RESPONSE OF TWO REINFORCED CONCRETE BUILDINGS IN MEXICO CITY DURING THE PETATLAN EARTHQUAKE OF MARCH 14, 1979

JOSE LUIS TRIGOS (I)
WERNER HENTSCHEL (II)
Presenting Author: J. L. Trigos

SUMMARY

This paper summarizes a study on the behavior of two reinforced concrete buildings in Mexico City's soft clay zone, during the Petatlān earthquake of March 14, 1979 (M_S = 7.6), which suffered structural and non-structural damage. The response of each building was obtained using a three dimensional structural idealization, in terms of dynamic properties (verified with in situ period measurements), story displacements and mechanical actions, according to the static equivalent and the dynamic modal methods for Mexico City's 1966 and 1976 Codes, and seismic coefficients and acceleration spectra of the earthquake. Predicted behavior using the mathematical model agreed satisfactorily with observed damage.

THE PETATLAN EARTHQUAKE OF MARCH 14, 1979

The Petatlān earthquake occurred at 11h 07m 02s GMT on March 14, 1979. Its epicenter was located off the Pacific coast approximately (Refs 1,2) at 17.3°N and 101.4°W, Fig 1, in the zone with highest sismicity of Mēxico (Ref 3), with an estimated magnitude M_S = 7.6 and focal depth of 50 km. In the town of Petatlān there was severe damage on small adobe houses; in Zihuatanejo and Lāzaro Cārdenas, cities located at 40 and 100 km, respectively, from the epicenter, there were several medium-rise buildings with important damage and, as in Petatlān, some non-engineered adobe houses collapsed. There are three important facilities in the area, the Las Truchas steel mill and two earth dams, Infiernillo and La Villita, in which there was no damage. A maximum peak acceleration of 0.31g was measured in Lāzaro Cārdenas whereas in Mexico City, 320 km from the epicenter, it was of 0.058g and 0.021g, in soft clay and rock, respectively, with a sensible duration of about one minute, ($g = 9.81\ m/sec^2$).

The effects of this earthquake were studied on 60 buildings reported with damage in Mexico City (Ref 4). Among the most important highlights was that from 46 buildings, non-structural damage (NSD) was detected on 24 (8 light, 5 medium and 11 severe), and structural damage (SD), on 22 (10 light, 7 medium and 5 severe, of which a three level reinforced concrete structure collapsed and two had previous damage), Fig 2. This work describes the study of two structures: building A, with medium NSD and light SD; and B, with severe NSD and SD. For further details see Ref 5.

(I) Development Manager, Civil Engineering Department, Bufete Industrial, Apartado Postal 74-171, Mēxico 09080, MEXICO

(II) Eng. Dept., Nuclear Power Proj., Comisiōn Federal de Electricidad, MEXICO

STUDIED BUILDINGS: CHARACTERISTICS, DAMAGE AND RESPONSE

Building characteristics and observed damage

Building A consists of an eleven floor, lobby and basement reinforced concrete structure, with cast in place girders, columns and floor system formed by slabs and beams, with masonry walls on frames A and D, and in the services zone, Fig 3. The foundation mat is supported on piles resting on the City's first sand layer at a depth of 33 m. SD involved shear failure cracks on three girders of the third floor; NSD originated mainly on masonry walls in the services zone.

Building B was an eight floor with lobby-garage, reinforced concrete structure with cast in place columns and floor system formed by waffle slabs, and hollow masonry walls located as shown in Fig 4. Exterior non-structural masonry walls were provided for architectural purposes. The foundation was resolved by a two way inverted beam and slab system, supported by piles resting on the sand layer at a depth of 25 m. Observed SD was concentrated mainly on the first story and consisted of the failure of columns, in some cases on the upper as well as on the lower end, and local shear failure of the flexible waffle slab, due to the torsional excenticity induced by the quake as can be inferred by inspection of Fig 4. NSD was located mainly on exterior facade masonry walls and on the adjoining windows.

Comparison between predicted and observed responses

For purposes of analysis, the structures were idealized as three dimensional linear-elastic systems, with vertical frames and masonry walls, and horizontal rigid diafragms at floor levels, subjected to gravity and seismic loads, according to local building Codes of 1966 and 1976, Refs 6-8. The analyses were performed with the computer program TABS, Ref 9. The geometrical properties of the analytical models were determined based on construction drawings checked with measured dimensions: for columns, the moments of inertia and the axial areas were based on the uncracked transformed section, and the shear areas were taken to be 5/6 of the gross areas; for beams, the moments of inertia were obtained with the Code's stipulation on effective width and uncracked concrete section; the masonry walls were modeled by panel elements. Weight and mass properties were calculated according to Codes, distinguishing between dead, live and reduced seismic live loads. Seismic loads were defined as indicated by Codes, including static equivalent and dynamic modal spectral analyses; also, static equivalent (based on first mode period) and spectral analyses were performed using the registered motion on recording station number 6, located, as buildings A and B, on soft clay soil (Fig 2). Design spectra from both Codes, as well as those response spectra corresponding to accelerograms recorded on station 6, are shown in Fig 5.

The predicted response was defined in terms of dynamic characteristics, displacements and shear forces for each story on every frame and mechanical actions on all structural elements.

The calculated dynamic properties were the natural periods and corresponding modal shapes, and the participation factors for the first nine modes of vibration in each building. The responses of the nine modes were calculated and used on a dynamic spectral modal analysis based on root-mean-square criterium. Calculated periods were compared with measured periods based on vibration tests induced by low velocity wind recorded on a portable seismograph, and a reasonable agreement was found with a relative error smaller than 15 percent.

The calculated interstory drifts in building A satisfied the Codes' limitations for seismic loads as well as for earthquake induced loads. In building B it was observed that a permanent drift of the first level was originated by the combined effect of the earthquake plus the excentricity between the rigidity and shear centers and the more flexible first story that had no facade non-structural walls. However, in the Codes' and earthquake analyses the maximum calculated interstory drifts were of the same order than the stipulated limit values. This fact suggests that there is a point to be reviewed in the prevailing Code.

Concerning mechanical actions on structural elements (bending moments and shear and axial forces on columns, bending moments and shear and axial forces on girder and beams, and shear and axial forces on masonry panels), from the analyses of building A agreement was found between observed and predicted damage on girder C-D at level three of frame 3, in which shear cracks were present, and first story column A1, in which no damage was evident, as it is illustrated on the typical interaction axial force-bending moment diagram, P-M diagram, shown on Fig 6. For building B, Fig 6 shows typical interaction P-M diagrams for ground floor columns D-5 and H-1; the former did not suffer SD whereas the latter had SD, as did columns B-1, D-1, F-1, A-3, B-3, D-3, F-3, H-3, F-5, H-5 and H-7, with diminishing importance toward upper stories, fact that combined with the waffle slab damage and the NSD on all the facade walls and fenestration, led to the owners' decision of demolishing the building.

CONCLUSIONS

As a result of the studies performed, good general structural behavior in Mexico City was observed during the Petatlán earthquake. Buildings with important SD were very reduced in number and in most cases the damage was due to previous differential settlements and accumulated damage from preceding earthquakes. One exception was building B, described in this work, in which SD was due mainly to very important defective structural layout and design and non-adequate connection details in addition with poor construction and workmanship procedures. Concerning NSD, there was a tendency in the soft clay zone to magnify this kind of damage, as in the case of building A, described here, due to non-satisfactory non-structural detailing of this type of elements. At present, it is known that attention is being devoted to studying the Code's interstory drift limitations and hopefully, in a short time there will be results incorporated on the Code.

It is the authors' opinion that care should be focused on the waffle floor system use in view of its low rigidity desadvantage compared with its cost and low construction time advantages, from the point of view of good earthquake design practice, in the light of the well proven design philosophy called "strong column weak girder". Also, it is believed that more knowledge is necessary on items such as inelastic response, effective width, membrane effect, etc., of waffle floor systems subjected to earthquake loading.

ACKNOWLEDGEMENT

The authors acknowledge the facilities given during the first phase of the study, relative to data acquisition and preliminary analyses, by the Institute of Engineering of the National Autonomous University of México, UNAM, where the first author was a research professor and the second a research assistant.

REFERENCES

1. Alonso, L., et al, "Preliminary report on the March 14 of 1979 earthquake near the coast of Guerrero", (in Spanish), Report IPS-5A, Instituto de Ingeniería, UNAM, México, March 20, 1979.

2. Krishna, S. S. and Havskov, J., "Some seismological aspects of the earthquakes of Oaxaca (24 Nov 1978, $M_s = 7.8$) and Petatlán (14 Mar 1979, $M_s = 7.6$)", V National Congress on Earthquake Engineering, Guadalajara, México, 1979.

3. Trigos, J. L., "Determination of parameters for seismic design of structures in México", IV National Congress on Earthquake Engineering, Oaxaca, México, 1975.

4. Meli, R., Trigos, J. L., et al, "Evaluation of the effects of the March 14 of 1979 earthquake on the Mexico City's buildings", (in Spanish), Internal Report, Instituto de Ingeniería, UNAM, México, April 1979.

5. Trigos, J. L., "Study of the behavior of two reinforced concrete buildings in Mexico City, during the Petatlán earthquake of March 14, 1979", (in Spanish), to be submitted to the IV National Congress on Structural Engineering, León, México, March 1984.

6. Anon., "Mexico City Building Code", February 1966.

7. Anon., "Safety and service requirements for structures. Title IV, Mexico City Building Code", (in Spanish), Report 400, Instituto de Ingeniería, UNAM, México, 1977.

8. Rosenblueth, E., "Seismic design requirements in a Mexican 1976 Building Code", Intl. Jnl. Earthquake Eng. and Str. Dyn., V7, 1979.

9. Wilson, E. L., et al, "Three dimensional analysis of building systems", EERC Rep 72-8, Rev April 1979, Univ. of California, Berkeley, Ca.

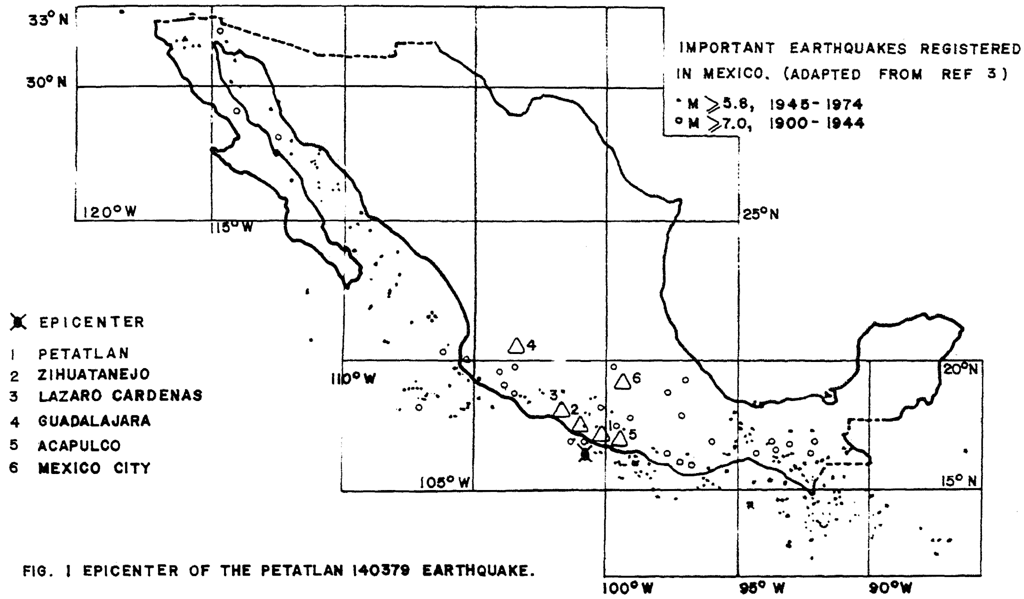

FIG. 1 EPICENTER OF THE PETATLAN 140379 EARTHQUAKE.

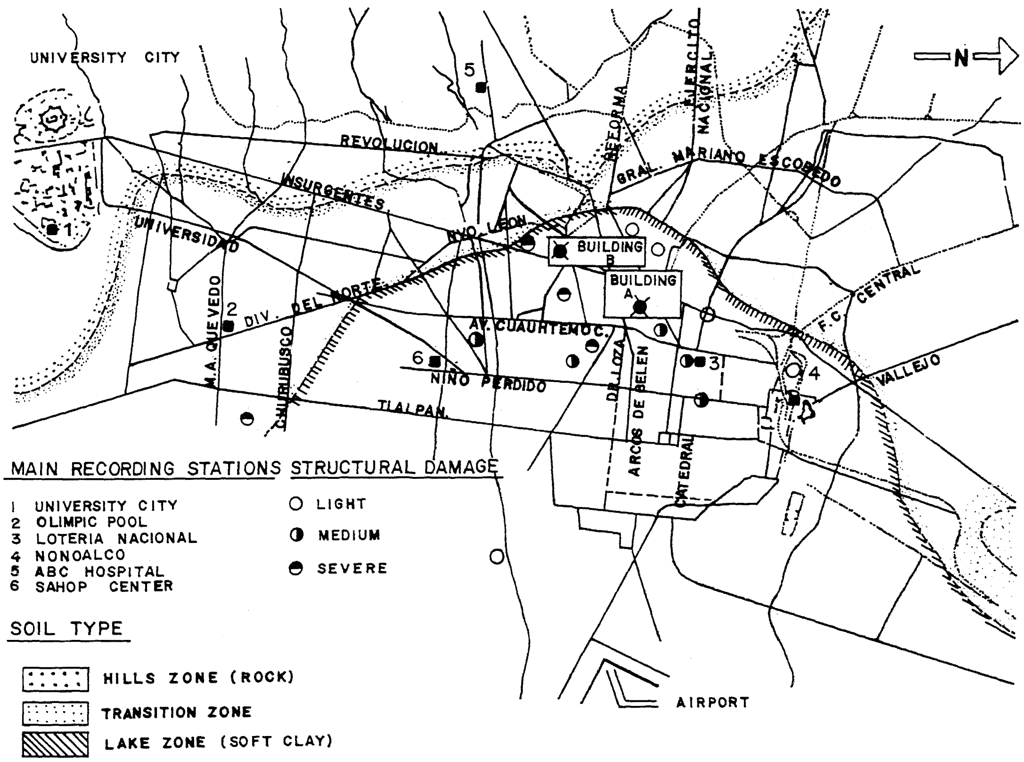

FIG. 2 STRUCTURAL DAMAGE DISTRIBUTION IN MEXICO CITY AND STUDIED BUILDINGS (ADAPTED FROM REF 4)

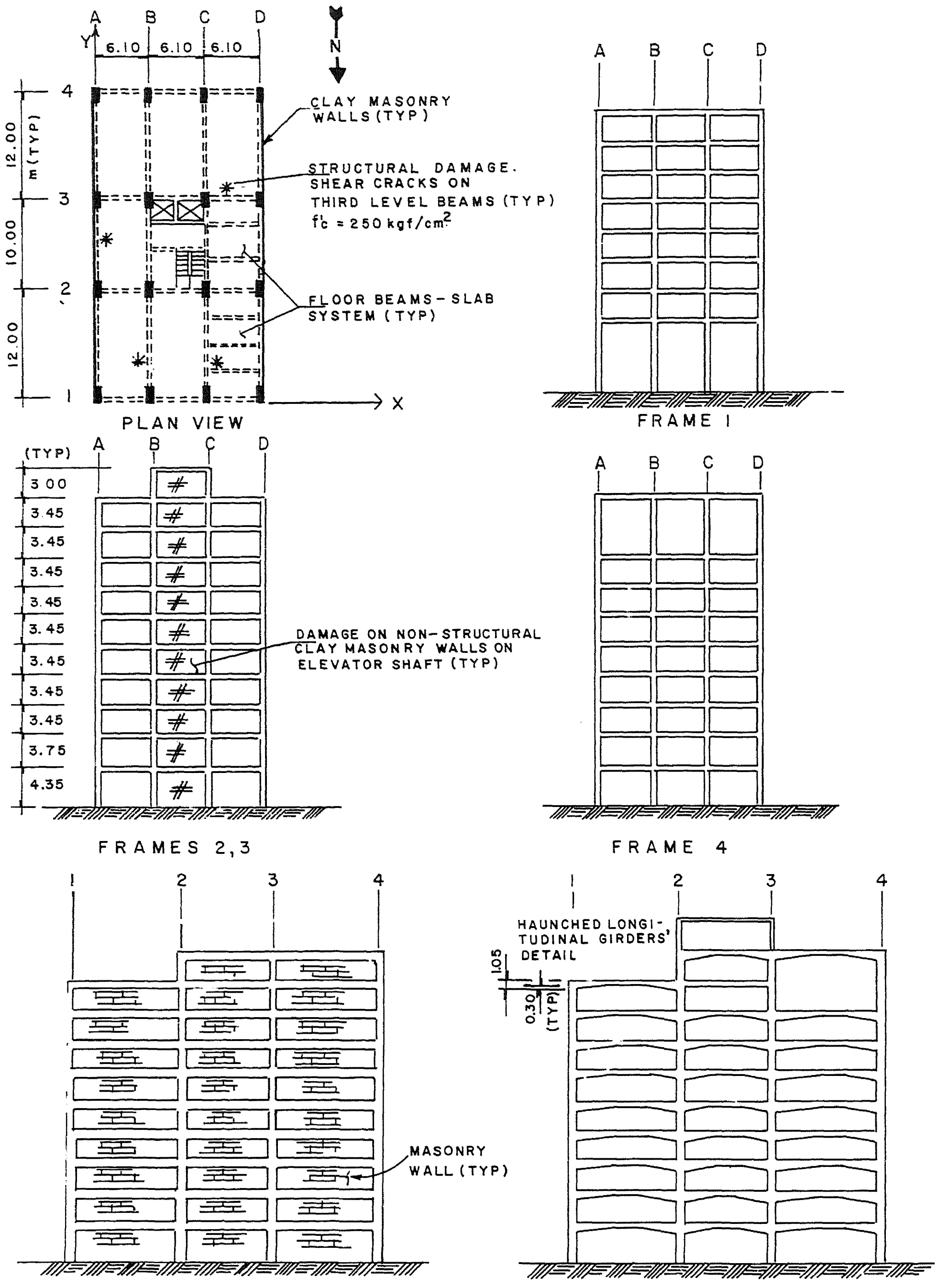

FIG 3. BUILDING A. GENERAL CHARACTERISTICS AND OBSERVED DAMAGE.

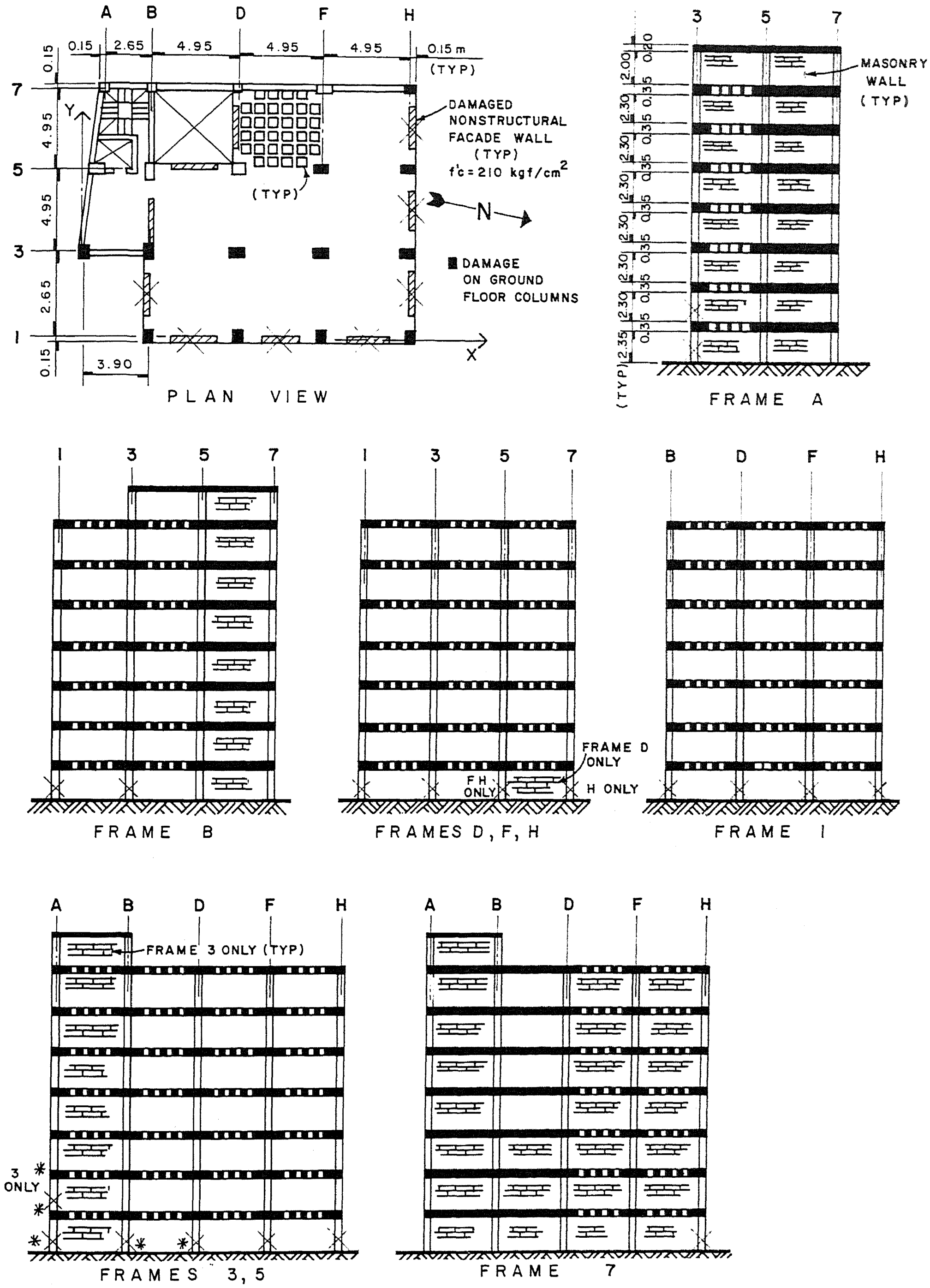

FIG. 4 BUILDING B. GENERAL CHARACTERISTICS AND OBSERVED DAMAGE.

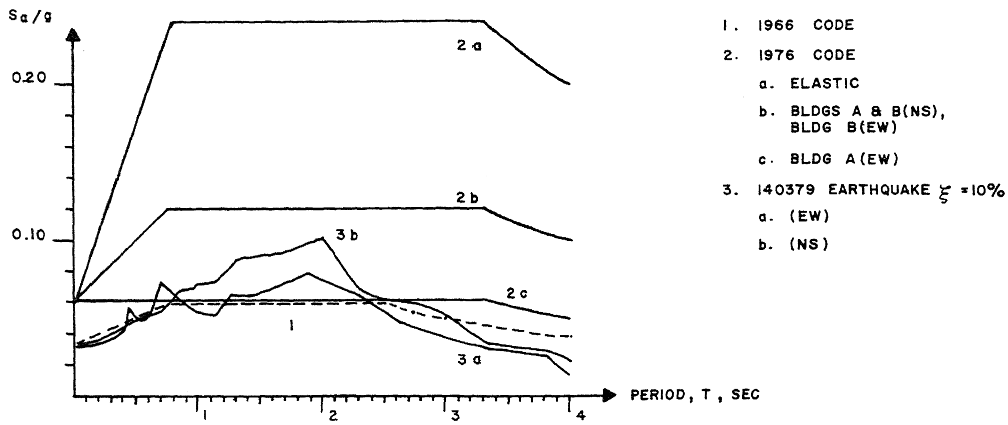

FIG.5. DESIGN AND RESPONSE SPECTRA USED IN THE ANALYSES.

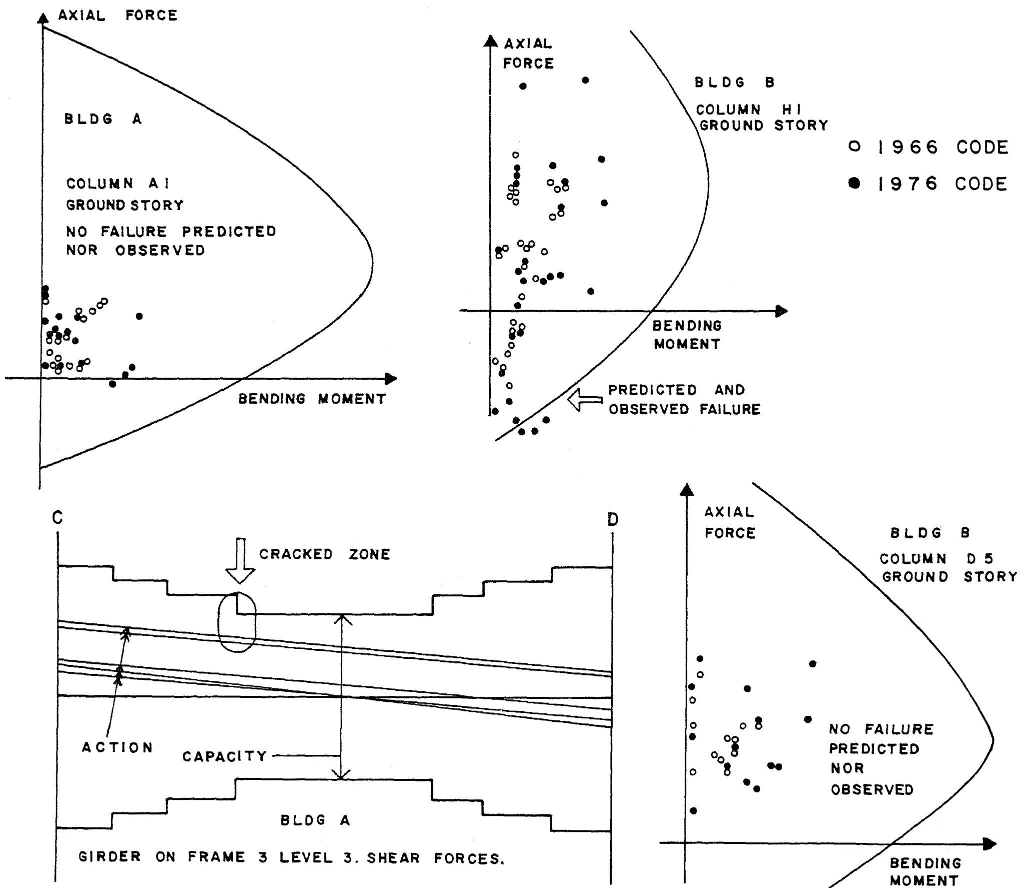

FIG.6. TYPICAL INTERNAL ACTIONS CAPACITY COMPARISONS FOR SELECTED LOAD COMBINATIONS IN BUILDINGS A AND B.