Seismic Assessment of a 15-Story Building Damaged in the Chile Earthquake of February 27th 2010

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SUMMARY

This paper considers the seismic assessment of a 15-storey RC structural wall building, Alto Rio, located in the city of Concepción that collapsed in the M8.8 Chile earthquake of Feb 27th 2010. Different methods of assessment, based on FEMA356, are applied to examine if the observed damage could have been anticipated. Results show that FEMA356 assessment methods would have identified that the building was likely to be damaged in a major earthquake. Severe irregularities and discontinuity of walls at the base of the structure, in general, and lack of ductile detailing in walls and coupling beams are identified as the likely reasons for the damage. In addition to this assessment, the design procedures that are expected to have been used for the original design of the building are reviewed. The results showed that in applying the code design procedure, wall axial tensile forces were probably underestimated and improved capacity design procedures should be implemented.

Keywords: Seismic assessment, Chile earthquake, AltoRio building

1. INTRODUCTION

Seismic assessment of the Alto Rio building that collapsed in the Chile earthquake of 27th Feb 2010 is provided in this paper. The earthquake was the sixth largest earthquake ever to be recorded. The building was located in the city of Concepcion 100km from the epicentre of the earthquake. At the time of the earthquake, 87 people were inside the building among whom eight people died and 79 survived (52 escaped on their own efforts, 27 were rescued) (Elnashai et al., 2010).

A number of buildings in the vicinity of the Alto Rio building were badly damaged by the very intense and long shaking duration, but did not collapse like the building as shown in Figure 1.1. As reported by (Elnashai et al., 2010), the building appears to have structurally failed at the connections between the first floor and the two basements. Subsequent motions then caused the building to collapse on its side as shown.

The question was raised whether the collapse of this building was caused by the large co-seismic displacement that may have generated a global overturning of the building in a manner similar to toppling of a chair when the rug under it is suddenly pulled away; or if the inertial earthquake forces that caused failures of other buildings in Concepción and elsewhere in Chile were the main cause of the collapse. However, Alimoradi and Naeim (2010) showed that the co-seismic displacement was most probably not the cause of the building collapse.

Figure 1.1 Picture of Alto Rio building before and after the Chile earthquake, Feb 2010 (Concepción under Construction, 2010 left; El Periodista, 2010 right)

Investigating the latter type of collapse, which is the more probable type, is the purpose of this study. In order to examine if the available code based assessment methods would have been capable of anticipating the observed failure, a code based assessment according to FEMA356 is performed. Both the Linear Static and Nonlinear Static analysis methods are applied in order to address the problem as well as to gauge the applicability of the methods. In addition, in order to examine the reason why the observed fragility of the building was overlooked in the original design, the design procedure that might have originally been applied by designers of the building is reviewed.

2. DESCRIPTION OF THE STRUCTURE AND SEISMIC ACTION

2.1. Alto Rio Building

The Alto Rio building possessed two basements surrounded by retaining walls of thickness 25cm and 15 stories above the ground level consisting of shear walls with thickness of 20 cm, seen in Figure 2.1.

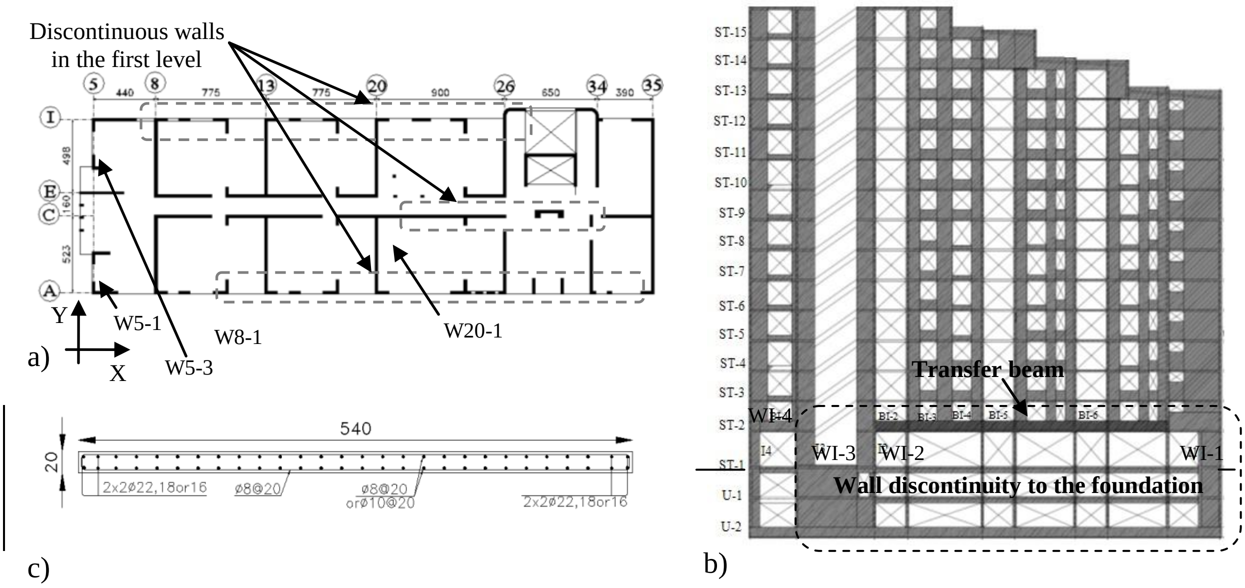

Figure 2.1 Building geometry: a) Typical story plan, b) Most critical elevation (Elevation I) with wall discontinuity to the foundation, c) Typical wall cross sections (e.g. W8-1 and W20-1)

The story height was 252cm, except for the first storey which was 306cm. The floor system is concrete two-way slab with 15 cm thickness. The typical building plan and what the authors consider to be the most critical building elevation are shown in Figure 2.1. In the X direction, the system is more a coupling wall system with strong beams that connect the walls. In the Y direction, two single walls in each elevation form the structure. In this direction, the connection of slabs to the wall is expected to have provided a coupling effect between the walls.

The structure is irregular in height because over the upper stories (12 to 15) it is set back in plan (see Figure 2.1). The plan of the structure in the underground levels is rectangular with dimensions of 45.6x22.8m whereas the typical stories above the ground level have plan dimensions of 39.3x11.8m.

2.2. Seismic Action

For this study, a ground motion recorded in station “Colegio San Pedro de la Paz” 8 km away from the epicenter of the earthquake and 102 km away from Alto Rio building has been used. The location of the building with respect to the epicenter and record station are shown in Figure 2.3.

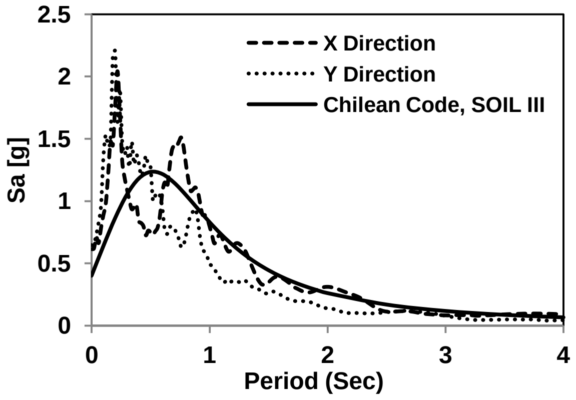

Figure 2.2 Response and Design spectra according to Chilean seismic code

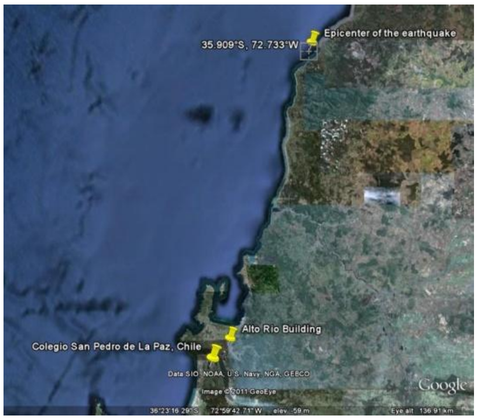

Figure 2.3 Location of *Colegio San Pedro de la Paz* station with relation to the site (Google earth)

Two response spectra of the records in the perpendicular directions (named as X, Y) are compared with the design spectrum of the Chilean code (NCh-433/96) in Figure 2.2. In calculating the design spectrum for this study, zone 3 with soil type III according to the Chilean seismic code is considered. An interesting observation is that the design spectrum of NCh-433/96 matches closely with the response spectrum obtained from a recording of the earthquake 8km from the site. The 27 February 2010 event could be considered a design intensity scenario for the city of Concepcion.

2.3. Modeling assumptions

SAP2000 (computers and structures, 2010) is used in order to conduct various analyses of the building. A 3D model is developed for the modal and linear analysis. Nonlinear Static Analysis is performed for two separate 2D models for the X and Y directions. For modelling the wall elements and the coupling beams, frame elements are used and each element is positioned at the centroid of the section. The cracking effect in RC elements was taken into account by a reduction factor of 0.5 of the cross section stiffness ($0.5E_c\ I_g$). It is noted that more accurate estimates of the cracked stiffness could be obtained (see Priestley et al. 2007) but the 50% reduction was adopted here as an approximation that was considered reasonable given the other arguably greater uncertainties in the seismic assessment. A global view of the 3-D model in SAP2000 is shown in Figure 2.4.

In order to take into account the effect of intersecting walls acting as T-shape, L-shape and U-shape sections, the vertical beam elements (walls) are linked together by rigid arms (rigid link beams) at each floor level. It is important that these rigid arms are modeled with the properties that are relatively rigid in the plane of each wall panel but not out of plane. To achieve this, a reasonable starting point is to assume that the rigid arm properties are based on a section with depth equal to the floor to floor height or vertical spacing of rigid arms and thickness equal to that of the wall, and then adjust the properties of the rigid elements as follows (in line with the recommendations of Arnott, 2005): 1) Ix- Torsion constant (out of plane effects) is reduced by a factor of 10 and 2) Iy- In plane stiffness is increased significantly by a factor of 100.

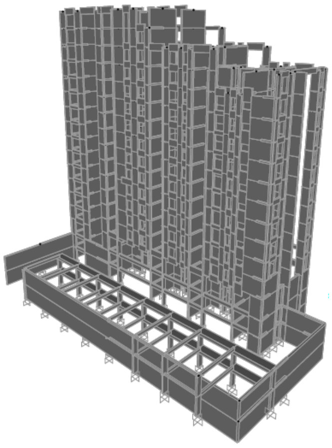

Figure 2.4 3D view of the model in SAP2000

The coupling effect of the slabs and the beams with T shape cross section are not considered in the modeling. The building in the underground levels is surrounded by retaining walls with a thickness of 25 cm and therefore the retaining walls up until ground level would provide a relatively rigid support for the structure. As such, in the model the first two stories are laterally restrained.

3. Analysis Results

From Eigen value analyses of the structure model described in the previous section it is found that the first mode of the structure is translational in the X direction (longer side) with a period of 1.09s. The second mode is translational in the Y direction (shorter side) with a period of 1.07s and the third mode is determined as rotational with a period of 0.83s. The relatively short periods of the 15-storey structure obtained from the modal analysis are typical of the constructional practice in Chile (Boroschek, 2010), since by the application of many structural walls, the stiffness of structures in Chile tends to be very high and consequently short periods are observed.

3.1. Linear Static Procedure

According to FEMA356, the pseudo lateral load in a given horizontal direction of the building is calculated according to the following equation.

$$V = C_1 C_2 C_3 C_m S_a W \quad (3\text{-}1)$$

The values of the coefficients C_1, C_2, C_3, C_m are defined according to FEMA356 and the definition of each one can be found in the code, S_a is the spectral acceleration and W is the total weight of the building. For the current structure, the base shear is calculated using the following coefficients: C_1 =1.0, C_2 =1.0, C_3 =1.0, C_m =1.0, S_a = 0.68.

The deformation controlled actions of each element can be calculated as follows:

$$Q_{UD} = Q_G \mp Q_E \tag{3-2}$$

In which, Q_{UD} is the design action due to gravity loads and earthquake loads, Q_E is the action due to the design earthquake loads and Q_G is the action due to design gravity loads. Therefore, the Demand Capacity Ratio (DCR) for the deformation controlled actions can be calculated according to the following equation:

$$DCR = \frac{Q_{UD}}{mkQ_{CE}} \tag{3-3}$$

In which, m is the Component or element demand modifier (factor) to account for expected ductility associated with the action at the selected Structural Performance Level which is an indirect measure of the nonlinear deformation capacity of the component or element. The m factors are adopted from FEMA356 considering the CP (Collapse Prevention) limit state (and are shown later in Table 3.1). k is the knowledge factor which for the case of the current structure is considered equal to one, finally, Q_{CE} is the Expected strength of the component or element.

On the other hand, force controlled actions, Q_{UF}, can be calculated using a number of methods. It shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering load to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building. Alternatively, Q_{UF}, can be calculated according to the following equation:

$$Q_{UF} = Q_G + \frac{Q_E}{C_1 C_2 C_3 J} \tag{3-4}$$

In which Q_E is the action due to the design earthquake loads and Q_G is the action due to design gravity loads. C1, C2 and C3 are the values defined in FEMA356 which are the same as indicated in Eq. (3.1). J is a Force-delivery reduction factor, greater than or equal to 1.0, taken as the smallest DCR of the components in the load path delivering force to the component in question. Alternatively, values of J equal to 2.0 in Zones of High Seismicity, 1.5 in Zones of Moderate Seismicity, and 1.0 in Zones of Low Seismicity shall be permitted when not based on calculated DCRs. In this study, a value of J=2 is used. According to FEMA356, in the case of concrete structural walls, moment and shear actions are considered as deformation controlled actions and axial load as a force controlled action. However, in the case of concrete beams, only moment is suggested to be deformation controlled and shear should be considered as a force controlled action.

The results of Linear Static Assessment are shown in Table 3.1. The name of each wall and beam was indicated earlier in Figure 2.1. The results indicate the evaluation of the wall sections in the first floor of the building, where the formation of plastic hinges and expected failure is more likely to occur. The analysis is performed for both positive and negative directions and the results for the most critical cases are shown here. The main problem observed in the wall sections in the X direction emerges from the axial tension forces developed in the coupled walls. For example, in walls WI-1 to WI-4, the axial tension demand coming from earthquake and gravity loads are higher than the maximum axial tension capacity of the wall calculated using the longitudinal reinforcement indicated in construction drawings. The tension yielding of the reinforcing bars could cause significant bar elongation and probably, in the next cycle of oscillation, the reinforcing bars would then have experienced a buckling failure in compression leading to the failure of the whole section. In some exaggerated cases, the tension force in the wall is so high that the rupture in the reinforcing bars is likely which can be considered as the wall failure. On the other hand, even for the cases where the axial tension demand doesn't surpass the axial tension capacity, the flexural capacity of the section reduced to very low values. The same issue is observed for the walls in elevation "A" and "C" where, similarly, the walls are not continuous into the foundations.

Table 3.1 Results of Linear Static Analysis According to FEMA356

	Demand[1]			Capacity				Demand/Capacity (DCR)		
Wall Name	P [6]	M	V	P[2]	M [3]	V [4]	m	P	M	V
W-I1	6417	3245	1148	1577	0	853	2.0	3.9	NC [5]	0.7
W-I2	1924	1357	1576	1493	0	694	4.0	1.1	NC	0.6
W-I3	2233	3122	1229	1493	0	788	2.0	1.3	NC	0.8
W-I4	1928	534	487	874	0	265	2.5	1.9	NC	0.7
W5-1	2278	790	144	937	0	338	2.0	2.2	NC	0.2
W5-3	1840	874	282	937	0	338	2.0	1.7	NC	0.4
W8-1	-50	28815	1578	2541	13396	2594	3.0	-	0.7	0.2
W20-1	-68	25267	2546	2541	14550	2695	3.0	-	0.6	0.3

1- Axial load, moment and shear demands derived from linear analysis (kN, kN. m)
2- Maximum axial tension capacity of the wall
3- Maximum moment capacity of the wall corresponding to the axial load derived from moment-axial load interaction curve
4- Shear capacity of the member according to ACI-318-02
5- NC=No moment Capacity due to excessive axial load
6- Positive values are tension and negative values are compression

For the walls in the Y-direction, in elevations 8, 13, 20, 26, 34 and 35, which have two walls of length 540 cm without many effective discontinuities and irregularities, the values of DCR for flexure and shear are less than 1.0 except for the walls in elevation 5. For example, in walls W8-1 and W20-1, the expected seismic axial forces are very small and thus negligible. Even though minimum shear reinforcement was typically provided to the walls, it was found that for the walls under consideration in the first floor there were only three cases in which a shear DCR of higher than 1.0 is observed. Therefore, it can be concluded that the occurrence of shear failure should not be expected to have preceded the axial-flexural failure of the examined cases.

3.2. Potential impact of the Chilean Code on the design of the Alto Rio Building

In order to examine the probable design procedure that was used for the original design and construction of the building, a code based design approach is adopted in this section and then comparisons are made with FEMA356 linear assessment method. Chilean seismic code (NCh043, 1996), suggests a Modal Response Spectrum analysis method rather than linear analysis for structures higher than 6-stories. Therefore, in the following, the MRS method has been applied for the analysis of the structure as this is likely to have been the method used for the design of the building. The design spectrum according to NCh-043 can be derived as follows:

$$S_a = \frac{I A_0 \alpha}{R^*}, \ \alpha = \frac{1+4.5\left(\frac{T_n}{T_0}\right)^p}{1+\left(\frac{T_n}{T_0}\right)^3} \qquad (3\text{-}5)$$

In which, S_a is the design spectral acceleration, I is the importance factor which for the residential buildings can be considered equal to 1.0. A_0 is the maximum effective acceleration according to the national seismic zoning. For the city of Concepcion (zone 3) A_0=0.4g can be considered, α is the amplification factor which accounts for the spectral shape. In which, T_n is the vibration period of mode n, T_0 and p are parameters relative to the foundation soil type. According to NCh-043, these values can be assumed equal to 0.75 and 1.0, respectively. R^*, is a reduction factor that for structural wall systems can be calculated as follows:

$$R^* = 1 + \frac{NR_0}{4T_0R_0+N} \tag{3-6}$$

In which, N is the number of stories and R_0 is a modification response factor. For structural wall buildings a value of 11 is suggested for R_0. Using N=15 and T_0= 0.75, a value of R*=4.43 is obtained and with the application of the design spectrum an MRS analysis is performed. The two load combinations of $U1: 1.2D + E, U2: 1.2D - E$ adopted from ASCE7-05 were observed to be the most critical load cases and thus the results of these two load combinations are reported in Table 3-2. The last three columns in the table show the capacity over demand ratio of the walls in shear and flexure.

Table 3-2 Simulated design summary of walls I-1 to I-4 and 5-1 to 5-4 according to Chilean code, NCh-043

Wall Name	P_D, T^1	P_D, C^2	$M_D{}^3$	$V_D{}^4$	M_N, T^5	M_N, C^6	V_N	$\frac{M_D}{\emptyset M_N, T}$	$\frac{M_D}{\emptyset M_N, C}$	$\frac{V_D}{\emptyset V_N}$
WI-1	1692*	-3270	843	266	0	3374	872	0.0**	0.28	0.40
WI-2	47	-1137	933	422	4234	4550	678	0.24	0.23	0.83
WI-3	27	-1193	185	327	355	630	702	0.59	0.32	0.63
WI-4	220	-1471	149	146	264	670	436	0.63	0.24	0.45
W5-1	216	-1018	133	118	244	550	387	0.63	0.27	0.40
W5-3	118	-920	219	123	255	560	395	1.00	0.43	0.42

1-Axial load, load combination U2, kN

2-Axial load, load combination U1, kN

3-Design Moment, kN. m

4- Design shear force, kN

5-Nominal moment resistance for load combination U2, kN. m

6-Nominal moment resistance for load combination U1, kN. m

* Positive values are tension and negative values are compression forces

** Axial tension force exceeds the axial tension capacity; therefore, no flexural resistance is expected.

As can be seen, except for the case of wall WI-1, the ratio of demand over capacity is less than one. This general trend could raise doubts as to whether a force reduction factor of 4.4 was actually adopted because if it had been, one could expect the demand to capacity ratios to equal 1.0. However, lower values may be indicative of the typical practice of providing more strength than necessary when sizing reinforcement and seeking standardized construction details. Note also that in the linear assessment procedure of FEMA356, the *m* factors suggested for the consideration of the nonlinear behavior of the wall elements were in range of 2-2.5 for most of the cases, which does appear to match the construction solution well. This value (R*) is used to reduce all actions consisting of axial load, moment and shear.

As it was mentioned before, according to FEMA356, the axial load action in the structural walls is considered force-controlled and therefore it is suggested to reduce the axial force coming from earthquake by a J=2 factor for assessment of the element. As it was observed in Table 3.1, in most of the critical walls, the axial tension load demand exceeds the capacity and consequently results in zero moment capacity. However, in the design procedure, all of the actions- axial load, moment and shear, coming from the earthquake would be reduced by a factor of R*=4.43. This would lead to a significant reduction in the tension axial load on walls in design compared to the axial loads predicted from the FEMA356 assessment approach. This highlights an issue with the design approach since it is evidently not conservative to assume that flexural strength can be set greater than the design flexural demand without also checking the consequences on the strength-dependent actions, such as axial forces. This reflects a potential shortcoming with the capacity design procedures used in the design of the Alto Rio building, since a rigorous capacity design approach (see, for example, guidelines provided in Priestley et al. 2007) would have established peak axial forces based on the strengths provided to potential plastic hinge regions.

An alternative explanation for the apparent underestimation of axial forces may have been an error in interpreting the axial forces obtained from modal response spectrum analysis in which computer software makes a combination of axial force computed and reports the absolute values. Without care, the SRSS or CQC axial forces may be interpreted as forces in compression rather than tension. As the bending resistance of RC walls greatly reduces when walls are subject to tension, it is apparent that this could be a reason for the poor seismic performance observed of the building.

It is worth mentioning that FEMA356 suggests m values of 4 and 6 for concrete structural walls in collapse prevention limit states for the cases in which a confined boundary element is provided for the section and the axial compression force is less than *0.1* and *0.25* $f'_c A_g$, respectively. However, appendix B of the Chilean seismic code explicitly mentions that "*When designing reinforced concrete walls, it is not necessary to meet the provisions of paragraphs 21.6.6.1 through 21.6.6.4 of the ACI 318-95 code.*" This part of the code is about the consideration of the confinement and boundary elements for the walls in high seismic zones and as was mentioned before, in the design of the current structure, very little confinement and no boundary elements were provided for the wall sections, which may also have been a critical factor in the building failure.

3.3. Nonlinear Static Procedure

The nonlinear static (pushover) procedure of FEMA356 requires definition of the nonlinear load deformation relations according to the tables provided in this code. For each wall and beam section in each story, the moment-axial load interaction is derived using CUMBIA [Montejo and Kowalsky, 2007] and two moment-axial load hinges have been defined for two ends of the elements. In addition, shear strength is calculated according to ACI-318-02 and brittle shear plastic hinges are defined in the middle of each wall or beam element. The capacity curve of the structure is derived based on a modal load distribution. The target displacement, δ_t, at each floor level is calculated in accordance with the following equation:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad (3\text{-}7)$$

C_0, C_1, C_2, C_3 are modification factors defined according to FEMA356, S_a is the spectral acceleration and T_e is the effective fundamental period of the building. The values of target displacement demands for two orthogonal directions have been derived equal to 0.26m and 0.43m for X and Y directions, respectively. In Figure 3.1 pushover curves are depicted. In addition, the stage in which the first element reaches the predefined local limit state is shown. For the X and Y directions the first elements that pass the limit states are WI-4 and W5-1, respectively.

In the X-direction, as it was also suggested by the linear static analysis, the wall elements that suffer most are the ones that are subject to high axial tension loads especially in the first storey of the building. On gridlines "I" and "A" (see Figure 2.1a), because of the coupling behavior of the walls, the axial load has exceeded the axial tension load capacity of the walls in the first storey. Whereas, the walls that have been subject to axial compression, have rarely passed the LS (Life Safety) limit state. As such, at target displacement, most of the walls in the first story in elevation "I" and "A" pass the CP (Collapse Prevention) limit state. This fact is also observed for the transfer beams in elevation "A" and "I" in which most of the beams pass in to the CP limit state.

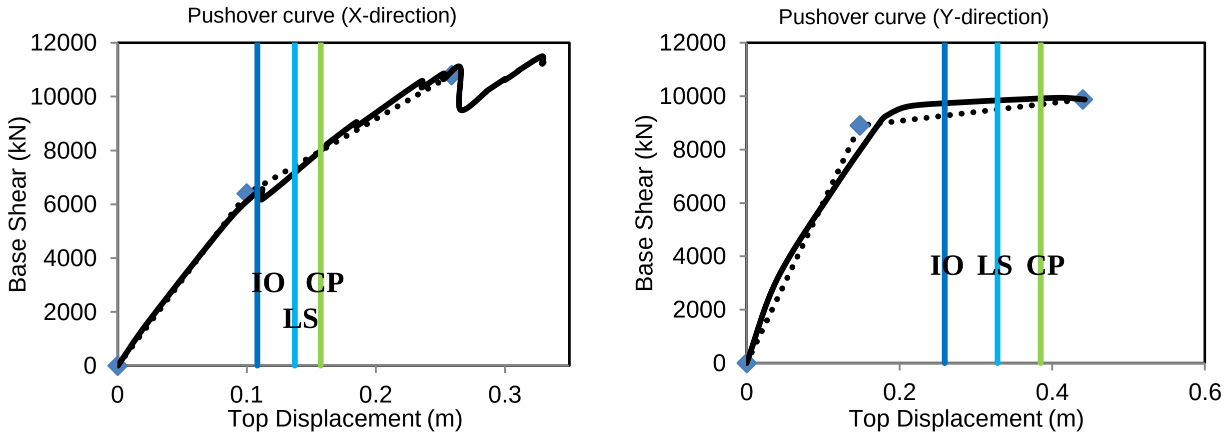

Figure 3.1 Pushover curves of the structure for X and Y direction

In the Y-direction, at the target displacement all of the walls on gridlines 8, 13, 20, 26, 34 & 35 (see Figure 2.1a), have remained in the IO (Immediate Occupancy) limit state except in one case at the second storey of grid 8, where the LS limit sate is reached. In this direction, only the walls at the base of the second storey of grid 5 exceed the CP limit state due to high axial tension forces. Shear hinges were not observed in both directions except for a few cases in elevation "A" and "I" in which shear plastic hinges reached the IO limit state. In addition, some of the coupling beams in elevations "A" and "I" developed shear hinges but demands did not exceed the IO limit state. According to these analyses, in the X-direction of the building in which there are severe irregularities over the height of the structure, more damage is predicted and is concentrated in elevations "A" and "I". In contrast, all the structural walls in the Y-direction, except those on grid "5", successfully satisfy the LS limit state.

These observations would suggest that failure of the building was expected in the X direction. However, the photos of the building after collapse suggest that the collapse occurred in the positive Y-direction (refer Figure 2.1a). Although severe damage and failures could have been predicted in the structure on grids "I" and "A" by the assessment procedure, it might be questioned: "how can these results explain the collapse of the structure in the Y-direction?" One probable scenario is that, if the building experienced significant damage to the four walls of WI-1, WI-2, WI-3 and WI-4 and the transfer beams of BI-2, BI-5 and BI-6 (see Figure 2.1) of the structure on grid I the gravity load carrying capacity along this side of the building could have been lost, provoking the collapse of the building in the Y-direction, in a manner like removing two legs of a chair on one side.

4. CONCLUSIONS

The seismic assessment of the Alto Rio building that collapsed in the 2010 Chile earthquake has been undertaken. The building is located in the city of Concepcion, Chile. Assessment has been carried out using linear static and pushover analyses based on FEMA356. The assessment methods confirm that the building should not have been expected to be able to sustain the seismic shaking, which interestingly, appears to have been of a similar intensity to the code design spectrum. In light of this, efforts have been made to consider how the Chile design practice may have provided the building with insufficient resistance. It is hypothesized that the designers may have underestimated the large tensile forces that can develop in walls due to coupling actions. Reasons for this may be related to poor interpretation of modal analysis results or inadequate capacity design procedures.

Interestingly, while common pushover analyses of a 15 storey building could often be considered of limited value, since they do not account for the higher mode effects, the building under examination was so stiff that it is expected that higher modes did not play a significant role in the response. This was supported by the results of modal analyses. It is also interesting that because of Chilean design and construction practice, the large number of walls in the structure would suggest that the seismic demand should not be excessive in the walls. However, because of the discontinuity and irregularity of the walls in the first storey of the building that have caused concentration of demands on the remaining walls, shear and flexural demands would have exceeded the capacity in the walls of this storey which could have led to the failure of the entire building.

In closing, while these analytical studies have identified some possible reasons for the observed damage, the actual behavior of the complex structural system used for the building cannot be known with certainty and other reasons for the damage (such as soil structure interaction effects, poor construction detailing, etc) should also be considered as part of future research.

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REFERENCES

ACI Committee 318 [2005] *Building Code Requirements for Structural Concrete (ACI 318-05) andCommentary (ACI 318R-02)*, American Concrete Institute, Farmington Hills, MI.

Amr S. Elnashai et al. [2010], *The Maule (Chile) Earthquake of February 27, 2010 Consequence Assessment and Case Studies*, MAE Center Report No. 10-04.

Alimoradi, A. & Naeim, F "*Did the Large Coseimic Displacement Cause the Global Overturning Collapse of the Alto Rio Building During the 27th February 2010 Ofshore Maule, Chile Earthquake*?" Journal of the Structural Design of Tall and Special Buildings, Volume 19, Issue 8, Article first published online: 29 NOV 2010.

ASCE (2000). *"FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings"*, ASCE for the Federal Emergency Management Agency, Washington, D.C., November 2000.

Concepción under Construction. 2010. http://concepcionunderconstruction.blogspot.com/search/label/Alto%20 Rio [August 2010].

Computer and Structures, Inc., [2010] *CSI Analysis Reference Manual for SAP2000, ETABS and SAFE*, Computer and Structures, Inc., Berkeley, California

El Periodista (EP). [2010]. http://elperiodistaonline.cl/ [August 2010].

H. Kato, S. Tajiri & T. Mukai, [2010] *Preliminary Reconnaissance Report of the Chile Earthquake 2010*, Building Research Institute, Japan, http://www.kenken.go.jp/english/.

Arnott, K., [2005], Shear Wall Analysis-New modelling; Same answers, Journal of Structural Engineering-February. Also in CSC (UK) Ltd., February

Montejo, L. A and Kowalsky, M. J, CUMBIA- *Set of Codes for the Analysis of Reinforced Concrete Members*, Report No. IS-07-01, Constructed Facilities Laboratory, North Carolina University, Raleigh, NC, 2007, 41 pp.

NCh 0433-1996, *Official Chilean Standard for Earthquake Resistant Design of Buildings (English translation)*, Instituto Nacional de Normalización (INN).

Priestley, M. J. N., Calvi G. M., and Kowalsky M. J. [2007] *Displacement Based Seismic Design of Structures*, IUSS Press, Pavia, Italy.

R. Boroschek [2010], *Chilean Design and the Mw=8.8 Feb 27 2010 Chile Earthquake, ROSE School Seminar, EUCENTRE, Pavia, Italy*