Behaviour of an aseismic structure during Caucete earthquake of November 23, 1977

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S U M M A R Y

After the Caucete earthquake, of intensity 9, on the modified Mercalli scale, only minor damage is seen in Cristo Rey Church's reinforced concrete structure. Dynamic modal analysis using recorded spectrum gives a ductility ratio of 2.6, while structural features allow a much larger ductility.

1. INTRODUCTION

The district of Caucete, Province of San Juan, Argentina, was affected on November 23, 1977, by an earthquake of intensity 9, on the modified Mercalli scale.

Information about this earthquake, including description of damages and records, is available from Reference (1). Spectrum calculated by IDIA (Instituto de Investigaciones Antisísmicas "Ing. Aldo Bruschi" of the Universidad Nacional de San Juan) since their accelerograph record is given in said Reference.

Behaviour of the Cristo Rey Church's reinforced concrete structure is reported through this communication. This church is one of the Caucete buildings which withstood that earthquake with only minor damage.

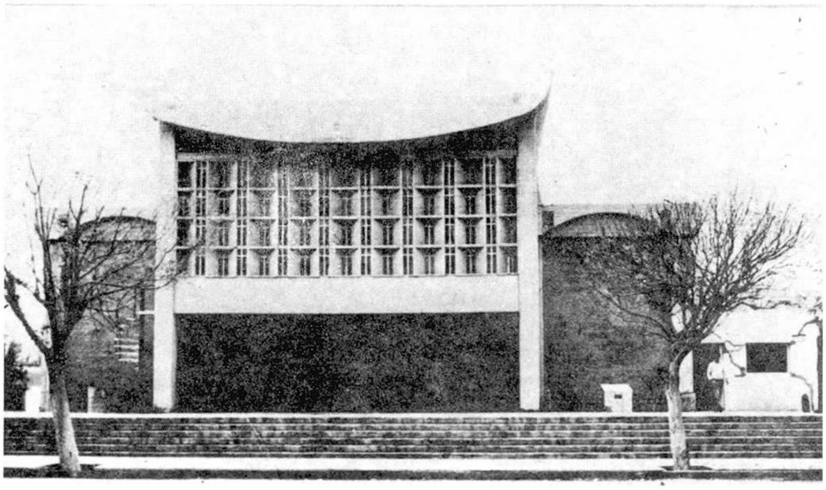


Fig. 1 - Front of the Church

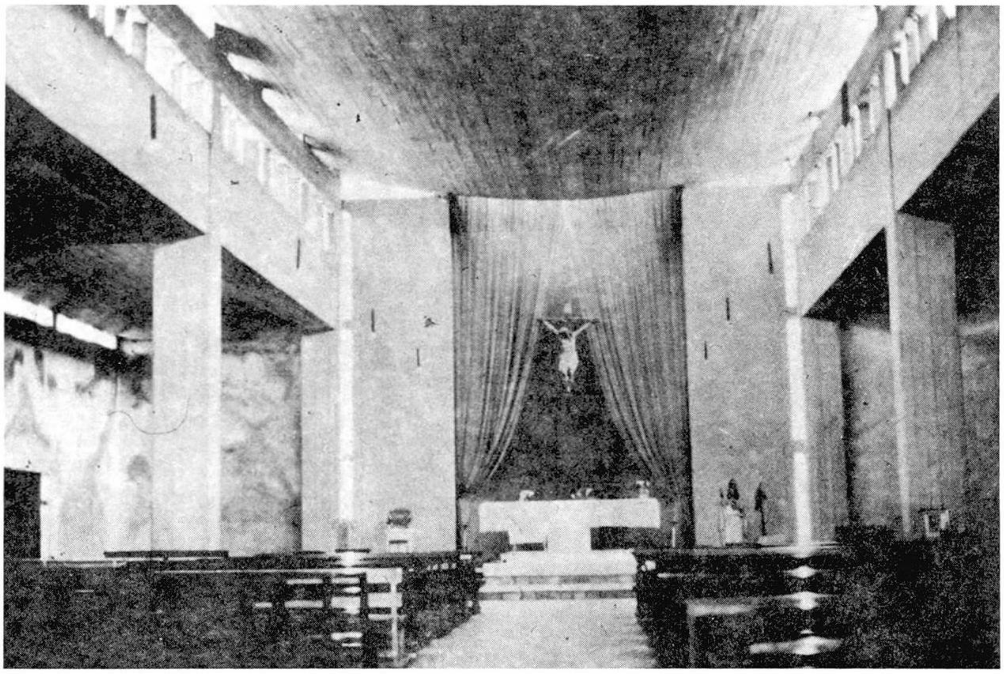


Fig. 2 - Central Nave

To investigate this behaviour we used the spectrum shown on fig. 3, which was obtained from the spectrum calculated by IDIA, by multiplying its ordinates by the ratio between the records of two seismoscopes, one located in Caucete town, about 200 m distant from the structure, and the other beside the IDIA accelerograph in the city of San Juan,

$$\frac{R_{\text{seism. Caucete}}}{R_{\text{seism. San Juan}}} = \frac{0.52}{0.26} = 2$$

These spectra are calculated for a damping ratio of 5% of critical damping. This figure is assumed to occur in reinforced concrete structures reaching yield, according to records from San Fernando earthquake of 1971.

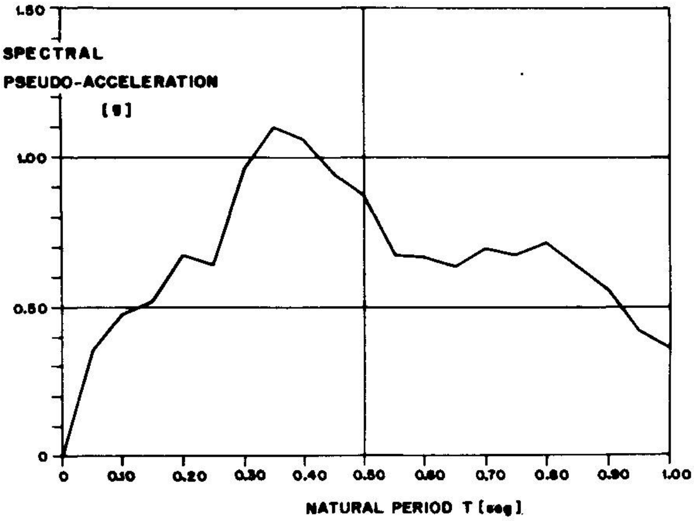


Fig. 3 - Pseudo-accelerations spectrum of Caucete 23.11.77 earthquake

2. DESCRIPTION OF THE BUILDING

The parish church of Cristo Rey has a rectangular plan of approximately 20 m by 35 m, with a nave of 10,30 span and approx. 10 m height, covered by a concave traction shell, while each aisle is covered by a cilindrical, flattened vault.

In the original design, the nave roof was a shell of a different convex shape. Structural design was performed by the writer in 1958. Later, for aesthetical reasons, the roof design was modified, without changing the other structural features. Church was built in 1965.

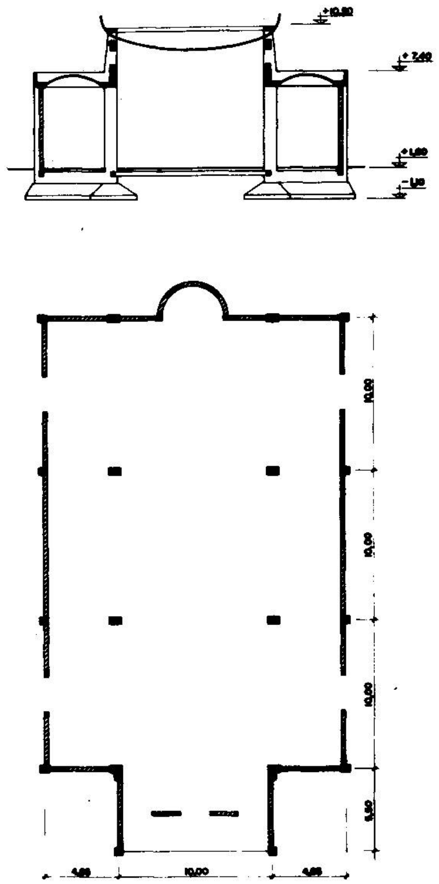
The main structure is formed by four transverse and four longitudinal frames. Perimetral frames are stiffened by brick masonry walls, while the inner ones are free of elements which could affect their structural behaviour.

Building examination after the earthquake reveals no damage in the reinforced concrete elements. The front brick walls show limited cracks, while there are no apparent cracks in lateral walls, but inner plaster spalled in wide areas. Some glass breakage in windows occurred (approx. 10 %), and light damages in masonry can be seen at both sides of apse, where roof shell touches the wall.

In the original conception of the structure, earthquake-resistant elements in the transverse direction were formed by two simple frames in each line, joined together only by the roof whose stiffness in the plane of frames was negligible. These frames were designed in accordance with the Seismic Regulations in force as of 1958, using a seismic coefficient of 0.15; and an increase of 33 % for allowable stresses. Main columns, 50 cm by 80 cm in cross section, were reinforced with a total area of steel of 1.2 % of concrete area; shear reinforcing in provided by stirrups and bent bars in both directions.

Yield stress of steel is 2200 kg/cm2; concrete has a characteristic ultimate stress of 222 kg/cm2, deduced from sclerometer measurements after the earthquake.

The two columns of each frame have a common footing, designed for a maximum pressure on soil of 1.1 kg/cm2. The large stiffness of this foundation ensures a practically perfect fixed end for both columns. Due to the change of roof design these frames were joined by an element of certain stiffness, formed by the shell plus the top compression beams.



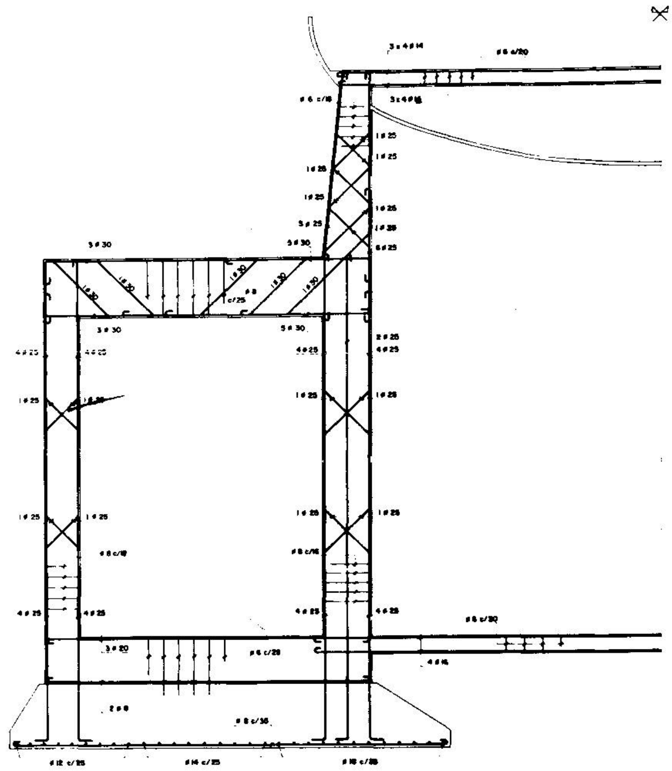


Fig. 5 - Reinforcing of transverse frame

Fig. 4 - Plan and elevation of the building

3. ANALYSIS OF BEHAVIOUR

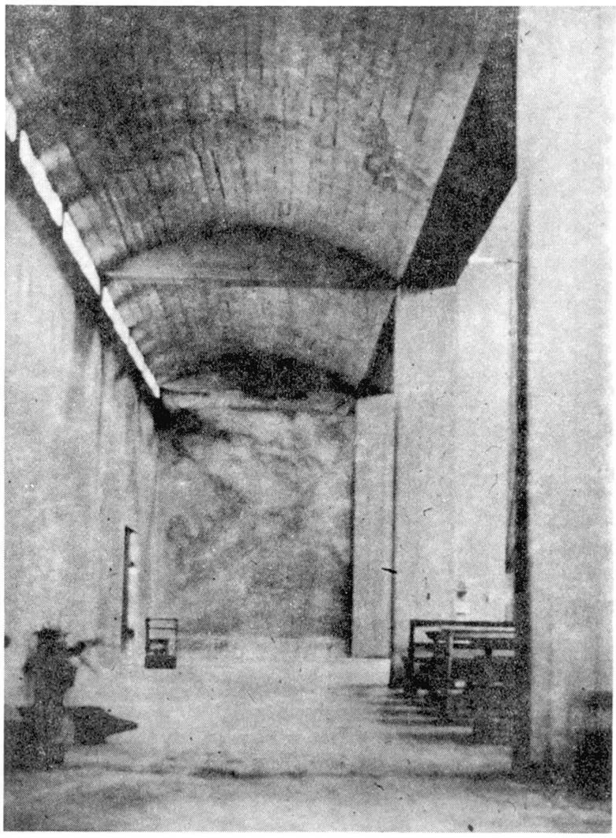
Analyzing the structure as built, the following is observed:

a. Under horizontal static loads the complete frame behaviour, due to slenderness of top beams, is almost the same as originally foreseen. Highest stresses appear at the foot of the inner columns. In this section reinforcing yield would be reached under horizontal loads which corresponds to a seismic coefficient of 0.33. The structure horizontal displacement, under these horizontal loads, would be 1.5 cm.

b. Natural period of structure first mode is 0.43 sec.

c. Dynamic analysis of the structure, assumed fully elastic and subject to spectrum of pseudoaccelerations of fig. 4, gives the following results:

- Maximum displacement of highest points is 3.9 cm.
- Maximum horizontal acceleration of the same point is 0.89 g.
- Bending moments at joints, of columns and beams, are much larger than ultimate moments of that sections.



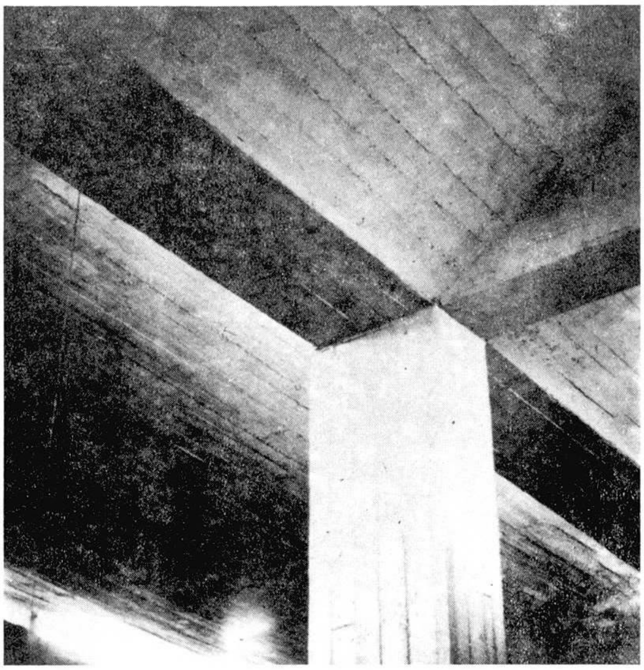


Fig. 7 - Detail of main frame joint

Fig. 6 - Lateral aisle

d. Results of dynamic analysis for elastic structure, inconsistent with its strength and with absence of damages, obviously show that actual behaviour of structure during the earthquake exceeded the elactic range. Assuming that, nevertheless, the actual displacement reached a maximum value very near to the one given by the elastic dynamic analysis, it is possible to deduce that structure has developed a ductility whose value is the ratio between this displacement and the displacement corresponding to the beginning of yield in the most stressed section of the structure:

$$\mu = \frac{3.9 \text{ cm}}{1.5 \text{ cm}} = 2.6$$

4. CONCLUSIONS

This analysis, even being very simplified, allows the following conclusions:

1) Ductility developed by the structure during this earthquake, not very high, means that the structure remained still far from its ultimate resistant capability.

2) A more detailed analysis, taking into account masonry walls as earthquake resistant elements, would give an even smaller value of ductility ratio.

3) Quantities and disposition of steel reinforcing, specially of those for shear, indicate that structure should develop a ductility considerable larger than the value obtained in this study.

4) Previous considerations justify in a satisfactory way the absence of damages in the structure and limited damages observed in masonry walls.

REFERENCE

(1) - J.S.Carmona - I.D.I.A. - El sismo de Caucete, San Juan, Argentina, del 23. XI.77 y la seguridad que proveen las normas sismorresistentes. Seminario Internacional de preparación para atención de Catástrofes - Viña del Mar, Chile, Marzo 1978.

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