ABSTRACT
of the paper

IMPLICATIONS ON SEISMIC STRUCTURAL DESIGN OF THE EVALUATION OF DAMAGE TO THE SHERATON-MACUTO

by

M. A. Sozen, N. M. Newmark, G. W. Housner

The main building of the Sheraton- Macuto Hotel is an eleven-story reinforced concrete structure. A significant feature of the structural system is that, in one direction the lateral forces are carried in the top seven stories by a pair of parallel slender shear walls which are supported by heavy columns in the lower four stories. During the earthquake of 29 July 1967, severe damage was sustained by the columns supporting the shear walls.

Analyses of possible causes for the column failures suggest that the columns failed primarily because of the lack of ductility, a shortcoming aggravated by the increase of axial load and the failure of some of the connecting girders in shear.

The damage to the building emphasizes the importance of considering in design the deformations of the entire structure under the influence of earthquake forces to make certain that all structural elements have compatible ductility and of insuring that enough web reinforcement is provided to develop the flexural capacity of beams and columns.

IMPLICATIONS ON SEISMIC STRUCTURAL DESIGN
OF THE EVALUATION OF DAMAGE TO THE SHERATON-MACUTO

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SYNOPSIS

An analysis of the causes of damage to the Sheraton-Macuto Hotel in Caraballeda during the Caracas Earthquake of 29 July 1967. The paper evaluates the strength of the structure to resist lateral loads and discusses a possible sequence of events leading to the failure of the main columns.

INTRODUCTION

One of the major structures sustaining serious structural damage during the Caracas Earthquake of 1967 was the eleven-story Sheraton-Macuto Hotel in Caraballeda.

This paper provides a brief description of the structure, the structural damage, and the results of studies made to understand the causes of damage.

It is assumed that the general characteristics of the earthquake and the nature of the damage in Caracas and its vicinity will be adequately covered by other papers in this session. Consequently, this paper concentrates on the study of the damage to the Sheraton-Macuto and the implications of the damage for structural design.

THE STRUCTURE

The main building of the Sheraton-Macuto Hotel in Carabellada was designed in 1955. The column layout of the entire structure, which covers an area of 99 by 18 meters center-to-center of columns, is shown in Fig. 1. The longitudinal axis of the building is inclined at approximately 65° to north. The building is sited on fill and closely parallels the shoreline. The beach is on the north side of the hotel.

Figure 2 shows a cross section of the building. The footings are supported on piles. Heavy columns extend up to level 4. Between levels 4 and 5, the columns meld into shear walls as shown in Fig. 2. Table 1 gives the dimensions and reinforcement for the members of the bent on column line 8.

As shown in Fig. 1 the structure is broken up into three independent parts separated by expansion joints. A plan of the central portion along with the sizes of the columns between levels 3 and 4 is shown in Fig. 3.

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The compressive strength of concrete specified for the building was 210 kg/cm^2 as determined by 6 by 12-in. cylinders. The specified yield stress of bars was 2400 kg/cm^2. A wide assortment of bars was used in the building including plain and deformed bars.

Floors were typically one-way joist floors. Two-way joists were used in span D-E (Fig. 3).

STRUCTURAL DAMAGE

Severe structural damage occurred in three classes of members: (1) interior columns at level 3, (2) floor girders connecting the main columns in levels 2 and 3, and (3) in columns on lines A and B (Fig. 2) at level 2. The critical failure was in the columns at level 3. It could have led to collapse of the whole structure.

The overall features of the column failures at level 3 were identical. An example is shown in Fig. 4. There was crushing of the concrete on the north side of the column at the junction of the column with the girder at level 4 (not seen in the photograph because of the false ceiling.) Wide cracks perpendicular and inclined to the column axis extended from the crushed concrete toward the south edge of the column. The orientation of the cracks and the location of the crushed concrete indicated that the major axis of the floor deflection at the time of the failure coincided with the magnetic north-south axis. The columns also had a network of cracks compatible with a floor deflection toward the south, but these were minor and did not lead to structural damage.

There was no distress at the junctions of the columns in level 3 with the floor girders. There was no structural distress elsewhere in columns on lines C, D, E, and F.

The locations of the damaged columns at level 3 are shown in Fig. 1. It is noteworthy that all but one of the failures occurred in line E. Only two of the columns in line E remained intact. One failure was observed in line D. (It has been reported that column D-10 was also found to have failed. The writers cannot confirm this observation because the column was covered during their inspections.)

Several of the deep floor girders connecting the columns on lines C-D and E-F at levels 2 and 3 were observed to have failed (Fig. 1). The failures were characterized by wide inclined cracks near the beam-column junctions (Fig. 5).

At level 2, all columns failed on lines A and B. These failures were also characterized by a wide inclined crack (Fig. 6) compatible with a south deflection of level 3 with respect to level 2.

No significant cracking was observed in the twin shear walls above level 4.

As would be expected, a considerable amount of cracking and crumbling occurred in partition walls, skin walls, and staircase details. None of this was structurally significant.

Direct Implications of the Structural Damage

It was stated above that the nature of the damage to columns in level 3 was similar and that the cracks indicated a deflection of level 4 with respect to level 3 in the north-south axis with level 4 moving south. The same motion is implied by the column failures in level 2. Furthermore, a one-story pergola in the hotel courtyard fell toward the south, its fall axis paralleling the damage axis of the columns in the building. This evidence indicates strongly that the major structural damage occurred simultaneously during a "spike" in the ground motion toward the north, an acceleration pulse which was not exceeded at any other time during the earthquake.

The primary cause of the column failures in level 3 is not identified categorically by the state or the failed column. There is no question that the columns were subjected to severe combinations of axial load, moment, and shear. Axial load may be eliminated as the primary cause of failure. The bias in the inclination of the cracks attributes more than secondary effects to bending and shear.

On the basis of qualitative evidence alone, it is not too difficult to make a case for shear as the primary cause of failure. The cracks are typical of shear failures. Even if it has to be admitted that all the interior columns carried the same shear, it is not awkward to claim that the maximum shear was approximately equal to the average shear strength of the columns thus causing about half of them to fail. However, the explanation is weakened when it is considered that at the assumed moment of failure columns on line E were likely to have been subjected to considerably larger axial forces (but not near the axial-load capacity of the columns) than those on line D. With shear as the primary cause of failure, more columns should have failed on line D than on line E.

A failure with bending as the primary cause is plausible in relation to the observed state of the columns after failure. The crack pattern is compatible with that observed in laboratory specimens subjected to similar loading conditions but failing under the primary influence of bending (1). As it was in the test specimens, the "faulting" in the columns may have been the effect of shear after destruction of the compressed concrete. Furthermore, the likelihood of failure on line E is stronger because the ductility of these columns is reduced as a result of the axial load.

It is apparent from Fig. 4 that the conditions of restraint of the columns were not identical at the top and bottom joints. This is also corroborated by the fact that the floor beams failed at levels 2 and 3, and the lighter columns in level 2 failed as a result of relative floor movements without distress in the heavier columns at the same level.

An attempt is made to develop a quantitative evaluation of the causes of failure in the following chapters.

STRENGTH OF THE STRUCTURAL ELEMENTS

Estimates of the strength and ductility of certain structural members of the building were made using the specified strengths for the steel (f_y = 2400 kg/cm^2) and concrete (f'_c = 210 kg/cm^2). The dimensions of the members and amounts of reinforcement were obtained from the structural drawings.

Columns at Level 3

The capacity of the round columns at level 3 (Fig. 3) to resist combinations of axial load and bending moment was calculated on the basis of the following assumptions: (a) linear strain distribution over the depth of the column, (b) elasto-plastic stress-strain curve for the steel with $E_s = 2\times10^6$ kg/cm^2, and (c) an equivalent rectangular stress block for the concrete with the maximum stress at 0.85 x 210 kg/cm^2, the limiting strain at 0.004 and the strain at initiation of stress equal to 0.15 x 0.004.

The computed interaction diagrams for column types I through V (Fig. 3) are shown in Fig. 7. The broken curves represent the calculated limiting curvature.

Shear Strength of Interior Columns

The calculation of the shear strength of the round interior columns is subject to more uncertainty than the calculation of the combined flexural and axial-load capacity of the columns. However, a good estimate of the shear strength may be obtained with the use of the data from round columns presented by Faradji and de Cossio (2) and reproduced in Fig. 8.

Referring to a column of Type IV and for a moment to shear ratio of 0.25m (which represents approximately the values calculated later in the paper for the interior columns), the shear strength of the column is estimated to be 85 tons for no axial load and 120 tons for an axial load to shear ratio of ten. Any direct contribution of the ties is not included in this estimate.

Shear Strength of Floor Girders

The dimensions of the floor girders at levels 2 and 3 are given in Table 1. The girders between column lines C-D and E-F for bent No. 8 have No. 3 stirrups (four legs) at 30 cm. Assuming in accordance with reference 3 that

$$V_u = 0.53 \sqrt{f'_c} + A_v \, f_y \, \frac{d}{s}$$

where V_u: shear capacity, f'_c: concrete strength in kg/cm^2, A_v: steel area in one stirrup, f_y: steel yield stress, d: effective beam depth, and s: stirrup spacing, the shear strength is estimated to be 70 tons for level 2 and 60 tons for level 3. The flexural capacities of the beams are adequate to develop the shears calculated.

DISCUSSION OF THE CAUSES FOR THE DAMAGE

The damage to the building was the result of the interaction of a complex system, a structure of nonuniform strength and stiffness, with a complex effect. It is necessary to make certain simplifications to understand the causes for the damage. Only one bent of the structure is considered in the following discussion: bent No. 8 (Fig. 3). The structure is assumed to be linearly elastic. The inertia effect is represented by static lateral loads acting parallel to the plane of the bent.

Gravity Loads

The gravity loads on the columns at level 3 were computed from the dimensions given in the drawings and the specified live loads. Unit weight of the concrete was taken as 2.4 tons/m^3. The floor filler tiles were assumed to weigh 55 kg/m^2. The live loads were 180 kg/m^2 for the roof, 600 kg/m^2 for level 11, and 300 kg/m^2 for levels 4 through 10. In determining the tributary areas for the columns, the interior spans were assumed to be simply supported. The resulting axial loads were 500 tons for columns D and E, and 400 tons for columns C and F.

Computations of The Effect of Lateral Loading

In determining the moments, shears, and axial loads in the frame members, the stiffnesses of the members were based on their transformed sections (modular ratio = 8.0). The member lengths were based on center-to-center distances.

The shearing and axial deformations of the members were considered in the analysis (with the shearing modulus taken as half the "elastic" modulus) as well as the flexural deformations. Columns at level 1 were assumed to be fixed at the top of the foundation.

Moments, shears, and axial forces were obtained for concentrated lateral loads placed successively at all levels of the bent. Different positions of the lateral load above level 4 make little difference in the joint moments at level 4. The axial loads in the columns are obviously sensitive to the location of the lateral load. Analysis of the idealized frame for its fundamental frequency (1.1 cps) indicated that the moment-to-axial load ratios at the joints of level 4 for the natural mode corresponded to those for a horizontal load placed at level 9. To simplify further analyses of the building, it was assumed that the mass of the building was concentrated at level 9.

Figure 9 shows the moments and shears in the columns and the shears in the beams for 100 tons applied at level 9. The moments are inscribed on the side of the column where they tend to create compression. Inspection of Fig. 9 reveals that the level-3 columns are subjected to greater moments at the top than at the bottom, an observation which follows readily from the geometry of the building, that the exterior-column moments are approximately 60 percent of the interior-column moments. The column loads indicate the nearly independent action of the two shear walls.

Stages in the loading of Columns at Level 3

Initially, it is assumed that the columns at level 3 are subjected to axial loads which result from the gravity loads. These loading conditions are indicated in Fig. 7 by the numeral 1.

The loading conditions on level 3 columns for a lateral load of 250 tons plus the gravity loads are indicated in Fig. 7 by the numeral 2. From the information in Fig. 7, it appears that the columns should be able to resist the load. The conditions for column F imply that this column would shed some of its moment because of increased flexibility. The shear in column E of about 76 tons is well within the computed capacity of the column. The calculated beam shear caused by the lateral load at level 2 is 57 tons. If calculated in reference to the clear span of the beam, this shear would be increased by 20 percent. Even if the calculated shear capacity of 60 tons is taken as a hard figure, it does not require hindsight to conclude that the level-3 floor girders will fail at or slightly above 250 tons. These will be followed by failures of floor girders at level 2 and columns on lines A and B. The series of failures will, in effect, rearrange the structure as shown in Fig. 9b.

The moments listed in Fig. 9b for a lateral load of 100 tons indicate that, for 250 tons, the remaining girders at levels 2 and 3 would yield. However, yielding of these girders have relatively little effect on the column moments at level 4, as shown by the solution in Fig. 9c. Also, it is assumed that the calculated base moments at level 1 are considerably larger than the actual ones because the damaged members are not entirely without resistance as implied by the structure in Fig. 9b.

The loading conditions for the top joints of level-3 columns corresponding to the rearranged structure are shown in Fig. 7 by the points marked by the numeral 3. It appears that by the time Stage 3 is reached, column F should have yielded. This is debatable. Column F may never have picked up that much moment because of a progressive reduction in stiffness. But it should be pointed out that even if it did yield, it would not have failed before the interior columns because of its greater capacity for deformation. After the ground motion stopped, any marks of yielding would be covered by the prestressing effect of the gravity load.

In reference to the Stage-3 data in Fig. 7 and assuming that column F would deform more or less plastically transferring the moment to the other three columns, columns C and D appear to be closer to failure than column E. However, on the basis of available ductility, column D is in a more favorable condition. It can shed excess moment without serious distress. Column C does not possess more ductility, but is restrained less rigidly than column C. Consequently, column E is more likely to fail first.

Admittedly the speculations about the response of the building beyond Stage 2 are softer than those about the response before Stage 2. Nevertheless, it appears that the frequency of failures in column line E and the attendant damage in the structure do fit into an intelligible scheme. It can be concluded that these events would have occurred at a shear at level 4 of approximately 250 tons corresponding to 14 percent of the weight of the building above level 4. The base shear coefficient must have exceeded 10 percent during the earthquake. In view of the other effects of the earthquake, this figure is credible. It would take a "third" of the 1940 El Centro N-S component to develop a spectral acceleration of approximately 0.1g in a SDF model with its frequency equal to the calculated natural frequency of bent No. 8 (1.1 cps) and a damping of 10 percent of critical. It may be an oversimplification to represent the building, especially with its progressively fracturing and yielding elements, by a SDF model, but it is plausible to conclude that the maximum ground acceleration was approximately 0.1g during the earthquake.

CONCLUDING REMARKS

The structural damage observed in the Sheraton Hotel at Caraballeda and its analysis lead to conclusions relating to the design of earthquake-resistant structures in general.

Considering conditions at level 3, the intact structure appears to have had adequate strength to resist a shear on the order of 25 percent of the weight above. However, failure of some of the horizontal elements and lack of ductility in the columns led to serious damage before the full strength of the structure could be developed. The behavior of the Sheraton Hotel reemphasizes what has been emphasized before: (a) girders and columns should have adequate web reinforcement to develop the flexural capacity of the member, (b) columns should receive special attention in design to make sure that they have the requisite ductility at increased loads (the estimated load on the damaged column in the structure was almost doubled at the time of failure), and (c) the deflected geometry of the structure should be considered in design as well as the forces in the structure.

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2. M. J. Faradji Capon and R. Diaz De Cossio, "Tension Diagonal en Membros de Concreto de Seccion Circular," Ingenieria (Mexico), April 1965, pp. 257-280.

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TABLE 1

COLUMNS FOR BENT NO. 8

	C	D	E	F
Level 4				
	1.00 Round 30-1	1.10 Round 27-1.25	1.10 Round 33-1.25	1.00 Round 25-1.0
Level 3				
	1.10 Round 30-1.0	1.15 Round 27-1.0	1.10x1.10 36-1.25	1.10x1.10 36-1.25
Level 2				
	1.10x1.10 30-1.0	1.15x1.15 27-1.25	1.15x1.15 36-1.25	1.10x1.10 36-1.25
Level 1				

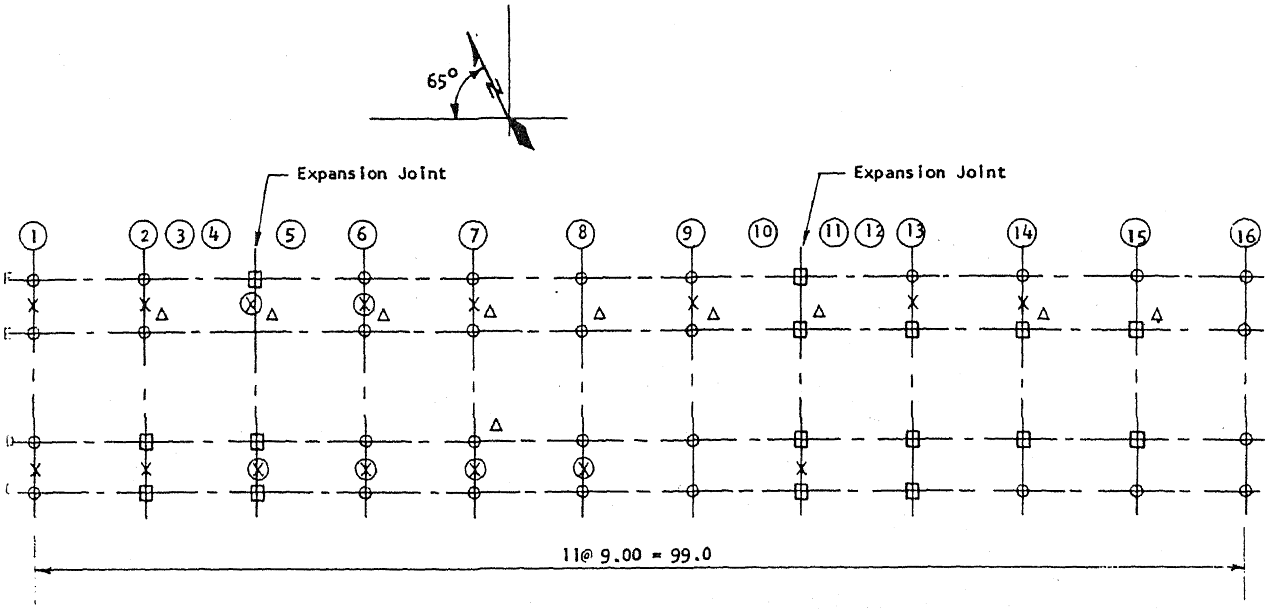
Note: 30-1 indicates 30 reinforcing bars with a diameter of 1.0 in.
27-1.25 indicates 30 reinforcing bars with a diameter of 1.25 in.

Level	4-5	5-6	6-7	7-8	8-9	9-up
	0.45	0.40	0.35	0.30	0.30	0.25

BEAM DIMENSIONS

Level 2	0.90 by 0.80 deep
Level 3	0.80 by 0.80 deep
Level 4	0.40 by 0.30 deep
Level 5-10	0.70 by 0.70 deep
Level 11	0.60 by 0.70 deep
Level R	0.50 by 0.70 deep

All Dimensions in meters



△ *Severely damaged columns at level 3

X Damaged beams at level 3

O Damaged beams at level 2

FIG. 1 COLUMN LAYOUT AT LEVEL 3, MAIN BUILDING, SHERATON-MACUTO

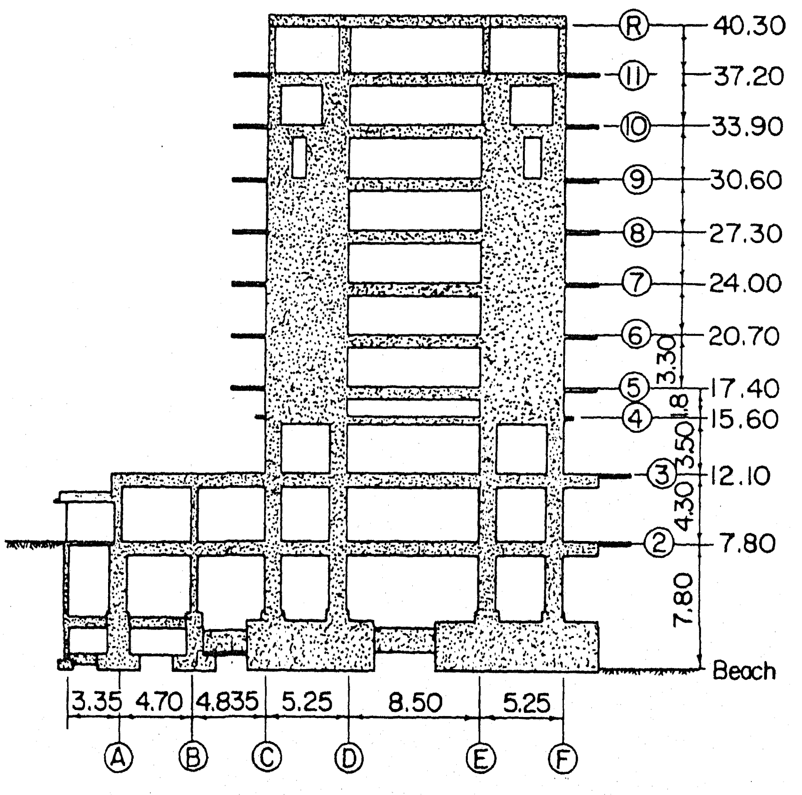


FIG. 2 CROSS SECTION OF THE MAIN BUILDING, SHERATON-MACUTO

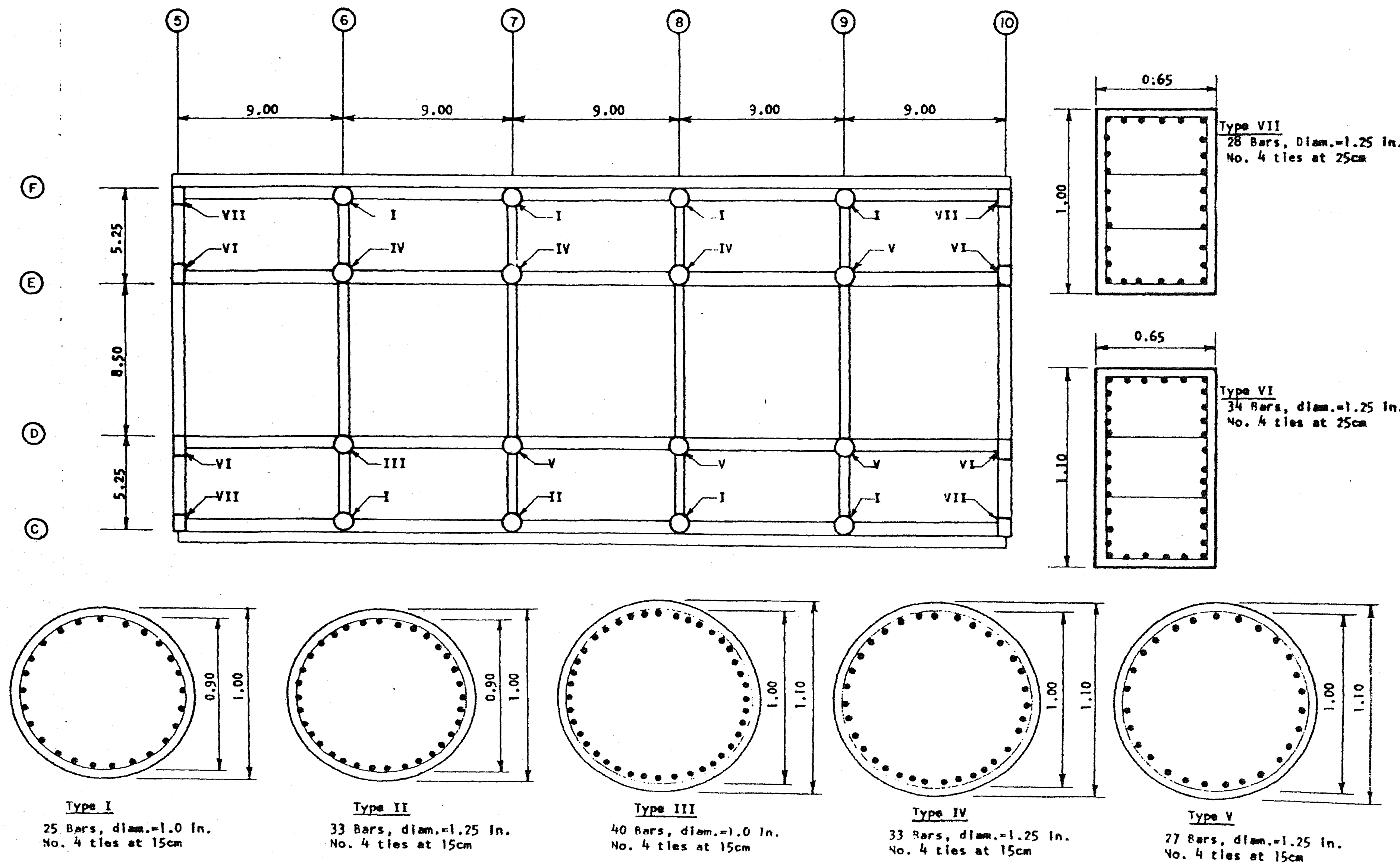


FIG. 3 PLAN OF THE CENTRAL PORTION, MAIN BUILDING, SHERATON-MACUTO

FIG. 4 COLUMN FAILURE AT LEVEL 3
(COLUMN D7, West View)

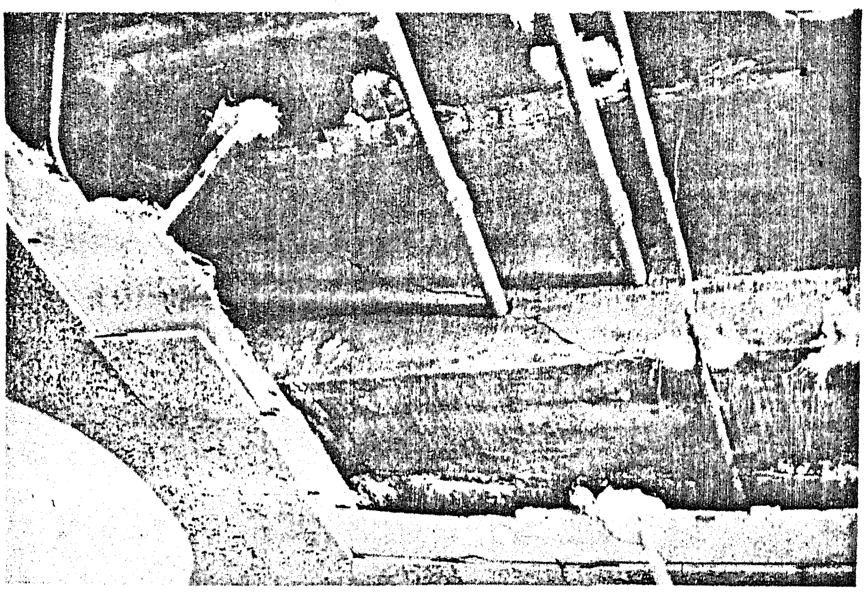


FIG. 5 INCLINED CRACK IN FLOOR GIRDER

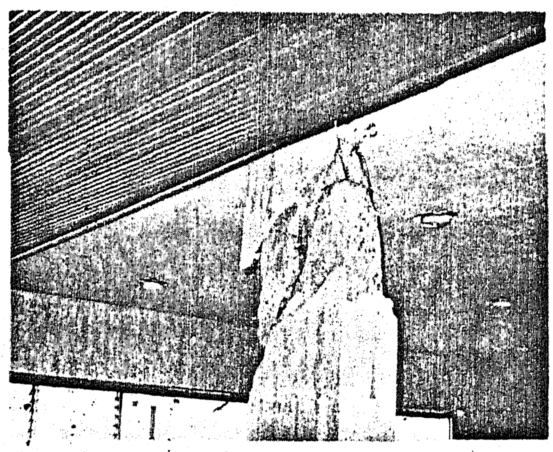


FIG. 6 COLUMN FAILURE AT LEVEL 2, LINE B

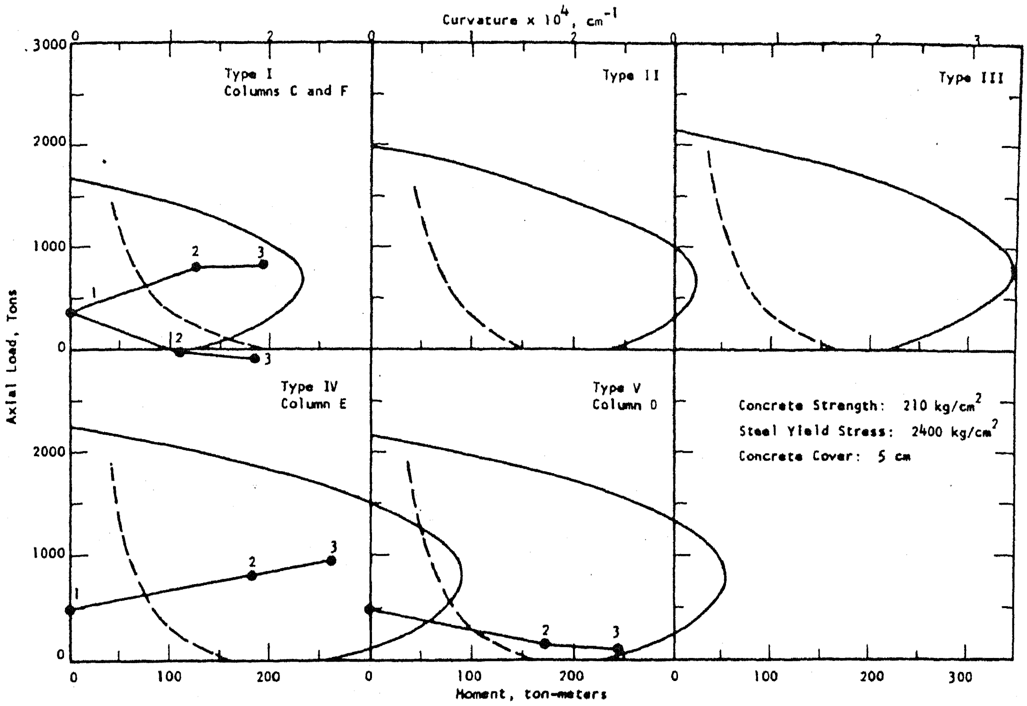


FIG. 7 INTERACTION DIAGRAMS OF LOAD, MOMENT, AND CURVATURE FOR THE ROUND COLUMNS AT LEVEL 3

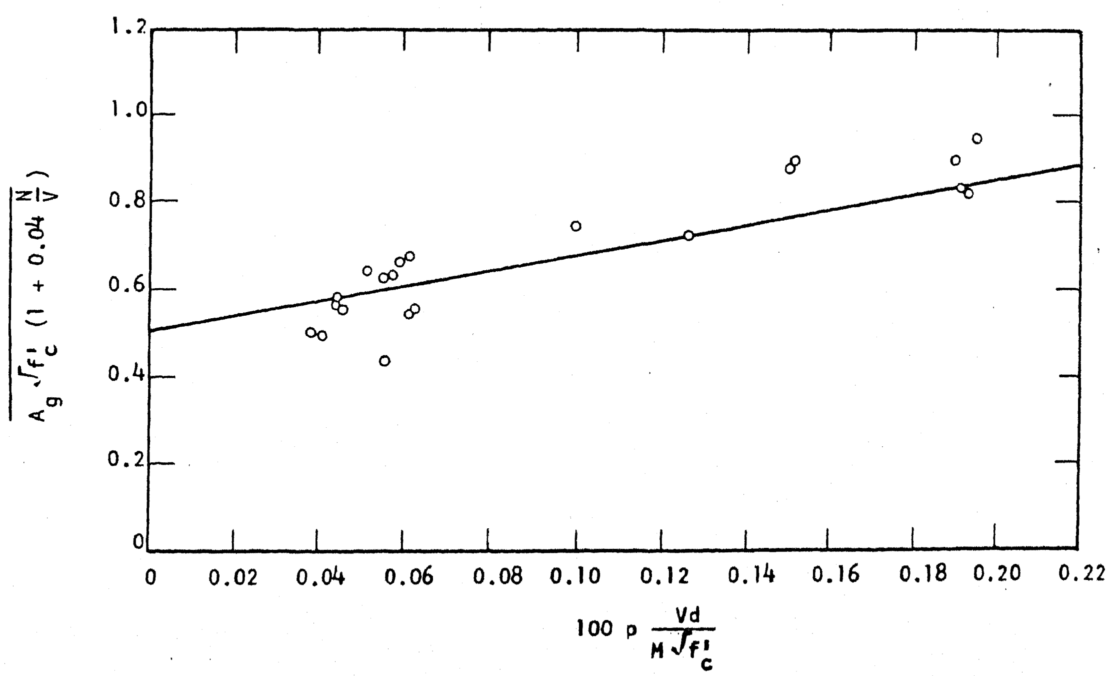


FIG. 8 SHEAR STRENGTH OF ROUND COLUMNS
(After Faradji and de Cossio)

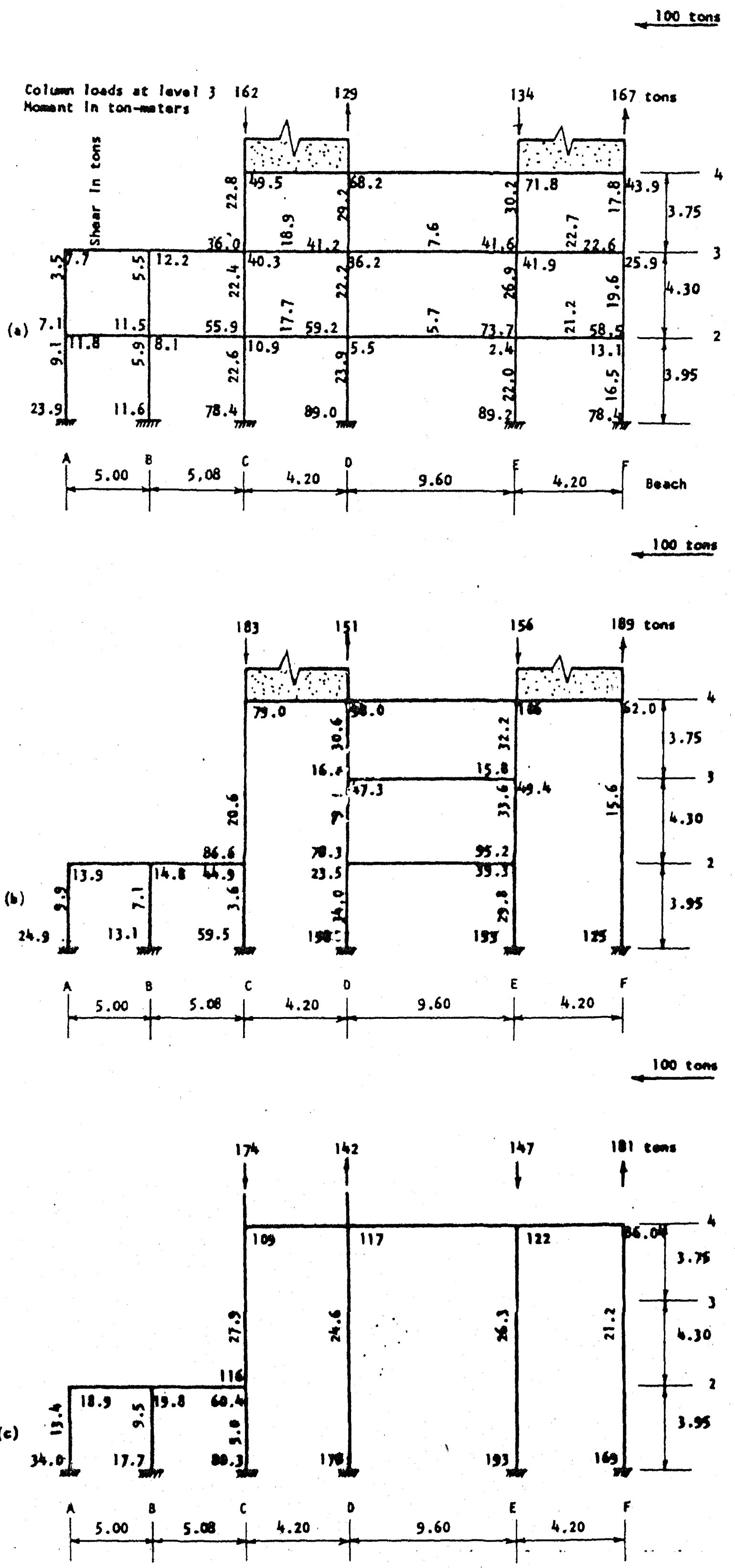


FIG. 9 RESULTS OF ELASTIC ANALYSES FOR A LATERAL LOAD OF 100 TONS APPLIED AT LEVEL 9