

# RESPONSE OF A REINFORCED CONCRETE SHEAR WALL STRUCTURE DURING THE 1972 MANAGUA EARTHQUAKE

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## ABSTRACT

The 1972 Managua, Nicaragua earthquake was a severe test of modern earthquake resistant design and construction procedures. This paper examines the behaviour of the 18-story reinforced concrete Banco de America building which performed exceptionally well during the earthquake. Although the building suffered some structural and non-structural damage, its large symmetrically located, coupled shear walls limited this damage to levels significantly below those observed in more flexible structures. Several linear elastic and nonlinear analyses were conducted to evaluate the building's behaviour and determine the probable cause of the observed damage. Both static and dynamic elastic analyses were used to determine the members that would have failed and the consequence of these failures on the subsequent dynamic response. The effects of biaxial ground motions, foundation flexibility and ground motion characteristics were considered in the elastic investigations. To get a better idea of the dynamic behaviour of the principal lateral force resisting system considered in the design, nonlinear analyses were performed for the coupled shear wall cores as constructed and for the idealized case where the coupling girders were assumed to have unlimited ductility. Even code type static analyses satisfactorily identified the damaged regions. The principal design deficiency was the low shear strength of the coupling girders. However, the nonlinear results indicated that had these girders been able to develop their flexural capacity they would have suffered substantial numbers of reversals and the shear walls would have been subjected to undesirable states of stress. The analytical results as well as the building's performance demonstrated that buildings with coupled shear walls combined with moment resisting frames offer excellent protection against seismic excitations, minimizing nonstructural damage while providing several lines of defense in the event of localized failure. Design and repair recommendations are offered.

## KEYWORDS:

reinforced concrete; buildings; earthquakes; dynamics; ductility; elastic analysis; plastic analysis; failure; shear walls; girders.

## INTRODUCTION

Although the Managua Earthquake of December 23, 1972 had a relatively moderate magnitude (surface wave  $M_s = 6.2$ ), it was particularly devastating because it occurred at shallow depth and four related surface faults extended through the city. Furthermore, within 50 minutes of the main shock two aftershocks occurred that were severe enough to cause additional structural damage (1). This earthquake was a very significant real test of the two most common structural systems that are currently used for reinforced concrete buildings; shear walls and ductile moment-resisting frames.

Preliminary studies of the earthquake damages (1,2) indicated that while buildings with properly detailed and constructed moment resisting frames could sustain large seismic excitations without collapse, several developed such large deformations

that they did not protect the contents of the building, made repair infeasible, and could also have endangered life safety. On the other hand, buildings with properly designed, detailed and constructed shear walls performed exceptionally well. The more symmetrical the plan of the building, and the greater the ratio of shear wall area to gross floor area, the better the building's performance.

Because seismic behaviour of shear wall buildings is not well understood, the Managua earthquake provided an excellent opportunity to investigate this behaviour. Thus, several shear wall buildings in Managua were included in a continuing program of investigation being conducted at Berkeley on buildings damaged during earthquakes.

## OBJECTIVES AND SCOPE

This paper summarizes the results obtained in an investigation of only one Managua building, the "Banco de America". The main objectives of this investigation

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were :- (1) to determine the probable cause of the observed damage; (2) to assess whether present analytical models and techniques could predict the damage observed in its shear wall structural system; (3) to identify the principal factors which can affect the seismic behaviour of coupled shear walls; (4) to recommend practical methods for repairing the structure, and (5) to suggest improvements in the aseismic design of similar structures.

#### BUILDING DESCRIPTION

The building is made up of two principal sections, a double level basement and an 18 story tower. The tower is the main concern of this study (Fig. 1). For a detailed description of the building see references 1 and 2.

Foundation: A 2-m. thick mat foundation is used to support the tower (Fig. 2).

Vertical Load Supporting System: Closely spaced perimeter columns and four large interior core shear walls support the tower floor slabs (Figs. 2 & 3).

Structural Floor System: Above ground 20-cm thick two way flat slabs are directly connected to the vertical supporting elements, except for girders which couple the core walls. The ground and basement floors have thicker slabs (Fig. 2).

Lateral Force Resisting System: The designer considered that all lateral forces were resisted by the coupled core walls and the perimeter columns were apparently intended only for vertical load support and architectural facade. However, these columns were attached to the exterior edge of the slab by connections that were capable of transferring both torsional and bending moments; and as such they resisted significant lateral forces by tube framing action. Even if shear lag would have reduced the efficiency of the tube action, the short column to core spans would have contributed additional framing action. Thus, the perimeter columns undoubtedly interacted with the coupled shear walls and contributed to the total resistance. Although each core wall could individually offer considerable lateral resistance, the initial lateral stiffness was increased tremendously by coupling the cores at each floor level with the floor slab and pairs of coupling girders as indicated in Figs. 2 and 4. All girders between the first and penthouse levels were pierced for ventilation ducts (Fig. 4).

#### SURVEY OF DAMAGES

Although one of the earthquake faults extended along the sidewalk adjacent to the building, only minor damage (1,2) occurred.

Nonstructural Damage: The damage to non-structural elements as well as to the contents of the building was very minor because the few nonstructural elements present were relatively flexible and properly attached to the structure; also they were not subjected to large deformations due to the high stiffness of the structural system. Although the elevators were inoperable, the stairs were completely free from debris.

Structural Damage: While the overall damage was very light, significant damage occurred in the coupling girders. These girders failed in shear where they were weakened by duct openings (Fig. 5(a)). This damage was concentrated in the girders oriented along the E-W axis of the building. Extensive diagonal cracking occurred in the deeper unpierced coupling girders of the 17th floor (Fig. 5(b)). The floor slabs were cracked where the coupling girders failed in shear, around the column-slab connections (Fig. 6) and at some of the slab-shear wall connections. No damage was observed in the columns, except for very slight tension cracks, which could have occurred due to shrinkage or temperature effects. The damage in the shear walls was very light, being visible only in the upper stories in the form of diagonal and horizontal cracks (Fig. 7) and vertical cracks over the doorways.

#### MECHANICAL CHARACTERISTICS OF THE STRUCTURAL ELEMENTS

To predict building performance it is necessary to know the mechanical characteristics (force-deformation relationships) of each of the different structural members and of their connections and supports. These have been estimated using the material properties and the structural details specified by the designers (1,2). Examples of some of these characteristics are given in Figs. 8 and 9. Because of uncertainties regarding the actual mechanical characteristics of the materials used and the quality of workmanship, bounds on the mechanical characteristics, rather than unique values, have been used for most of the elements.

#### STRUCTURAL CHARACTERISTICS OF THE BUILDING

Realistic prediction of the elastic or inelastic behaviour of a real structure using mathematical models depends upon the accuracy of the modelling. To achieve the desired accuracy, it is necessary to have a quantitative understanding of the contribution of each structural element to the overall structural stiffness and strength. A convenient way of estimating this contribution is by analyzing the elastic behaviour of several structural models under static forces representative of those expected during the actual response. These models were obtained in this case by removing from the total structural system those subsystems which, according to the results of analyses, the strength capacity estimates and the observed damages, might have suffered significant deterioration in their strength or stiffness.

For these analyses two types of frames were considered: the coupled core shear wall frame (Fig. 10(a)) and the periphery frame (Fig. 10(b)). Based on these frames several two and three-dimensional structural models were formulated and analysed (2) to determine their elastic response to the static loading and their dynamic characteristics (mode shapes and periods). While a complete discussion of these results is presented in Reference 2, some of the findings are summarized below:

(1) foundation deformations had little effect on the response; (2) despite a torsional mode occurring in the first 5 modes of the 3D response the translational degrees of freedom were virtually uncoupled; (3) as can be seen in Fig. 11 the failure of different structural elements had a significant effect on the dynamic characteristics of the building; viz: a) an increase in period from below 1 sec. to 1.35 sec. when all perimeter frame columns were ineffective; b) a similar increase when the coupling girders were removed; c) the simultaneous removal of elements in a) and b) increased the period to 3.3 secs. (an increase of over 200% from the initial value); and d) simultaneous removal of the coupling girders and columns of the periphery frame (Fig. 10(b)) increased the period to 2.4 secs. From the results of these analyses, the damage survey and in situ periods measured after the earthquake (1) the final condition appears to be close to the structural system described in (3) d) above with some further reduction in stiffness due to hingeing in the slab-column connection in the shear core frame (Fig. 10(a)). Dynamic response analyses were then carried out to allow comparison with this predicted behaviour.

#### EARTHQUAKE GROUND MOTIONS

The only ground motion record available was obtained at the ESSO Refinery (1), 5 km. away. Thus, analyses were carried out using the two horizontal components of this record. Some analyses were also made using the 1940 El Centro, N-S component record.

#### DYNAMIC LINEAR ELASTIC RESPONSE

Time history analyses of the response of the different models considered above were evaluated including the case of biaxial ground motions and foundation flexibility. However, bounds on the actual response can be directly estimated from the elastic response spectrum of Fig. 12. According to the period corresponding to the initial state of the structure,  $T \approx 1$  sec., and assuming completely elastic behaviour, the maximum inertia forces and displacements that would have been developed correspond to a pseudo-acceleration (PSa) of 0.34g and a spectral displacement (Sd) of 8.5 cm. However, these values could not have been developed because the real strength and ductility capacity of many structural elements would have been exceeded causing significant deterioration in their contributions and a corresponding increase in the building's period. The maximum probable elastic strength of the core walls was controlled by the shear cracking capacity which was estimated to be reached at a value of PSa  $\approx 0.10g$  for the initial elastic structural system. Under this magnitude of PSa most of the coupling girders would have failed, increasing the fundamental period to a value of  $T \approx 1.35$  secs. and significantly increasing the forces acting on the periphery frames. Under these circumstances the connections between perimeter columns and slab would have also begun to deteriorate at forces corresponding to PSa  $\approx 0.15g$ . If the stiffness and capacity for moment transfer of these connections would have disappeared

completely, the period would have increased further to  $T \approx 2.4$  secs. (Fig. 11). A structure with this period under the ESSO ground motion, even if higher mode effects are considered, could not have developed inertia forces significantly larger than that corresponding to PSa = 0.14g, with a maximum displacement of Sd = 21 cm. Thus, a considerable decrease in the effective inertia forces, without the building reaching excessive deformations, was possible due to the failure of several structural fuses built into the structural system.

#### NONLINEAR RESPONSE

The elastic analyses indicated that the framed tube had a substantial effect on the overall response of the building. However, the analyses and damage observations indicated that the final resistance of the tube had significantly deteriorated. Thus, it was decided to initially concentrate on the nonlinear response of the coupled shear wall cores acting alone and these results are summarized below. In the structural models used for the nonlinear studies (Fig. 13) the shear walls were treated as wide columns in which flexural and axial deformations were considered and bilinear, hysteretic yielding was controlled by the interaction curves shown in Fig. 9. Two bounds were considered for the possible nonlinear behaviour of the coupling girders: (1) a brittle failure based on estimates of the shear capacity of the beams as constructed (Fig. 8) and (2) a ductile inelastic flexural mode based on the flexural capacity of the coupling girders assuming unlimited flexural ductilities and shear strengths. All shearing deformations were disregarded. Gravity loads were considered to initialize the member forces and account for "P- $\Delta$ " effects. While more detailed discussions of the computer program and the nonlinear results may be found in Ref. 4, the general elastic and nonlinear behaviour of the coupled shear wall cores when subjected to the east component of the ESSO Refinery record are compared and summarized below.

Lateral Displacements: The maximum floor displacements and story drift indices for the model with the brittle coupling girders were generally more than twice those for the elastic and ductile inelastic models (Fig. 14). Thus, from the point of view of controlling damage caused by large drifts, brittle failure of the girders must be avoided. The maximum displacements for the ductile inelastic model were slightly less than those for the elastic response of the coupled wall or the entire building. As will be discussed later the ductile girders maintained sufficient coupling between the cores to prevent a large increase in the period and displacement.

Shear Distribution: The design shears were substantially lower than those indicated by the elastic or inelastic analyses as seen in Fig. 15. Elastic results obtained for the entire building indicate that the cores resisted most of the story shears and that, except for some higher mode effects, these values were similar to those resisted by the elastic coupled

shear wall model. The nonlinear results were generally smaller than the elastic values but still substantially larger than the design values indicating the conservative design of the shear cores. Even though the brittle girder model developed much higher displacements, the shears were generally smaller.

Overturning Moments and Axial Forces: The trends in the maximum story overturning moments (Fig. 16) are similar to those observed for shears. While the total overturning moments resisted by the brittle nonlinear model are less than those for the ductile inelastic or elastic models, more than 75% of the maximum overturning moments in the elastic and ductile inelastic models were resisted by the axial force couple produced by the coupled cores. Thus, the bending moments developed in the cores for the brittle nonlinear model were substantially higher than for the cases where coupling was maintained.

Because of the low shear capacity of the coupling girders used in the brittle nonlinear model, the maximum axial forces in the cores are very close to the gravity forces (Fig. 16). However, both the ductile inelastic and elastic analyses produced large variations in the axial forces, including substantial tensile loads. These fluctuations in axial force can play an important role in determining the behaviour of the shear cores and must be considered in their design. Tension reduces the flexural and shear capacities of the cores and may result in fracture of the longitudinal reinforcement (Fig. 9); compression reduces the available flexural ductility. To insure ductile behaviour in the walls it is necessary to limit the shears acting in the girders by adjusting the flexural capacity or length of the coupling girders.

Member Strengths and Ductilities: The moments acting in the coupling girders were very sensitive to higher mode effects. While Fig. 17 shows that for the ductile inelastic model the roof displacement and the axial load in the cores oscillate at nearly the fundamental frequency, the moments in the girders change much more rapidly (at the second and third mode frequencies). Thus, for each cycle of roof displacement, the girders may yield many times including a substantial number of reversals. In fact, Fig. 18 indicates that most of the girders suffer about 25 yield excursions (some more than 60) during only 10 seconds of the main shock record. This effect is a consequence of the structural system and the occurrence of a peak in the earthquake response spectrum coinciding with the second mode's period. Since yielding was controlled by the higher modes, all the girders seldom yielded simultaneously. Thus, the stiffness and period of the system and the core internal forces were similar to the elastic values.

The cyclic curvature ductilities required in the ductile girders are about 8 for all the girders except those at the first floor and roof (Fig. 18). The variations from floor to floor are due to differences in the stiffness and strength of the girders. These computed ductilities are

substantially higher than the overstress ratios (the maximum moment/flexural capacity) obtained from the elastic analyses for each girder. Careful design detailing is required if the effects of the rapid deterioration of strength and stiffness that accompanies inelastic reversals under high shear<sup>(3)</sup> are to be mitigated. Most of the girders in the brittle nonlinear model failed in shear very early in the response (Fig. 19) without yielding. Once these failures occurred the period and magnitudes of the displacements rapidly increased. In spite of the extensive yielding occurring in the ductile girder model, Fig. 19 indicates that its displacement response was similar to the elastic one.

While none of the cores yielded for the ductile inelastic model, the loss of coupling action for the brittle nonlinear girder model increased the moments acting in the cores sufficiently to induce yielding at the base and in story 12 (Fig. 18). In the actual structure the moments in the cores would have been reduced due to the incomplete loss of coupling and the additional restraint offered by the framed tube. Even if yielding had occurred in the cores for the case of brittle girder failure, the cores would not have failed because they had large ductility capacities. However, if the girders were ductile the cores would have probably failed had the ground motion been severe enough to induce yielding in them due to the detrimental effect of the large changes in the axial forces.

Additional research is in progress to investigate the effect of stiffness degradation, ground motion characteristics and shear wall-frame interaction on the response of these types of systems.

## CONCLUSIONS

Behaviour of Shear Wall Systems (SWS) vs Ductile Moment-Resisting Space Frame Systems (DMRSFS): (1) While properly detailed and constructed DMRSFS may prevent collapse, heavy nonstructural and structural damage may make repair infeasible, and endanger life. Similar buildings with properly designed and constructed SWS fared exceptionally well, generally without damage that could not be repaired. (2) The more symmetric the plan of the building and the greater the ratio of shear wall area to gross floor area, the better the building's performance. (3) The combination of coupled slender ductile shear walls with a ductile framed tube, like that used in the Banco de America, appears to be an excellent system to resist strong earthquakes since structural fuses incorporated in the design provide several defense lines by which the behaviour of the whole system can accommodate the demands of different kinds of severe earthquake ground motions. Such a system, when properly designed and detailed, can be more properly called Ductile Bending (rather than Shear) Wall-Frame System.

Banco de America - Cause of Observed Damage: (1) The main damage was caused by the shear weakness of the coupling girders. This was created by the combination of the

following factors: a) design - the flexural capacity was too high for the relatively short span and end conditions (a potential for high shear); b) detailing - inadequate shear reinforcement; and c) construction - duct opening. However, even without the duct openings the shear reinforcement was not sufficient to prevent a shear failure. Furthermore, had the reinforcement been designed according to present ACI (71) provisions, serious problems still could have developed because the shear corresponding to the flexural capacity was too high for the required ductilities and numbers of reversals. (2) The degree of damage observed around the slab-column and slab-wall connections would have been minimized if failure of coupling girders had been prevented.

Reliability of Present Techniques to Predict Seismic Behaviour: Because of the shear weakness of the coupling girders (brittle failure), their damage could have been predicted by comparing their estimated strength capacities with the forces obtained from a static elastic analysis of the structure using design response spectra or even the UBC. The minor damage observed in the walls and around the slab connections could similarly be traced using the dynamic characteristics of progressively degrading structural systems. Selection of these systems required a thorough estimation of the possible bounds of the strength and ductility capacities of the different structural elements and connections. It is doubtful that the overall behaviour could be traced from just one dynamic linear elastic time history. The possible inelastic deformation demands (ductilities and number of reversals) at the critical sections could not be estimated from the elastic analysis. Although present inelastic computer programs can give a good idea of such demands, there is need for further refinement.

Seismic Behaviour of Coupled Shear Walls: The mechanical characteristics of the coupling girders significantly affect the behaviour of this system. Very little girder stiffness is required to achieve substantial coupling of the walls. Ductile behaviour of these girders is desirable, thus, their shear capacity must be sufficient to allow inelastic flexural deformations without substantial loss of strength and stiffness even under large numbers of reversals. Increasing the flexural capacity of the coupling girders generally reduces the drift of the structure and the elastic moments induced in the walls. However, the larger the flexural strength of the coupling girders, the larger the change in axial forces induced in the walls. Although large compression can increase the wall's bending and shear capacities, it can reduce its flexural ductility. On the other hand, if tension axial forces are developed they will reduce the wall's bending and shear capacities. Furthermore, the larger the flexural strength of the coupling girders, the larger their shear, and this is not desirable since they may undergo a large number of reversals.

#### RECOMMENDATIONS

Repair of Structure: In view of the overall

success of the behaviour, it would appear reasonable to just restore the girders to their original form if the maximum earthquake expected during the remainder of the service life is similar to that which occurred on December 23, 1972. However, if earthquake motions with different characteristics (large pulses) could be expected, it is suggested: (1) to decrease the flexural strength of the coupling girders to avoid any tension in the cores; (2) to eliminate duct openings; and (3) to provide sufficient web reinforcement to protect against the maximum possible shear that can be developed and the degrading effect of the large number of reversals.

Improvement in the Seismic Design of Similar Structures: (1) Take advantage of the framed tube to develop larger stiffnesses by increasing efficiency of slab connections (perhaps adding ductile edge beams) and by designing them as DMRSF<sub>E</sub> (which may permit a reduction in the area of the cores); (2) Design the coupling girders with: a) the smallest stiffness necessary to obtain definite coupling (beyond this stiffness there is little further increase in the overall stiffness); b) flexural strength such that the maximum shear forces that can be developed will neither lead to nominal shear stresses larger than  $5\sqrt{f'_c}$  (3) nor induce large changes in the wall axial forces; and (3) Design should be based on the use of response spectra rather than present code provisions.

#### ACKNOWLEDGEMENTS

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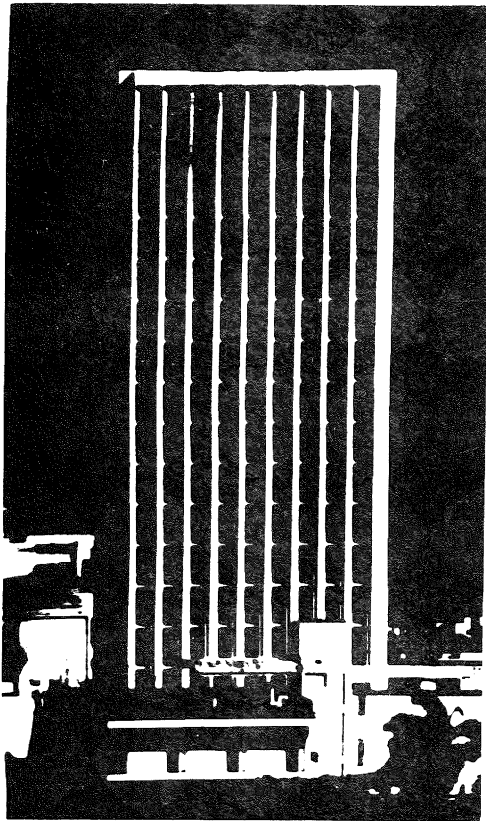


FIGURE 1: GENERAL VIEW

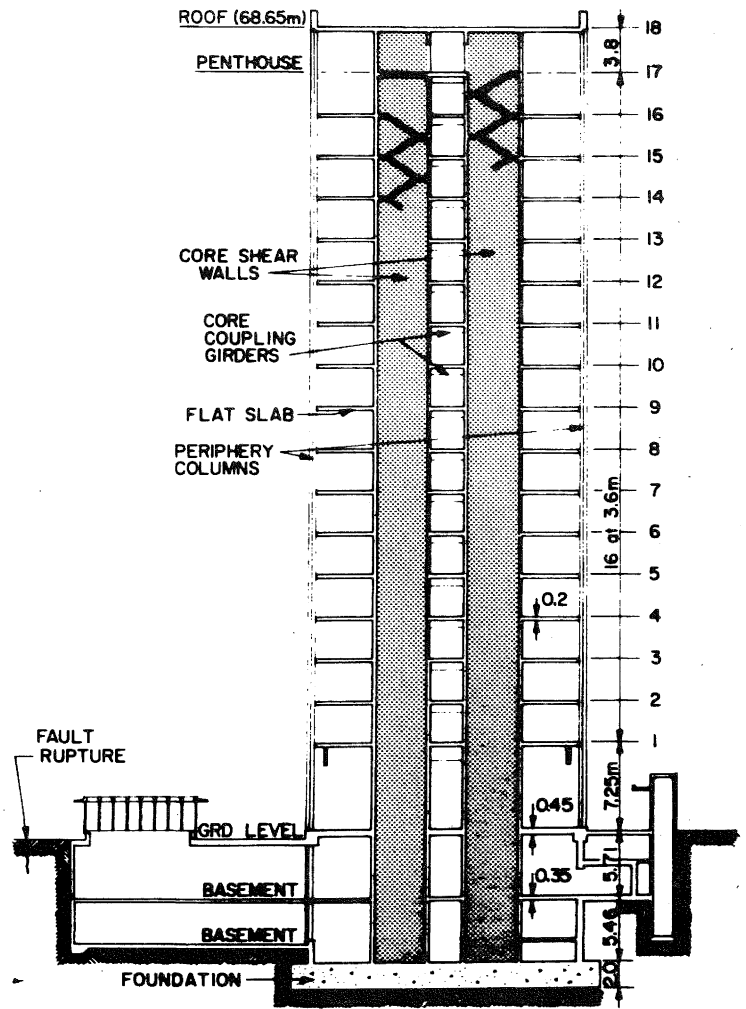


FIGURE 2: E-W SECTION THROUGH BUILDING

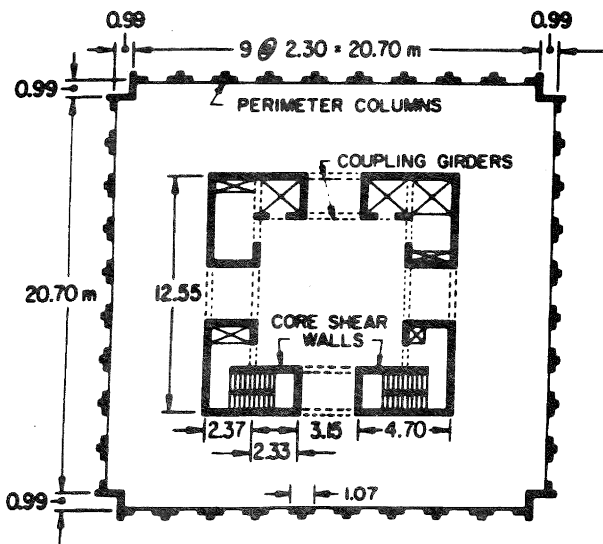


FIGURE 3: TYPICAL FLOOR PLAN

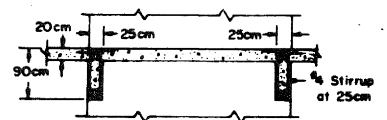


FIGURE 4a: TYPICAL SECTION OF CORE COUPLING GIRDERS

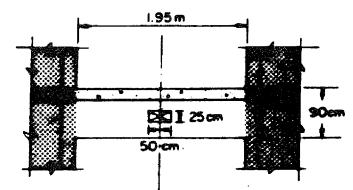


FIGURE 4b: OPENING IN CORE COUPLING GIRDERS

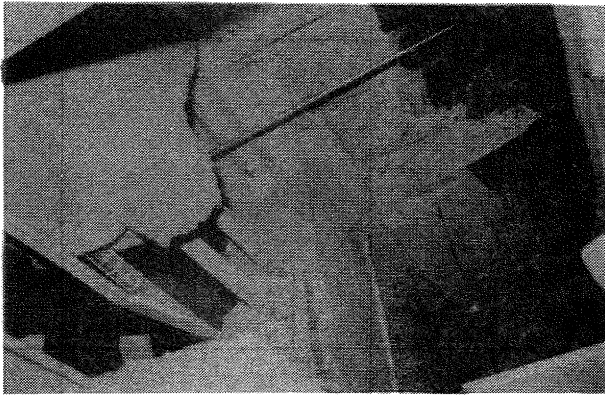


FIGURE 5a: SHEAR FAILURE OF TYPICAL COUPLING GIRDER



FIGURE 5b: SHEAR CRACKING IN PENT-HOUSE COUPLING GIRDER

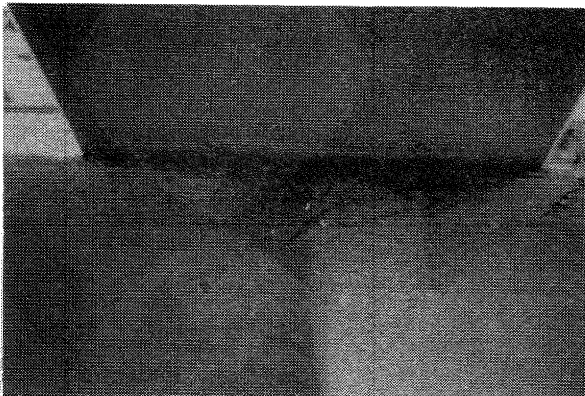


FIGURE 6: DAMAGE TO PERIMETER COLUMN SLAB CONNECTION

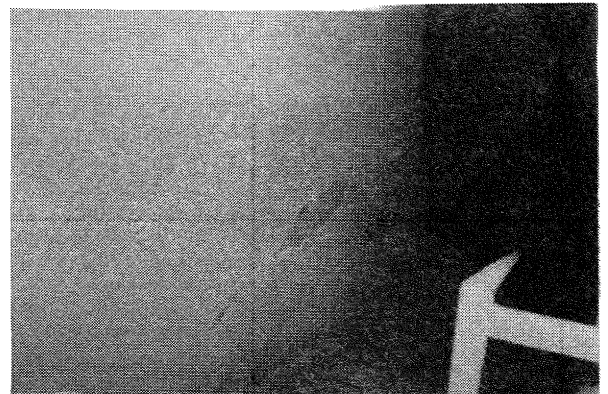


FIGURE 7: CRACKING IN CORE SHEAR WALLS

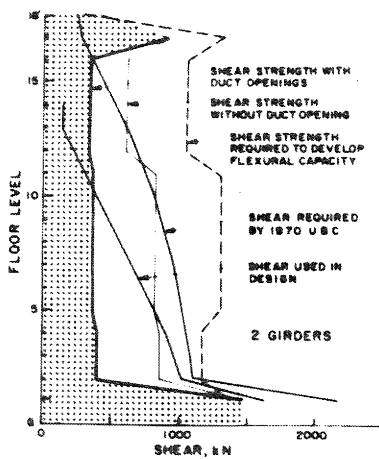


FIGURE 8: COMPARISON OF SHEAR STRENGTH OF COUPLING GIRDERS WITH DESIGN REQUIREMENTS

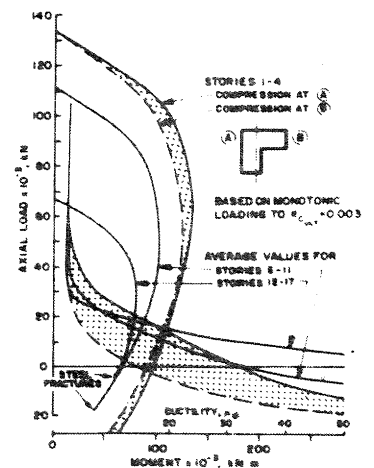


FIGURE 9: ESTIMATED STRENGTH AND DUCTILITY OF CORE SHEAR WALLS

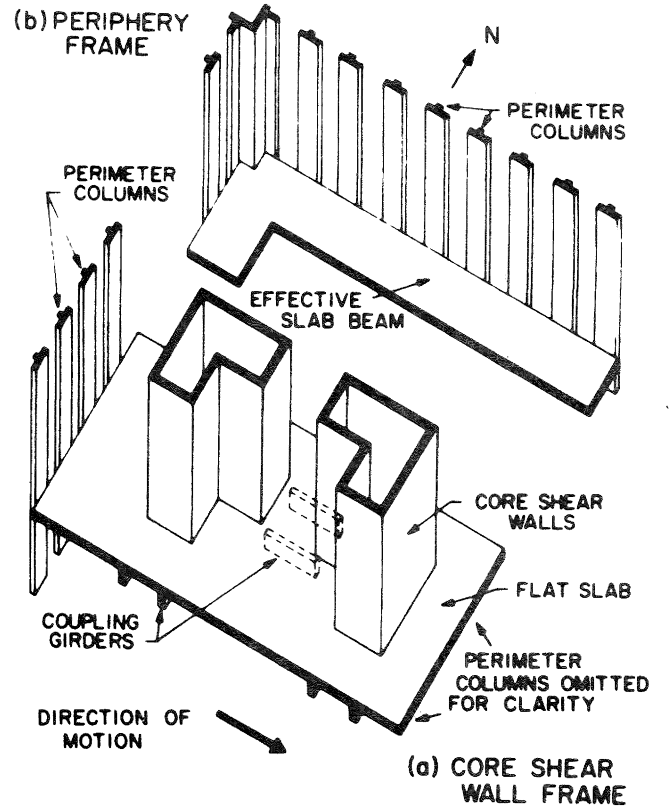


FIGURE 10: FRAME MODELS FOR ELASTIC ANALYSES

| MODE | VALUE OF THE PERIOD IN SECONDS FOR |       |          |  |                       |                  |                                  |   |   |
|------|------------------------------------|-------|----------|--|-----------------------|------------------|----------------------------------|---|---|
|      | WHOLE STRUCTURE WITH FOUNDATION    |       |          | 2 DIMENSIONAL MODEL (RIGID FOUNDATION)-EW MOTION<br>NEGLECTING FRAMING ACTION OF |                       |                  |                                  |   |   |
|      | RIGID                              |       | FLEXIBLE | COLUMNS ON N & S FACES   | ALL PERIPHERY COLUMNS | COUPLING GIRDERS | COUPLING GIRDERS AND ALL COLUMNS | COUPLING GIRDERS AND COLUMNS ON N & S FACES | COUPLING GIRDERS AND SLAB ALONG N-S FACES |
|      | 3 DIM                              | 2 DIM | 2 DIM    |  |                       |                  |                                  |   |   |
| 1    | 0.984                              | 0.983 | 1.161    | 1.220  | 1.350                 | 1.347            | 3.300                            | 2.400                                       | 2.340                                     |
| 2    | 0.256                              | 0.252 | 0.269    | 0.277  | 0.285                 | 0.364            | 0.532                            | 0.484                                       | 0.474                                     |
| 3    | 0.128                              | 0.125 | 0.134    | 0.132  | 0.134                 | 0.167            | 0.228                            | 0.194                                       | 0.190                                     |
| 4    | 0.083                              | 0.081 | 0.086    | 0.085  | 0.085                 | 0.097            | 0.126                            | 0.107                                       | 0.105                                     |
| 5    | 0.046                              | 0.059 | 0.063    | 0.061  | 0.062                 | 0.066            | 0.082                            | 0.071                                       | 0.069                                     |

FIGURE 11: NATURAL PERIODS FOR DIFFERENT STRUCTURAL MODELS

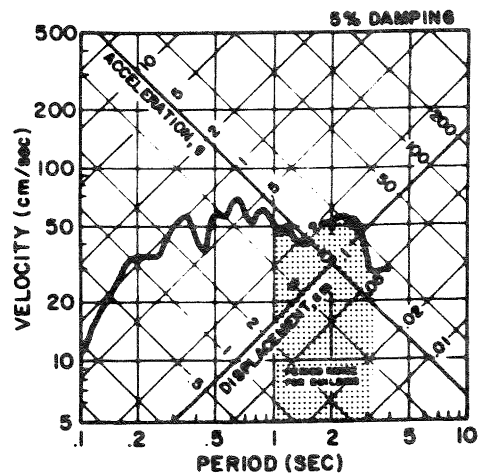


FIGURE 12: RESPONSE SPECTRUM FOR ESSO REFINERY ACCELEROGRAM - EAST COMPONENT OF MAINSHOCK

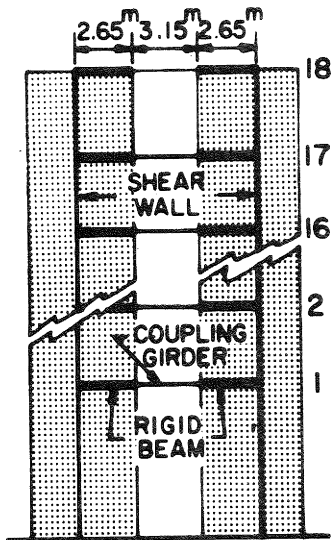


FIGURE 13: STRUCTURAL IDEALIZATION FOR NONLINEAR ANALYSES

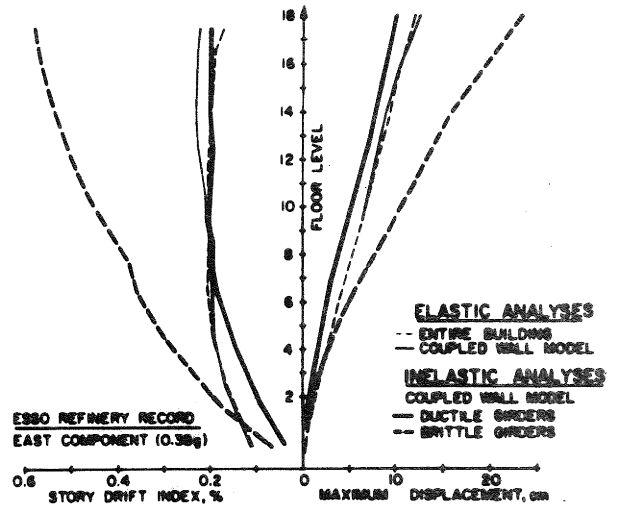


FIGURE 14: ENVELOPES OF MAXIMUM FLOOR DISPLACEMENTS AND STORY DRIFTS - COUPLED SHEAR WALL MODELS

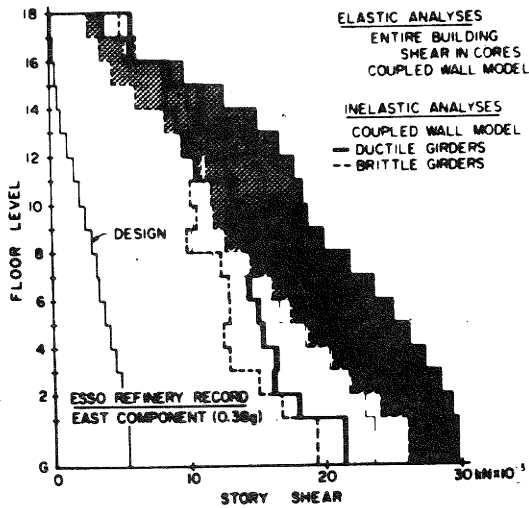


FIGURE 15: ENVELOPES OF MAXIMUM STORY SHEARS - COUPLED SHEAR WALL MODELS

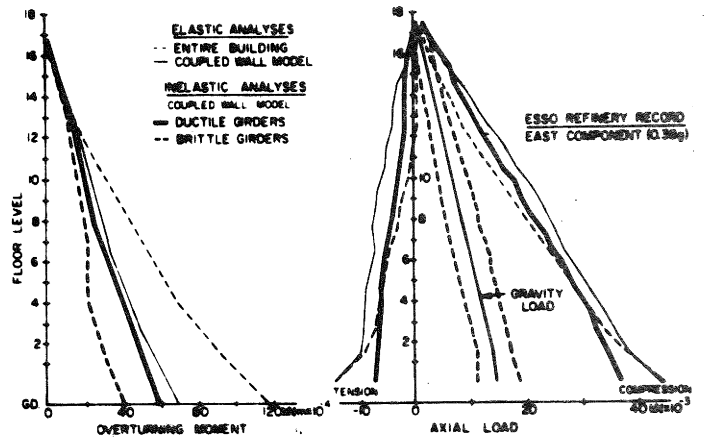


FIGURE 16: ENVELOPES OF MAXIMUM OVERTURNING MOMENTS AND CORE AXIAL FORCES - COUPLED SHEAR WALL MODELS

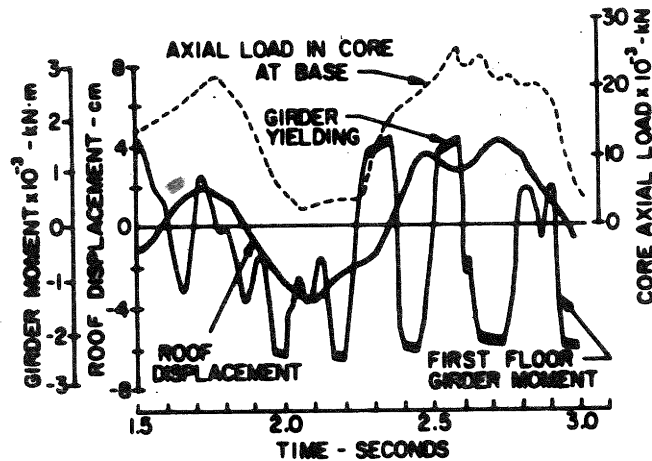


FIGURE 17: RESPONSE TIME HISTORIES FOR NONLINEAR MODEL WITH DUCTILE COUPLING GIRDETS

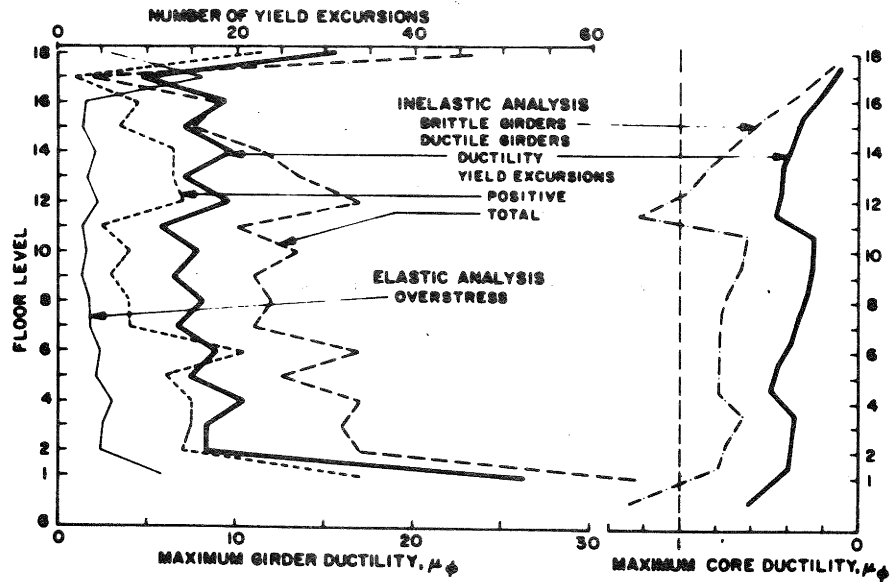


FIGURE 18: MEMBER DUCTILITIES FOR COUPLED SHEAR WALL MODELS

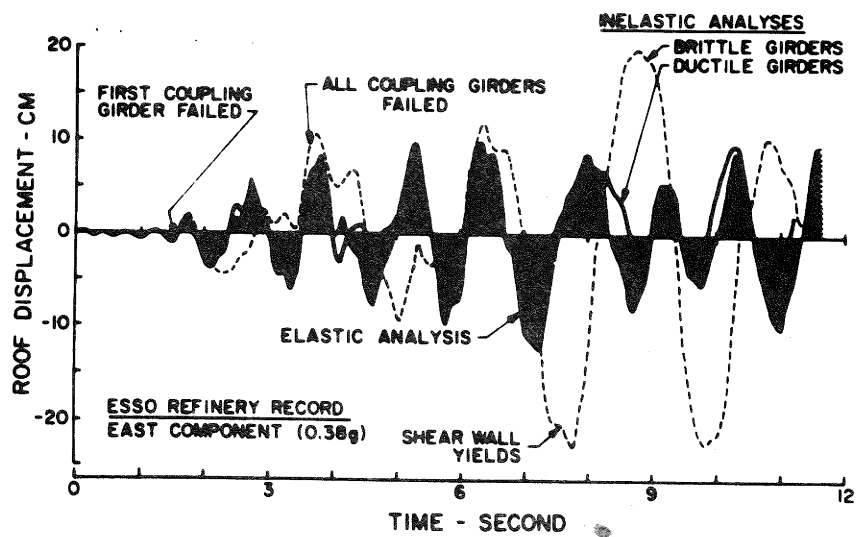


FIGURE 19: TIME HISTORY OF ROOF DISPLACEMENTS FOR COUPLED SHEAR WALL MODELS