SEISMIC STUDIES OF PARQUE CENTRAL BUILDINGS

by

M. Paparoni [I], J. Ferry Borges [II], A. Ravara [III] and Robert V. Whitman [IV]

Synopsis

Comprehensive seismic studies were made in connexion with the design of a group of 13 apartment and office buildings, 44 and 62 stories high, included in Parque Central project in Caracas, Venezuela. Not only were many different aspects of the expected seismic response studied in detail, but in addition these studies were closely co-ordinated with the progress of planning and design.

The subjects were: intensity of ground motion for design; relation of earthquake response to foundation design; dynamic soil-structure interaction; linear and non-linear analysis of the seismic response for translational and torsional vibrations; elastic model tests; tests of reinforced concrete models for ductility assessment; and the observation of structural behaviour during construction.

This paper describes the methods that were used during the different stages of the structural design regarding the planning and the coordination of the research involved, gives a brief account of the major results obtained in the analytical and experimental studies, and outlines the progress of design as a consequence of these results.

1 - INTRODUCTION

The Parque Central Development in Caracas, Venezuela, comprises an occupation area of approximately 130, 000 m2 encompassing a total of 11 residential buildings (44 stories), 2 office towers (62 stories) and several. other minor buildings for a grand total of 1, 200, 000 m2 of enclosed space.

The development is now completing its first phase, totalling 4 finished apartment buildings, and two under construction (Fig. 1). The present paper deals with the studies connected with the apartment buildings, the ones relating to the office towers being now under way.

I - DELPRE, C.A. Ingenieros Constructores, Caracas.

II - Associate Director, Laboratório Nacional de Engenharia Civil, (LNEC), Lisbon.

III - Head of the Applied Dynamics and Applied Mathematics Divisions, LNEC.

IV - Professor of Civil Engineering and Head of the Structures Division of the Dept. of Civil Engineering, M.I.T., Cambridge, Mass.

What makes these studies particularly interesting from Earthquake Engineering point of view are the following points:

i) The project was started not so long after the city had suffered a severe earthquake (29th July 1967), with local codes still under discussion.

ii) The construction methods selected were developed in non-seismic countries, mostly in France, and there was no previously known experience in the full shear-wall type of construction either in the heights reached or in the lateral load requirements for design.

iii) There was no important previous local experience or trained personnel in this type of industrialized construction.

A multidisciplinary local group was assembled in a brief time and an international group of advisors was formed, once the initial ideas proposed by the developer, Delpre, C. A., a private firm, were accepted by Centro Simon Bolivar, the government institution in charge, who is at the same time the land owner and the body in charge for city renewal. This paper tries to retrace the main experiences gathered during the project stage, most of them resulting from interactions between geographically separated groups.

In terms of relative influence on the final solution, one could establish a scale of relevance by listing the main decisions which conformed the finally chosen structural solution as follows:

i) Rejection of systems based on prefabricated panels in favour of cast-in-place type of construction, using a side sliding tunnel formwork.

ii) Comercial use for the lower stories and office use for the intermediate stories, instead of totally separated types of buildings.

iii) Influence of the type of soils on the intensity of ground motion.

iv) Assessment of ductility factors.

v) Type of foundation.

2 - GENERAL DESCRIPTION OF THE STRUCTURE

The structure of the apartment buildings consists in transverse shear walls spaced 5.70 m and two longitudinal shear walls along the central corridor as shown schematically in Fig. 2. The building height is about 125 m. Floors are 16 cm thick reinforced concrete slabs.

The total permanent load considered in the dynamic analysis ammounts to about 66, 000 t, including 25% of the live load according to the Venezuelan seismic code.

At the lower six stories longitudinal shear walls are replaced by frames and large openings in the transverse walls increase the overall ductility of the structure.

The buildings are directly founded on the ground by means of a ribbed mat (Fig. 2).

Concrete with a characteristic compressive strength 3500 N/cm2 (5000 psi) and reinforcing steel with a characteristic 0.2 proof-stress 42000 N/cm2 (60000 psi) were normally used.

3 - SOIL CONDITIONS AND FOUNDATION CHOICE

The Valley of Caracas is underlain by intermingled cohesive and cohesionless soils laid down both in river flood plains and in alluvial fans emanating from the steep mountains bounding the Valley. The pattern of these deposits is erratic; at any depth at any site either clay or sand is equally likely. The depth of these soils also varies greatly over the Valley, with a maximum depth of over 300 m. At Parque Central, the depth to bedrock ranges between 40 and 100 m.

The soils within the top 10 m generally are too loose or too soft to support heavy loads directly at ground surface, and most tall buildings in Caracas are supported upon piles with a length of 10 to 20 m. However, bellow 10 m the soils become denser and firmer, suggesting the possibility that the Parque Central apartment buildings could be supported by rigid mats founded at that depth. Such a foundation had another potentially important advantage in connection with this unusual building: the action of a mat foundation during earthquakes is better understood than is the action of a pile foundation.

Detailed studies were made to determine the feasibility of a mat foundation. Analysis of static settlements and of resistance to overturning indicated that a mat would be adequate and safe if it had a width of about 30 m with a static bearing stress of about 35 N/cm^2. Soil exploration showed that the site could be dewatered economically by means of deep wells, and that there was no danger that dewatering would cause damage to nearby buildings. Thus a mat foundation placed in a deep excavation was feasible.

At the same time, other factors also indicated the desirability of deep excavation and of widening the lower part of the structure. Excavation of the entire site provided desirable underground parking space. Widening the lower part of the structure was desirable from the structural viewpoint and provided useful commercial and office space. Hence, the final solution was a mat 32 m wide founded 12 m below ground surface.

4 - INTENSITY OF GROUND MOTION FOR DESIGN

At an early stage of the project three types of seismic spectra were independently proposed by consulting engineers Abenante and Brewer, by Whitman and by the LNEC. All spectra were related to the seismic intensity of the 1967 Caracas earthquake, their main characteristics being as follows:

Abenante and Brewer (1) assumed a seismic intensity similar to the 1967 earthquake and determined a response spectrum for soft soils using Esteva and Rosenblueth propagation expression for a Richter magnitude 7, an epicentral distance of 50 km, and a focus depth of 20 km.

Whitman's response spectrum was based on a seismic intensity of about twice the 1967 earthquake (2) and on a predominant period of

1 s for the soil.

LNEC suggested an acceleration power spectrum with a constant density of 350 $gal^2\ Hz^{-1}$ in the frequency range of 0 to 5 Hz. The maximum intensity assessed to the zone of Palos Grandes during 1967 earthquake was little above this value (3).

For the comparison of the three types of ground motion it was necessary to derive relationships between power and response spectra of acceleration. This was done at the LNEC (4) by means of a formulation and a computer program that plots acceleration response spectra produced by any type of power spectra of acceleration. Fig. 3 presents the three response spectra. It is seen from the figure that Whitman's spectrum exceeds the other two for periods near 1 s, since this was the value estimated by that author for the fundamental period of the soil at Parque Central. The incidence of this value on the seismic design of the structure is critical because it is about the same as the fundamental period of the building in the longitudinal direction. It was thus decided to perform measurements of soil vibrations in several zones of Caracas, in order to relate the predominant period of the soil and the depth of alluvium. These measurements were performed in June 1970 (4), using displacement meters to record ambient and pile-driving vibrations, the main results being presented in Table I.

TABLE I - Predominant soil periods in Caracas.

Test Site	Depth of Deposit (m)	Frequency (Hz) measured	Frequency (Hz) corrected	Period (s) corrected
Palos Grandes	~ 230	1.0 to 1.5	0.7 to 1.0	1.0 to 1.5
Parque del Este	~ 120	1.5 to 3.0	1.0 to 2.0	0.5 to 1.0
Parque Central	~ 70	2.5 to 5.0	1.5 to 3.5	0.3 to 0.7
Caurimare	0 (rock)	5.5 to 7.5	3.5 to 5.0	0.2 to 0.3

The "measured" values were corrected by a 50% increase of the period, in order to extrapolate microtremors to large amplitude vibrations. The value of this correction factor was selected on the basis of previous studies on the 1967 Caracas earthquake, particularly those concerning the Palos Grandes zone (1).

The soil measurement results were in some way confirmed by dynamic tests of a tall building located at Parque Central. This building, with fundamental periods of about 1 s was but little affected by the 1967 earthquake.

As a result of the studies mentioned most of the dynamic analysis of the structure used as input an envelope of the three spectra proposed, with an intensity reduced to the LNEC spectrum, the variance and the maximum values of the ground acceleration being given by:

$$\bar{a}^2 = \int S(f)\, df = 1750\ \text{gal}^2 \quad \ldots\ldots\ldots\ldots \quad 1)$$

$$a_{max} \simeq 3\sqrt{1750} = 0.13\ g \quad \ldots\ldots\ldots\ldots \quad 2)$$

At this level of acceleration structural behaviour is non-linear.

5 - DUCTILITY ASSESSMENT

Static rupture tests on reinforced concrete models (5, 6) were carried out to determine the ductility of the transverse walls and lintels. Two models having the same geometry but different amounts of reinforcement were used in both studies.

5.1 - Shear wall model tests

The 1:20 models of the lower portion of the transverse walls (Fig. 4) were subjected to cycles of vertical and horizontal forces combined so as to correspond to increasing values of seismic coefficients. In Model 1 most of the reinforcement was uniformly distributed as a double mesh, the total ratio of vertical reinforcement being about 1%. In Model 2 the reinforcement was distributed as a double mesh with aditional bars and hoops placed at the extremes of the shear walls. The total ratio of reinforcement in this model was 2%.

The main conclusions of the tests were: in the elastic range both models exhibited cantilever behaviour; yielding started for seismic coefficients of 0.20; cracking developed at the base and around the corners of the openings and progressed uniformly in both models; the structural behaviour after yielding was little affected by cracking; rupture occurred in Model 1 due to tension in the base for a seismic coefficient of 0.36; and in Model 2 due to compression of the concrete at the base for a seismic coefficient of about 0.50.

By extrapolating the test results to the prototypes an overall ductility factor of 1.5 to 2.0 was ascribed to the transverse shear walls, within the limits of practical interest for the seismic intensity considered.

5.2 - Lintel model tests

The lintels designed was based on the testing of two reinforced concrete models, scaled to 1:2. The surrounding slabs were also taken into account in these models, in which two consecutive door openings were reproduced. The models were tested vertically.

One model was reinforced with crossed bars and another with an embedded I-beam, together with hoops placed on the surrounding concrete. The latter solution was chosen owing to its good cracking pattern and higher ductility.

The models showed a relatively early loss of rigidity in the lintel beams. This fact required a reexamination of the structural analysis schemes, a proper reduction in the lintel rigidities being taken into account.

The confirmed reliability of the lintel performance was a basis for further redesigns of the buildings, including a substantial increase in the number of door openings.

6 - DYNAMIC LINEAR AND NON-LINEAR ANALYSES

The seismic response of the structure to translational and torsional oscillations was investigated by experimental and analytical studies. Tests on an elastic model of the whole building to a 1:40 scale were carried out at the ISMES in Bergamo (7). These tests, fully described in another paper presented at this Conference, were particularly useful for studying torsional frequencies and modes and the coupling between translational and torsional vibrations, due to a slight building asymmetry.

In the analytical studies (4) the following problems had to be considered in detail.

In the structural idealization the pseudo-frame approach (Fig. 7) proved accurate enough. It was complemented by finite element analysis for studying the longitudinal structure in the transition zone between frame and shear wall.

The influence of soil deformability upon the dynamic response was studied by including a foundation rocking spring in the pseudo-frame idealization. The values of the spring constant were estimated by using the elastic theory together with typical values of in-situ shear wave velocities for the soils of Caracas (1) and also dynamic tests in a nearby building (4). The range of spring constants covered the uncertainties inherent in such estimates (8). The resulting spring constants corresponded to values of subgrade modulus from 10 to 100 N/cm^3. Since inclusion of rocking decreases the stresses but increases the displacements in the structure, the largest estimated rocking spring constant was used for computing stresses, while the smallest estimated value was used for determining total motion at the top of the structure.

It was concluded from the dynamic analysis that the soil-structure interaction could be disregarded in the longitudinal direction; in the transverse direction it was quite important, reducing seismic stresses near the base of the structure by about 20%.

Measurements during and after construction confirmed the reasonableness of the foundation design studies. The measured static settlements were, as predicted, smaller than 8 cm, while the measured dynamic foundation motion corresponded to a subgrade modulus of 60 to 100 N/cm^3 (9). A viscous damping of 0.05 for the first and second modes was assumed in all the calculations.

Modal analysis techniques and step-by-step numerical integration (4) were used in the study of linear and non-linear seismic responses. Torsion due to the phase-lag propagation of the seismic waves was also studied (4). It was concluded that torsion could increase seismic stresses in the transverse extreme walls by about 20%.

Table II presents values of natural frequencies, as predicted in the analytical and experimental studies, and as measured after completion of the structure, in June 1972 (9).

TABLE II - Predicted and measured natural frequencies.

Type of movement	Vibration mode	Natural frequency, Hz		
		Dynamic analysis	Elastic model tests	Measured
Transverse translation	1st 2nd	0.52 1.94	0.54 1.88	0.58 1.88
Longitudinal translation	1st 2nd	1.14 4.00	0.66 2.12	0.87 2.44
Torsion	1st	-	0.59	0.42

Maximum horizontal displacements of 15 to 20 cm (transverse) and of 5 to 7 cm (longitudinal) were obtained at the top of the structure just below the penthouse.

The penthouse steel structure was subjected to a careful dynamic analysis since it was foreseen that whiplash effects could occur.

7 - SEISMIC DESIGN CRITERIA

The results obtained in the dynamic analysis of the structure, performed in close coordination with the structural engineers of Delpre, C.A. led to the following basic criteria.

The transverse shear walls were designed for a triangular distribution of seismic coefficients corresponding to a base shear of 14% of permanent loading. The axial seismic forces acting on the columns were reduced so as to resist the overturning moments given by the dynamic analysis.

The longitudinal frames and shear walls were designed using seismic coefficients varying linearly from 0.08 at the base to 0.16 at the top.

The penthouse was designed for a seismic coefficient of about 0.50.

These load distributions are in good agreement with the results of the dynamic analysis.

8 - GENERAL CONCLUSIONS

The seismic studies of Parque Central 44-story apartment buildings in Caracas performed by a multidisciplinary team of advisors co-ordinated by the Venezuelan engineers in charge of the structural design afforded an excellent opportunity for exchange of information and experience in the field of Earthquake Engineering. Thus useful results were obtained for the design and construction of these large reinforced concrete shear-wall buildings.

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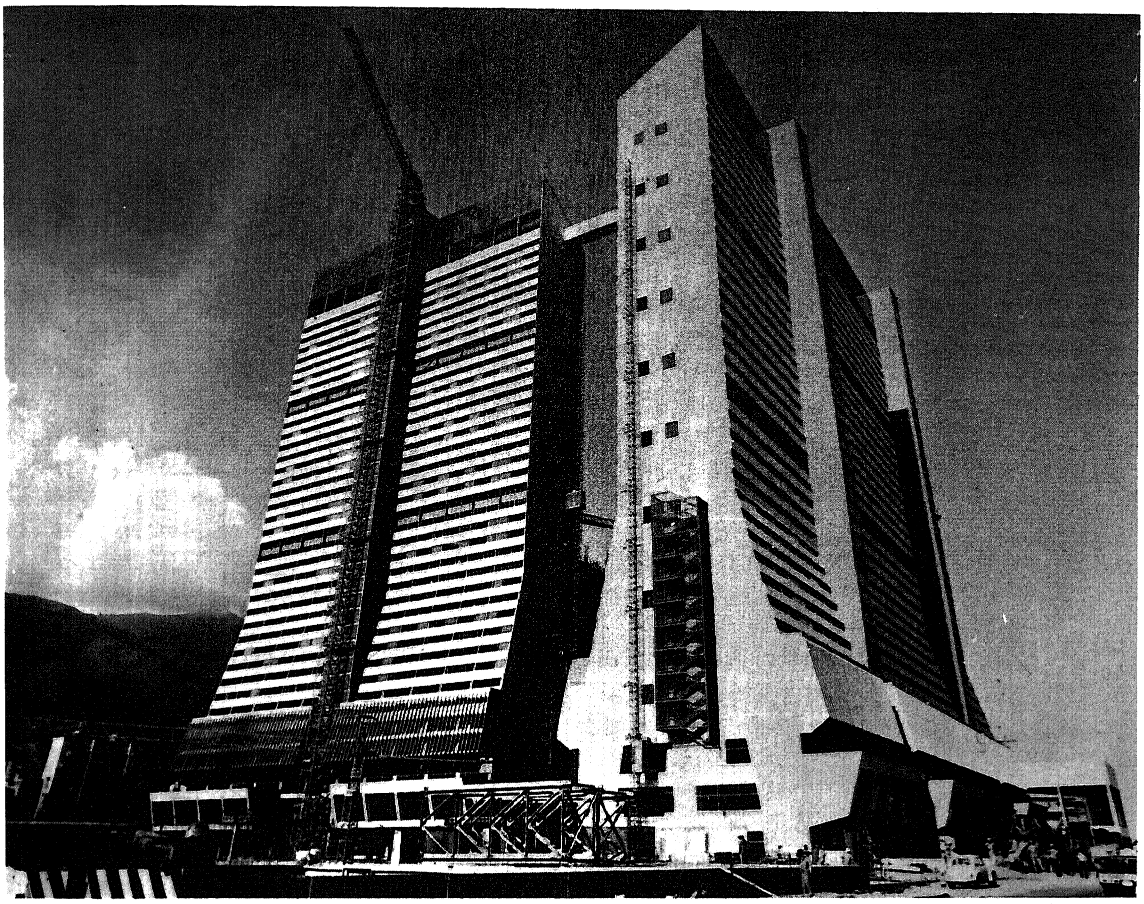

Fig. 1 - View of Parque Central Buildings, November 1972.

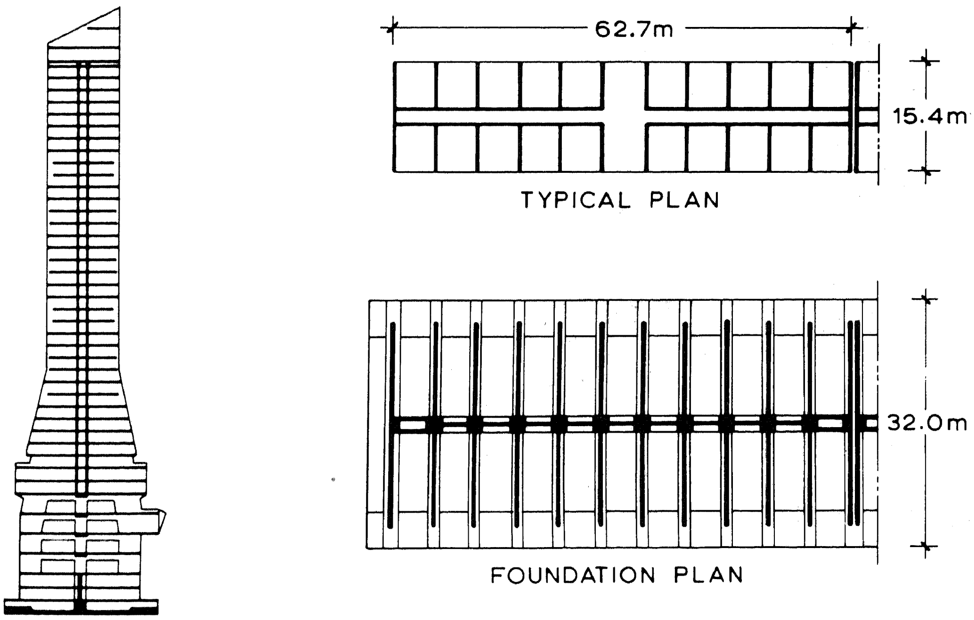

Fig. 2 - Elevation, typical plan and foundation plan of the buildings.

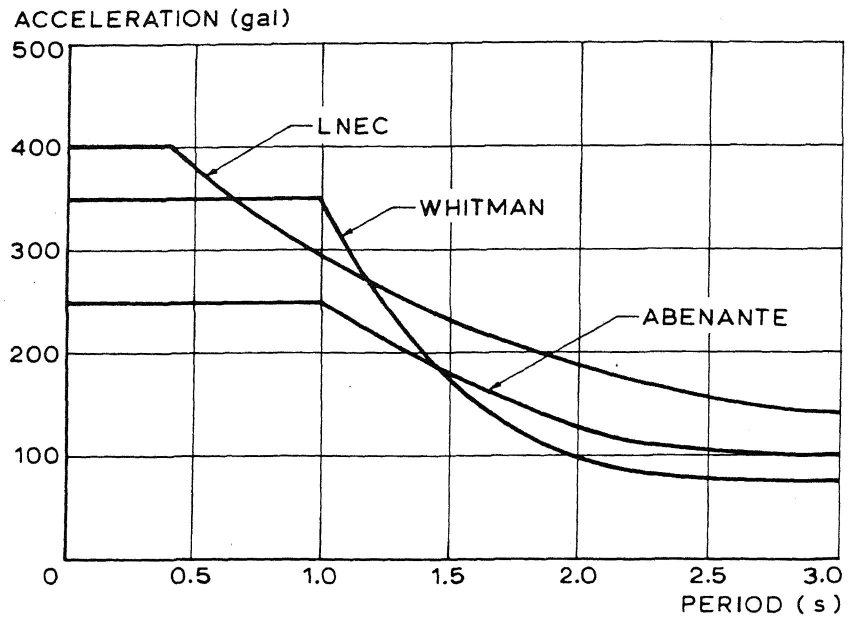

Fig. 3 - Acceleration response spectra.

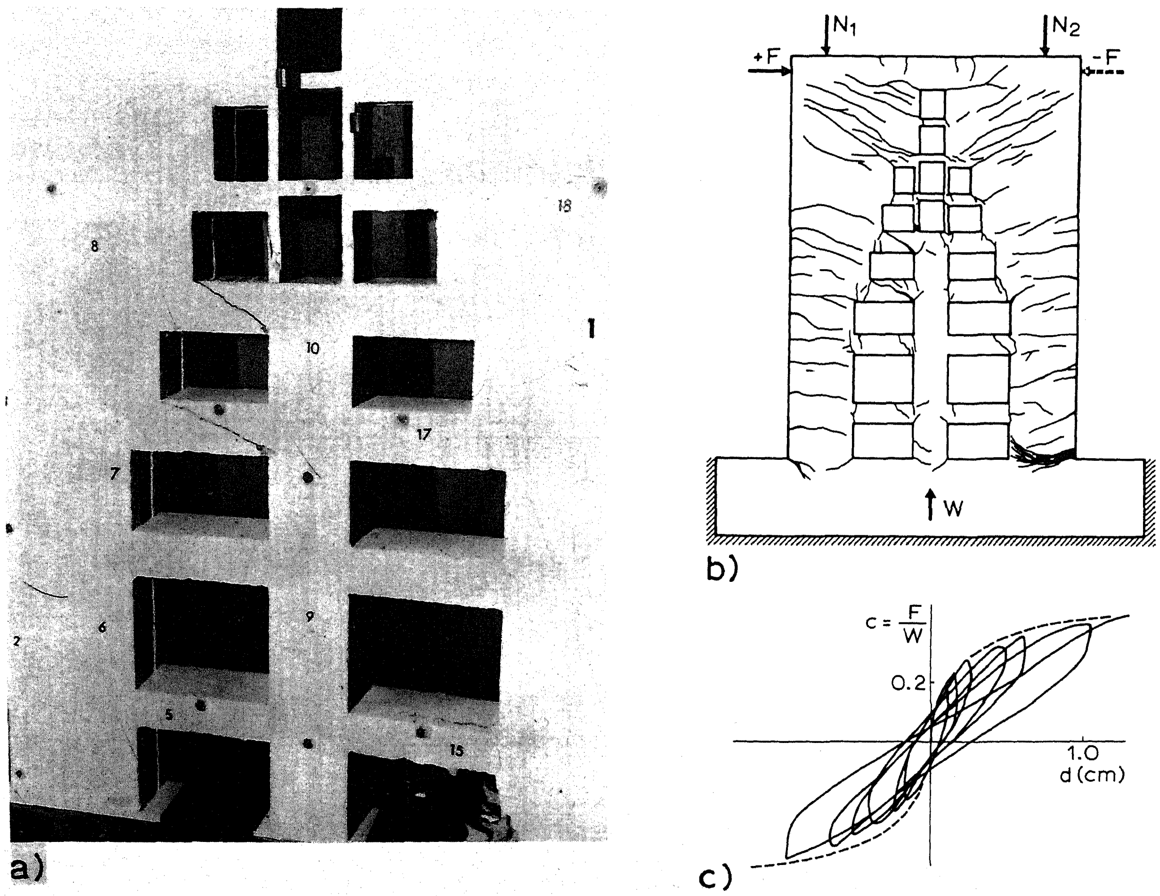

Fig. 4 - Test of reinforced concrete Model 2 (shear walls with openings).