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Performance of R/C buildings in the 1985 Chile earthquake

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ABSTRACT: One of the most important features of the March 3, 1985 earthquake in central Chile was the performance of moderate-rise reinforced concrete buildings in the coastal city of Viña del Mar. Most of the buildings were designed with structural walls to resist both gravity and seismic loads. Strong ground motion occurred in Viña del Mar, but only a few buildings sustained serious structural damage. In this paper the general characteristics of the building inventory in the city are described, and the results and conclusions of detailed analyses of the three most severely damaged buildings -Acapulco, El Faro, and Hanga Roa- and two undamaged buildings -Torres del Sol and Torres de Miramar- are presented.

1 INTRODUCTION

On Sunday, March 3, 1985, an earthquake off the coast of Chile, with a surface wave magnitude of 7.8, caused significant damage that was officially estimated in US$ 2000 million, amount that represented about 10% of the Gross National Product of the country. Reported Modified Mercalli Intensity in Viña del Mar was VII-VIII, and an acceleration record featuring a 0.36g peak, and more than 40 seconds of strong motion duration, was obtained in the basement of a 7-story building in the city. Elastic and inelastic spectral ordinates for the Viña del Mar record are comparable to those for the NS component of the 1940 El Centro record, as shown in Figure 1.

At the time of the earthquake, more than 400 reinforced concrete buildings ranging from 5 to 23 stories were located in Viña del Mar (Riddell et al. 1987). About 80% of the inventory corresponded to buildings with 5 to 9 stories; 58 buildings ranged from 10 to 14 stories, while 15 to 23-story buildings were 19. Aside from the concentration of damage in five-story buildings of the Canal Beagle housing project, an area where significant topographical amplification effects were reported (Celebi 1986), and although there were cases of severe damage, the majority of the buildings experienced slight or no damage. Thus, the inventory in Viña del Mar provides a unique opportunity to evaluate the methods of structural analysis and design and their accuracy in predicting the seismic behavior of buildings. Towards this general objective, detailed analytical studies of five buildings were conducted as reported herein; parallel efforts with similar purposes have been carried out and reported elsewhere (Wallace and Moehle 1989; Wood 1991).

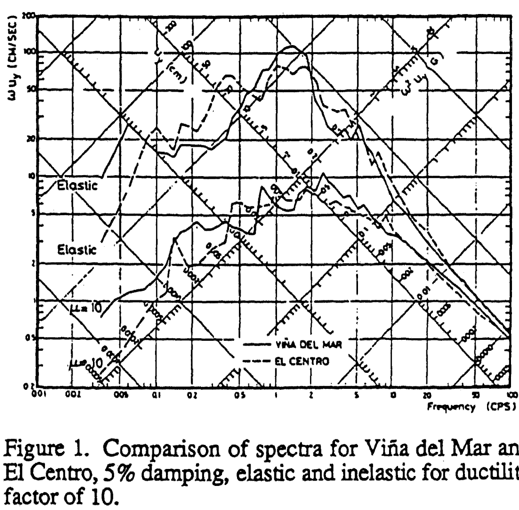

Figure 1. Comparison of spectra for Viña del Mar and El Centro, 5% damping, elastic and inelastic for ductility factor of 10.

2 BUILDING INVENTORY SURVEY

An overall statistical survey of the building inventory in the city was carried out with a twofold purpose: a) to identify the typical characteristics of the buildings, and b) to describe the nature and distribution of earthquake damage (Riddell et al. 1987). Basic information on the date of construction, number of stories, height of the building, total plan area, material properties, structural system, type of foundations, and extent of damage was collected. Data was available for 178 different buildings, which represented a total of 322 buildings since some projects included two or more identical buildings.

2.1 Structural characteristics

Only three of the 322 buildings could be classified as frame structures. Structural walls were used in all other buildings for seismic resistance. The dominant characteristic of these buildings is their relatively high proportion of walls. An interesting index is the ratio of the total cross-sectional area of walls at one level to the floor area. Figure 2 shows an histogram of the number of buildings with given wall indexes; it can be seen that for most buildings this index ranged between 3 and 8%. It was also found that the amount of wall area was nearly independent of the height of the building with an average value of about 6%. The ample wall area results in relatively stiff buildings with calculated periods of vibration being nearly N/20, where N is the number of stories. Drift is controlled in stiff buildings, therefore damage is likely to be limited.

A rough estimate of the design shear stress can be obtained by dividing the total weight of the building above certain level by the available area of walls in one direction at that level. For most buildings in the inventory this index is less than 50 kg/cm^2 (Figure 3). If one assumes a design base shear of 10% of the total weight of the building, the previous index indicates that most buildings were designed for moderate average shear stresses (less than 5 kg/cm^2). In turn, the review of structural drawings for many buildings reveals that walls are relatively lightly reinforced, and boundary elements or special details to provide confinement of the concrete at the ends of the walls were rarely used.

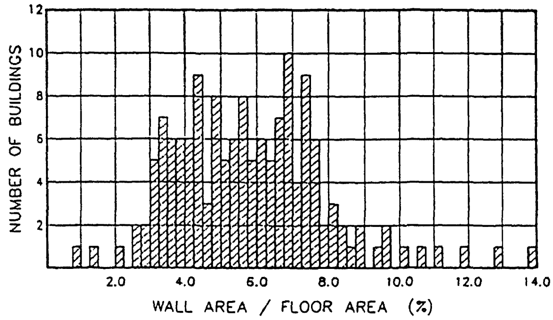

Figure 2. Number of buildings as a function of the amount of cross sectional area of walls.

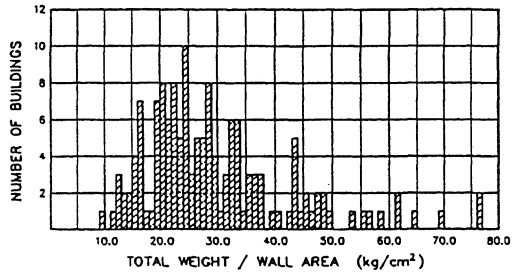

Figure 3. Number of buildings as a function of shear index.

2.2 Distribution of damage

The extent of structural damage in each building was classified in four categories: none, light, moderate, and severe. Excluding the above mentioned damages in Canal Beagle, only 5 buildings experienced severe damage, 7 buildings sustained moderate damage, and 26 buildings were slightly damaged. Although buildings with 12 to 15 stories presented a higher rate of damage, and buildings with 16 or more stories suffered light or no damage, there were no easily identifiable building attributes or indexes that could be well correlated to observed damage. Three of the five most severely damaged buildings are included in the case studies presented hereinafter.

3 CASE STUDIES

As mentioned before, the Viña del Mar building inventory succesfully passed a severe exam, and noteworthy, there were no deaths, and there was no information of injuries, caused by structural or non-structural damages in buildings. The most damaged buildings apparently feature similar characteristics to the rest of the inventory; for example, the shear stress index (Fig. 3) is 39, 33, and 20, for the Acapulco, Hanga Roa, and El Faro buildings respectively, thus indicating typical amounts of available wall area, while the undamaged Torres del Sol and Torres de Miramar present higher indexes of 76 and 61 respectively. However, the structural irregularity of the three damaged buildings suggests that configuration plays an important role in the seismic performance of buildings.

The buildings selected for study are founded on similar materials and conceivably experienced a similar ground motion. Four of the buildings are located along the beach in Viña del Mar (Figure 4); these buildings are between 1200 and 1600 meters away from the site where the earthquake record was obtained. The foundation material for these buildings corresponds to dense coarse/medium sands and fine gravel. The El Faro building, 4 km north of the previous buildings, was founded on top of a cemented sand dune.

The methods used for the analysis of the buildings ranged from routine design-office methods to slightly more sophisticated procedures. The methods used were: equivalent lateral force code-type analysis, modal superposition analysis, elastic response history analysis to actual ground motion, and static limit analysis to determine the ultimate capacity to lateral loads. Since the behavior of some of the buildings raised questions about the way spatial characteristics of buildings structured mainly with walls are taken into account in common practice, pseudo-tridimensional (P3D) and tridimensional (3D) models were used. Satisfaction of compatibility conditions of wall segments adjoined to form C, L, and T-shaped, or other irregularly shaped walls is particularly relevant in this case. A brief discussion of the limitations of the P3D model is given by Riddell and Vasquez (1984). The design requirements were verified using allowable stress design according to Chilean codes, and ultimate strength design according the ACI code. The main objective of studies using these methods was to assess their reliability in identifying structural weaknesses. Detailed reports of the analyses are available (De la Llera and Riddell, 1988, 1989; Marin et al. 1990; Huerta and Riddell, 1990a, 1990b).

3.1 Acapulco building

It is a 15-story one-basement building constructed in 1962-1964. Its lateral resistance is provided by a system of structural walls arranged in plan in a

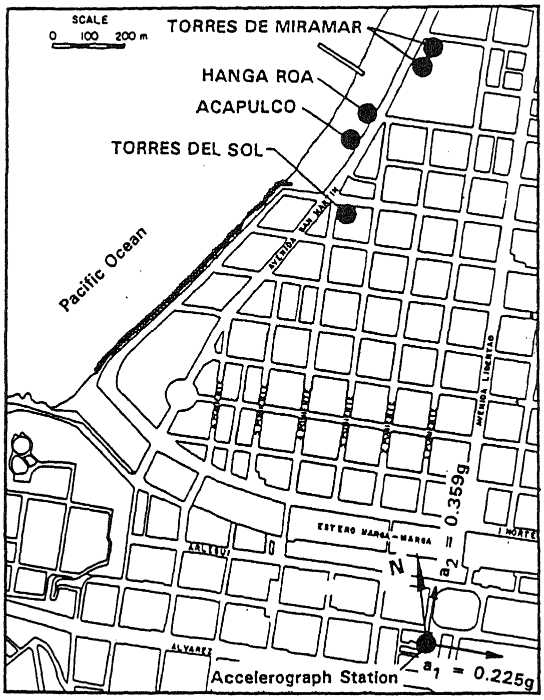

Figure 4. Location of analyzed buildings and accelerograph station in Viña del Mar.

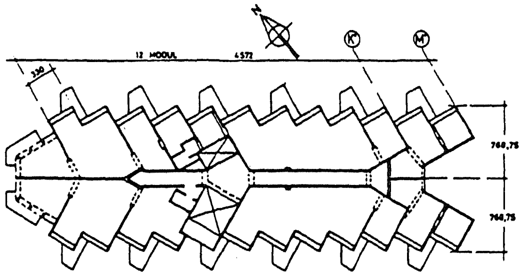

Figure 5. Acapulco building floor plan. Levels 2 to 6.

herring-bone pattern as shown in Figure 5. Walls are not connected by beams, except by shallow ones in a few locations. Properties of the materials specified for construction were: concrete cubic strength of 225 kg/cm^2 (equivalent to 190 kg/cm^2 cylinder strength) and steel yield stress of 2800 kg/cm^2. Earthquake inspection revealed that twisted bars with yield stress of 5100 kg/cm^2 had been also used, but there was no evidence of the extent it substituted or supplemented the original reinforcement; therefore the analysis of the building was based on the original structural drawings.

Walls and slab-wall connections sustained extensive cracking during the earthquake. Wall M' was badly damaged: crushing of the concrete, yielding and buckling of the main reinforcement, and slippage along the construction joints was observed between the ground and fourth levels; wall K' also showed crushed concrete at the ground level. The analysis revealed that these walls, as well as several other structural elements, were underdesigned according to both allowable stress and ultimate strength methods. The results of modal superposition and response history analyses correlate well with observed damage; the latter method is particularly effective in this case in identifying a more critical condition of the building at 1/3 and 1/2 of its height in the transverse and longitudinal directions respectively.

This building is quite flexible for Chilean standards. The periods of vibration for the first three modes were 1.79, 1.18, and 1.16 seconds for the P3D model, where the first two modes corresponded to transverse translation combined with torsion, and the third corresponded to translation in the longitudinal direction. The interstory drift exceeded the Chilean code (INN 1972) limit of 0.2%, reaching 0.43% at the top of the building in the northwest end of the plan.

The calculated ultimate capacity of the building to uniform lateral load corresponded to base shears of 0.096W and 0.124W (W=total weight of the building) in the transverse and longitudinal directions respectively, for flexural mechanisms of failure forming at intermediate stories. Ultimate strengths associated to shear modes of failure were 0.30W and 0.36W respectively. If a triangular distribution of load is used instead of the uniform one, the base shears associated to flexural modes of failure become 0.07W and 0.09W. The very low strength of the building is consistent with large ductility demands and significant flexural damage. Also note that the base shear according to the Chilean code (INN 1972) is 0.06W (allowable stress design).

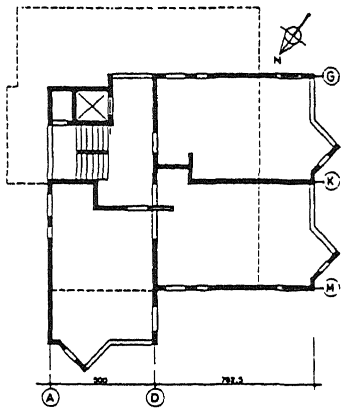

Figure 6. El Faro building. Typical story plan.

3.2 El Faro building

This was an eight-story one-basement building built in 1981. Materials specified for construction were: concrete cubic strength of 225 kg/cm^2 and steel yield stress of 4200 kg/cm^2. The unsymmetrical arrangement of walls resulted in great eccentricity and significant torsional response. The typical floor plan is shown in Figure 6, where dashed lines delineate the basement plan. The building was severely damaged during the earthquake. There was no failure of the soil or the foundations. Damage concentrated in the first story: the wall along

axis M and its adjacent wall in axis D collapsed and the building leaned sharply as seen in Figure 7. Since collapse was imminent, the building was dynamited a few days after the earthquake.
The first three periods of vibration, calculated for a 3D model of the building, were 0.57, 0.3 and 0.23 seconds; coupling of translations in both directions and rotation was present in each of the three modes. The calculated ultimate capacity of the building to uniform lateral load corresponded to a base shear of 0.15W in both horizontal directions for a flexural failure mechanism. Ultimate strengths associated to shear modes of failure were 0.64W and 0.55W in directions parallel to axes K and D respectively. The maximum interstory drift for the Chilean code seismic load was about 0.05%, hence much less than the allowable 0.2%.
The previous parameters reveal stiffness and strength apparently enough to expect good seismic performance, however, torsional response results in uneven force and deformation demands upon the various structural walls. Indicative of the considerable torsion in this building is the fact that several walls have higher stresses for earthquake analysis in the direction perpendicular to the walls than for the earthquake acting in the direction parallel to them. In turn, this is a typical case in which the effect of two simultaneous orthogonal earthquake components, and consideration of the critical angle of incidence for the earthquake motion, are relevant.
Evaluation of individual members shows a lack of flexural reinforcement in several walls of the first story, including walls M and D, while concrete compression and shear are moderate. Results of the analysis and observed damage correlate well. Wood et al. (1991) investigated the behavior of this building and concluded that the building collapsed after the longitudinal reinforcement fractured in wall M in the first story; the brittle failure was related to underreinforcement, condition that was aggravated by the torsional response.

Figure 7. El Faro building. South-west elevation.

3.3 Hanga Roa building

It is a 15-story two-basement building constructed in 1970-1971 with unusually shaped plan as shown in Figure 8. The curved walls along the central corridor have doorway openings staggered in the vertical direction. The resulting one-story-deep lintels suffered severe damage featuring cracks up to 50 cm wide. Reportedly, there had been damage in these elements in a previous earthquake -occurred just after construction was completed in 1971- and had not been repaired but cosmetically. There was also significant cracking of the slab. The building was extensively repaired and retrofitted after the 1985 earthquake.
A 3D model is inevitable in this case. The first three periods of vibration were 0.84, 0.69, and 0.56 seconds; the first period corresponded to translation in the direction of axis F coupled with torsion, the second combined translation in both directions with torsion, and the third was mainly a translational mode in the direction perpendicular to F. Maximum interstory drift was 0.06%. Again, the provided stiffness alone was not sufficient to prevent damage.
The analysis indicated that the nominal shear capacity of the lintels varied between 40% to 80% of the required code strength. The lintels also presented insufficient flexural capacity. Again, the results of the analysis correlated well with the observed damage.

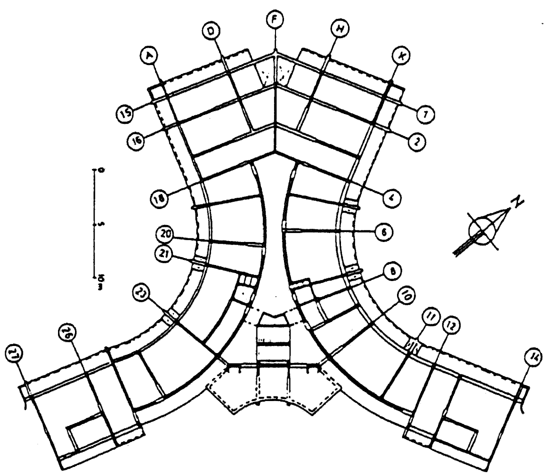

Figure 8. Hanga Roa building. First story plan.

3.4 Torres the Miramar buildings

These are two twin towers with 21 stories and one basement constructed in 1974-1976. The shape of the plan is an equilateral triangle with 34.9 m side. Figure 9 shows the floor plan of levels 1 and 2, and Figure 10

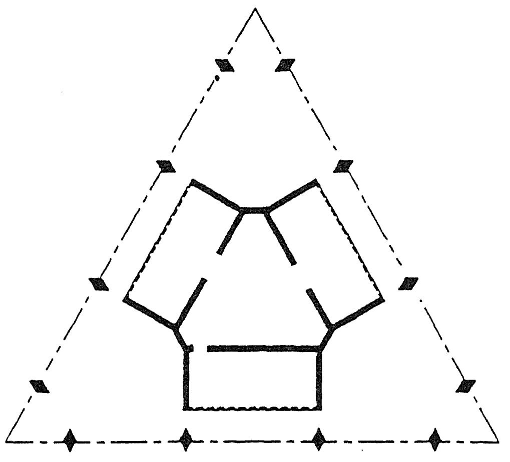

Figure 9. Floor plan of Torres de Miramar building, levels 1 and 2.

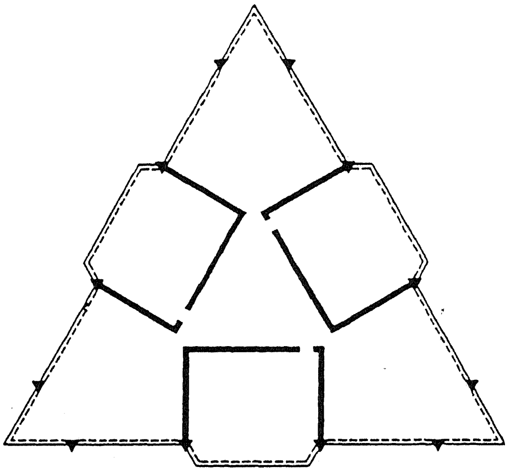

Figure 10. Floor plan of Torres de Miramar building, odd numbered levels from 3 up.

shows the plan of odd numbered floors from 3 up; even numbered floors differ to the previous in that door openings are in the opposite side of the core walls. No structural damage was observed in these buildings.

The total cross sectional area of walls is 6.6% of the plan area in the most critical level (2nd floor). The calculated periods of vibrations, for a fixed-base model of the building, for the first three modes, were 1.19, 1.02, and 1.01 seconds; the first mode was a pure torsional mode as expected from the symmetry and the relatively low torsional stiffness of the building. By means of ambient vibration tests Midorikawa (1990) measured periods of 1.32, 1.07, and 1.06 seconds, which are in close agreement with the calculated ones. The maximum interstory drift, measured at the center of mass, was 0.07%, while peripheral displacements were 50% larger than at the mass center (accidental eccentricity was introduced by shifting the mass centers in 5% of the plan width).

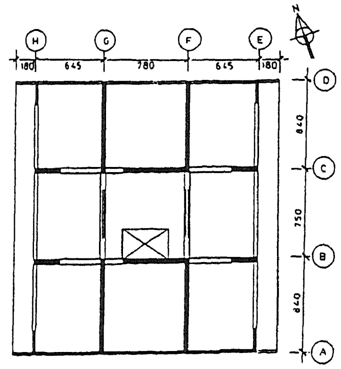

Figure 11. Floor plan of Torres del Sol building, levels 3 to 22.

The design base shear according to the Chilean code was 0.066W. For the corresponding lateral loads strength requirements were satisfied in structural walls and columns, except in the first story columns near the middle of the sides where the demand was 5.3 and 8.7% larger than the capacity. However, soil structure interaction effects significantly influenced the dynamic characteristics of this buildings according to Wallace and Moehle (1989); they found that the translational period increased up to 43% when soil flexibility was considered in combination with intense ground motion. The elongation of the period resulted in about 45% reduction of the base overturning moment and about 25% reduction of the base shear, for response history analyses.

The calculated ultimate strength of the building to a triangular distribution of lateral load corresponded to a base shear of 0.18W for a flexural mode of failure associated to total plastification of the third level. The ultimate strength related to shear capacity corresponded to a base shear of 0.23W; the weakest level was also the third. Considering a capacity of 0.18W, 5% damping, and the inelastic response spectrum for the ground motion recorded in Viña del Mar, ductility requirements associated to a global ductility larger than 3 are inferred for this building, which should have manifested in terms of damage; however, if a 43% increase in period is considered, the ductility demands reduces to a global ductility of 1.5 which is more consistent with the observed behavior of building.

3.5 Torres del Sol building

This is a 22-story building with a penthouse and two basements that experienced only light structural damage during the earthquake. It was built in 1981-1982. The first three periods of vibration of 1.18, 1.17, and 0.79 correspond to uncoupled translational and rotational modes; the periods increase to 1.52, 1.42, and 0.85 respectively if a P3D model of the building is used. The building is almost symmetric in plan as shown in Figure 11. The structure is regular in height, except that it has a significant discontinuity in the second story of axes F and G as shown in Figure 12. The second story walls worked as deep beams coupling the walls below. These coupling beams exhibited incipient shear cracking after the earthquake, although they were not underdesigned.

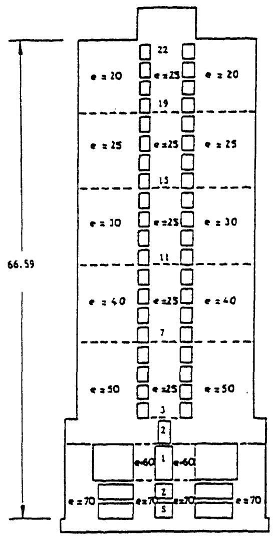

Figure 12. Torres del Sol building. Elevation of axes F and G.

The analysis shows that the reinforcing steel in the exterior walls in axes F-G at the basement and 1st-story levels exceed the allowable stress; these walls are subjected to flexure combined with axial tension for lateral earthquake loading. However, no damage could be seen in these walls even though they must have been driven within the inelastic range. Compression and shear stresses in concrete were in general quite low. Similar proportions to the Torres de Miramar buildings suggest that this building may have also experienced favorable soil-structure interaction effects. This condition, the low concrete stresses, and the symmetric configuration, may be possibly considered as beneficial factors justifying the good behavior of the building.

CONCLUSIONS

Based on the inventory survey and the previously presented cases the conclusions of the study are the following: a) The good earthquake performance of multistory buildings in Viña del Mar has shown that the use of generous wall areas results in an effective earthquake resistant structural system, b) however, abundant cross sectional area of wall does not guarantee good seismic behavior, c) Damage distribution does not correlate well with simple structural indexes or straightforward causes, however, irregularity was a typical characteristic of damaged buildings, d) Standard methods of analysis succeeded in identifying the weaknesses that probably caused the poor behavior of the most severely damaged buildings, e) Actual tridimensional modeling of the structure, including compatibility at joints common to two frames, is necessary in buildings presenting strong translational-torsional coupling, f) Seismic design codes shall include provisions for taking into account the effect of two simultaneous earthquake components, and the effect of the most critical earthquake direction, g) Buildings presenting lateral-torsional coupling are particularly vulnerable to the effects mentioned in the previous item.

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REFERENCES

Celebi, M. 1986. Seismic site-response experiments following the March 3, 1985 central Chile earthquake. Report 86-90, USGS, Menlo Park, Ca.

De la Llera, J.C., Riddell, R. 1988. El terremoto de 1985 en Chile, análisis del edificio Acapulco. Depto. de Ingeniería Estructural, Universidad Católica de Chile, DIE Nº 88-1 (in Spanish).

De la Llera, J.C., Riddell, R. 1989. El terremoto de 1985 en Chile, análisis del edificio El Faro. Depto. de Ingeniería Estructural, Universidad Católica de Chile, DIE Nº 89-1 (in Spanish).

Huerta, A., Riddell, R. 1990. El terremoto de 1985 en Chile, análisis del edificio Hanga-Roa. Depto. de Ingeniería Estructural, Universidad Católica de Chile, DIE Nº 90-9 (in Spanish).

Huerta, A., Riddell, R. 1990. El terremoto de 1985 en Chile, análisis del edificio Torres de Miramar. Depto. de Ingeniería Estructural, Universidad Católica de Chile, DIE Nº 90-6 (in Spanish).

Instituto Nacional de Normalización (INN) 1972. Cálculo antisísmico de edificios. NCh 433-72, Santiago, Chile (in Spanish).

Marin, A., Riddell, R., Vásquez, J. 1990. El terremoto de 1985 en Chile, análisis del edificio Torres del Sol. Depto. de Ingeniería Estructural, Universidad Católica de Chile, DIE Nº 90-4, (in Spanish).

Midorikawa, S. 1990. Ambient vibration tests of buildings in Santiago and Viña del Mar. Depto. de Ingeniería Estructural, Universidad Católica de Chile, DIE Nº 90-1 (in English).

Riddell, R., Wood, S., De la Llera, J.C. 1987. The 1985 Chile earthquake: Structural characteristics and damage statistics for the building inventory in Viña del Mar. Civil Engineering Studies, SRS Nº 534, University of Illinois, Urbana, Illinois, U.S.A.

Riddell, R., Vasquez, J. 1984. Existence of centers of resistance and torsional uncoupling of earthquake response of buildings. 8th. WCEE, San Francisco, California, Vol. 4, pp. 187-194.

Wallace, J.W., Moehle, J.P. 1989. The 1985 Chile earthquake: An evaluation of structural requirements for bearing wall buildings. EERC-89/05, University of California, Berkeley.

Wood, S.L. 1991. Performance of reinforced concrete buildings during the 1985 Chile earthquake: Implications for the design of structural walls. Earthquake Spectra, EERI, Vol. 7, Nº 4.

Wood, S.L., Stark, R., Greer, S.A. 1991. Collapse of eight-story RC building during the 1985 Chile earthquake. J. of Struc. Div., ASCE, Vol. 117, Nº 2.