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ANALYSIS OF A COLLAPSED BUILDING USING THE DISPLACEMENT SEISMIC DESIGN APPROACH

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ABSTRACT

The Villa Olímpica, a five-story R/C building constructed in 1961 in Santiago, Chile, suffered partial collapse during the 1985 earthquake, caused by the failure of the facade columns at the lowest story. Linear-elastic analyses showed that the structure complied not only with the design requirements that were in force at the time building was constructed, but also with the current code provisions. Since the code design procedures were not able to detect the potential weakness of the building, the structure was analyzed by using the displacement design approach. The inelastic deformation capacity of the building under lateral actions, was estimated by performing an incremental collapse analysis and by simple calculations on the collapse mechanism of the failed structural facade. The inelastic displacement demand imposed on the structure was estimated by performing non-linear analyses of the building subjected to the earthquake records, and by using non-linear displacement response spectra of the same records. The estimated displacement demand was very close or even larger than the building deformation capacity. By comparing displacement capacity and demand, the weakness of the building was detected in a straightforward way.

KEYWORDS

Seismic design; Reinforced concrete buildings; Displacement seismic design

INTRODUCTION

The Villa Olímpica, a condominium of several five-story R/C buildings in Santiago, Chile, was constructed in 1961. The buildings, as shown schematically in Fig. 1, had R/C and masonry walls resting on columns at the bottom story. One of those buildings suffered partial collapse during the 1985 earthquake, caused by the failure of the facade columns, as shown in Fig. 2. The upper part of the wall fell to the ground, causing progressive collapse of the floor slabs between axis 15 and 16. The columns of three other buildings of similar characteristics, presented some degree of damage without reaching collapse. The cracking pattern suggested the columns had reached their shear strength and that the axial forces due to the overturning moment would have been important.

Monge et. al. (1986) suggested the collapse was mainly due to the effect of the torsion in plan on the building response, increasing the demand on the elements along the perimeter of the building plan. Cruz et. al. (1988) mentioned the lack of appropriate requirements in the chilean seismic code to design buildings with such type of irregularities on the height.

The observed type of failure is typical of buildings with a soft story. High levels of ductility demand are generated on the elements of that story due to the concentration of non-linear deformations. A brittle failure can occur due to the lack of a capacity design and adequate reinforcement detailing, as well as due to the degradation of shear strength caused by repeated cycles of non-linear deformations. It is demonstrated in this work that current code design procedures are not able to detect the potential weakness of the building, as it is done in a straightforward way by using the displacement design approach.

CODE TYPE ANALYSIS

In order to study the building design, code-type analyses were performed using seismic and R/C design codes that were in force in Chile at the time the building was constructed and the current ones.

Seismic Analysis

Linear dynamic analyses were performed according to the Chilean Seismic Design Code NCh433 (INN 1972, 1993) in the two orthogonal directions X and Y, as indicated in Fig. 3. The seismic weight of the building was 647 [T], and the first translational periods were 0.32 [sec] in the longitudinal direction (X) and 0.18 [sec] in the transverse direction (Y). Since the collapse of the building showed the transverse direction as the critical one, only the results of the analysis on that direction are presented.

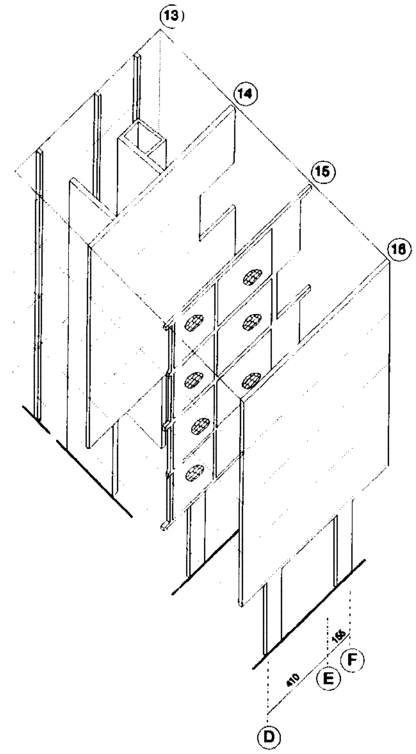

Fig. 1. Schematic view of the Villa Olímpica Building

1972 Seismic Code Analysis. The base shear equal to 9.5% of the seismic weight was larger than the minimum code value of 6%. The maximum value of the inter-story drift ratio, 0.13‰ at the bottom level, was less than the code maximum of 2.0‰. Due to torsion in plan, the shear forces on the columns along the perimeter of the building plan (Axis 16) were increased up to 100% over the value due to pure translation, which is just the maximum increase accepted by the code.

1993 Seismic Code Analysis. Since the calculated base shear was lower, it was adopted the code maximum value of 8.2% of the seismic weight for this building. The distribution of the relative displacements on the height, in terms of inter-story drift ratio, are presented in Fig. 4. As shown in that figure, the maximum value of 0.11‰ that occurred at the bottom level, resulted less than the code maximum of 2.0‰. The plan

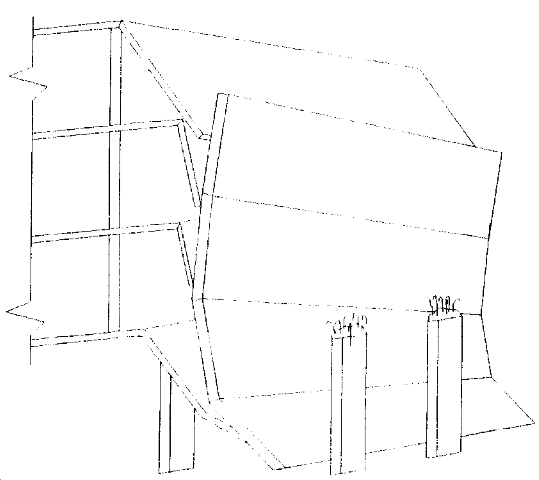

Fig. 2. Schematic view of the collapse during the 1985 earthquake

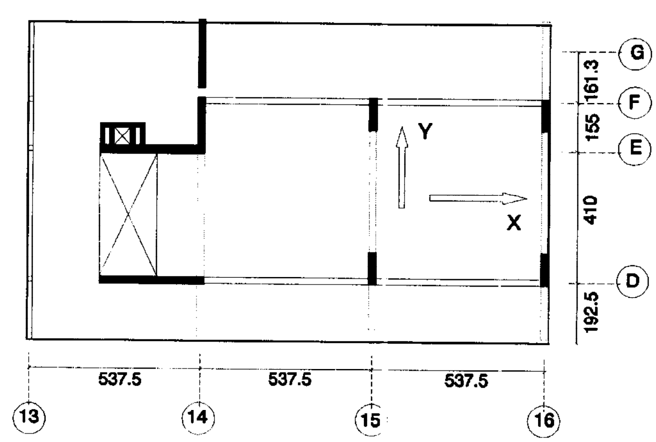

Fig. 3. Plan view of the first story

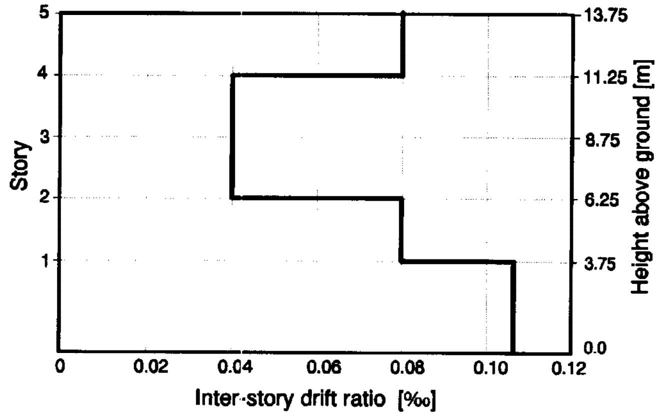

Fig. 4. Inter-story drift ratio, NCh 433-1993

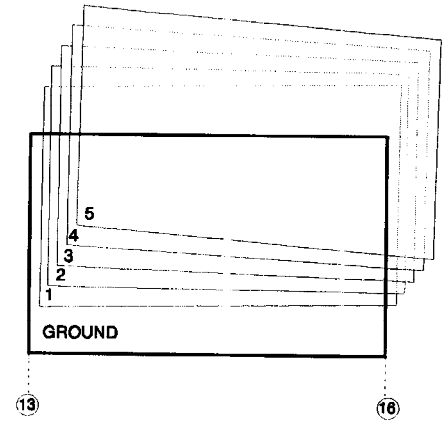

Fig. 5 Floor slab displacements, NCh 433-1993

view of the position of each story slab is shown in Fig. 5. For the sake of clarity, rotations of the plans are not scaled. The increase in displacements on the columns along the perimeter of the building plan, due to the torsion in plane effect, was less than the maximum value of 1‰ of the story height specified by the code.

R/C Element Design

In order to check the design of the R/C critical elements, i.e. columns of axis 15 and 16, the internal forces on those elements were compared with the allowable and strength values of the R/C design codes.

1957 Chilean Code NCh429. This code, in force at the time the building was constructed, was based on the allowable stress design method and it did not include any special provisions for seismic design. Allowable internal forces for the critical columns, calculated with a reinforcement allowable stress of 2000 [Kg/cm^2] and a concrete allowable stress of 105 [Kg/cm^2], are presented in terms of the interaction diagram of Fig. 6. Seismic forces on the columns, calculated with the 1972 seismic code, have been plotted on the same figure. The following combinations of dead load D, live load L, and earthquake load E were considered: (D + L) and (D + 0.25L + E). As it can be seen in the figure, all the pairs Moment-Axial force so calculated satisfied the code requirements.

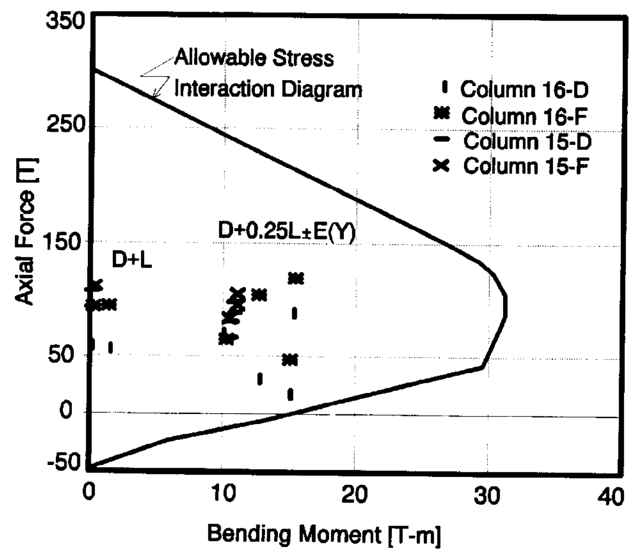

Fig. 6. Interaction diagram of allowable forces and seismic forces on critical columns

ACI 318-89. Since the ACI requirements have been adopted as the official R/C design code in Chile, the design of the building was reviewed with respect to this code. According to the original design specifications, a concrete characteristic strength of 160 [Kg/cm^2] and reinforcement yielding strength of 4000 [Kg/cm^2] were considered. Nominal strength of the elements under combined flexure and axial load, calculated according to the usual assumptions for this type of elements, is presented in terms of the interaction diagram (M_n, N_n) in Fig. 7. Shear nominal strength V_n of the columns was calculated according to the usual provisions of the ACI Code, considering the contributions of the concrete V_c and the transverse reinforcement V_s. In order to present the shear strength in the same figure, the moment M_v associated with the shear strength was calculated by assuming the columns were in double curvature. The results, in terms of the interaction diagram (M_v, N_v) are shown in Fig. 7. The design strength interaction diagram are represented by the curve (ϕM_n, ϕN_n) and (ϕM_v, ϕN_v), where ϕ is the appropriate ACI strength reduction factor. Finally, the curve (ϕM_s, ϕN_s) in the same figure represents the design shear strength interaction diagram assuming V_s is the only contribution to the shear strength. The required strength, in terms of the pairs (M_u,N_u), was calculated for the seismic actions obtained from both versions of the seismic design code, considering the ACI loading combinations. The results, for the column 16-F, are shown in Fig. 7.

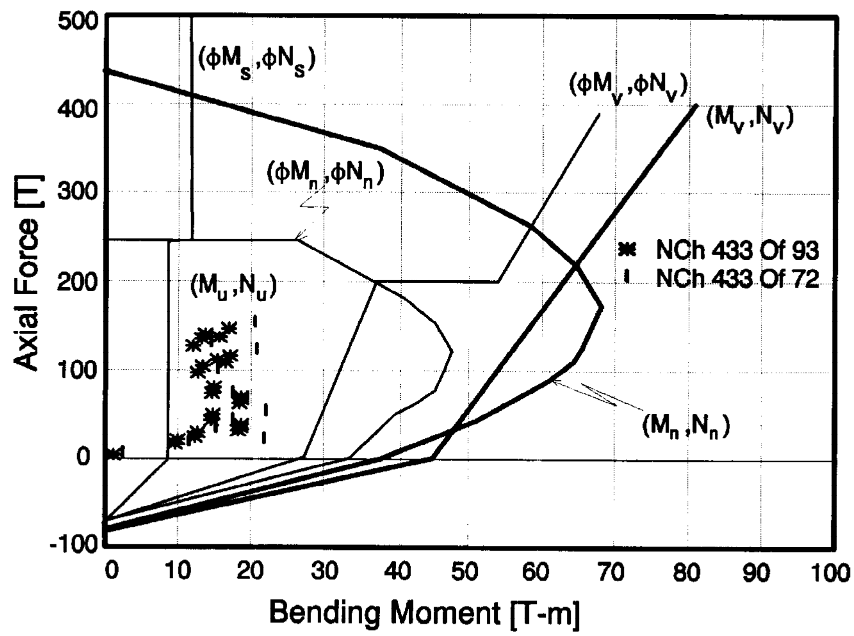

Fig. 7. Nominal and design strength M-N interaction diagram and seismic forces on column 16-F

As it can be seen in Fig. 7, the combined flexure-axial load design strength and the shear strength are large enough to provide the required strength. However, when the contribution of concrete is neglected, the calculated shear strength was lower than that required. These results were the same for all first story critical columns. In addition to that, the columns did not satisfy the ACI specifications regarding the minimum reinforcement ratios for columns supporting walls in seismic zones.

NON-LINEAR DISPLACEMENT CAPACITY

Non-linear analyses of the building, subjected to a monotonically increasing lateral loading distribution in the Y-direction, were performed by using the DRAIN-2DX computer program. Uniform and linear distribution of lateral loads on the building height were considered. Given the slenderness of the critical columns, it was appropriate to consider only flexural non-linear behavior, in terms of the typical moment-curvature diagrams shown in Fig. 8. Crushing of concrete at a ultimate deformation of 3.8‰ and fracture of longitudinal reinforcement at an ultimate deformation of 4% were considered as failure criteria for the originally designed cross-sections of the columns under combined flexure-axial load. The moment-axial load

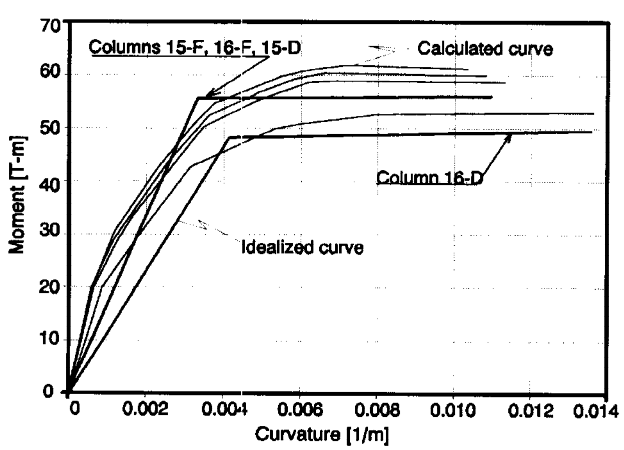

Fig. 8. Moment-curvature diagrams for critical columns

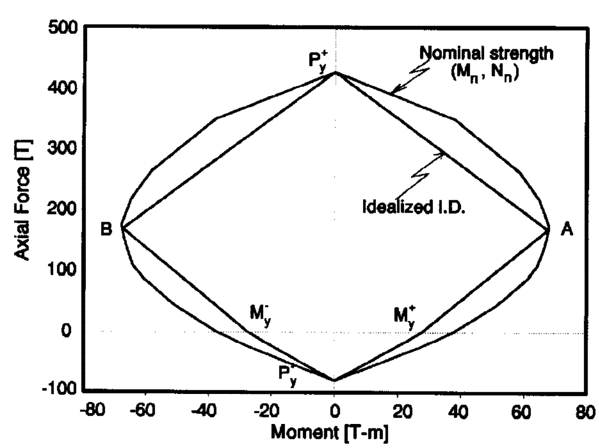

Fig. 9. Moment-axial force interaction diagram for critical column cross-sections.

interaction diagrams of those elements is shown in Fig. 9. Shear strength was calculated as described previously.

Since the results obtained from the analyses of both cases of lateral loading were very similar, only those from the uniform distribution will be discussed here. The base shear-overall drift ratio curve, representing the displacement capacity of the building, is shown in Fig. 10. The curves corresponding to each one of the building axis have been plotted in the same figure. The main events have been identified with number marks on the curves. The first elements to reach their capacity were the columns at the tensioned side of the building (marks 2 and 1'). However, experimental evidence has shown that, after developing its flexural strength, a tensioned element is able to behave as a simple tensioned tie with a high level of ductility. Under this assumption, collapse of the building was defined by shear failure of the columns at the tensioned side, reaching an overall drift ratio of 1.09‰ and 1.24‰, and a base shear of 189.7 [T] and 156.8 [T] for each direction of analysis respectively (marks 4 and 5').

By assuming the columns had enough transverse reinforcement to avoid a shear failure, the capacity of the building was defined by the flexural strength of the columns at the compressed side of the building, reaching an overall drift ratio of 1.96‰ and 1.31‰ and a base shear of 204.5 [T] and 159.3 [T] for each side respectively (marks 7 and 6'). The same analyses previously described were carried out assuming the critical columns had been detailed according to ACI 318-89 Code, i.e., they had enough transverse reinforcement to provide confinement and concrete would reach an ultimate deformation of 8‰. In this case, the building reached an overall drift ratio of 3.27‰ and 3.05‰ and a base shear of 201.8 [T] and 164.8 [T] for each side respectively (marks 8 and 7').

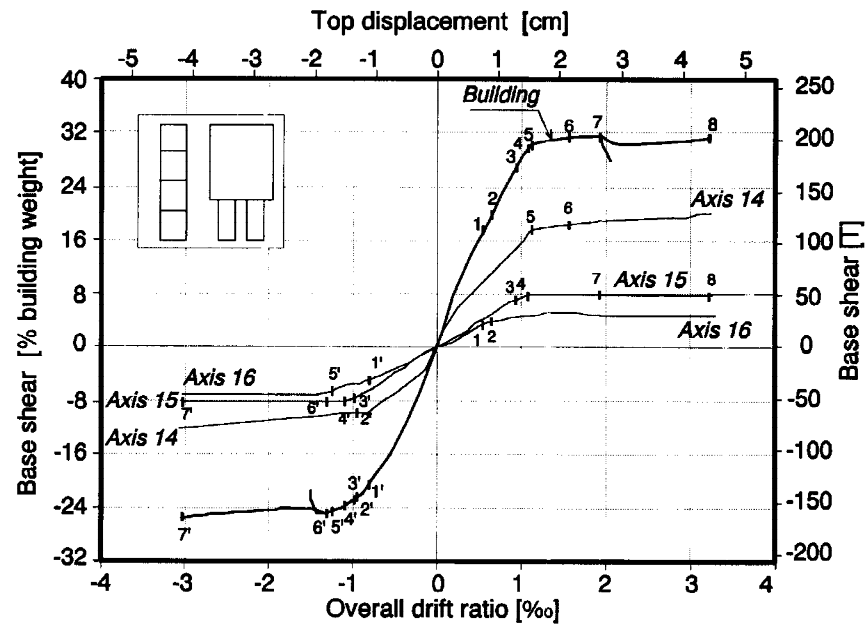

Fig. 10. Displacement capacity under uniform lateral load: Base shear-overall drift ratio curve

As expected, the building behavior was governed by the non-linear deformation capacity of the first story columns, remaining practically elastic the elements of the upper stories. The distribution of displacements on the height at the time the building reached its capacity, for the cases of the original reinforcement and assuming confining reinforcement is added, are shown in Fig. 11. The estimated displacement capacity of the building was much lower than that of the order of 1 to 2% expected for R/C buildings before reaching collapse.

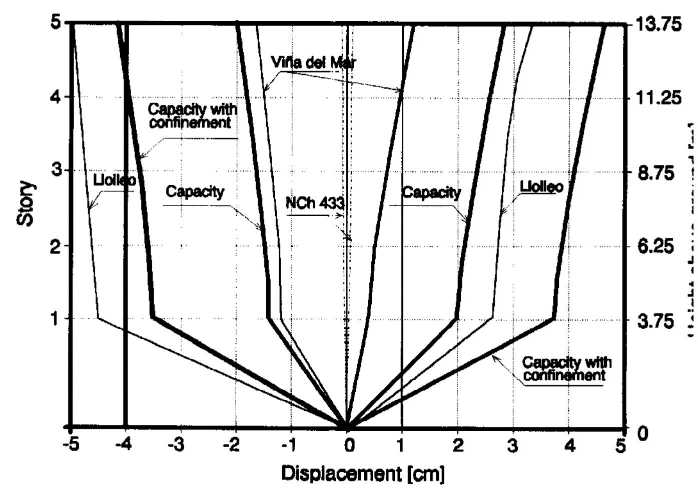

Fig. 11. Distribution of capacity and demand of displacements on the building height.

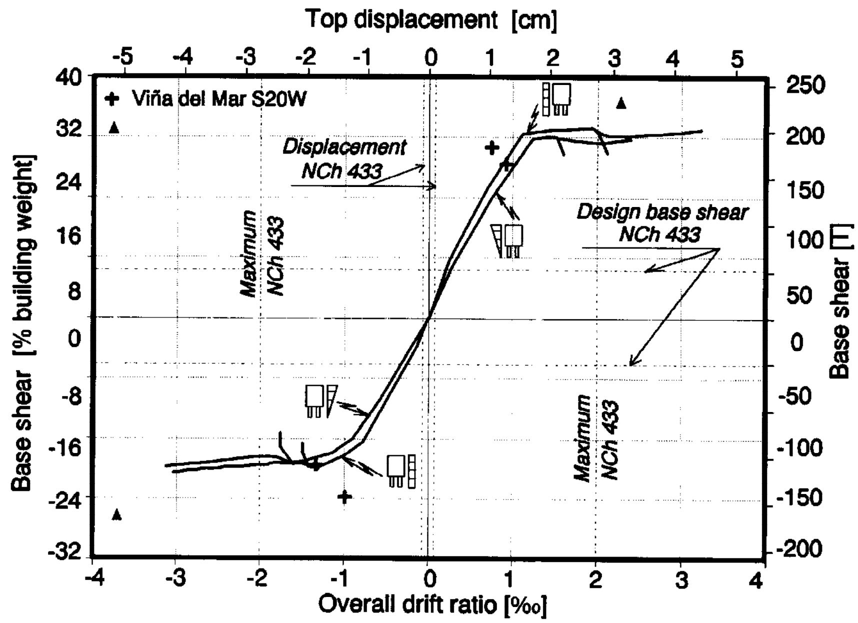

Fig. 12. Non-linear displacement demand and capacity

The inelastic displacement demand imposed on the structure was estimated by performing non-linear dynamic analyses of the building subjected to the earthquake records of Viña del Mar (GPA of 0.36g) and Llolleo (GPA of 0.67g) (Ridell and Cruz, 1994). The analyses were carried out by using the DRAIN-2DX computer program, considering the same assumptions previously described about the behavior of the elements. The results, in terms of the distribution of the maximum values of the displacements on the building height, are shown in Fig. 4.11. The displacements obtained from the linear elastic analysis, according to the chilean seismic code NCh433, have been also plotted in the figure. The same results, in terms of the base shear-overall drift ratio values, are compared with the building non-linear deformation capacity in Fig. 4.29. As it can be seen in these figures, the demand imposed by the Llolleo record is larger than the building capacity. In the case of the Viña del Mar record, the building would have been just on the limit of its capacity.

The displacement demand was also estimated by using results obtained from analyses of SDOF systems subjected to the same 1985 earthquake records, in terms of non-linear displacement response spectra, as shown in Fig. 4.13 (Marillanca, 1996). Spectral displacement was obtained using the building period T, calculated assuming uncracked sections, and ($\sqrt{2}$)T, corresponding to sections with reduced stiffness. The results are summarized in columns (d) and (e) of Table 1. Ratios between displacement obtained from the non-linear analysis and spectral displacements are presented in columns (f and (g). As it can be seen in that table, there is a good agreement between the results of both methods.

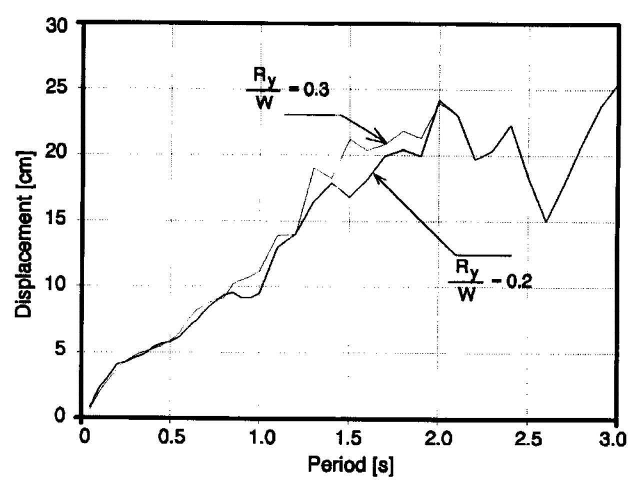

Fig. 13. Non-linear displacement response spectra, Lloleo N10E record.

CONCLUSIONS

Linear elastic analysis showed the building satisfied the requirements of the chilean seismic design code and the R/C code. However, the critical columns reinforcement did not satisfy the ACI requirements for detailing of elements in seismic zones.

Due to the presence of the soft story, the building had a very low deformation capacity compared with that expected in R/C structures.

Table 1. Displacement demand obtained from non-linear analises and from displacement response spectra, 1985 Chile's earthquake

Record	Direction	Non-linear analysis displ. [cm]	Spectral displacement [cm]		Non-linear analysis displ. / Spectral displacement	
			T	$T\sqrt{2}$	T	$T\sqrt{2}$
(a)	(b)	(c)	(d)	(e)	(f)	(g)
Llolleo	+	3.28	3.38	4.36	0.97	0.75
Llolleo	-	4.93	3.50	4.31	1.41	1.14
Viña	+	1.20	0.80	1.19	1.50	1.01
Viña	-	1.66	1.35	2.50	1.23	0.66

The displacement demand imposed by the earthquake would have been larger than the deformation capacity of the building as originally designed.

If the building had been designed according to the ACI requirements for R/C structures in seismic zones, the deformation capacity increase would have not been enough to satisfy the earthquake demand.

The potential weakness of the building was detected in a straightforward way by using the displacement design approach.

ACKNOWLEDGMENTS

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