A performance assessment of a R/C frame building using a record obtained at its foundation

P. Bonelli & M. González
Departamento de Obras Civiles, Universidad Técnica Federico Santa María, Casilla 110 V, Valparaíso, Chile, Fax: +56-32-654115, Email: patricio.bonelli@usm.cl
R. Boroschek
Departamento de Ingeniería Civil, Facultad de Ciencias Físicas y Matemáticas, Universidad de Chile, Blanco Encalada 2002, Santiago Chile, Email: rborosch@ing.uchile.cl

ICCRRR International Conference Concrete Repair, Rehabilitation and Retrofitting Nov. 2005 South Africa.

ABSTRACT: A six story R/C frame building suffered non structural damages under the 2001, Mw=8.4 South of Peru Earthquake, losing functionality. One Earthquake record was obtained close to the building base with a local peak acceleration of 0.3 g. The building is a typical 1970's limited ductility moment resistant frame. After the 2001 event the building was abandoned and a retrofitting project was proposed. In this paper, the damage pattern of the building after the 2001 event is compared with results of a non-linear analytical model made with the record obtained at the site. In general, results of the analysis match the observed behavior. Computed shear demands in columns were lower than the estimated column shear capacity, cracks appeared in beams where yield was expected. Maximum interstory drifts can be related to damage in non structural elements.

1 INTRODUCTION

Different modes of failure have occurred in limited ductility reinforced concrete frames during earthquakes. Typically they are: shear failure in columns and joints, sliding of longitudinal reinforcement due to insufficient splice development and local buckling in longitudinal reinforcement.

An essential hospital in the northern part of Chile shares the above characteristics. The Hospital complex is built on one of the largest subduction thrust earthquake areas of the world.

A Mw=8.4 earthquake occurred in the South of Peru on June 23, 2001, producing moderate structural and non structural damage in the Hospital. Before the 2001 event detailed experimental and analytical studies had been performed to evaluate possible failure modes (Boroschek 2000). Designed in the seventies, the structure appeared as very vulnerable to a brittle shear failure in columns at very low lateral displacement demands. Post the 2001 earthquake, linear and nonlinear studies were carried out to propose a retrofit strategy (Bonelli and Boroschek, 2004).

This paper presents results of a non-linear analysis using a record obtained close to the building during the 2001 earthquake. Predicted damages are compared with observed ones. Additional studies were performed considering a more severe event expected for the area.

2 THE BUILDING

Figure 1 shows a typical building plan and figure 2 a cross section of a column. Table 1 shows the amount of reinforcement and column cross sections.

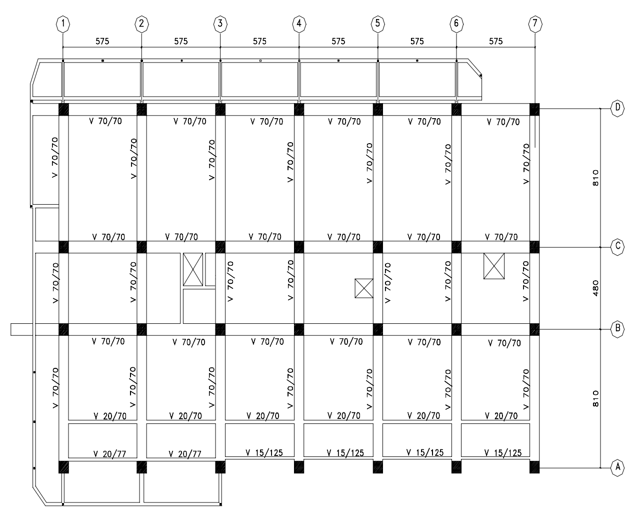

Fig. 1- Plan view

3 THE ARICA HOSPTAL RECORD OF THE PERU 2001 EARTHQUAKE

A record of the Peru 2001 Earthquake, the Arica Hospital record (AH), was obtained close to the building site 400 km. south-east from the epicentre (Boroschek et al, 2001). Peak ground acceleration

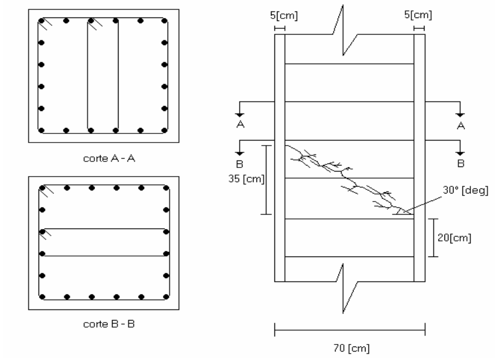

Fig. 2- Cross section of the columns and ties arrangement.

Floor	Dimension [cm]	Longitudinal Reinforcement	Transversal Reinforcement
1	70x70	$36\phi26$	$2E\phi10@20$
2	70x70	$20\phi26$	$2E\phi10@20$
3	60x60	$28\phi26$	$2E\phi10@20$
4	60x60	$20\phi26$	$2E\phi10@20$
5	60x60	$16\phi26$	$2E\phi8@20$

Table 1- Column dimensions and reinforcement.

and velocities at site were 0.28g and 19 cm/sec respectively. Strong Motion Duration above 0.05g (bracket duration) was 22 seconds.

Moderate structural damages were observed but non-structural damages resulted in the building functionality loss. The Hospital was evacuated, patients were moved to an old 1940's two-story confined masonry building next to the Hospital that suffered no damage. Figure 3 and figure 4 compare capacity spectra of the AH record, Llolleo 1985 record and the Chilean Code Spectra. These figures also include the building capacity curve. Natural periods were obtained from analytical and ambient vibration studies.

4 DRIFTS AND DAMAGES

Non linear analyses were carried out with the Ruaumoko computer program. The AH record was used to verify if analytical results matched the observed behavior. The Llolleo 1985 record was applied to evaluate the possible building performance under a severe earthquake that could affect the area. Figure 5 shows lateral displacement envelopes and figure 6 the maximum interstory drifts calculated for both records. Thin cracks appeared in the building at the beam-column face specially on the first and second floors but according to the analytical predictions most beams of the upper floor should have yielded under the Arica Hospital record. Figure 7 compares the bending moment demands with flexural capacity in columns showing that they effectively remained elastically during the earthquake, but could have yielded in upper floors under a Llolleo earthquake. Unfortunately, the low column shear resistance does not allow the bending capacity to be developed. The failure mechanism is controlled by column and beam shear failure at very low lateral displacements.

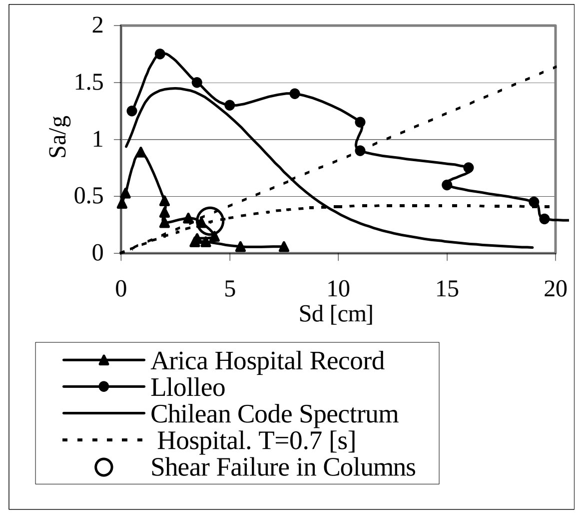

Fig. 3- Capacity Spectra . ξ= 2 %

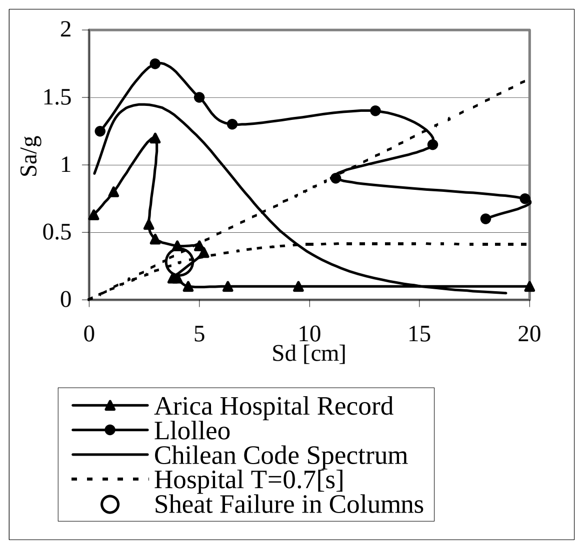

Fig. 4- Capacity Spectra. ξ=5%

5 DEMANDS OF EXISTING COLUMNS

5.1 *Column Shear Capacity*

Existing columns have double 10-millimeter diameter ties spaced every 20 centimeters. Interior ties have been placed alternating their direction. Thus if shear forces produced a crack with an inclination of 30 degrees, the arrangement of the reinforcement requires only two of 10-mm diameter to resist the shear forces, as shown in figure 2. Severe corrosion of the transverse reinforcement

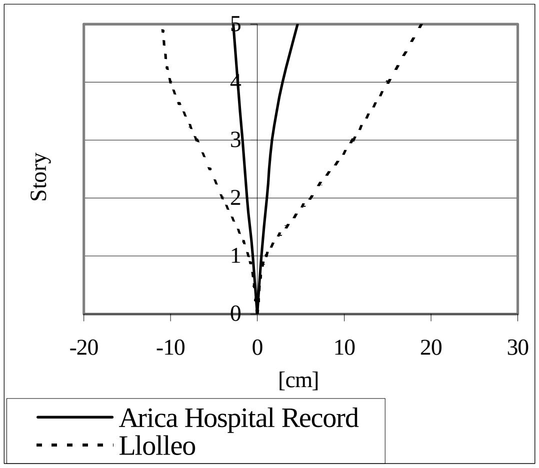

Fig. 5- Lateral Displacement Envelopes

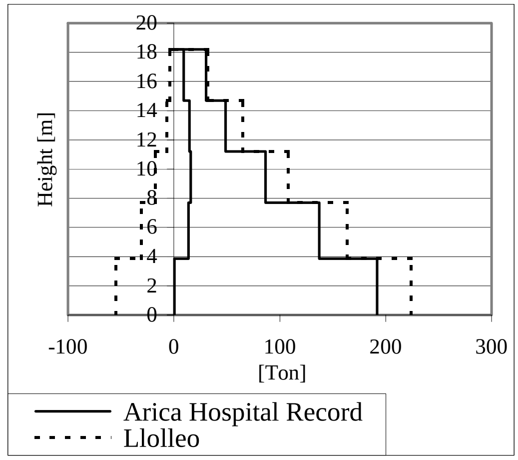

Fig. 8- External Axial Force

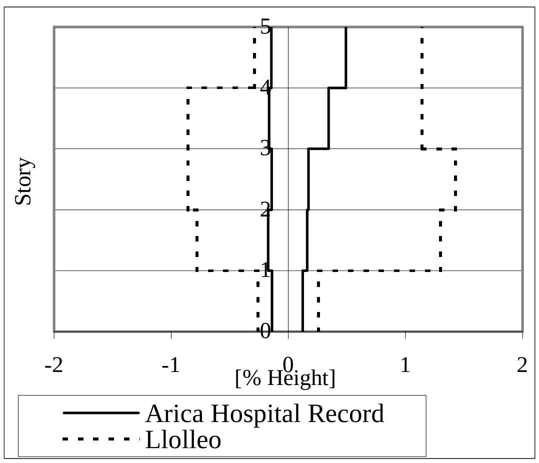

Fig. 6- Maximum Interstory Drifts

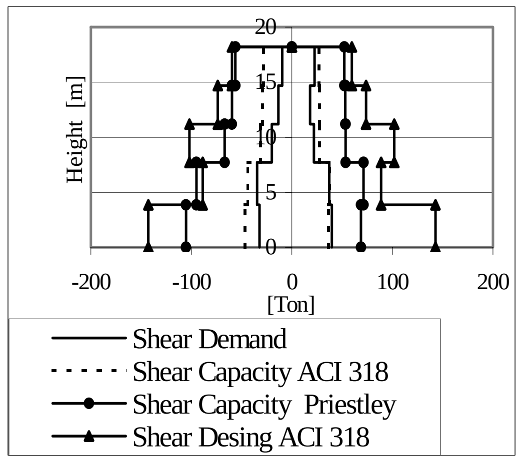

Fig. 9- External Column Shear. Arica Hospital Record

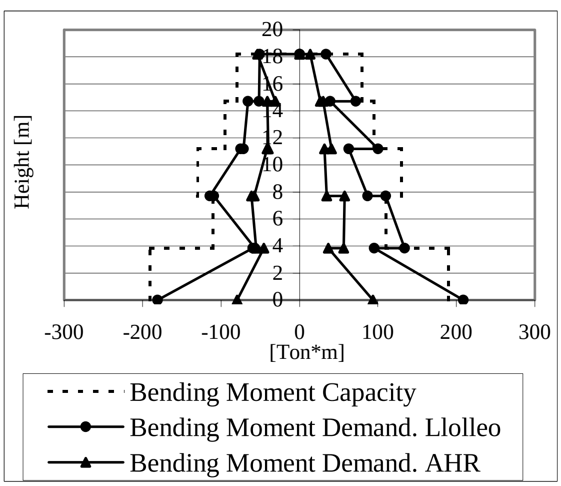

Fig.7.- External Column Bending Moments

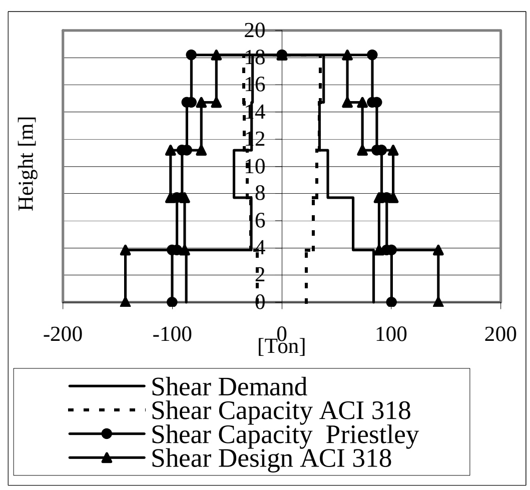

Fig. 10- External Column Shear. Llolleo

at the base columns was observed during an inspection of the building (Boroschek, 2000). Also the number and disposition of existing ties did not comply with modern ACI 318 codes. Therefore, the analysis considered that the shear strength of the main element is only due to the contribution of the concrete.

The shear capacity was determined from an expression proposed by Ang et al,(1989), and Priestley et al (1994). The concrete shear resistance mechanism can be estimated as:

$$V_c = 0.37 \cdot \alpha \cdot \left(1 + \frac{3P}{f'_c \cdot A_g}\right) \cdot \sqrt{f'} \cdot A_g \qquad [MPA] \qquad (1)$$

where

$$\alpha = \frac{2}{\left(M / (V \cdot D)\right)} \geq 1$$

A_g is the gross area, A_e the effective area (0,8 Ag), M the bending moment, V the shear and D the diameter of the confined concrete area. P is the axial compression force. The shear capacity was evaluated for the corresponding axial force obtained by the non linear analysis. Figure 8 shows that for the Llolleo record axial forces are slightly higher than for the Arica Hospital record.

5.2 *Column Shear Demands*

Figure 9 contains four curves The maximum shear in external columns obtained from the non linear analysis is compared with the shear capacity calculated in two different ways: i) the ACI318 Code shear capacity taken from expressions (11-7) and (11-8), art. 11.3.2.2 of ACI318-2002, only considering the concrete contribution to shear resistance, and ii) application of the expressions proposed by Priestley et al (1994). The demanded shear forces in columns were calculated with the AH record. The results revealed that the shear demand was very close to the calculated shear capacity. This means that some columns could have failed in shear under this earthquake according to the ACI318 expressions. The performance point of the original structure was drawn in figures 3 and 4 proving that it is very close to the seismic demand. However Priestley's expressions give higher values to the shear strength in columns, being the response satisfactory for this record as actually occurred in the building. Nevertheless the columns do not comply with ACI318-2002.

Similar event characteristics and the soil type suggest that the 1985 Llolleo record could be a realistic record to estimate the demand for the area.

Figure 10 presents the shear demands for the Llolleo record, confirming that a brittle shear failure in columns could be expected under a severe earthquake in the area. In consequence, a retrofit strategy should be implemented or the building must be demolished.

6 CONCLUSIONS

Observed damages under the 2001 South of Peru Earthquake in the building match the analytical results obtained using the record registered close to the building. Cracks observed in beams of the building suggest that they effectively yield. Columns appeared undamaged but they would have been very close to a shear failure. The vulnerability study proved that they are very vulnerable to a brittle catastrophic shear failure under more severe earthquakes

7 ACKNOLEDGEMENTS

Research students José Luis Contreras and Macarena González worked at their Civil Engineering Degree thesis at Federico Santa Maria Technical University, producing most of the results shown in this paper.

8 REFERENCES

– *Ang. B.G., Priestley. N.J., and Paulay. T., 1989.* Seismic Shear Strength of Circular Reinforced Concrete Columns –. ACI Structural Journal, V.86,No.1, Jan-Feb 1989,pp.45-59.
– *Bonelli, P., Boroschek. R., 2004.* Reinforced Concrete Structural Walls as Solution to Retrofit a R/C Frame Building, Performance-Based Seismic Design Concepts and Implementation, Proceedings of an International Workshop, Bled, Slovenia. June 28-July1, 2004, pp433-444.
– *Boroschek. R., 2000.* Estudio de la Vulnerabilidad sísmica del Hospital Juan Noe Crevani de Arica: Un modelo. Fundación para la Transferencia Tecnológica Universidad de Chile. Santiago de Chile.
– *Priestley. N.J., Seible. F. Xiao. Y. and Verma. R., 1994.* Stell Jacket Retrofitting of Reinforced Concrete Bridge Columns for Enhanced Shear Strength – Part 1: Theoretical Considerations and Testing Design. ACI Structural Journal, V.91, No.4, July-August 1994,pp.394-405.