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INFLUENCE OF VERTICAL IRREGULARITIES IN THE RESPONSE OF EARTHQUAKE RESISTANT STRUCTURES

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ABSTRACT

The nonlinear response of reinforced concrete buildings with irregularities in elevation is studied. Two cases of irregularities have been considered: i) Walls interrupted in height, and ii) Buildings having a soft first story. The buildings were first designed using the Chilean code NCh 433. of 72 for earthquake resistant buildings, and the ACI 318-89 code, applying a linear-elastic dynamic analysis by spectral mode superposition. The strength and displacement capacity of those buildings were obtained by a nonlinear static incremental analysis; the earthquake demand was obtained from a time response dynamic analysis using acceleration record from the March 3, 1983 Chile earthquake. The results showed that irregular buildings not having walls in some stories have a brittle mode of failure and should be avoided.

KEYWORDS

Irregular buildings; earthquake design; nonlinear response; soft story; stiffness irregularity.

INTRODUCTION

Symmetry and regularity are usually recommended for a sound design of earthquake-resistant structures; however, in many cases, these two requirements cannot be met. Vertical irregularities in buildings are imposed by city regulations, and the structural designers have to assess their effects on the earthquake response in addition to code prescriptions.

Observations of structural damage due to strong earthquakes show that the class of buildings with irregularities in height systematically have had an unsatisfactory behavior, even though they comply with code regulations. This fact makes a deeper study on this subject essential, especially because the literature on irregular buildings is rather scarce and some isolated contributions can be cited.

Dolce and Simonini (1986) study the influence of stiffness variation in elevation upon the dynamic response of buildings, in order to evaluate the static methods of analysis. They conclude that the code requirements for the static analysis are highly restrictive. Jain and Sharma (1988) compute the dynamic response of a building with a long and narrow first story base and a slender tower. In their analysis, they assume a flexible slab on the first floor, finding that this flexibility has a big influence on the building response. Aranda (1984) uses Takeda's hysteresis curves to determine the inelastic response of R/C frames to vertical irregularities, thus reaching values of ductility demands. It was found that irregularities may increase the ductility demand by a factor close to 2. Similar results where found by Costa et al. (1988). The failure suffered by the columns of the Imperial County Services building during the 1979 Imperial Valley earthquake was studied by Kreger and Sozen (1983). The six-story R/C building had a wall sustained by two columns on the first floor, and another wall along a parallel axis to take the shear. A reduced - scale model was tested as part of this study. The conclusion was that the columns failed under a combination of axial load and moment, due to the biaxial flexure produced by the two orthogonal components of the earthquake (N-S and E-W). Finally, it was noted that current design codes do not allow for the design of structures such as those described in the study.

Currently the codes typify the irregularities and recommend appropriate methods of linear analysis. Certain quantitative criteria are mentioned in UBC (1982) for detecting irregularities in elevation, and thus force the designer to engage in a dynamic analysis. These criteria are based on the definition of lateral stiffness, mass, geometry and strength values in order to identify the irregularities.

In this work two types of vertical irregularities in height in R/C buildings are studied; namely, i) building 1: walls interrupted in height, and ii) building 2: soft story building.

CHARACTERISTICS OF BUILDINGS

Figures 1 and 2 show the buildings under study. Both are R/C, ten-story, symmetrical buildings. The symmetry is imposed to avoid torsional effects and enhance the irregularities in elevation. Building 1 has walls in axes B and C interrupted at the 5th floor, defining an irregularity of stiffness. Building 2 has a soft first story made of only frames, while the other stories have walls.

Both buildings were designed using the linear dynamic method of analysis of Chilean code NCh 433. of 72 (1972) and the ACI 318-89 (1989) code.

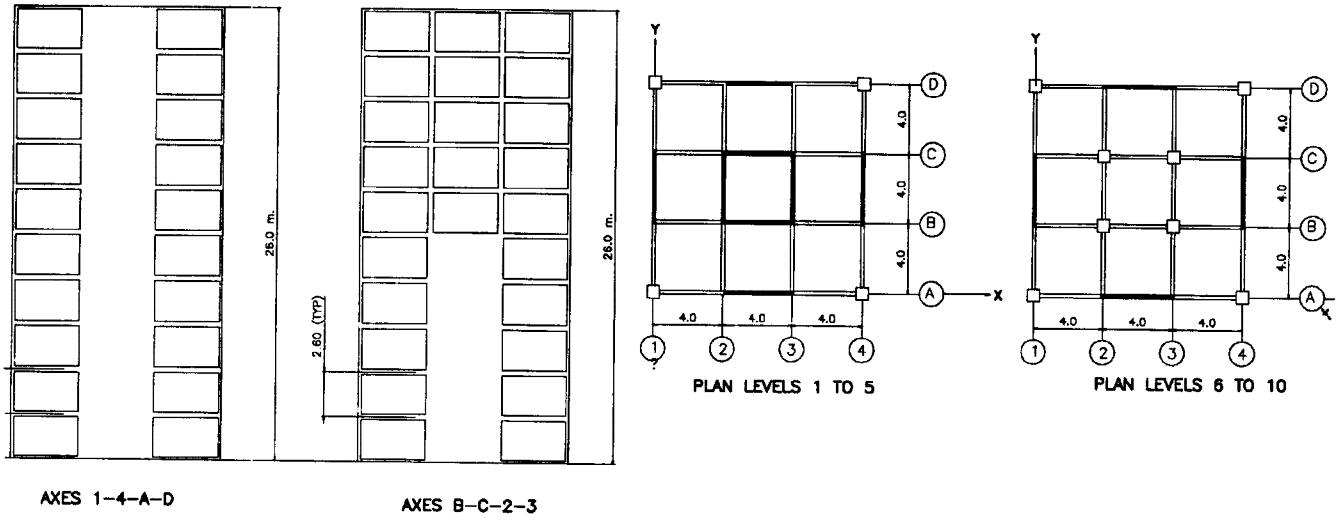
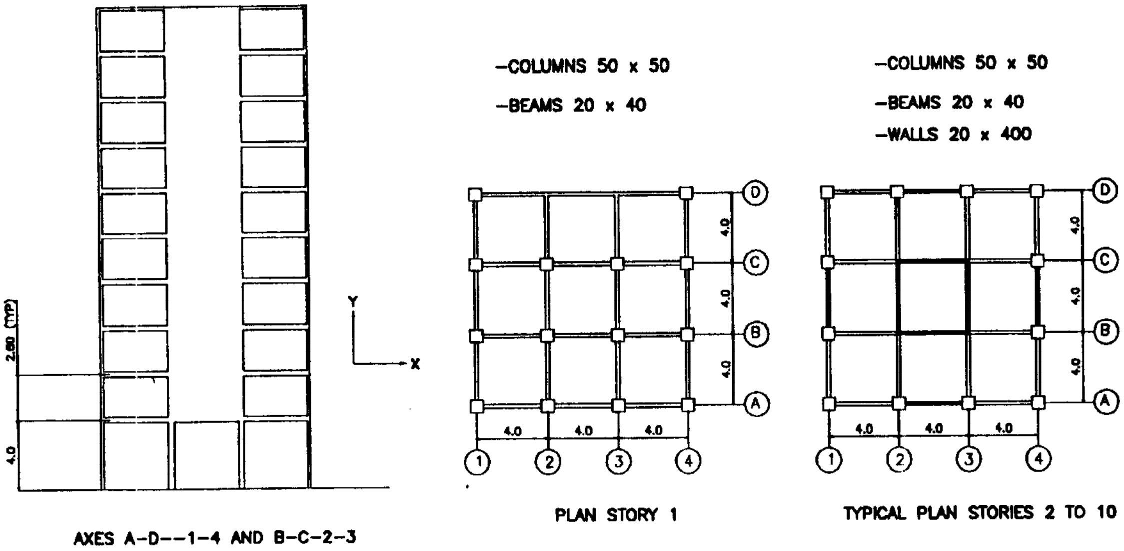


Fig. 1. Building 1: Ten story building with walls interrupted in height.



AXES A-D--1-4 AND B-C-2-3

PLAN STORY 1

TYPICAL PLAN STORIES 2 TO 10

Fig. 2. Building 2: building with a soft first story.

LINEAR ELASTIC RESPONSE

Figures 3 and 4 depict the force and displacement linear elastic responses of the two buildings under study. Fig. 3 shows the amplification of shear and bending moment at level 5 in walls A and D, which span all the building height. On the 5th floor, where the singularity occurs, there is asignificant transfer of shear forces between the tall and low walls. This transfer is performed through the slab, which must be properly designed. In building 2, on the other hand, there are no sudden changes of bending moment or shear at the interface between stories 1 and 2 since there is no transfer of forces among the different vertical substructures due to the fact that all of then vary in the same way (see Fig. 4). However, due to the big change of stiffness between the soft first story and the others, the interstory drift between the first and second stories is very high in comparison with the other stories' drift.

STRENGTH AND DEFORMATION CAPACITY

Both the estimation of the demands from an earthquake and the evaluation of the capacity of the building to deform to the point of rupture were made using the program Drain - 2DX (Osteraas, 1987). A 5% damping ratio with respect to the critical was used for the calculation of the nonlinear response in the dynamic case.

The application of the program Drain - 2DX to an incremental nonlinear static analysis led to Figs 5 and 6, which show the curves for base shear versus top displacement for both buildings. The curves are referred to two different lateral force distributions: uniform and triangular.

The collapse mechanism shown in Fig. 5 for building 1 corresponds to $Qb/W = 48\%$ and $\delta/H = 0.85\%$ for uniform forces and $Qb/W = 38\%$ and $\delta/H = 0.98\%$, for triangular forces, where Qb is the base shear, W the total building weight, δ the top lateral displacement, and H the total building height. This strength can be improved by adequate wall confinement.

Figure 6 shows the results of the incremental analysis done for the soft story building. Failure occurs with the fracture of the steel confinement ties in the compressed columns of the first story. This failure is due mainly because the interstory drift is highly concentrated between the first and second stories. The dotted line in Fig. 6 indicates the behavior the structure would have presented if the columns had allowed it

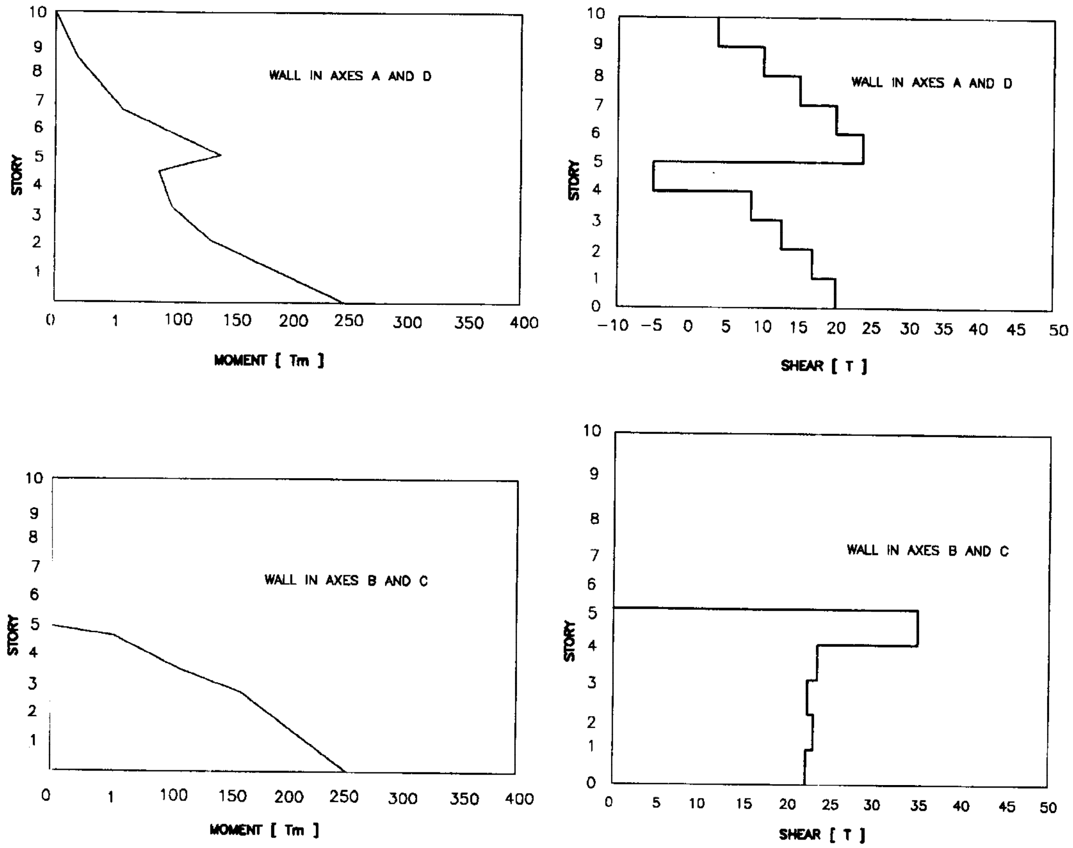


Fig. 3. Shear and overturning moment in different walls of building 1. Linear elastic dynamic analysis.

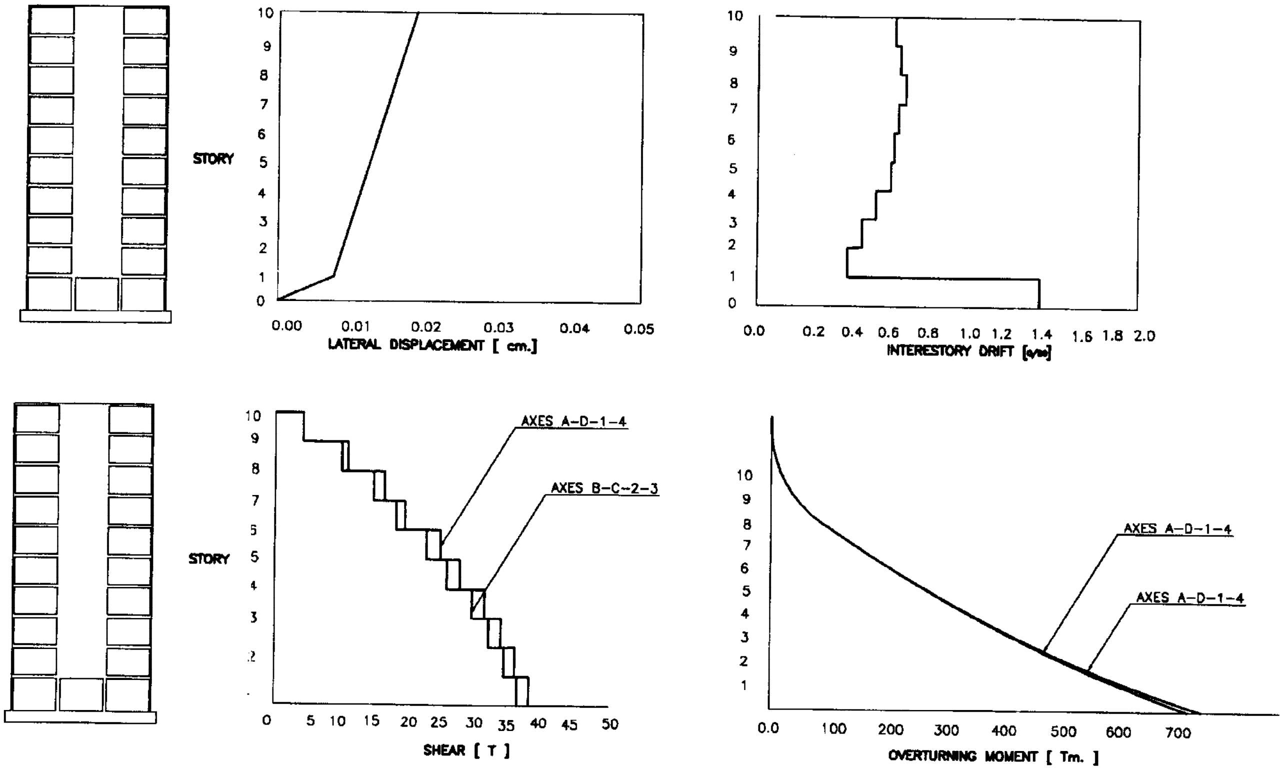


Fig. 4: Shear and overturning moment in building 2. Linear elastic dynamic analysis.

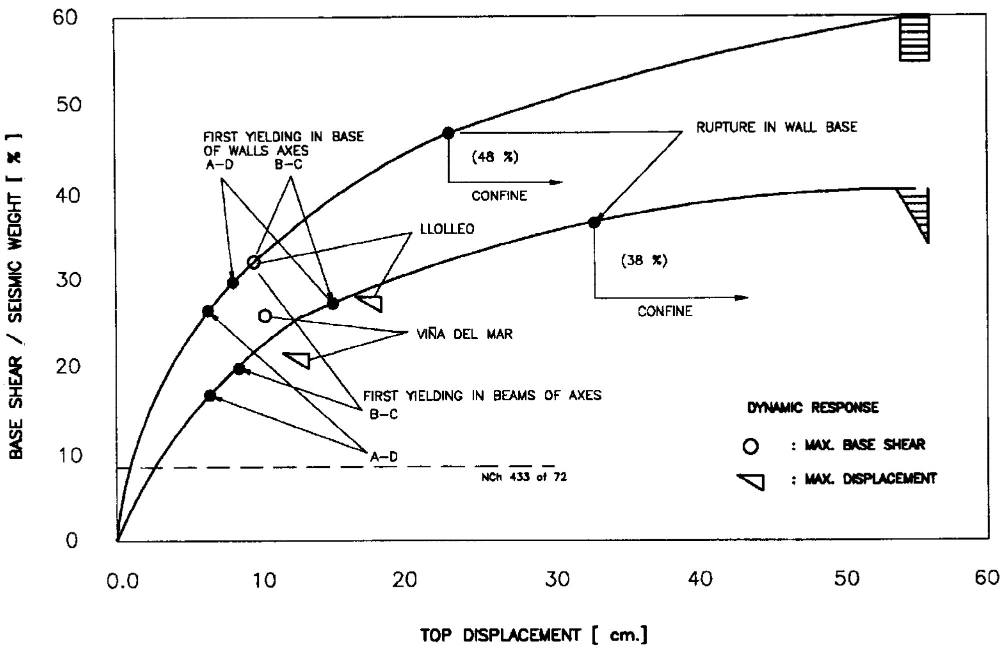


Fig. 5. Base shear vs top displacement; building 1.

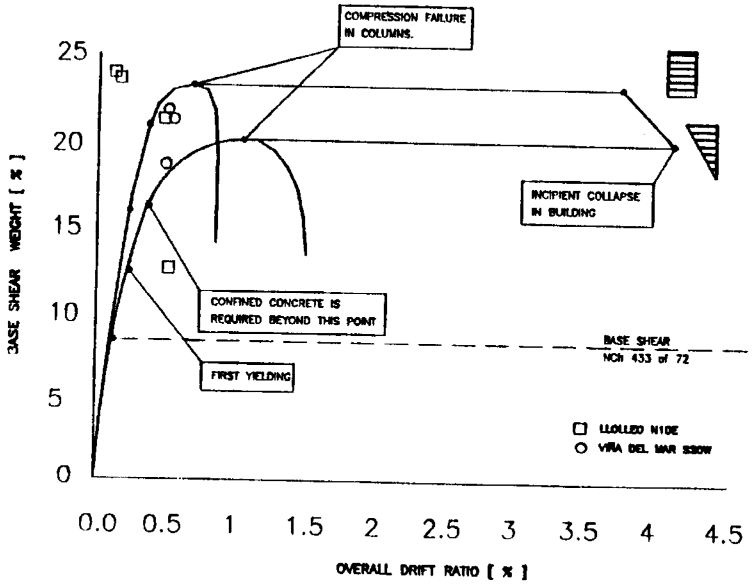


Fig. 6. Base shear vs top displacement; building 2.

without failing in compression (a greater resistance to axial stresses is required). In fact, the flexural mechanism could have occurred at a lateral displacement close to 4,5% of the building height, had the brittle failure in compression not previously occurred. The prevention of this response is not possible without changing the structural system.

At failure, the base shear reached a value equal of 24% of the structure's weight. The first elements to yield were the columns located under the walls. From the curve shown in Fig. 6, it may be inferred that the low lateral displacement capacity is due to the concentration of the damage in the columns. If the bending collapse were permitted, plastic hinges would form at the bases of the columns at the first and second floors. In addition, the beams on the upper floors would suffer significant damage.

From comparison between Figs. 5 and 6 it may be observed that the calculated displacement capacity is considerable less in the building with a soft first storey. This is due because building 1, although is irregular in height, it has some uniformly continuous walls all along the elevation.

DEMAND FROM REAL EARTHQUAKES

Once the deformation capacity of the structure is known, a good estimate for the demand of the expected earthquake is necessary. Earthquake demands were determined in this paper by means of a nonlinear dynamic analysis for some records available from the 1985 Chile earthquake; namely, those from Viña del Mar and Llolleo, which correspond to points close to the epicenter and located in cities with a large number of buildings. A time response was obtained for both records.
The maximum values for the earthquake response in displacements and shears calculated from dynamic analysis were included in Figs. 5 and 6 over the curves of the incremental analysis. In general, maximum displacements did not occur simultaneously with maximum base-shears for the two records used, but it was observed that the incremental analysis gives a good bound to the maximum shear.

Inspection of Figs. 5 and 6 show that building 1 would have enough strength to resist both earthquake inputs, whereas building 2 would have failed in a brittle way without adequate column confinement, and even if they had been properly confined, earthquake demands would have been close to the building failure.

CONCLUSIONS

Building 1: some walls interrupted in height

a) The distributions of shear and overturning moment in the vertical substructures present great variations in the zones of irregularities. Significant transfer of forces between the vertical substructures are produced there making necessary a careful design and detailing of the horizontal diaphragms (slabs) and walls in that part of the structure.

b) The earthquake response of this irregular building was good because some walls spanned all the height. Thus interstory drift may be controlled and so the damage; on the other hand, ductility demands kept within an acceptable range of values.

Building 2: soft first storey

c) Large values of interstory drift are concentrated between the soft first story and second story.

d) In this kind of building it should be remarked that there is not a desirable collapse mechanism. In effect, it shows a brittle failure mode characterized by the failure of the lateral reinforcement in hooped columns under compression. Due to the high level of axial load, longitudinal steel may also fail in tension. If these failure modes are prevented by increasing the lateral or longitudinal steel, a flexural ductile mode of failure could be reached but it would demand large deformations, impossible to be accepted in real structures. The foregoing features of failure modes in soft story buildings lead to a rejection of them unless adequate changes in the structural pattern may be done, as is the case when some structural walls are extended to the first story, thus providing enough stiffness and strength to that soft and weak story.

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