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# Lessons learned from the March 3, 1985 Chile earthquake and related research

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ABSTRACT: Some lessons learned from the March 3, 1985 Chile earthquake are discussed. They are based on direct observation of structural behavior and damage after the earthquake as well as on a related program of analytical and experimental research. Inelastic analysis of typical structural systems is performed using some records of the March 3 earthquake as input. Results of analysis of real structures of Viña del Mar are also shown. Relevant parameters of seismic behavior, as strength, ductility, structural response factor, and drift are computed. As a result, the advantages of the structural wall buildings used in Chile as earthquake resistant structures are described.

## 1 INTRODUCTION

The March 3, 1985 earthquake that ocurred off the coast of Central Chile was a major event having a magnitud Ms of 7.8. It affected the most populated area of Chile having many kinds of constructions: housing, buildings, industrial facilities, bridges, highways, port facilities, storage tanks, lifelines. In addition to theses facts, this earthquake has been of big significance due to its long duration and because of the fact that it was recorded by a major strong motion instruments network.

The damage was concentrated on old non engineered brick or adobe dwellings, port facilities, some bridges and lifelines. The modern building structures, with few exceptions, behaved very well, showing even less damage than it was expected according to the design code philosophy.

Almost all medium and high rise buildings in Chile are constructed of reinforced concrete. In them, earthquake resistant responsibility is essentially given to the structural walls. Compared to the construction practice in some other countries, Chilean RC buildings have a much higher density of walls.

Dwellings and low rise buildings made of framed or reinforced masonry behaved well due also to the amount of walls in addition to the beneficial effect of beam-column framing or to an adequate amount and distribution of reinforcing steel.

Many real buildings located in the affected seismic area were accurately studied by several researchers: Bonelli (1989), Blondet et al (1989), Klingner et al (1989), Cassis et al (1988), Wallace and Moehle (1989), Riddell et al (1987), Wood et al (1987), Monge et al (1986), Astroza and Delfin (1986), and others. Extensive analytical and experimental work was done to assess the seismic behavior of buildings and compare the results with the damage observed after the earthquake, leading to important conclusions.

## 2 THE CHILEAN SEISMIC CODE

The present Chilean Code NCh 433. of 72 has been used for almost 30 years, including a period of preliminary draft. It prescribes seismic forces of similar levels to those used in high seismic activity areas of the United States (see Wallace and Moehle 1989). Presently, some alternative RC design codes can be used in Chile. The code in force is based on an old German code using the working stress method, which leads to an effective load factor of 1.7. However, there is now in Chile a pronounced tendency to use the ACI 318 design code, with strenght design leading to 1.4 and 1.7 factors for dead load and earthquake, respectively. According to this, strength requirements for most Chilean buildings are similar to those for U.S. structural wall buildings designed according to UBC-85, but somewhat less than the requirements of UBC-88 (Wallace and Moehle 1989).

Design base shear in the Chilean code has a lower bound of 6 % of building weight. This code also limits the lateral interstory desplacements resulting from the code forces to 0.2 % of the story height.

This bound raises up to 0.4 % if non-structural partitions are adequately separated from the structure.

In Chilean practice, longitudinal steel reinforcement in walls is located at the edges having neither special requirements for confinement nor a wider concrete section than the web. Wall stability is implicitly provided by the requirement of minimum wall thickness of 20 cm. Wall reinforcement is usually equal to 0.2%. In opposition to U.S. codes prescriptions, sizing and detailing of walls have no stringent demands in the Chilean code. This is in agreement with the customarily used high density of walls and with some analytical results that will be described later.

In what follows, an analysis of the main features of RC Chilean buildings in connection with their seismic performance is done. Quantitative results are extracted from research.

## 3 ANALYSIS OF SEISMIC STRUCTURAL RESPONSE OF TYPICAL RC STRUCTURAL SYSTEMS

Viña del Mar, a coastal resort city of Central Chile, is located 80 Km from the 1985 earthquake epicenter, and at that time it had about 400 modern RC buildings. A great amount of them was inspected after the earthquake showing a good earthquake behavior. Due to this fact, the S20W acceleration record obtained in Viña del Mar (on sand), was utilized in this study; also, the records of Llolleo N10E (on sand), the closest to epicenter and where the intensity was the highest, and Valparaiso N70E (on rock), located about 3 Km from Viña del Mar record, were used. Figure 1 shows the records of these motions and Figure 2 shows the 5% damped elastic response acceleration spectra of the 3 aforementioned records.

Comparing the neighboring records from Viña del Mar and Valparaiso it can be seen the remarkable influence of soil on amplification of motion.

To evaluate the relative seismic behavior, 4 different structural systems have been selected for analysis. They typify distinct systems found in practice. They are wall, dual wall-frame, and frame systems. The wall systems differ in the ratio of area of walls over area of floor. Plans and typical elevations of the 4 systems are described in Figure 3.

The four structural systems shown in Figure 3 were subjected to the motions shown in Figure 1 and their responses were obtained using the general purpose computer program DRAIN 2D for Dynamic Analysis of Inelastic Plane Structures developed by Kanaan and Powell (1973).

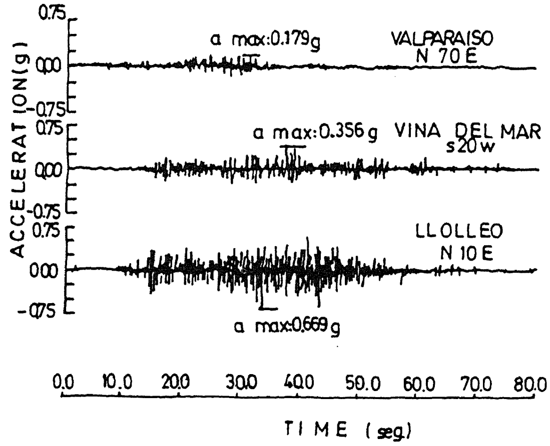


Figure 1. Ground acceleration records.

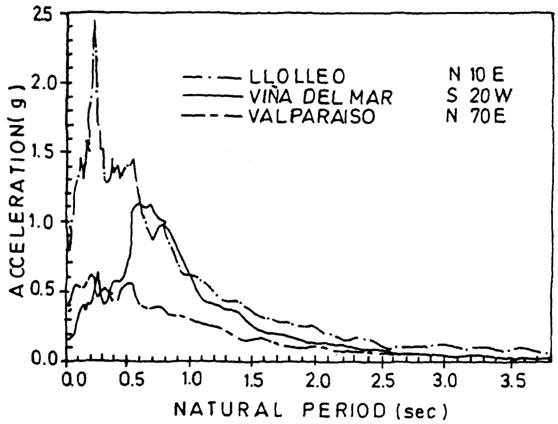


Figure 2. Elastic response acceleration spectra for 5 % damping.

Response parameters like strength, ductility, structure response factor R, roof and interstory drifts are computed and compared with supplied strenghts and ductilities. The latter correspond to properties of structures designed according to the Chilean seismic Code NCh 433. of 72 Diseño Antisismico de Estructuras from Instituto Nacional de Normalización INN (1972).

Results are fully described in Bonelli (1991). A summary of them is given next.

### 3.1 Ductility requirements and response modification factors

Global displacement ductility requirements were computed for the 4 structural systems and the 3 acceleration records already mentioned. The values were obtained using inelastic response spectra from a procedure

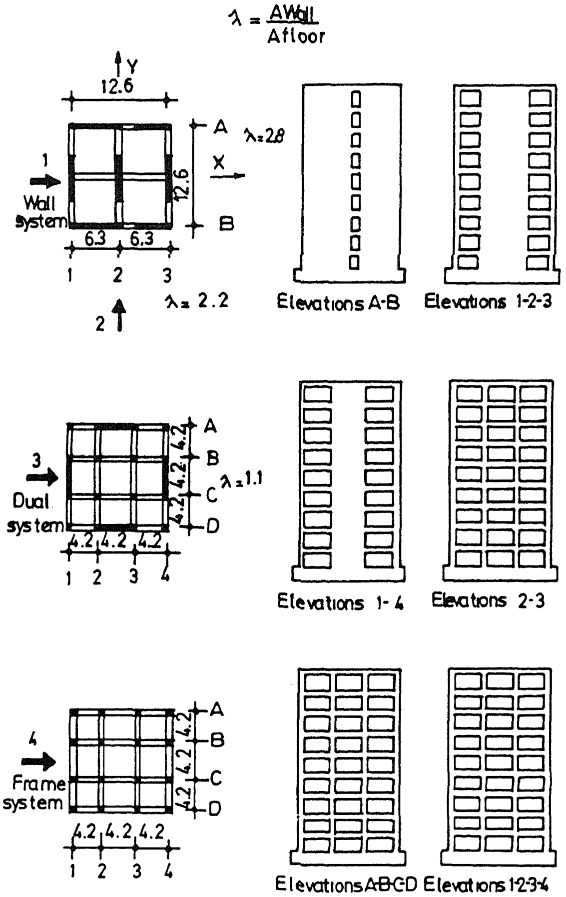


Figure 3. Plan and elevation views of 4 structural systems.

recommended by Newmark and Hall (1982) relating the spectral values to calculated base shear strengths (see Table 1).

R factors, used in modern codes to derive an inelastic design response spectrum from a smoothed linear elastic design response spectrum, were calculated in two ways: i) dividing the linear elastic acceleration spectra from Figure 2 by the ordinates of the design spectrum proposed in the Chilean code NCh 433. of 72, and ii) dividing those spectra by the actual values of strengths instead of code design forces, the ratios called Ra (see Table 1).

Comparison of R and Ra values from Table 1 clearly shows the overstrength of real buildings. Ductility demands for Viña del Mar have values of 1.4 for rigid structures and up to 3 for flexible structures.

## 3.2 Lateral displacements

Computation of inelastic lateral displacements is important in earthquake

Table 1. Global ductility and response modification factors.

| Record | Structural system (Figure 3) | Global displacement ductility ratio | R | Ra |
|---|---|---|---|---|
| Valparaiso N70E | 1 | 1.1 | 5.3 | 1.2 |
| | 2 | 1.1 | 6.2 | 1.1 |
| | 3 | 1.2 | 5.6 | 1.1 |
| | 4 | 1.0 | 2.0 | 0.6 |
| Viña del Mar S20W | 1 | 1.4 | 5.6 | 1.2 |
| | 2 | 1.4 | 6.7 | 1.2 |
| | 3 | 2.5 | 18.0 | 3.7 |
| | 4 | 3.0 | 6.8 | 2.0 |
| Llolleo N10E | 1 | 6.0 | 15.9 | 3.4 |
| | 2 | 3.5 | 14.1 | 3.0 |
| | 3 | 3.0 | 16.6 | 3.3 |
| | 4 | 3.0 | 9.0 | 2.6 |

resistant design because they provide indices related to structural damage. Building top drift and relative interestory displacements (which are a measure of distortions), together with ductility ratios, are relevant parameters to assess the degree of damage for both structural and non-structural members.

Figure 4 shows inelastic lateral displacement envelopes, measured as a ratio with respect to building height, for the 4 structural systems and the 3 ground motion records under study. Roof drift ratios range between 0.33 and 0.77 percent, and they would be expected to cause some non-structural damage, as it was observed after the March 3 earthquake in Viña del Mar. The figure makes apparent the importance of wall *amount to limit damage.*

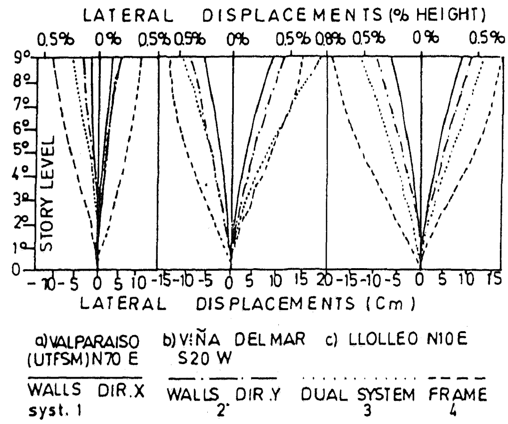


Figure 4. Inelastic lateral displacement envelopes.

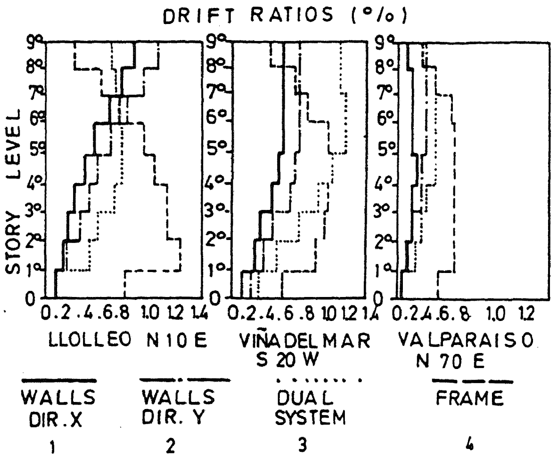


Figure 5. Interstory drift ratios.

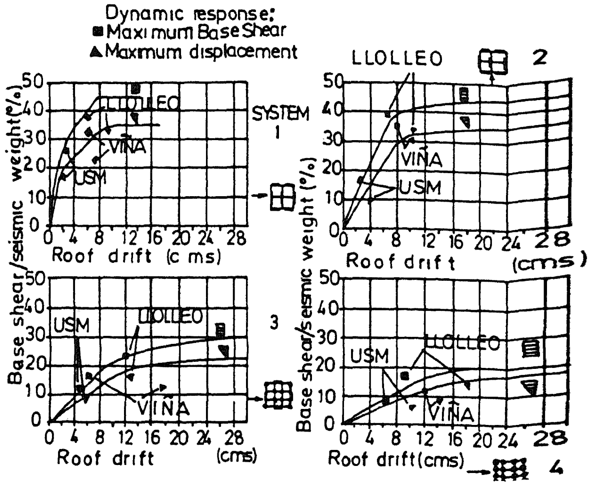


Figure 6. Base shear-top lateral displacement relations.

Figure 5 depicts the plots for the envelopes of interstory drift ratios. From there it is again evident the importance of the density of walls to control damage.

### 3.3 Base shear-roof drift relations

A static inelastic incremental analysis was performed for the 4 structural systems to determine the sequence of damage up to reach the strength of each structure. This is plotted in Figure 6, relating the ratios of base shear/building weight versus top displacement/building height. The analysis was done using alternatively a triangular and a rectangular force distribution in height.

An inelastic dynamic analysis was also performed for these systems using the Takeda (1970) hysteresis model. The ground motions of Figure 1 were used as inputs. Points corresponding to maximum base shear and maximum top displacement from the dynamic analysis are also included in Figure 6. The figure indicates that maximum capacity of the structures was not reached under any of the ground motions, showing a strength reserve. It is also noted that the points corresponding to maximum base shear are closer to the curve associated with a uniform lateral load while the points corresponding to maximum top lateral displacement are close to the curve associated with a triangular lateral load.

The same features are manifested in shear-displacement relations for actual buildings. Figure 7 shows these relations for two structural wall buildings located in Viña del Mar (Bonelli 1989). It should be noted that in both cases the base shear strengths are of the order of 30% of the buildings weight W. These high values of strengths as well as the good general behavior of structural wall buildings have been confirmed by experimental tests (Bonelli and Tobar 1991).

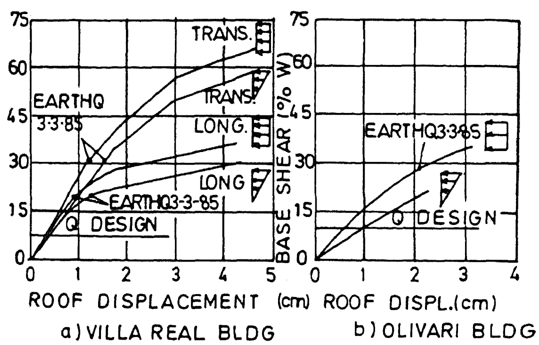


Figure 7. Base shear vs roof drift in real buildings.

## 4 LOCAL CURVATURE DUCTILITY AND DETAILING REQUERIMENTS

Observations of damage in buildings after the March 3 earthquake supports the Chilean practice of non considering special detailing for ductile seismic design of shear walls. To study the necessity of providing ductile design it is convenient to relate global displacement ductility with wall curvature ductility. In this procedure the inelastic curvature accumulates at wall base and has a length lp relative to the wall length and height. No confinement of

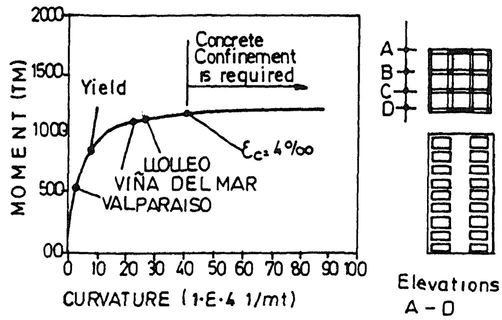


Figure 8. Moment-curvature relations for wall base.

wall is required if the maximum concrete strain is less than 0.004 (Paulay 1986).

Figure 8 and Table 2 show the results of using the aforementioned method to the structural systems under study. Table 2 includes the supplied curvature ductility ratio. In all cases, concrete strains were smaller than 0.004, not requiring concrete confinement, and supplied curvature ductilities were greater than required curvature ductilities. These results are in agreement with the satisfactory performance of buildings under the March 3 earthquake.

Table 2. Curvature ductility ratios

| Structural system | Record | Supplied curvature ductility ratio | Required curvature ductility ratio |
|---|---|---|---|
| 1 | Viña | 8.4 | 1.5 |
| | Llolleo | 8.4 | 2.4 |
| 2 | Viña | 4.3 | 2.2 |
| | Llolleo | 4.3 | 2.2 |
| 3 | Viña | 5.5 | 3.0 |
| | Llolleo | 5.5 | 3.5 |

The record of Valparaiso (in rock) did not demanded ductility.

## 5 INFLUENCE OF THE STRENGTH ON THE STRUCTURAL RESPONSE

*Delgado (1991), studied the inelastic* seismic behavior of two RC buildings designed using two spectra having significant differences in ordinates. It was concluded that the strength increase has almost no influence on lateral displacements, although it somewhat reduces the ductility requirements. Consequently, damage should be controlled acting directly on the lateral drift. On the other hand, it is necessary to provide a minimum strenght for the tensile reinforcement in order to prevent excessive deformation or failure of the tensile steel.

## 6 CAUSES OF OBSERVED DAMAGE IN BUILDINGS

The analysis from the preceding sections has shown the qualities of structural wall buildings as earthquake resistant systems. That is why it is important to mention the *causes of damage for the few buildings that* suffered it.

Let us consider two 15 story RC buildings located next door on the beach area of Viña del Mar: Hanga-Roa and Acapulco. Both suffered big damage in walls, coupling beams and slabs. Some common causes of their poor performance are:

i) Both were designed during the decade of 1960, using a preliminary draft of the present Chilean seismic code NCh 433. of 72, and hand calculations.

ii) At the time of the March 3, 1985 earthquake both had non-repaired damage from a past earthquake in 1971, having a reduced strength.

iii) They have inadequate wall layout, causing torsional problems, since walls do not have special ductility details.

iv) They have strenght deficiencies like bar cutoff, and scarce reinforcement in wall edges.

In the particular case of Hanga-Roa building, in addition to having an inadequate distribution of openings in height, their two main longitudinal walls have a big curvature in plan. This is a nice but too audacious design that caused many problems after the earthquake. In turn, the Acapulco building has steel reinforcement with plain and twisted bars, which produced problems of bond and supplied ductility, respectively (Monge et al 1986).

## 7 CONCLUSIONS

The following general conclusions can be extracted from this study.

a. Structural wall buildings used in Chile, characterized by their high density of walls are very adequate as earthquake resistant structures, due to their low shear stresses, high stiffness and strength, low *requirements of ductility*, good control of deformations causing scarce problems to non structural members, non susceptibility to collapse, easiness to be repaired.

This good behavoir is sustained by two prescriptions of the Chilean seismic code NCh 433. of 72: the minimum design base shear of 6 % of the total building weight, and the limitation to 0.2 % of the story height for interstory displacements. The first rule prevents low resistance, while the second rule controls the non structural damage. Medium rise structural buildings with high density of walls can easily accomplish these requirements.

b. These buildings do not need to have a design and construction with strict demands of detailing and inspection.

c. To keep these non stringent demands, structures must be regular, with appropriate distribution of strength, and must have flexural failure modes, avoiding failure modes of brittle nature.

d. The Chilean customary high density of walls in low rise framed or (suitably) reinforced masonry buildings also produces well behaved earthquake resistant structures.

## 8 ACKNOWLEDGMENTS

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