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REINFORCED CONCRETE STRUCTURAL WALLS AS SOLUTION TO RETROFIT A R/C FRAME

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ABSTRACT

A retrofitting methodology applied to a six-story R/C frame building is described. The original building is a typical limited ductility design moment resistant frame of the seventies, i.e., with insufficient splice length for longitudinal reinforcement, insufficient amount of ties in columns and no reinforcement in joints. A previous vulnerability study showed that the structure could have a brittle failure. The frame could be reinforced jacketing all the columns and reinforcing all the external joints, but large drifts could make the building loose its functionality under an expected earthquake in the area.
Adding reinforced concrete walls in facades and at interior frames was selected as the final solution after a cost-benefit analysis, since lateral displacements can be reduced as stiffness increases, decreasing ductility demands in joints and shear stresses in beams and columns, so that the frames mainly acted as gravitational systems. Acceptance criteria for the retrofit are based on functional restrictions and drift control.

Keywords: Retrofitting, structural walls, brittle frames, shear demands, non-linear analysis, and column jacketing.

1. INTRODUCTION

Different modes of failure have occurred in earthquakes in the 1970's type of reinforced concrete frames. Typically they are: shear failure in columns and joints, sliding of longitudinal reinforcement due to insufficient splice development and local buckling in longitudinal reinforcement in columns. In Chile there are a few reinforced concrete frame buildings that could show this behaviour.

This paper presents a procedure applied to an existing building to detect possible modes of failure and retrofit alternatives using Performance Based Seismic Design.

The case study is a Hospital in the North of Chile. The structure is a reinforced concrete frame building, designed in the seventies, that is vulnerable to brittle shear failure in columns at very low lateral displacements demands.

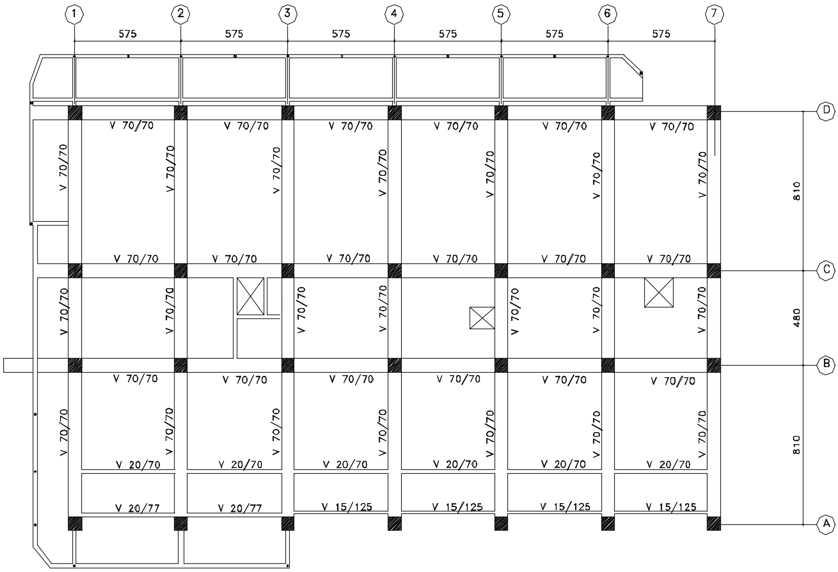
2.- CASE STUDY

A typical plan view and longitudinal elevation of one of the buildings are shown in figure 1. A cross section of a column is shown in figure 2. Table 1 shows the dimensions of the cross section and the amount of reinforcement in the columns.

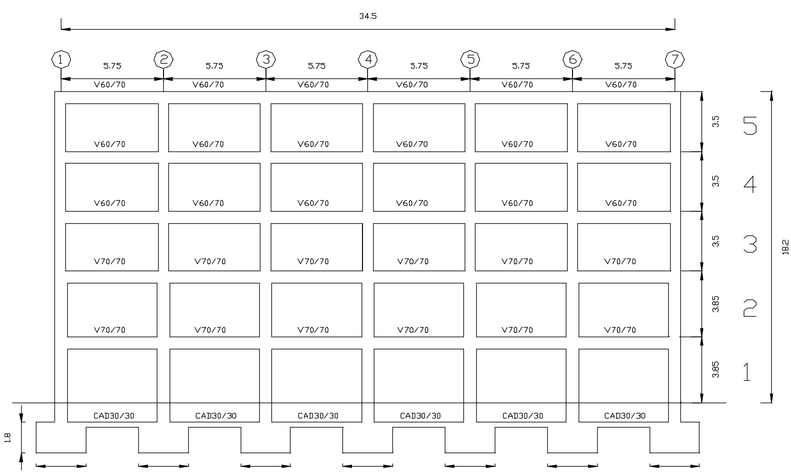
The Magnitude Mw = 8.4, 2001 Earthquake in the South of Peru caused important non-structural damages in the building, loosing its functionality, but with minor structural damages. Surgery rooms and in general partition walls suffered moderate and severe cracking. In this case it was considered that operation rooms could not be used if their walls were cracked. This fact showed the need to apply performance-based design criteria in the design and retrofitting of hospitals, as it is not enough to ensure the structural integrity, but to allow the functionality protection of the system. The Hospital was evacuated due to the pressure of the personnel that was worried about visible damage and loose of functionality. Patients were moved to an old 1940's two-story confined masonry building next to the Hospital that suffered no damage.

3.- SHEAR STRENGTH AND SHEAR DEMANDS OF EXISTING COLUMNS

Existing columns have double 10-millimeter diameter ties spaced every 20 centimetres. Interior ties have been placed alternating its direction. Thus if shear forces produced a crack with an inclination of 30 degrees, the arrangement of the reinforcement allows only two 10-mm diameter to resist the shear forces, as shown in figure 2 for a column of the second floor. Severe corrosion of the transverse reinforcement at base columns was observed during an inspection of the building. For this reason in the analysis it was considered that the shear strength of the main element is due only to the contribution of the concrete. Also the number and disposition of existing ties did not comply with modern ACI 318 codes.



a) Plan view



b) Elevation

Fig. 1- Plan view and longitudinal elevation

Floor	Dimension [cm]	Longitudinal reinforcement	Transversal reinforcement
1	70x70	36ϕ26	2Eϕ10@20
2	70x70	20ϕ26	2Eϕ10@20
3	60x60	28ϕ26	2Eϕ10@20
4	60x60	20ϕ26	2Eϕ10@20
5	60x60	16ϕ26	2Eϕ 8@20

Table 1. Dimensions and amount of reinforcement of columns

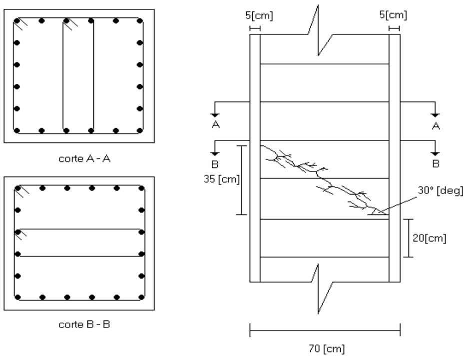


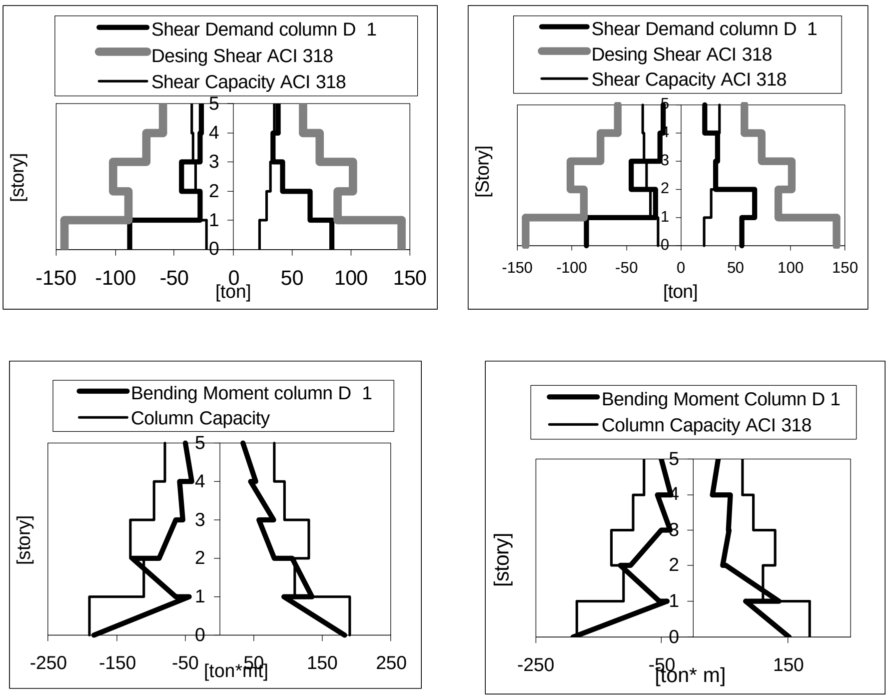
Fig. 2- Cross section of the columns and arrangement of ties

4.- SHEAR DEMAND CALCULATED WITH EARTHQUAKE RECORDS, BY NON-LINEAR ANALYSIS

4.1 Time history analysis

In this study, two Chilean March 3, 1985 earthquake records, Viña del Mar (0.35 g) and Llolleo (0.67 g) were chosen to estimate the possible demands. The fault mechanism of the expected seismic action at the site and its epicentral distance are similar to the ones that generated the selected records. In the vulnerability study of the building, a maximum acceleration of 0.55 g was estimated [1]. This value was used to scaled Viña del Mar S20W records to 055g. In the analysis, because the columns of the existing building have a low shear resistance and they could suffer brittle failure, a 2% damping ratio was considered.

Figure 3 shows the maximum bending moment and shear forces calculated with Llolleo N10E and Viña del Mar S20W scaled to 0.55g records. If columns had had larger shear strength, the bending moments drawn in figure 3 would have been reached, yielding some columns in the base of the third and fourth floors. Envelopes of shear calculated for a 5% damping ratio were very similar to the ones calculated with a 2% damping.



Llolleo Viña del Mar

β= 2%

Fig. 3- Shear and bending moment envelopes

4.2 Incremental analysis

Base shear versus roof displacement for the longitudinal direction of the building, calculated from applied uniform and inverted triangular distributed loads are shown in figure 4. Additional marks are included in the figure to show the maximum displacement demands for the records, calculated with a non-linear dynamic analysis, and the results of an elastic linear analysis with the elastic spectrum of the Chilean Code.

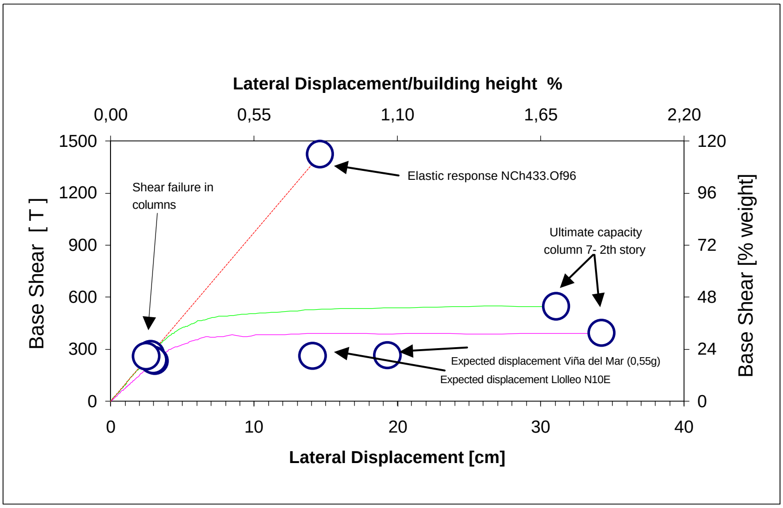


Figure. 4 – Basal shear v/s Displacement of the roof

5.- SHEAR STRENGTH IN COLUMNS

Only the concrete contribution, Vc, to shear resistance of columns was considered in the analysis. Since shear resistance depends on axial forces, their magnitude was obtained from the Llolleo record response, considering 2% of damping ratio. With these values available, shear resistance varies between 43 [tons] and 22 [tons]. The available shear resistance of the existing columns, calculated with ACI 318-99, does not allow the columns to reach their flexural capacities.

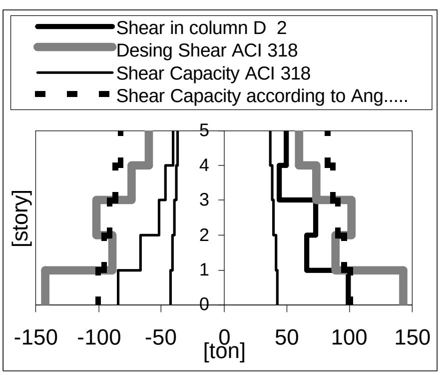
Since ACI 318 is conservative, to estimate the shear in columns that do not suffer tension axial forces, the internal column shear resistance was also calculated with expressions obtained experimentally by Ang, Priestley and Paulay [2],[3]. In this case the contribution of the concrete to the strength of the shear can be estimated as:

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

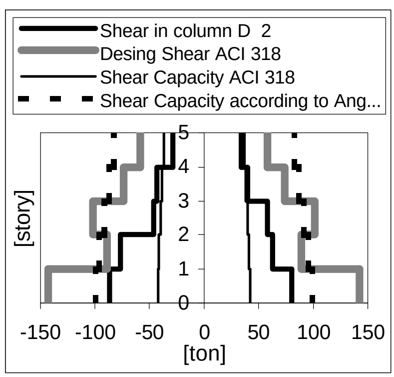
where:

$$a = \frac{2}{\left(M / (V \cdot D)\right)} \geq 1 \qquad \text{Eq.(2)}$$

P is the axial compression force, Ag the gross area, Ag the effective area (0,8·Ag), M the moment, V the shear and D the diameter of the confined concrete area.



Llolleo, β = 2%



Viña del Mar, β = 2%

Fig. 5 – Shear capacity and shear demands in columns

Column shear capacity of existing columns is compared in figure 5 to the shear demands of the considered records.

The exterior columns, which could be in tension during an earthquake, would have less shear resistance than the interior ones, which remain under compression during the whole response. The record of Viña del Mar S20W scaled to 0.55g has a displacement demand on the structure of 14.1cm (0.8% of height). If columns at the perimeter are not reinforced with jackets, they could resist the earthquake without a shear failure only for displacements below 2.5 [cm]. Therefore, all exterior columns of the building and in all floors must be reinforced

An option to meet the required shear strength, as indicated in ACI 318 -Chapter 21, would be to add jackets to the columns with reinforced concrete, steel plates or carbon fibers.

When adding jackets to columns, the building could sustain lateral displacements as large as 31.1 [cm] (1.71% of height), enough to withstand all the considered demands satisfactorily[4]. .

This type of analysis helps to decide if reinforcement of a building is mandatory and provides a criterion to decide whether to evacuate or not the building if a severe seismic action is expected. In this case, the probability to have a shear failure in columns is very high.

When reinforced concrete jackets are used, the requirements are met with single 16 mm diameter ties every 6 cm (E Φ16@6) in plastic hinges zones and every 9 cm (E Φ16@9) in the rest of the column. Use of carbon fibers was evaluated but discarded because their cost was four times the cost of the other solutions.

6.- WALLS AS AN ALTERNATIVE SOLUTION

Jacketing columns and reinforcing joints could be a reasonable solution to maintain the structure safe, but to avoid non structural element damage and assure functionality of the building it is wise to decrease drift demands and also to provide a corresponding seismic design to non-structural elements. Displacement demands can be decreased considerably, isolating the building at the base or increasing the stiffness of the building with structural wall or braces. Several options were examined and only one of the selected options is shown in Figure 6. It consists on adding reinforced concrete walls of 30 cm thickness in direction X and 40 cm in direction Y. This solution has been suggested because similar buildings that have this structural system had a satisfactory response in previous Chilean earthquakes. The use of structural walls in facades as retrofit strategies has several advantages: it lowers the cost of the final building because additional architectural facades are reduced because they are transformed into structural elements. The long length of these walls permits control of overturning moments, in this case if the walls are supported on a grade beam that connects existing footings, no additional foundations are needed. Concentrating the retrofitting elements on the perimeter of the building reduces functional interference.

The analysis of this solution was made with the Viña del Mar record scaled to 0.55g, for a damping ratio of 3%, because wall and beam cracks are expected, assuming that the columns will remain with minor cracking.

Figure 7 shows the required shear strength and the demands of the bending moment in an external column. Even though in direction X the proposed wall density is different than that the one used in direction Y, results were very similar.

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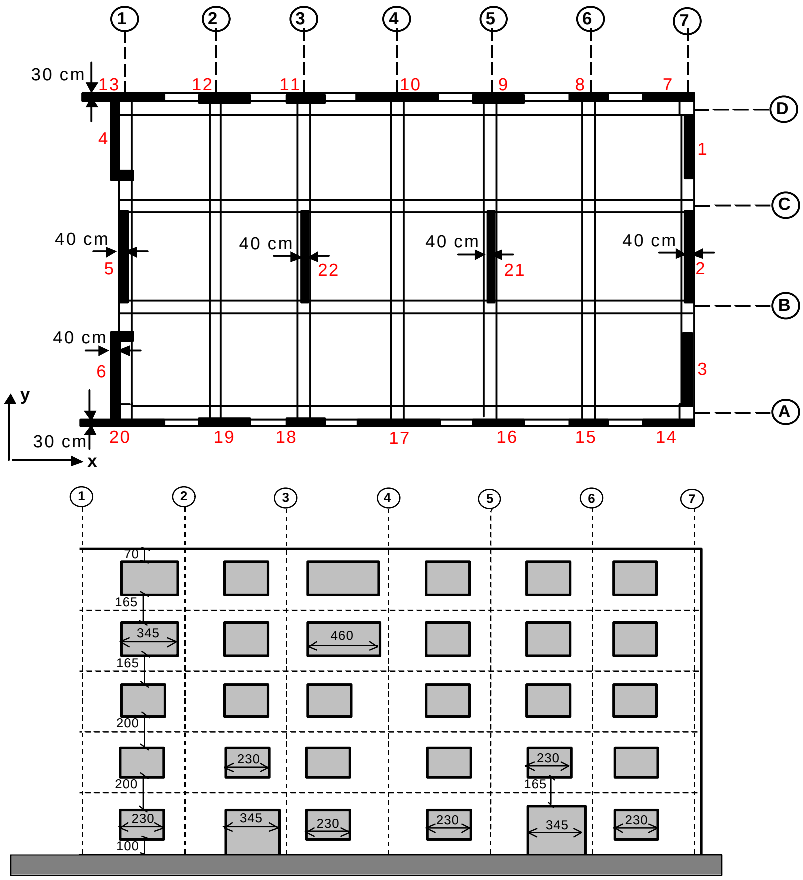

Fig. 6 – Plan view and Elevation

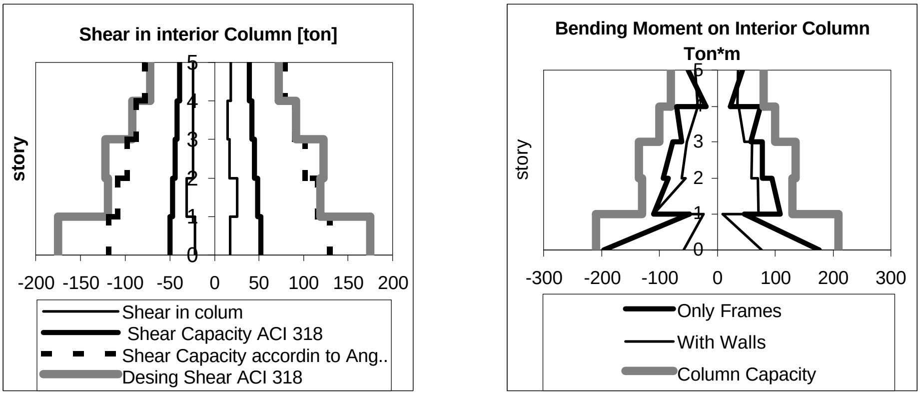

Fig. 7 – Shear and bending moment demands and lateral displacement envelopes.

Figure 8 compares the lateral displacement envelopes at the original frame building with the retrofitted structure. The abrupt change in stiffness and strength at columns at the first floor produces the observed reduction in drift at first level. Walls reduce overall displacement and change the deformed shape decreasing shear demands in every element and joints

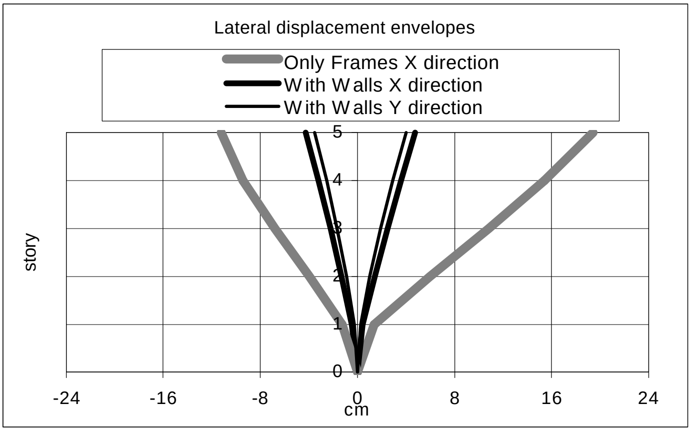

Fig. 8 – Lateral displacement envelopes

7.- CONCLUSIONS

In the analysed frame building, a shear failure in columns could occur for very small deformations. When structural walls are included, the demands of the displacements are drastically reduced, and the alteration of the deformed shape decreases the strength demands in the elements of the frames. The analysed building reaches expected lateral deformations before critical beam sections yield. Nevertheless, the increase in accelerations due to the increase in stiffness must be taken into account in the design of non-structural elements.

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