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THE DISPLACEMENT SEISMIC-DESIGN APPROACH APPLIED TO R/C BUILDINGS

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ABSTRACT

The objective of this work is to show the need to improve current design procedures to control damage by introducing the displacement control. This work demonstrates that in spite of the low ductility demands of recent Chilean earthquakes, certain buildings should be designed following capacity-design techniques.

A 16-story building has been designed according to the Chilean Code NCh433Of.72 and ACI 318-89 Code. No special calculations were done to improve ductility. Global displacements, forces, and drift ratios were lower than code limit values.

The resulting displacement capacity was limited to 0.5%. If capacity-design rules are used to prevent different types of failures, drifts no larger than 1% can be reached in this type of structure. However, lateral-displacement demands of the 1985 Chilean Earthquake did not exceed the estimated capacity of the building.

In conclusion, the current-code design procedures are not able to detect the potential weakness of this building. The weakness of the building was directly detected by using the displacement design-approach

Low demands obtained for the Chilean earthquakes could lead to the acceptance of brittle. However, the probabilistic nature of the phenomenon advises the use of ductile structures.

KEYWORDS

Damage; lateral deformations; earthquake-resistant design; ductility requirements; 16-story building; incremental collapse-analysis; capacity-design rules; March 3, 1985 Chilean Earthquake

INTRODUCTION

R/C wall-buildings have not experienced significant collapse during recent earthquakes. Relatively low damage was observed in these buildings during the March 3, 1985 earthquake in Chile. Wall-buildings, however, have been not immune to structural damage.

The satisfactory earthquake performance of most wall-buildings in Chile suggests that no special ductility details are needed. The Chilean Code NCh433.Of93 specifies reduction factors R. These guidelines specify that these values have been established for buildings designed with no special considerations to supply ductility.

This paper shows that even when regular walls are symmetrically positioned in plant, very brittle structures may result if no special considerations to supply ductility are taken into account. A review of some structural design guidelines established to achieve satisfactory earthquake performance, is presented. Finally, the same building is designed using a seismic design-method based on displacement control. The results show that ductile behavior can be easily achieved.

THE BUILDING

Elevations and a typical plant of the selected building are shown in Fig. 1. Differences in stiffness in two directions, X and Y, allow for a comparison of the structural behavior of two different structural systems. A large beam was added on top to diminish lateral displacements and shear stresses in coupling beams in the other stories.

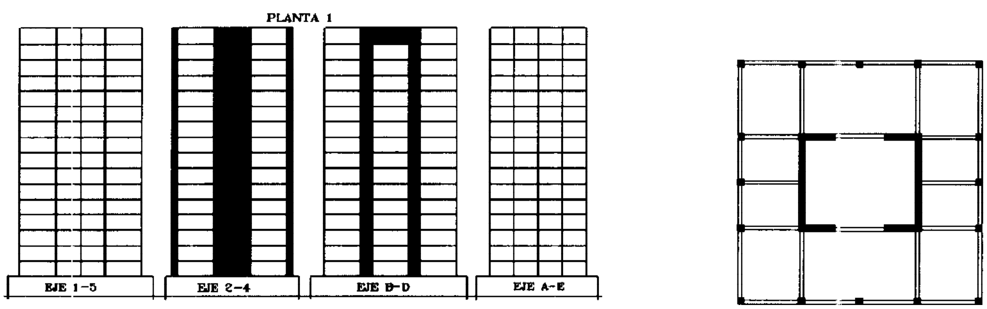

Fig. 1. Elevations and a typical plan.

Seismic design forces were calculated according to the old Chilean Code NCh433Of.72 for a soft soil (To= 0,9 secs). The main results of the analysis are shown in Table 1.

Table 1. Analysis with the old Chilean Code NCh433 Of.72.

Direction	Base Shear [Tons]	Base Shear/ Weight %	Period T [sec.]	Maximum interstory drift . % h*
X	534.5	8,6	0.881	0,083
Y	521.1	8,4	0.648	0,048

* **h** = a typical story height

The old Chilean Code was selected for analysis, because many buildings that withstood the 1985 earthquake were designed following this code. The recently approved code gives similar results in some areas for some specific types of soils. The methodology is practically the same in both codes. Both codes limit interstory drift calculated with reduced forces to 0.2%. The buildings used for the analysis, as well as many other Chilean buildings, largely satisfy the deformation. These test results did not detect any abnormality. Stresses determined from the analysis led to the final design. In this building, steel was determined following ACI318-89, revised 1992.

DEFORMATION CAPACITY

The deformation capacity of the structure can be assessed trough the use of an incremental analysis. Two lateral-force distributions in height, inverted triangular and uniform, were considered. Drain-2D, Version 1.10 Powell computer program was used in the analysis.
Since results are similar in the other direction, base-shear and lateral-displacements relations are shown in Fig. 2 only for direction Y. Differences and the damage sequences are shown in Table 2, which includes results for both directions of analysis.

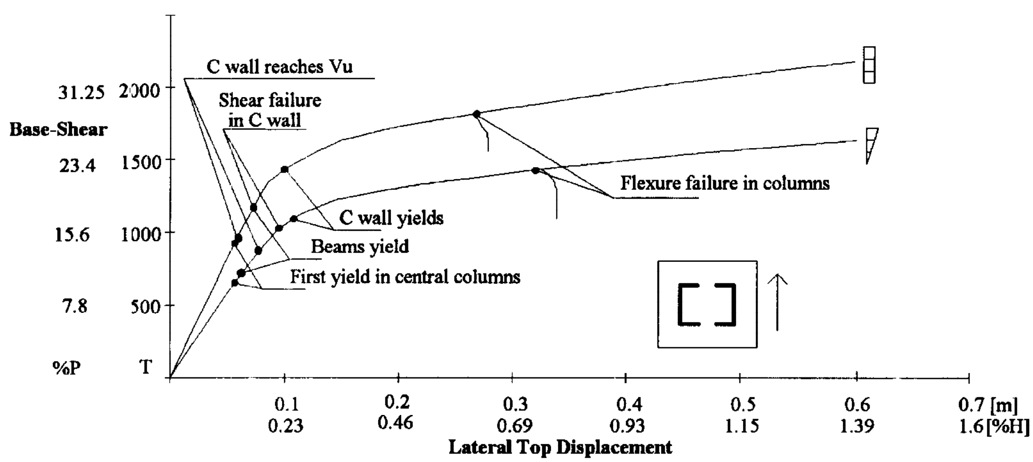

Fig. 2. Base-shear and lateral-displacements for direction Y for two lateral-force distributions.

Damage and Lateral Displacement

Figure 3 shows bending moment-curvature relations for walls in axes B and D. If unconfined reinforcement is used, brittle behavior can be observed. The types of failures associated with the top lateral-displacements are shown in Table 2. Note that if walls do not have an adequate reinforcement in shear, 0.2% drifts can be reached. In the X direction of the analysis, the displacement capacity was limited by wall flexural failure to 0.5% of the overall drift-ratio, due to the high level of compressive strains at the ends of the flanges of the C-cross section of the tensioned wall. If this type of failure is prevented by confining concrete in the compressive zone, lateral displacements in the order of 0.7% of overall drift-ratio can be reached; but beams fail in shear due to low shear reinforcement provided according to the Chilean Code. This type of failure could be easily avoided with additional stirrups that comply with ACI 318-89 ductility requirements (capacity design).

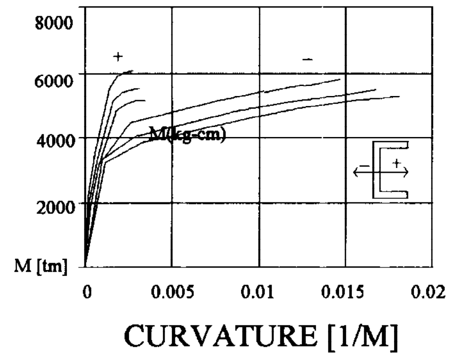

Fig. 3. Bending moment-curvature of wall in direction X.

If the transverse reinforcement in beams is increased according to the ACI 318-89 Code, the lateral displacement capacity will not be improved, because walls will fail in shear. If this shear failure is avoided by increasing wall thickness or shear reinforcement, then lateral displacements close to 1% of height can be reached, until the columns fail in tension. Shear failure was not detected for the Y-direction of analysis, but external tensioned columns failed at 0.7% of overall drift-ratio.

Table 2. Failure sequence and lateral displacement.

Direction	Lateral force-distribution		Description
	Uniform	Triangular	
	Lateral global drift %H		
	0.18	0.19	First yield in columns and coupling beams.
	0.18	0.21	Wall C reaches Vu NCh433 .
	0.19	0.25	Shear failure in walls
X	0.42	0.52	Flexural yielding at wall bases.
	0.66		Ultimate shortening strain in unconfined coupling beams.
	0.74		Flexural failure on tensioned column.
		0.97	Shear failure and flexural failure in coupling beams if concrete has been properly confined.
	0.14	0.14	First yield in columns and beams.
	0.15	0.18	Wall C reaches Vu NCh433 .
Y	0.16	0.21	Shear failure in walls
	0.23	0.26	Flexural yielding at wall C base and beams in facades.
	0.65	0.74	Flexural failure on tensioned column.

First yielding at wall bases occurred just at 0.52% drift. Confined concrete is required to reach drifts as large as 1%, conditions which are similar to the demands of any severe earthquake. The brittleness of the structure can not be detected directly from NCh433 Code results. Further analysis and improvements in the structural system are required to obtain a greater ductility. However, the C shape wall in direction Y has a great ductility capacity. Lateral displacements close to 1.5% of the building height could have been easily reached, but the building capacity-deformation in this direction was controlled by columns, which failed at drifts no larger than 0.74% of the building height. Applications of the capacity-design method are compulsory.

EARTHQUAKE DEMANDS.

Earthquake demands were determined through a nonlinear dynamic analysis using Drain-2D computer program. The March 3, 1985 Chile earthquake records for the cities of Vina del Mar and Llolleo, were chosen for analysis because these cities are the closest to the epicenter.

Global Results

Both records have similar deformation demands. Llolleo produces a greater number of nonlinear incursions than Vina del Mar, but the latter produces remnant lateral displacements.

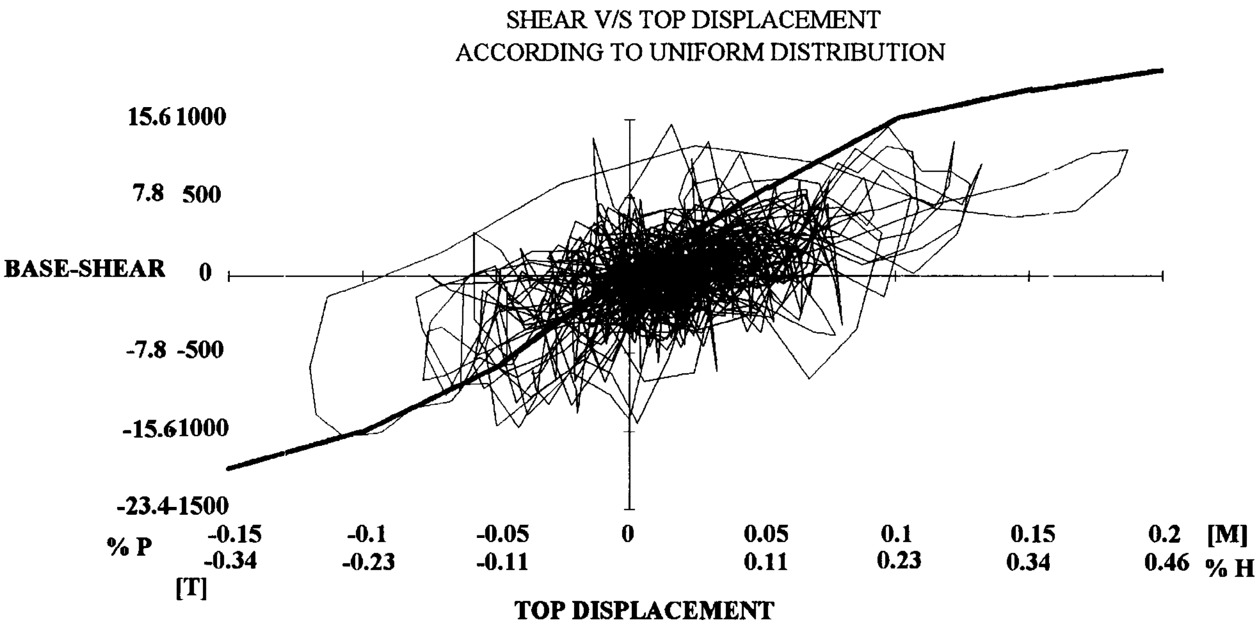

Fig. 4. Base-shear in direction X versus top lateral-displacements. Viña del Mar, S20W component- March 3, 1985 Chile earthquake.

Figure 4 compares base-shear calculated with incremental analysis and with a nonlinear analysis for the Viña del Mar record of the March 3, 1985 Chile earthquake. Fifty-four nonlinear incursions occurred according to the Viña del Mar record, a number which is 1.2 times the number of nonlinear incursions produced by the Llolleo record. Lateral-displacement demands of the 1985 Chilean Earthquake did not exceed the estimated building capacity. A large acceleration pulse observed in the Viña del Mar record (S20W), produced a maximum lateral-displacement as large as 18 cms (0.5%. of overall drift-ratio). The Llolleo record rendered similar, but lower results.

A SEISMIC DESIGN METHOD BASED ON THE CONTROL OF DISPLACEMENTS AND DEFORMATION CAPACITY

Many buildings that have survived earthquakes without collapsing, reveal that the global damage can be directly related to interstory displacements. The low level of damage caused to modern buildings by the March 3, 1985 earthquake, has been partly attributed to the high degree of stiffness of those buildings, due to the extensive use of walls as structural elements. Lateral displacements may be reduced by increasing the building's stiffness, which is achieved through an appropriate choice of the structural system. The structural system selection is the most important design decision. It requires close collaboration and mutual understanding between the contractor, the architect, and the engineer.

An assessment of the maximum lateral-displacement able to be sustained by a building during a severe earthquake, is a prerequisite for accepting a specific structural system. The structure must be able to deform beyond this estimated value, since there are many difficulties in characterizing an earthquake in a reliable way. Some relations between the maximum expected lateral-displacement, as a function of the initial fundamental period of the building, and the maximum strength to lateral forces, have been proposed by Shimizaki, Miranda, Qi and Moehle, and Giuliano and Amado. The maximum top lateral-displacement of the building analysed in this paper was taken equal to 1.3 times NCh433Of.93 the elastic-displacements spectra calculated for a 5% damping ratio shown in Fig. 5 (Decanini et al, 1993). An effective period based on cracked-section flexural properties must be considered; a simple appropriate approximation is T = 1.41*T. Table 3 shows calculated values for the building studied in this paper.

Table 3. Estimated lateral displacements from linear displacement spectra.

Direction	T	√2 * T	1.3*D Viña	1.3*D Llolleo	1.3*D NCh433
X	0.881	1.25	22.1	17.6	30.42
Y	0.648	0.92	22.1	14.3	25.09

T= Fundamental period of uncracked structure.[sec.]
√2 T= Effective period of the cracked structure.[sec.]
D.= Elastic-displacement spectrum for the effective período √2 * T.[cm.]

Note the similarities between the values proposed by the Chilean Code and those calculated using two March 3, 1985 Chilean earthquake records.

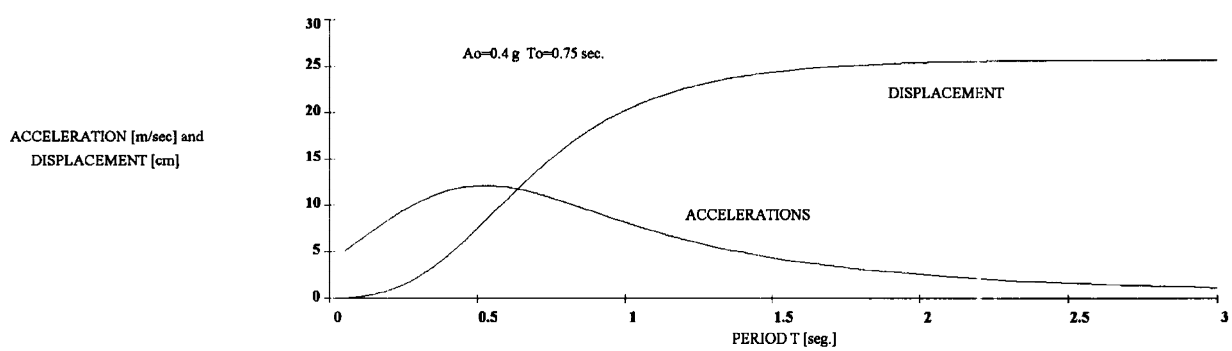

Fig. 5. Pseudo-acceleration and displacement linear spectra from NCh433Of.93.

Interstory drifts, measured as the ratio between the relative displacement and the story height, can be derived from the global distortion Dr, which is defined as the ratio between the top displacement and the building height. An approximate factor α, equal to 1.5, can be used for structures with walls, and 1.25 for framed buildings. The relationship can be obtained from a linear analysis. The linear dynamics analysis using the old Chilean Code NCh433Of.72 gives 1.31 and 1.25 for directions X and Y, respectively. Expected drift values would be 0.92 and 0.726 for directions X and Y, respectively.

Note the low drift-values obtained for the structures studied under Chilean earthquakes. These values are similar to the estimated values for actual Chilean buildings that behaved very well during the 1985 Chile earthquake. The determination of a limit to the calculated drift-value should serve as criteria for the acceptance or rejection of buildings. The control of the drift of a structural system under earthquake excitation is important to maintain architectural integrity, avoid damage to nonstructural components, limit structural damage, prevent structural instability, and ease human discomfort under frequent, minor or even occasional, moderate earthquake shaking. The limits depend on the acceptable amounts of damage. Chilean experience demonstrates that values as low as 0.5% easily can be reached by using the adequate amount of walls. Values close to 1% appear sufficient for design purposes. The present building, therefore, would have been satisfactory. If not, the stiffness should have been increased, thereby introducing changes to the structural system.

Deformation Capacity

If very large displacements occur, the structure must have a collapse mechanism that displays ductile behavior. The seismic demands should be less than those associated with the incipient collapse mechanism, but since is not possible to guarantee an upper bound to seismic behavior, it seems reasonable to evaluate the structure behavior at the ultimate point in which the collapse mechanism is activated. A suitable mechanism then is chosen, which defines critical sections where energy should dissipate. Figure 6 shows the selected collapse mechanism. Inelastic deformations should be concentrated at the base of walls and at the ends of the coupling beams. It is assumed that the walls rotate around the compressed ends of their bases. The plastic rotation capacity θpb of beams is:

$$\theta_{pb} = \left(1 + l_w / l_b\right) \frac{\Delta_p}{nh}$$

Where n is the number of stories, *lb* is the beam length, *lw* is the total depth of the wall section, h is the height of each story, and Δ_p is the top displacement..

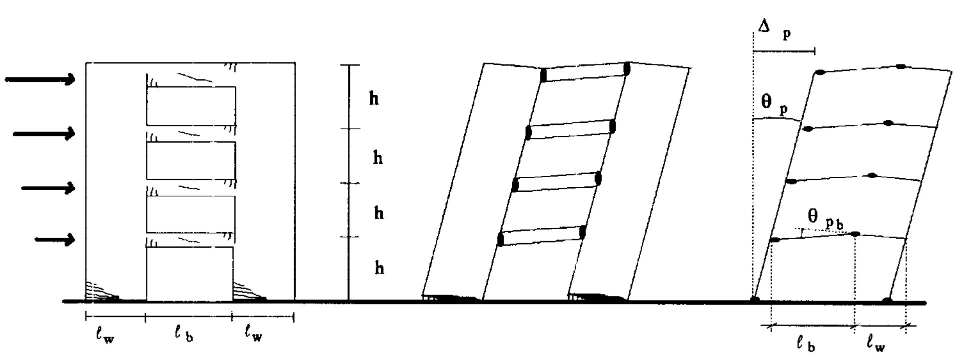

Fig. 6 Collapse mechanism for coupled walls.

Table 4 shows required plastic rotations in beams for the development of the estimated maximum top displacement Δmax.

Table 4 Maximum plastic rotations upheld by beams to sustain Δmax.

Direction	Beam	$l_{w\,[m]}$	$l_{b\,[m]}$	$\Delta_{p[cm]}$	θ_{pb}
X	interior	3.0	4.0	30.42	0.0123
X	extenal	3.0	5.0	30.42	0.0113
Y	external	8.0	6.0	25.09	0.0136

The required ultimate curvature must be enough to develop such rotations. Details of the section must be known at this stage. Moment-curvatures relationships obtained for different amounts of steel can help to select the final strength to be used. Alternatively, the required flexural strength could be determined from the applicable code.

A similar procedure should be applied to walls and other elements to determine the final design (Paulay, 1986), (Moehle, 1993).

CONCLUSIONS

The current code design procedures are not able to detect the potential weakness of this building. The weakness of the building was directly detected by using the displacement design approach, which compares displacement capacity and demand.

Low demands obtained for the two specific records used in these study, could lead to the acceptance of brittle designs as shown in this paper. Warnings must be established, given the probabilistic nature of the phenomena. Greater demands could easily be obtained in the future. In any case, a ductile structure is desirable.

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