fig. 1). This caused the corner of the building to drop down 75 mm resulting in a permanent drift of 0.44%.

The building has an overall floor plan of about 14 x 37 m. In the long direction, there are two 200 mm-thick concrete shear walls on either side of a 1.6 m wide central corridor that runs the full length of the building. In the transverse direction, there are seven slender shear walls on both sides of the corridor. Most of the transverse shear walls are 200 mm thick; however some are 150 mm thick. The transverse shear walls have a uniform length from above the ground floor to the top of the building; but in the two levels of parking below grade, these walls are narrower. Shear walls 13C, 15C and 20C failed in flexural compression immediately below the ground level (Fig. 2). After the earthquake, the residual vertical displacements of shear walls 13C, 15C and 20C were measured to be 24, 75 and 58 mm at the outside edge of the building (Fig. 3). The horizontal displacements of the building along these three shear walls were measured to be 100, 130 and 185 mm at the roof level.

Cracking of concrete walls (Fig. 1) and cracking of floor slabs occurred throughout the damaged corner of the building causing the entire building to be distorted. As a result, there was concern that it may not be possible to restore all horizontal and vertical alignments of the building within acceptable tolerances by jacking only at the base of the building. Analyses were done to simulate the building damage and to estimate whether jacking at the base could be used to reposition the building. Estimates of maximum jacking forces were used to design local structural modifications and the new

substructure needed to support the jacks and shore the building during repair. Throughout jacking of the building, the movements of the building were continuously monitored for reasons of safety and to make decisions about when to inject existing cracks with epoxy, when to apply FRP to walls to control additional cracking, and when to stop the jacking.

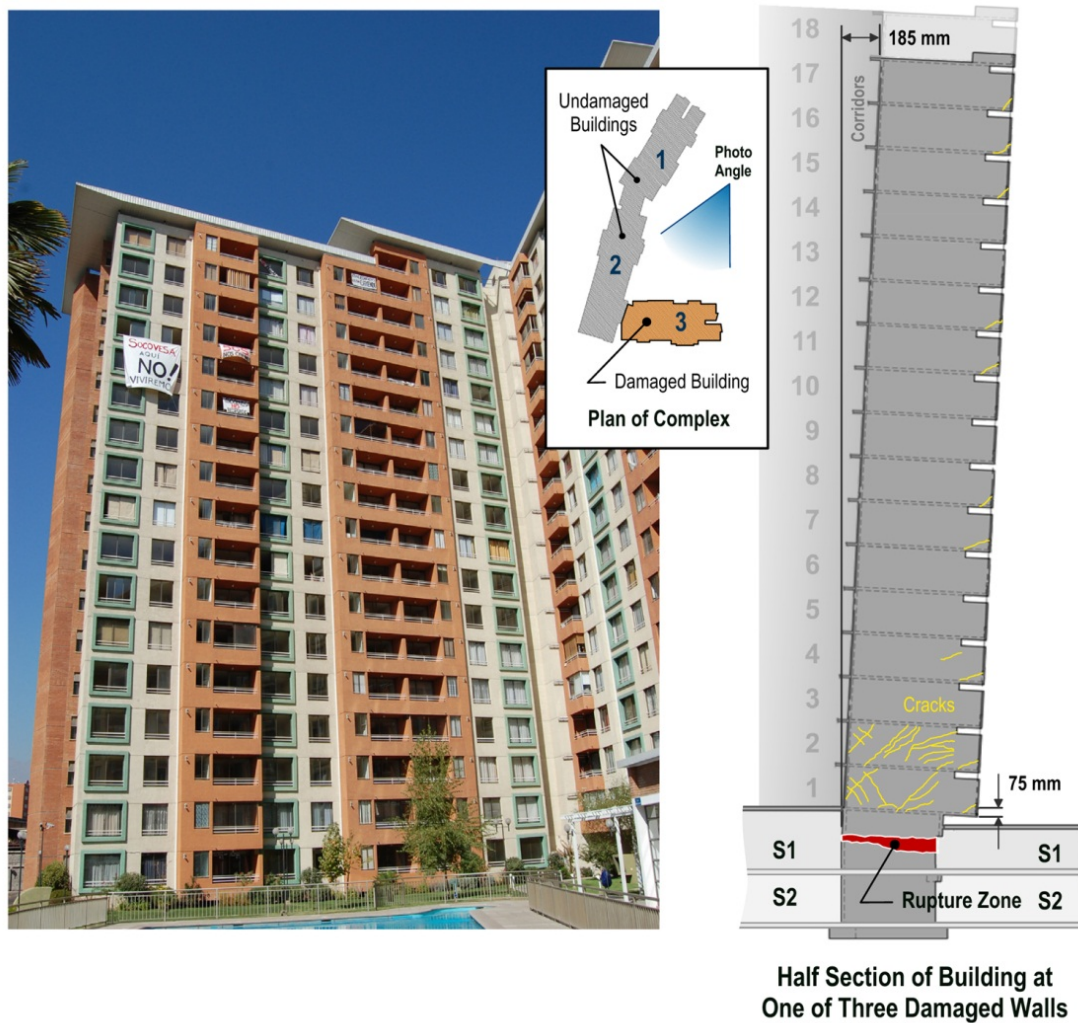


Figure 1. Earthquake damaged building

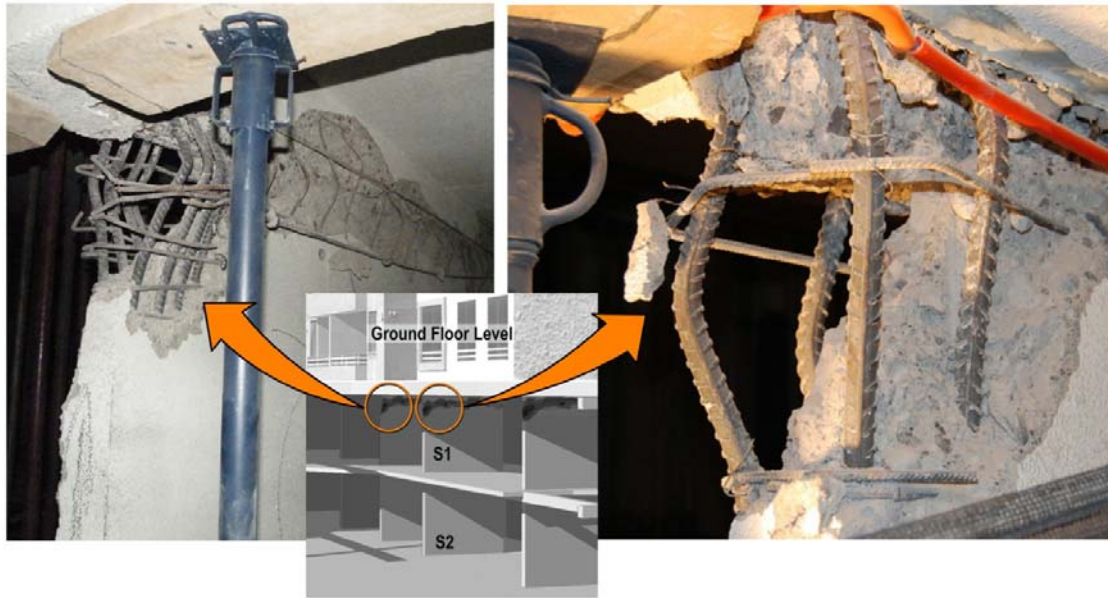


Figure 2. Flexural compression failure of shear walls in first level (S1) below grade

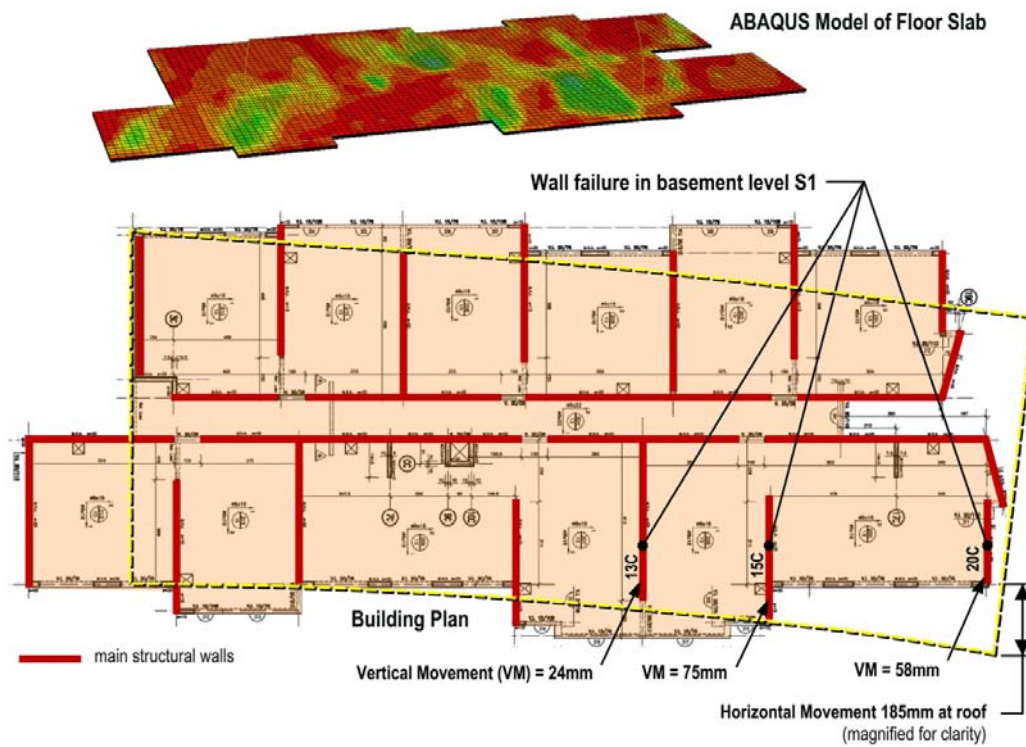


Figure 3. Plan view of building showing main structural walls, residual vertical movement of walls at grade level above damage, and outline of residual horizontal movement at roof level; finite element model of floor slab (top)

2. FLEXURAL COMPRESSION FAILURE OF THIN SHEAR WALLS

Nonlinear finite element analyses of the concrete shear walls (Adebar, 2012) indicated that the maximum concrete compression strain demands at the level immediately below grade are significantly magnified by a sudden increase in wall length above grade. The overhanging wall above causes a concentration of compression stress and strain at the outside edge of the wall below. This is exactly where the compression failure of the shear walls occurred in the building (Fig. 2).

Recent experiments on concrete shear wall elements have shown that 150 mm and 200 mm thick concrete shear walls with two layers of horizontal reinforcement may fail very suddenly at compression strains less than 0.0015 (Adebar and Lorzadeh, 2012). The reason for the failure at such low compression strains is that the horizontal reinforcing bars parallel to the wall faces create stress concentrations and micro-cracking in concrete surrounding the bars. As the concrete stress approaches the compression strength, the concrete surrounding the bar, and particularly the concrete cover outside of the bar, becomes unstable. In a thin wall, the concrete surrounding the two layers of horizontal reinforcing bars and the concrete cover outside these bars make up a very large portion of the wall thickness and so the wall fails when this concrete starts to become unstable. It is interesting to note that nominal column ties were found to stabilize thin walls and allow the walls to tolerate compression strains greater than 0.003 before failure (Adebar and Lorzadeh, 2012).

3. DESCRIPTION OF BUILDING

The building was constructed in 2006, and has approximately 7,000 m² of usable residential area in 18 stories; there are two levels of underground parking. The typical floor height is 2.5m, the first basement story (S1) height is 3.6m and second basement story (S2) height is 3.1 m. The top of the roof slab is 51.7m above the top of the foundation.

The building structural system consists of reinforced concrete flat plate slabs supported by shear walls. The two-way floor slabs are 150 mm thick and the one-way slab in the central corridor is 120 mm thick. Perimeter walls have spandrel beams under window openings. All slab reinforcement consists of 8 mm diameter reinforcing bars spaced at 180 mm in both directions top and bottom in two-way slabs and spaced at 220 mm in one-way slabs. The foundations under shear walls consist of 0.8 m deep strip footings that vary in width from 1.3 to 2.0 m.

The distributed reinforcement in the 200 mm thick shear walls consists of two layers of 8 mm diameter reinforcing bars spaced at 200 mm vertically and horizontally. Below the second floor level, the spacing of the horizontal wall reinforcing is reduced to 150 mm. The end of every wall has a minimum of four concentrated vertical reinforcing bars that are 25, 22, 18 or 16 mm in diameter. The concentrated vertical reinforcing bars do not have any ties to confine the concrete or prevent buckling of the vertical reinforcement. The horizontal reinforcing bars extend to the end of the walls and are anchored by a 90-degree bend around the last vertical bar. Based on cylinder tests conducted at the time of construction, the concrete compressive strength was assumed to be 25 MPa, and the Modulus of Elasticity of concrete was assumed to be 32,000 MPa.

4. ANALYSIS OF DAMAGED BUILDING

Detailed nonlinear analyses of the individual damaged shear walls and damaged floor slabs were used to develop a three-dimensional linear model of the entire building with reduced effective stiffnesses to account for damage combined with applied forces to simulate residual displacements. Nonlinear analyses were conducted on a number of two-dimensional frames consisting of a damaged shear wall connected to an undamaged shear wall by the floor slabs. The finite element models of each wall simulated the actual amount and arrangement of the wall reinforcement. Fig. 4 shows only a portion of one finite element model, for only the lower part of the damaged wall. The dead load acting on the

walls due to the wall self-weight and tributary area of the slabs was included at each floor level. Nonlinear finite element program VecTor2 (Wong and Vecchio, 2002) was used because it has state-of-the-art material models for cracked reinforced concrete subjected to shear combined with axial load and bending moment.

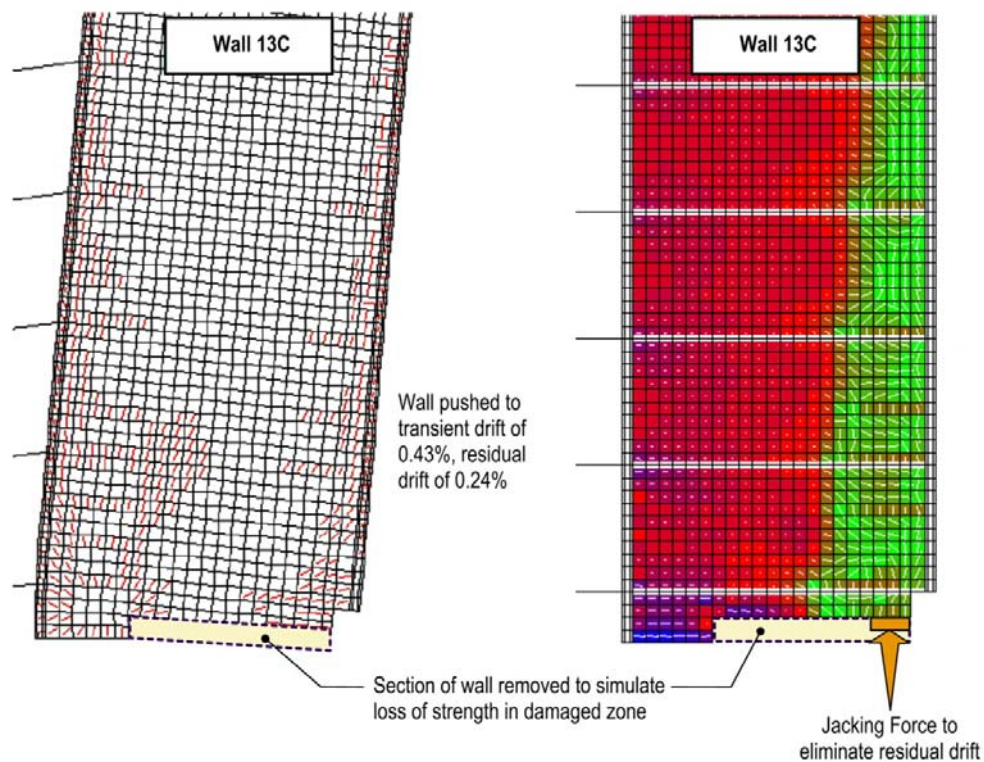


Figure 4. Nonlinear finite element analysis of damaged shear walls (small portion of walls shown): reverse cyclic loading to simulate earthquake damage (left); simulated jacking (right)

The two-wall frames were subjected to a three-step analysis. In the first step, the two walls started as undamaged and were subjected to reverse cyclic lateral loading. The maximum global drift that was applied was about 0.15% and this produced many of the cracks that were observed in the walls after the earthquake. This drift level corresponds to the expected demand based on a linear analysis of the building subjected to a typical ground motion recorded in Santiago during the earthquake. In the second step, elements were removed (Fig. 6) to simulate the damage observed at the base of one wall in each wall-pair. The two walls were then subjected to additional cycles of lateral load to a drift level that produced the correct level of residual horizontal displacement at the roof level and residual vertical floor displacement at the ground floor level. In the third step of the nonlinear analysis, the simulated applied vertical jacking forces were increased until the residual displacements of the walls were eliminated. For walls 15C and 16C the required jacking force was 306 tonnes, while for 13C and 14C the jacking force was 316 tonnes. Note that these are only one component of the total jacking force needed to straighten the entire building.

Nonlinear computer models, using ABAQUS, of two complete floor slabs were used to understand the observed residual deformations and to determine the effective stiffnesses of the slabs. The slabs were subjected to vertical and horizontal forces applied at the shear wall locations that when removed from the slabs, resulted in the observed residual deformations of the slabs at two representative elevations in the building. The nonlinear FE models were used to determine the secant stiffnesses used in a three-dimensional linear model that incorporated the damaged slabs.

The full three-dimensional linear model of the building was developed using ETABS. The effective stiffnesses of the damaged shear walls and the lateral loads used to simulate residual lateral displacements in the linear analyses were determined as follows. First two-dimensional linear models of the two-wall frames were developed similar to what was done in the nonlinear analysis of the shear walls. Elements were removed from base of the walls to simulate wall damage. All dead loads and jacking forces determined in the nonlinear wall analyses were applied to the linear wall models. The effective stiffnesses of the damaged shear walls were reduced a variable amount over the height of the wall so that areas with more visible damage had larger reductions in effective stiffness and the deflected shapes of the walls were the reverse of the observed residual displacement profiles. Lateral loads were then applied over the height of the shear walls until the lateral displacements of the walls were zero. With these effective stiffnesses and lateral loads applied to the linear models of the walls, the influence of the jacking force was very similar in the linear and nonlinear models – removing the jacking force caused the walls to have the observed residual displacements and applying the jacking force caused the walls to straighten.

The linear models of the damaged shear walls described above and similarly developed linear models of the damaged floor slabs were combined into a three-dimensional model of the entire building. A uniform 20% reduction in flexural stiffness was used for all undamaged shear walls to simulate the nonlinear flexural behaviour using a linear model. Additional lateral loads were applied to the building until the displacements were similar to the measured residual displacements of the building. The 3D linear model was now ready to be used to estimate the jacking forces accounting for the interaction of shear walls and slabs. A study was done with this model to determine the relationship between jacking forces and building movements. An upper-bound estimate of the jacking forces required to eliminate the residual drifts was determined to be 500, 600, and 500 tonnes for shear walls 13C, 15C and 20C, respectively. Note that the tributary gravity loads due to the dead load of the structure on these same three walls are 408, 306, and 265 tonnes, respectively. Additional information about the analyses that were done on the damaged building is given by Sherstobitoff et al., 2012.

5. TEMPORARY STRUCTURES

New concrete foundations were built around existing foundations of the three damaged shear walls. In addition, a 300 mm thick heavily reinforced concrete slab was cast on top of the new and old foundations. These slabs were used to support eight temporary steel HSS columns that extended through holes cut into the first underground level slab S1. A grillage of steel plate girders was used to transfer the loads from four jacks under each wall to the eight temporary HSS columns. This arrangement for the main jacks was typical at each of the three damaged walls.

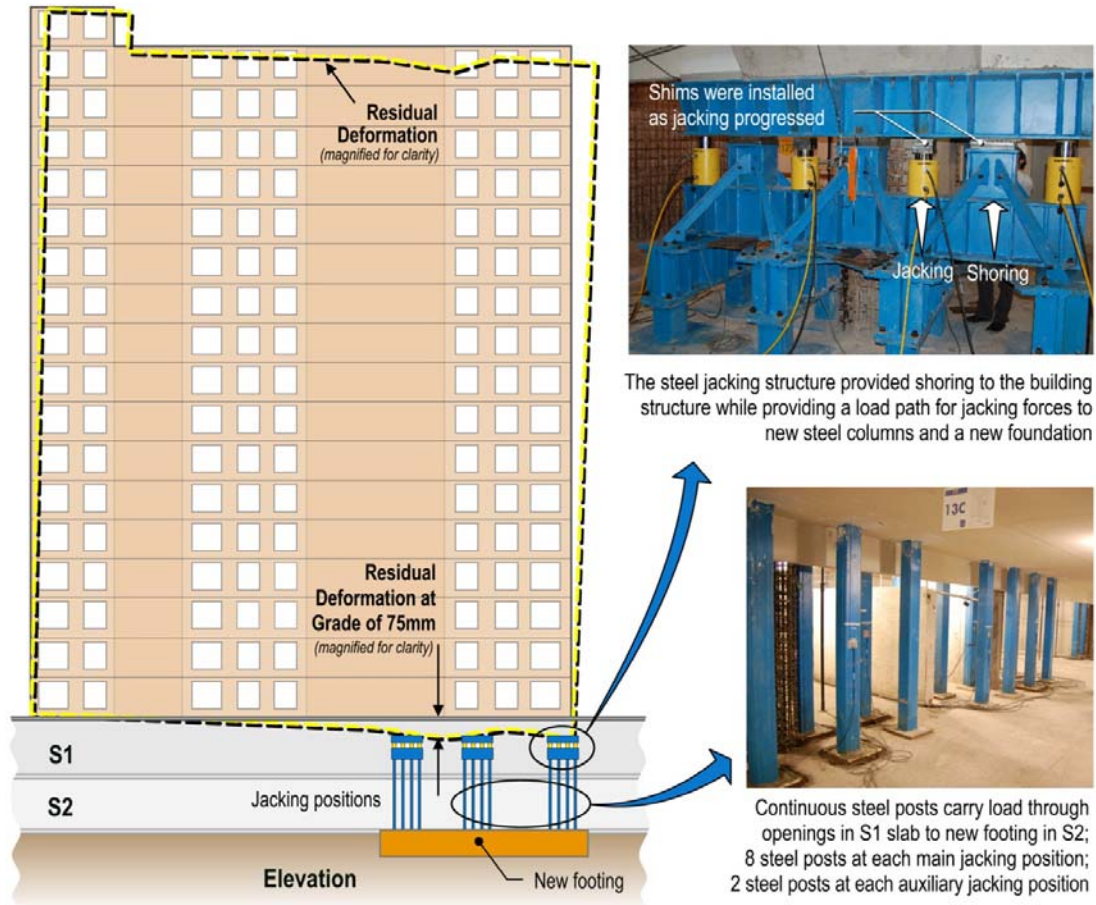


Figure 5. Repositioning of building by jacking at base

The jacking structure was designed so that supports adjacent to the jacks could support the walls when the jacks were lowered (Fig. 5). Shim plates were inserted above these supports as the wall moved upward. The hydraulic jacks were used to lift the structure a certain amount and then shims were inserted, and the force in the hydraulic jacks was reduced to transfer the force onto the steel girder/column jacking/shoring system. Two sets of jacks (Fig. 6) were used for damaged walls 15C and 20C (larger jacks at the outside end of the wall and secondary jacks at the inside end), while only one set of jacks was used for wall 13C. The same structure used for jacking was designed to shore the building while the damaged portions of the concrete walls were repaired.

In order to transfer the load from the jacks into the damaged concrete walls without crushing the concrete wall, new reinforced concrete corbels were constructed (Fig. 6). Finite element analysis of the corbels was used to ensure the concrete compression stresses in the walls remained below a safe limit so that the thin walls did not crush.

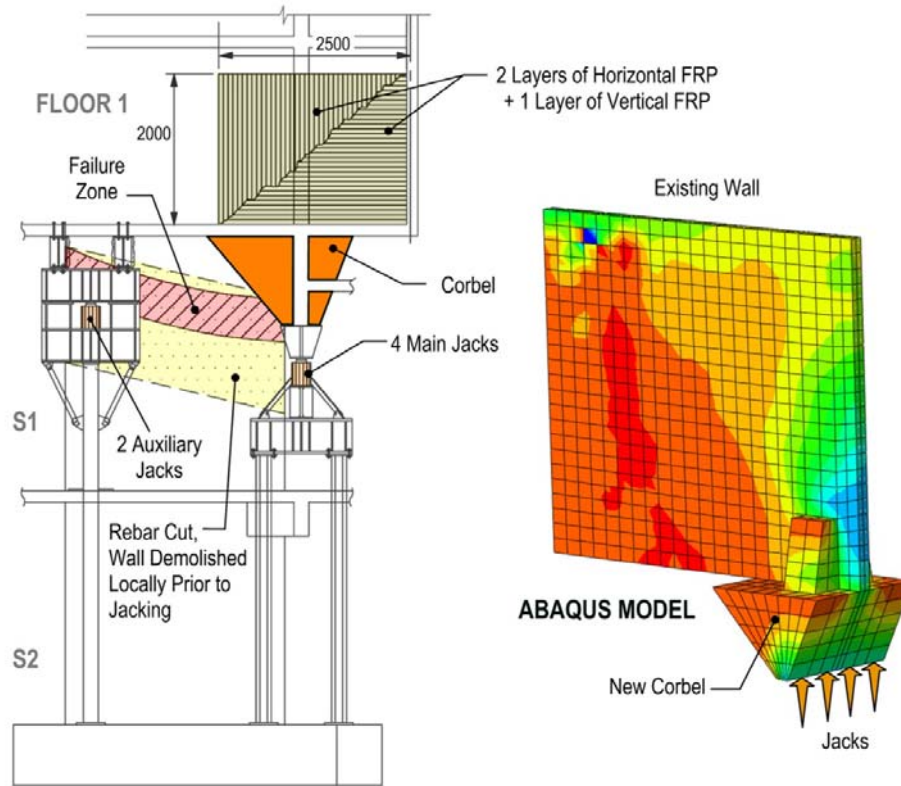


Figure 6. Details of jack set-up (left); finite element analysis of local concrete stresses due to jacking forces

Analysis indicated that the jacking forces could cause new cracks to form in the concrete shear walls. As the concrete walls are lightly reinforced, carbon fibre reinforced polymer (FRP) fabric was added to the highly stressed areas of the wall (Fig. 6). Together, the plies of FRP on both sides of the wall were designed to resist a total tension force per unit area of wall equal to 2 MPa. The distributed reinforcement within the shear walls had a tension force capacity per unit area of wall equal to 1.0 MPa. In addition to providing crack control, the FRP was also used to increase the ductility (strain capacity) of the walls in compression. FRP fibre-anchors connecting the fabric on the two sides of the wall were spaced approximately at 600 mm in the highly compressed zones of the wall.

All significant cracks in the concrete shear walls were repaired by epoxy injection. As the jacking was expected to close many of the existing cracks, the injection was not done until after the third day of jacking. At that point, the width of most cracks had reduced significantly. All cracks with a remaining width greater than 0.1 mm were injected.

6. INSTRUMENTATION

A survey team was used to monitor three sides of the building concurrently during jacking. Two survey control networks were established – one on the outside of the building and one on the inside of the building. This enabled surveyors to observe the select monitoring points outside the building at four different locations and at three different levels for a total of 24 observation points. The surveys inside the building measured floor slopes and deflections between the corridor and the outer wall, at various levels of the building.

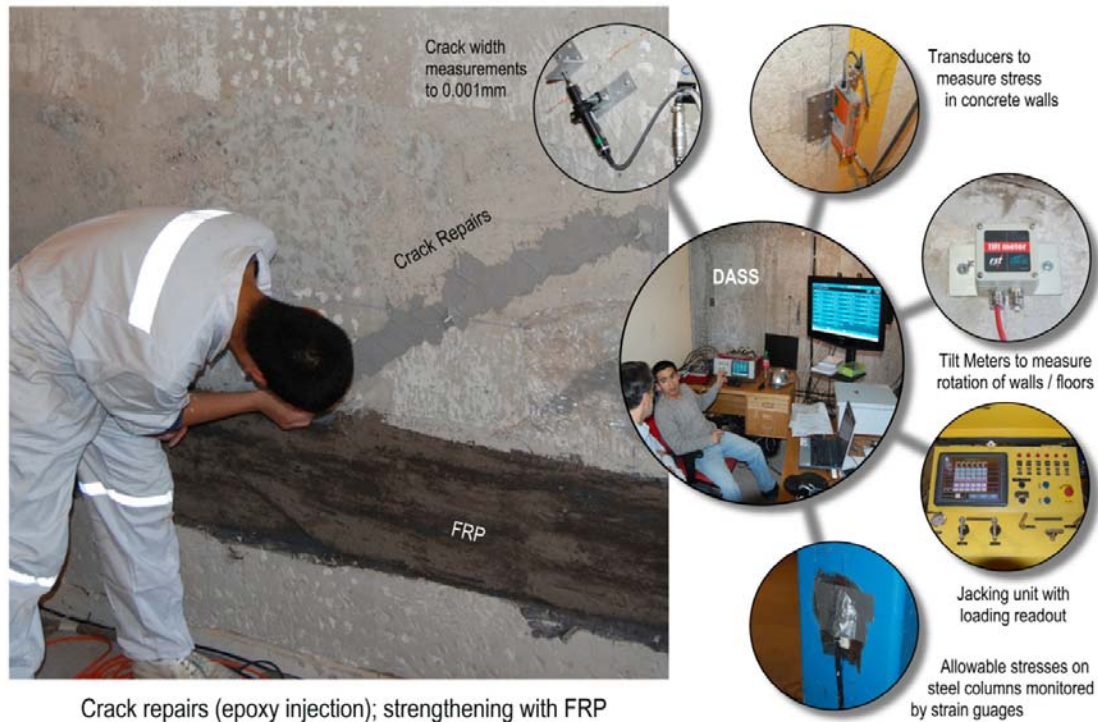


Figure 7. Crack repair (left); instrumentation (right)

The location of potential new cracks in the building due to jacking was known from the analysis. Finishes such as wall paper and tiles were removed from the surfaces and all existing cracks were marked to permit early detection of any new cracks. The widths of existing cracks in the shear walls were measured using 12 displacement transducers over a short gauge length across cracks.

The shear walls immediately above the jacking points were the most highly stressed portions of the building, and therefore were carefully monitored during jacking using 12 displacement transducers. These transducers were used over the first two stories to measure the maximum compression strains in the walls, as well as to verify that there was no uplift on the tension end of walls. Bi-axial tilt-meters were installed on damaged walls and slabs adjacent to these walls, at the ground-floor level and at the 15th floor, to permit the real time viewing of the change in floor and wall slopes.

The force in the hydraulic jacks was monitored during jacking. The steel columns that transferred the jacking loads from the first level below grade to the foundation (Fig. 5) were instrumented with strain gauges to monitor the axial load and bending moment in these members. The top of the new foundation used to support the jacks was monitored for vertical movement using survey equipment. All electronic instruments were monitored continuously from a central data acquisition station (Fig. 7).

7. MEASURED BUILDING MOVEMENTS

Jacking of the building took place over a seven day period. On the first day, the maximum jack forces reached 150, 150 and 50 tonne on walls 13C, 15C and 20C. As a result of these jack forces, the building moved up 7, 10 and 6 mm at walls 13C, 15C and 20C. During day 2, the maximum jack forces were increased by 180, 190 and 170 tonne, and the building moved upward an additional 11, 22 and 17 mm, respectively.

At the end of day 2, the average width of all cracks in the three walls had reduced to about 40% of the initial widths over the first two stories of the damaged walls. Over the next three days, FRP was applied to the highly stressed regions of the walls to control any new cracks that may form during additional jacking.

On the third day of jacking, the jack forces were increased by 20, 60 and 60 tonne, and the additional upward movement of the building was 4, 8 and 6 mm. The average width of cracks in the three damaged walls had reduced to 34% of the initial width over the first two stories of the walls. During the evening of Day 6, all shear wall cracks with a remaining width larger than 0.1 mm were injected with epoxy.

On the final day, the jacking forces were increased by 30, 280 and 140 tonne, and the additional upward movements were 9, 29 and 21 mm at the three wall locations resulting in final residual displacements of +7, -6 and -8 mm (+ indicates above position prior to earthquake damage).

8. CONCLUSION

The actual jacking forces on the three walls were 380, 680 and 420 tonne (1480 tonne total); while the predicted jacking forces to reach zero displacement was 500, 600 and 500 tonne (1600 tonne total). The distribution of jacking forces was somewhat different than predicted; but the total jacking force was 93% of the predicted total jacking force.

Vertical alignment of the building was restored to within 10 mm over 18 stories, and horizontal alignment of the ground floor slab was within 8 mm, both of which are well within acceptable tolerances for new construction. The horizontal alignment of the upper floors was good enough that a conventional repair could be used to level the floors.

ACKNOWLEDGEMENTS

Many people contributed to the successful repair of the building. The engineers at Ausenco Sandwell included Reza Mousavi, Sasan Iranpour and Zahra Riahi. Primo Cajiao was with Ausenco Sandwell during the course of this project. Stephen Mercer from The University of British Columbia conducted the VecTor2 nonlinear analyses, while Dennis Mitchell of McGill University, Montreal was a consultant. Construction work was done by SOCOVESA, Santiago. Input from original designer was provided by Gonzalo Santolaya of Santolaya Ingenieros Consultores of Chile. Jacking consultant was John Brise of Apex Industrial Movers, BC Canada. Surveying and monitoring services were provided by SOCOVESA and IDIEM, University of Chile, Santiago.

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