Case Study of the Performance of Prestressed Concrete Buildings During the 1985 Mexico Earthquake

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The overall effects of the 1985 Mexico earthquake on buildings are summarized, with specific consideration of the performance of prestressed concrete buildings. Then, for five typical prestressed concrete buildings, results of analyses of the dynamic response, with due consideration of the soil-structure interaction, are presented. In general, the computed response of the buildings under the effect of a ground motion simulating the 1985 earthquake, corresponded reasonably well with their observed performance. Nevertheless, in some cases the analyses indicated that the buildings should have experienced a greater nonlinear behavior than the ones perceived from their level of damage. Some reasons for these differences are discussed. Recommendations on earthquake-resistant design of prestressed concrete buildings are given. The importance of providing lateral stiffness by shear walls or bracing, and of achieving ductility and continuity through mild steel reinforcement, is emphasized.

Scarce evidence is available on the performance of prestressed concrete buildings subjected to severe earthquakes. The few well-documented cases of failures are related to gross errors in the conceptual design of the structural system and, mainly, to poor connections between precast concrete members.[1] Specifically, there are very few known cases of detailed analyses of prestressed concrete buildings in which their observed behavior under severe earthquakes was compared to that computed according to well-established principles regarding the seismic response of buildings.

The 1985 earthquake in Mexico City constituted a severe test in the ability of building structures to withstand seismic forces. Therefore, it offered a unique opportunity to evaluate current design practice and construction methods. Although not many multistory prestressed concrete structures existed in Mexico City, a few dozen buildings in the range from 4 to 12 stories had prestressed frame structures. Indeed, some of these buildings were located in the most severely affected part of the city and were subjected to very intense ground shaking.

The object of this paper is to first present an overview of the performance of prestressed concrete construction in Mexico City during the September 19, 1985, earthquake, and then to show the results of a case study of five buildings whose structural drawings could be obtained. Although none of these buildings fully complied with modern code requirements for earthquake resistance, they survived the earthquake with minor or no damage. Several practical recommendations, regarding different aspects of the earthquake-resistant design of prestressed concrete structures, are drawn from these evaluations.

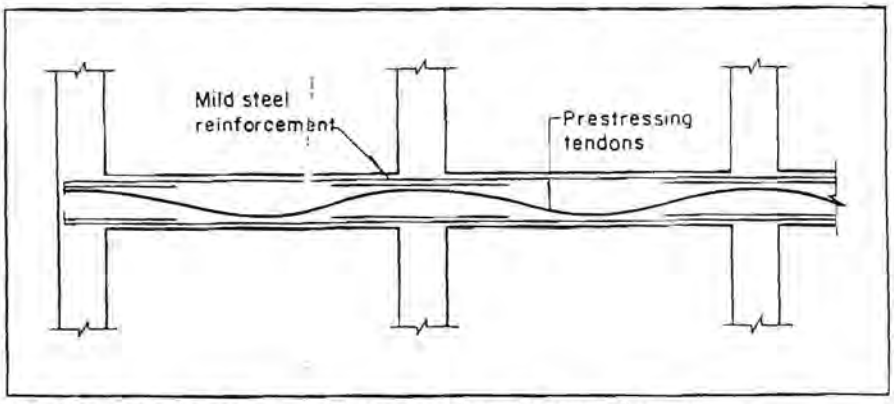

Fig. 1. Cast-in-place frames with post-tensioned concrete beams.

ASSESSMENT OF EARTHQUAKE DAMAGE

The earthquake of September 19, 1985, with a magnitude Ms = 8.1 (Richter Scale), originated near the Pacific Coast of Mexico and was felt with extraordinarily large intensities in some parts of the Valley of Mexico, approximately 400 km (250 miles) from the epicenter, causing the collapse or severe damage of many buildings.

The intensity of the ground motion varied considerably throughout the Mexico City area. Peak ground accelerations were less than $0.04g$ in sites of firm soil, while they reached $0.20g$ in some parts of the bed of an old lake which contains very deformable clay deposits. Unfortunately, this is also where the most populated part of the city is located.

A description of the main structural aspects of this earthquake can be found elsewhere.[2] Briefly, the long duration ground motion with prevailing long periods mostly affected multistory flexible buildings, whereas low-rise stiff buildings, even those apparently rather weak, suffered very little damage.

Reinforced concrete frame buildings of more than five stories were the most damaged structures. They had been typically designed for a base shear coefficient of 0.06 and, in most cases, the reinforcement in members and joints had not been detailed with the current strict code requirements for ductile frames.

Failures were mainly due to shear or eccentric compression in columns and to bond or shear in joints. Irregularities in the structural scheme, such as lack of in-plane symmetry and discontinuity or sharp changes in stiffness of structural members, frequently contributed to the failures. Hammering with adjacent buildings and excessive rocking of the foundations were other sources of damage and collapse.

As a result of the damage evaluation following the earthquake, design forces for seismic resistance have been significantly increased in the new Mexico Building Code, which also imposes stringent requirements for ductility in concrete structures.

PERFORMANCE OF PRESTRESSED CONCRETE BUILDINGS

Detailed evaluations of the performance of prestressed concrete buildings were published a few months after the earthquake.[3,4] Additional comments given here will augment the earlier reports.

Within the large variety of construction systems, including prestressed concrete members, that have been used for buildings in Mexico City, two major types can be distinguished:

1. Cast-in-place concrete frame structures, with beams that are post-tensioned in at least one direction (those with largest spans) — Continuity is provided by mild reinforcing steel and by draped prestressing tendons (see Fig. 1). As a variation of this system, flat plate or waffle slab floor systems are post-tensioned in a similar way.

2. Structures with cast-in-place concrete columns and precast, prestressed concrete beams of different shapes — Joint continuity is provided by a cast-in-place concrete topping with mild steel reinforcement (see Fig. 2). Frequently, long span prestressed concrete beams in one direction are combined with non-prestressed reinforced

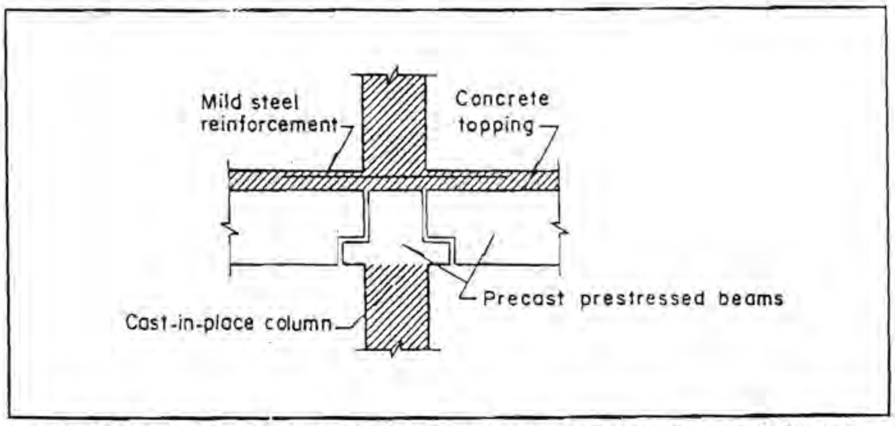

Fig. 2. Continuity for negative moments in precast, prestressed concrete frames.

concrete beams, of shorter span, in the other direction.

Most prestressed concrete buildings were of mid-rise construction, between four and eight stories. In some cases, the buildings were stiffened by shear walls, but more often frames were required to resist the total lateral force.

Some of the collapsed or severely damaged buildings contained precast structural elements. In most cases, the precast concrete members were part of the floor slab supported by cast-in-place reinforced concrete beams on column lines. Therefore, monolithic reinforced concrete frames constituted the main lateral load-resisting system. A rather thin reinforced concrete topping was typically cast over the precast floor to provide for in-plane continuity. Quite often, this type of floor showed signs of in-plane distortion, indicating that an effective diaphragm action was not achieved.

Efforts were made to retrieve the structural drawings of the five prestressed concrete buildings which, according to Fintel's evaluation,[3] had suffered severe damage or collapse. Only for one of these buildings could some information about the structural design be obtained.

This building was a six-story hospital with cast-in-place columns. The floor system was composed of precast T-beams. The beams on the column lines in both directions had a precast soffit with ducts for straight post-tensioned tendons. Mild steel reinforcement for positive and negative bending was placed inside stirrups protruding from the precast soffit. A practically monolithic frame was obtained in this manner.

The scarcity of available information did not allow a detailed evaluation of the seismic response and safety of this building. Therefore, it could not be included in the case studies. Nevertheless, approximate computations showed that the shear and bending strengths of the columns in the longitudinal direction were clearly insufficient to resist the forces induced by ground motion.

It can be concluded that, in this building — as it was in the large majority of the damaged reinforced concrete structures — the collapse was due to the weakness of the columns and was not related to the behavior of the precast or prestressed members nor to their connections to the columns. This conclusion is confirmed by the inspection of the remains showing intact beam-to-column connections.[3]

It can be argued, nevertheless, that the lack of thorough in-plane stiffness of the precast floor could have caused the concentration of shear forces in some of the column lines, thus somehow contributing to the failure.

The remainder of the prestressed concrete buildings did not suffer any significant structural damage, even those located in the most severely affected area. Nevertheless, several of the buildings showed signs of excessive lateral displacements, resulting in non-structural damage such as cracking of partitions and distortion and falling of ceilings. Some very slender buildings, founded on friction piles over very soft clay soil, suffered significant base displacements and rotations, and even some residual tilting.

CASE STUDY OF SPECIFIC BUILDINGS

A search for detailed information on the structural design of prestressed concrete buildings was undertaken to be able to fully evaluate whether their performance during the earthquake could be explained through analytical computations. About 20 prestressed concrete buildings were identified, but only for five of them could complete structural drawings be obtained. These five structures were analyzed in detail and the results are reported here.

Only one of these buildings was located in the area of heaviest damage. Three buildings were founded on soft soil deposits of moderate thickness, in the so-called transition zone, where the intensity of the earthquake was still high but less severe than in the lake area. The last building was located in an area of firm soil where the ground shaking was minor and no damage occurred.

Each building was inspected to detect damage caused by the earthquake, to check its properties against those reported in the drawings and to detect construction defects. The vibrations of the five buildings under normal conditions were measured to determine their dynamic properties and to calibrate the analytical models used to compute the theoretical response. The technique used for these so-called ambient vibration tests will be described in the next section.

Methodology

The methodology used in this study was similar to that applied to other types of buildings in evaluating the design methods and the required provisions of the building code for earthquake-resistant design. An evaluation of the performance of concrete and of masonry buildings can be found elsewhere (see Refs. 5 and 6, respectively). A detailed presentation of the evaluation of prestressed concrete buildings, summarized in this paper, can be found in the doctoral dissertation of the first author.[7]

Analytical models with different levels of sophistication were used to study the seismic response of the five buildings. Linear response was investigated using a three-dimensional model, with proper consideration for the stiffness of the joints. The computer program SUPERETABS was used for this purpose.

To take into account the relative deformations between the structure and the soft soil where it was founded, an artificial story was added at the bottom of the structure. In this story, the axial and lateral stiffnesses of columns were determined to reproduce those of equivalent springs representing the rotational and translational stiffness of the surrounding soil.[8] A stiff diaphragm was assumed to connect all the members at each floor.

The buildings were modeled as a frame structure where the contribution of the slab to the lateral stiffness was included in the moment of inertia of the equivalent beam. Proper considerations were made for the effect of masonry infill walls through equivalent diagonal members.

From the linear analyses, the dynamic properties of the building, i.e., vibration periods and modal shapes, were first determined and compared to those measured in the ambient vibration tests. The dynamic response of

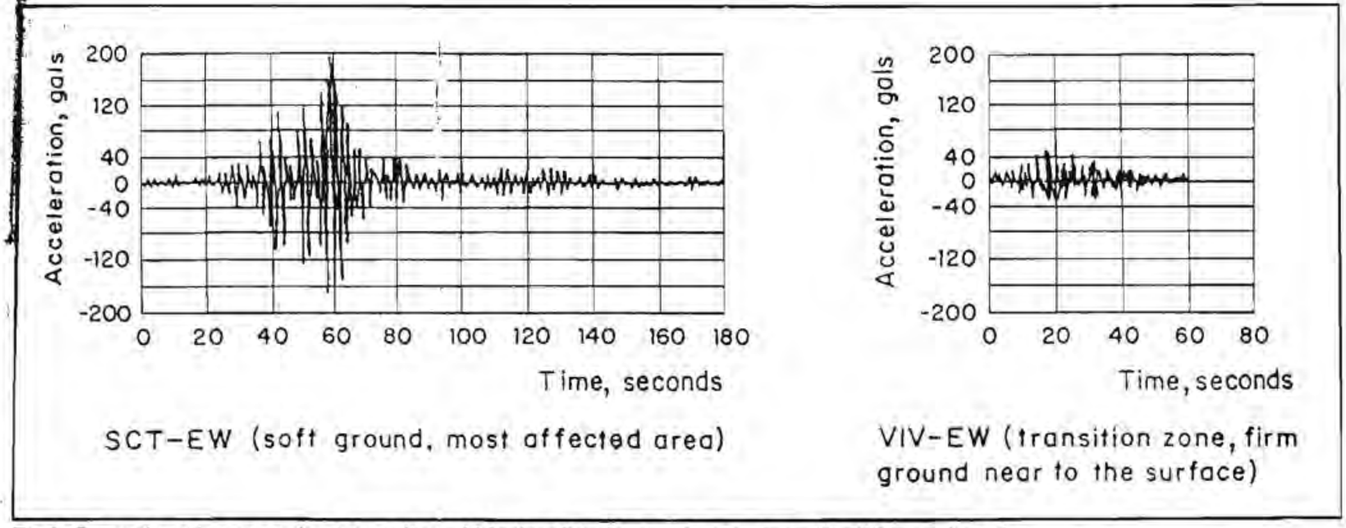

Fig. 3. Ground motion records at two sites of Mexico City for the September 19, 1985 earthquake.

each building to ground motions recorded in the 1985 earthquake for soil conditions similar to those of the sites was computed using a step-by-step dynamic response analysis.

From the available records of the 1985 earthquake, the SCT-EW record was selected as the most representative for the lake bed area and the VIV-EW for zones of firmer soil. Nevertheless, it must be taken into account that the characteristics of the ground motion varied considerably in each zone. Therefore, most buildings were analyzed for more than one ground motion to find bounds for the response. The accelerograms of the two previously mentioned records are shown in Fig. 3. The large differences in maximum amplitude, duration and frequency content between the records can be observed.

Lateral displacements and internal forces induced by the selected ground motions were computed and compared with those that could be resisted by the building. For this type of linear analysis, no specific difference was made between the model typically adopted for a non-prestressed, reinforced concrete frame and that corresponding to a prestressed concrete building.

Despite some evidence indicating that prestressed concrete structures have lower damping ratios than those of non-prestressed, reinforced concrete structures, the same damping ratio was assumed for both cases, i.e., 5 percent. It was assumed that the major source of damping in buildings is the friction between structural and non-structural members; therefore, the difference in damping between prestressed and non-prestressed concrete should not be significant. The validity of this assumption is discussed in the next section, based on the results of the ambient vibration tests.

For buildings in which the linear analyses indicated that the theoretical capacity of some structural members should have been exceeded for the ground motion considered, nonlinear analyses were performed on a planar model to ascertain the amount of inelastic behavior that should have occurred.

Several models were considered to represent the nonlinear behavior of a prestressed concrete member. The classic elasto-plastic model [see Fig. 4(a)] was compared to a stiffness degrading model [see Fig. 4(b)] and to the S-shaped model that has been proposed for prestressed concrete members [see Fig. 4(c)].

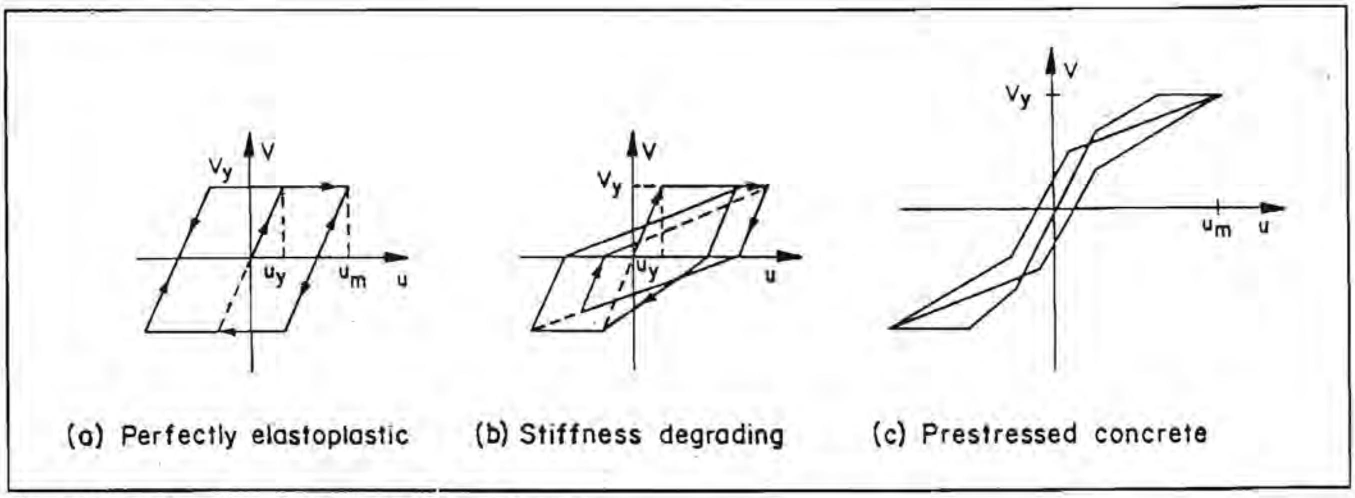

Fig. 4. Model of nonlinear behavior under cyclic loading.

The response of one-degree-of-freedom systems with the three models of nonlinear behavior to different ground motions has been studied. For the El Centro record, the difference in the maximum displacements (and the ductility demand) was negligible between the perfectly elasto-plastic and the stiffness degrading models. The response was significantly larger for the S-shaped model. Nevertheless, for systems with periods in the range between 1 and 1.5 seconds, the difference did not exceed 25 percent.

Analyses performed for the type of ground motion recorded in the soft soil of Mexico City indicated that, for periods larger than 1 second, the difference among the results for the three models was smaller than that obtained for the El Centro record. Since none of the buildings studied had shown significant evidence of inelastic behavior, the nonlinear analyses were intended to provide only an approximation to the theoretical inelastic deformation. Therefore, it was decided to use the elasto-plastic model because it required the least amount of computer time.

Another important simplification in the nonlinear analyses was the use of a planar model, which neglected the three-dimensional behavior of the building. A representative frame was selected for each orthogonal direction. A portion of the total mass of the building was assigned to this frame proportional to its relative stiffness. This simplification allowed the use of the widely known computer program called DRAIN-2D. Since the layout of the buildings was symmetrical, the planar system gave a reasonable estimate of the overall response.

In computing the strength of the members, the usual assumption of compatibility of deformations was used. All partial safety factors were eliminated. For beams, the yielding moment was used, and for columns, the complete interaction diagram for axial force and moment was defined by linear segments connecting three key points. Nominal values for the moduli of elasticity and the gross dimensions of the sections were assumed for the stiffness computation.

In a first stage computation, the nominal strengths of the structural members selected were those normally used in design practice. In a second stage computation, the expected (average) strengths of structural members were used to estimate the most probable response of the buildings. Therefore, average material strengths and ultimate member capacities were used instead of specified values. Also, the contribution of the slab reinforcement to the flexural strength of the member was considered.

AMBIENT VIBRATION TESTS

The availability of high sensitivity accelerometers has allowed the measurement of very small vibrations experienced by buildings under normal operating conditions. Traffic, wind and micro-seismic activity produce small amplitude vibrations which can be recorded by suitably distributed instruments. Using this technique, the major dynamic properties of buildings, such as periods, shapes for the first modes and damping coefficients, can be determined.

In the lake zone of Mexico City, ambient vibrations are greatly amplified due to the flexibility of the soil; thus, free-noise signals are obtained. The technique has been widely used to determine dynamic properties of structures for different purposes, i.e., to check the validity of theoretical computations, to ascertain the influence of some specific factors such as the soil-structure interaction, or to evaluate the effectiveness of rehabilitation schemes in increasing the lateral stiffness of the building. The signals of one or more sensors are recorded, filtered and amplified, and their power spectra are computed through a spectrum analyzer. Typically, a large sample of measurements is taken and average spectra are used as results.

Measurements were taken in the five prestressed concrete buildings studied. A detailed description of the techniques and results is found in Ref. 9. The main objective of these measurements was to validate the analytical models used in this study. Results are summarized in Table 1. As an example, in Fig. 5 the acceleration spectrum for the vibration in the transverse direction of the QRO Building is shown. At least three modal frequencies can easily be identified. In Fig. 6, the shapes of the first three modes in the transverse direction are shown for the same building. The translation and rocking of the building at the base can be clearly observed.

A study of the results shown in Table 1 depicts that, in general, the fundamental periods of the buildings are greater than those that would be desirable, at least for the direction with no infill walls. This indicates that the structural system adopted is very flexible and that large lateral displacements can be expected under seismic effects.

A good agreement is found between measured and computed periods, especially when the deformations at the base of the building are taken into account. The average error for the first translational mode is 1 percent and the coefficient of variation is 2 percent. Only for the buildings on soft soil were the vibration periods and the lateral displacements significantly increased due to the deformations (translation and rocking) at the base of the building.

It can be concluded that the model and the dynamic properties assumed for the analysis were adequate to represent the dynamic responses of the buildings. Nevertheless, it must be appreciated that for larger amplitudes of vibration, as those induced by severe earthquakes, the level of stresses will be greater, the structural stiffness lower and, therefore, the vibration periods longer. This consideration tends to indicate that the model adopted overestimates the actual stiffness of the structure under severe ground motions.

Damping coefficients were computed from vibration records. A significant variation was found in different records for the same buildings. Therefore, for some cases, instead of a single value, a range of variation of the damping coefficients is given in Table 1. The overall range is between 2 and 5 percent, and it is very similar to that obtained from measurements in reinforced concrete buildings.

Since structural damping increases with the amplitude of vibration, it can be concluded that the 5 percent damp-

Table 1. Comparison of computed and measured periods for five prestressed concrete buildings.

Building identification	Type of soil	Number of stories	Type of structure	Direction of measurement*	Computed period (seconds) Fixed	Computed period (seconds) Base displacement	Measured period (seconds)	Damping coefficient (percent of critical)
QRO	Soft	11	Post-tensioned beams (cast-in-place)	T L Θ	1.16 0.59 0.63	1.34 0.78 0.64	1.39 0.83 0.78	4-5
TAM	Transition	10	Post-tensioned flat plate (cast-in-place)	T L Θ	0.72 0.38 0.24	0.77 0.45 0.26	0.73 0.45 0.30	3
TAC	Transition	5	Post-tensioned beams (cast-in-place)	T L Θ	0.97 0.21 0.36	1.00 0.22 0.37	1.00 0.21 0.44	—
IMP	Transition	5	Prestressed columns and beams (precast)	L T Θ	0.50 0.77 0.52	— — —	0.43 0.78 0.52	2-3
SMO	Firm	7	Prestressed columns and beams (precast)	T	0.54	—	0.54	2

* T = transverse (short) direction. L = longitudinal direction. Θ = rotational, in-plane vibration.

ing ratio assumed in the analysis is a reasonable estimate of what can be expected for moderate earthquakes — and it is probably conservative for very severe ground motions. On the other hand, the fact that similar damping coefficients are obtained for reinforced and prestressed concrete buildings does not mean that the same result will be obtained for large amplitudes of vibration. It is expected that reinforced concrete structures will show greater damping when subjected to high levels of stresses, due to the energy dissipation provided by opening and closing of flexural cracks and to more stable and fat hysteresis loops for well detailed members.

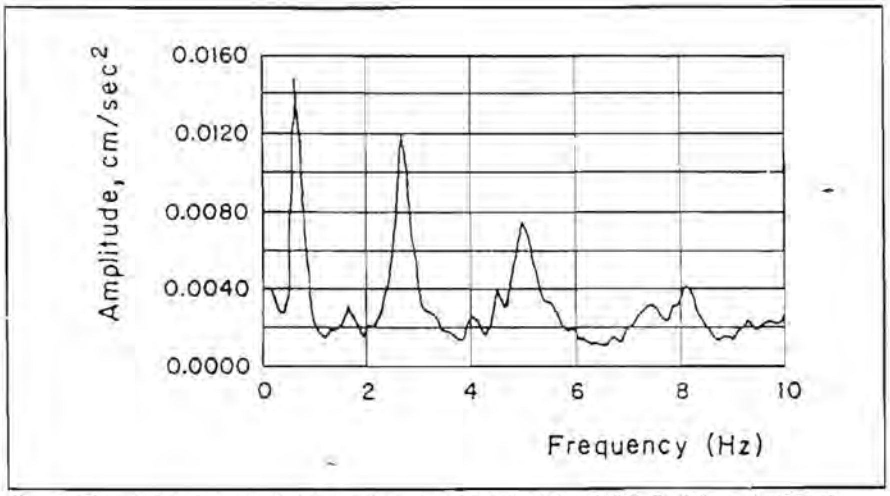

Fig. 5. Power spectrum of the motion at the roof of the QRO Building. Ambient vibration test in the transverse direction.

QRO BUILDING

This is a 10-story building located in the most severely damaged area of the city. Several buildings collapsed in a radius of 200 m (656 ft) from this structure. Its plan is relatively small and elongated, with one bay in the short (transverse) direction and five bays in the longitudinal direction (see Fig. 7).

The structure is cast-in-place with reinforced concrete columns and post-tensioned beams with grouted prestressing tendons. Mild steel reinforcement provides additional continuity at beam-to-column joints. All the bays of

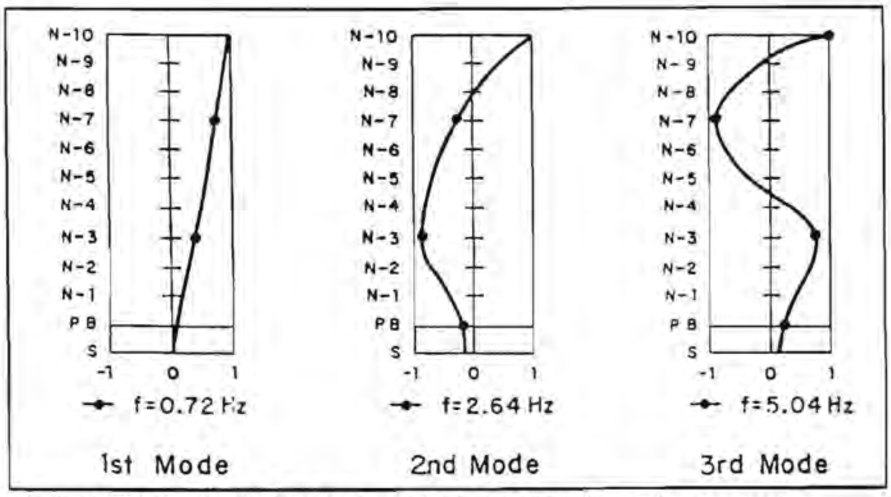

Fig. 6. Modal shapes of the vibration of the QRO Building in the transverse direction.

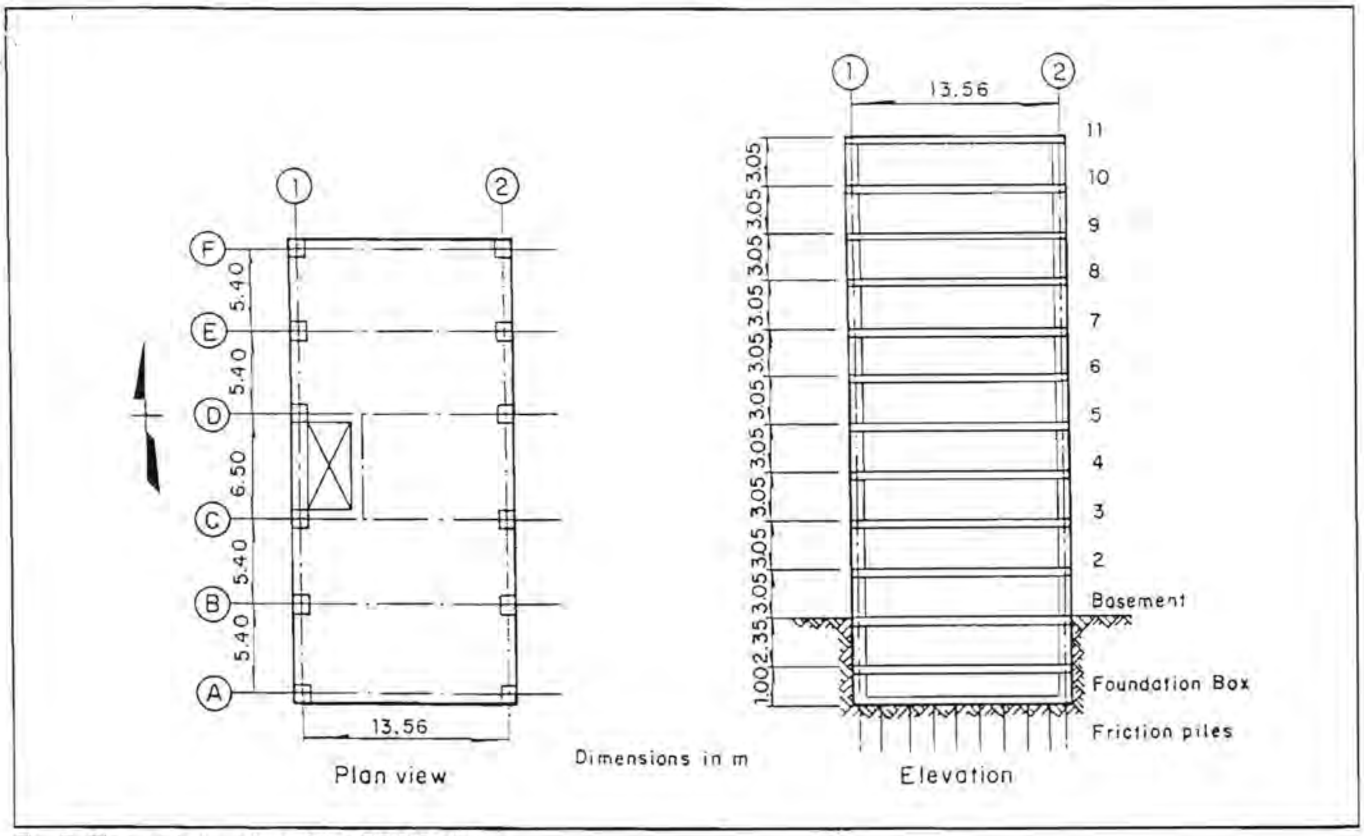

Fig. 7. Plan and elevation of the QRO Building.

the end frames in the longitudinal direction are infilled with brick walls, which provide a significant contribution to the lateral stiffness. Masonry walls also enclose the elevator and staircase shafts. The building is founded on friction piles beneath a basement and a foundation box. Typical reinforcing details of structural members are shown in Fig. 8.

An inspection of the building after the earthquake revealed extensive cracking in the masonry walls around the elevator shaft and distortion and falling of some ceiling panels. No

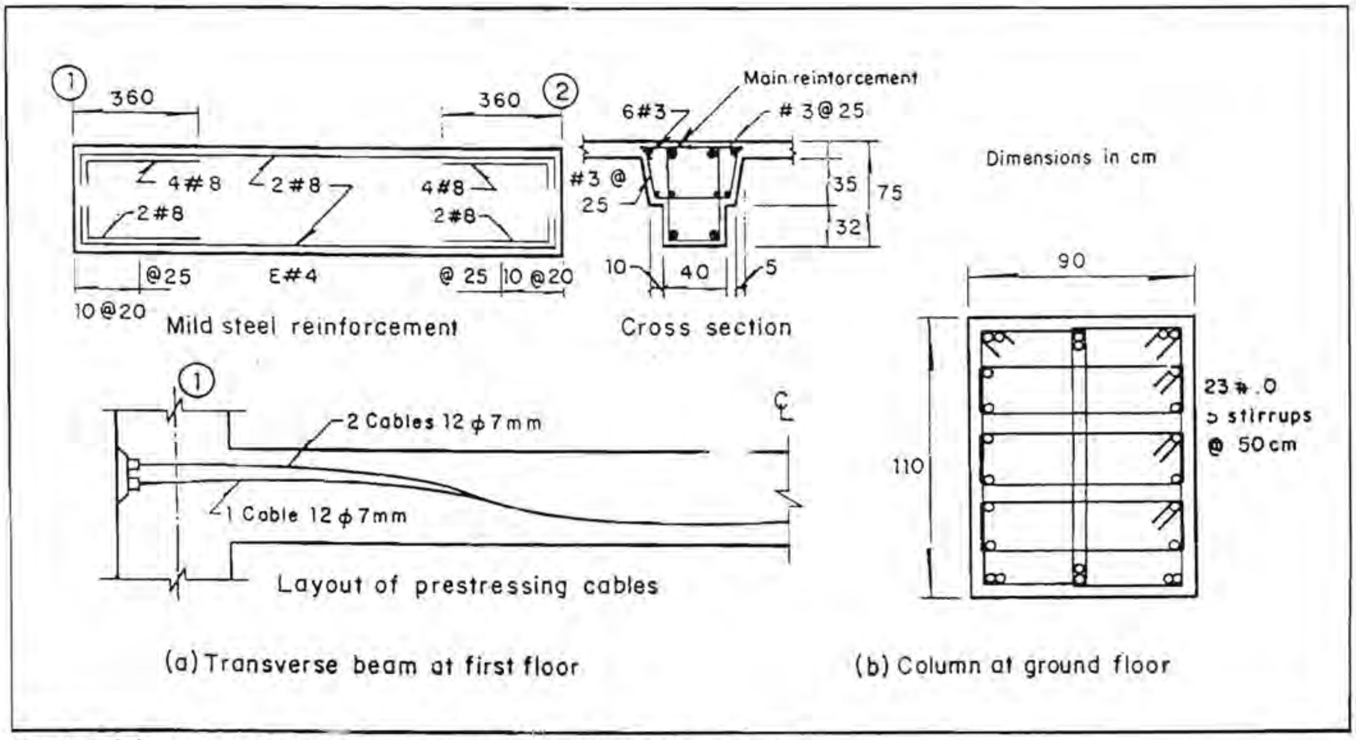

Fig. 8. Reinforcement details of structural members of the QRO Building.

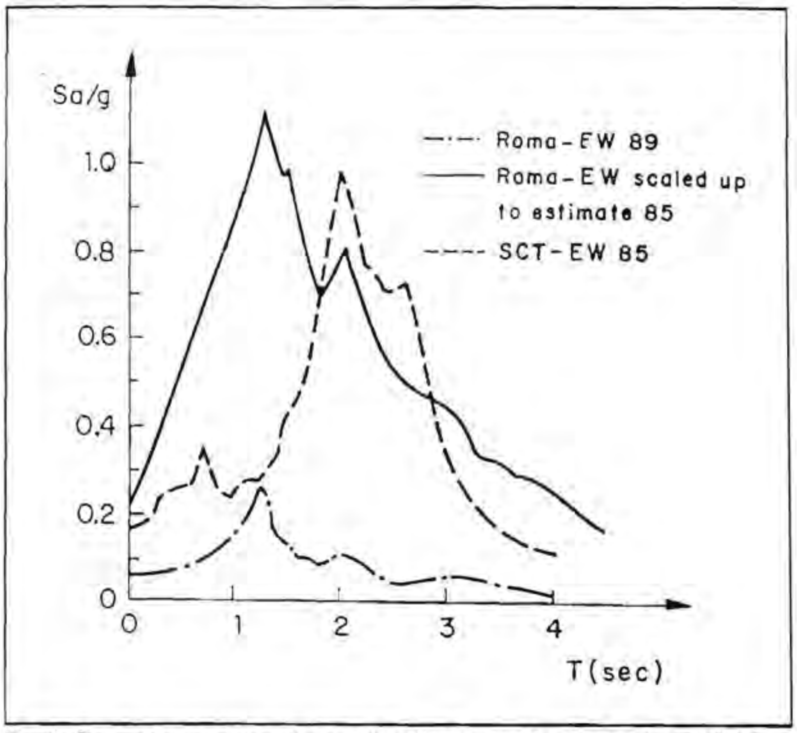

Fig. 9. Response spectra of ground motions used for the analysis of the QRO Building.

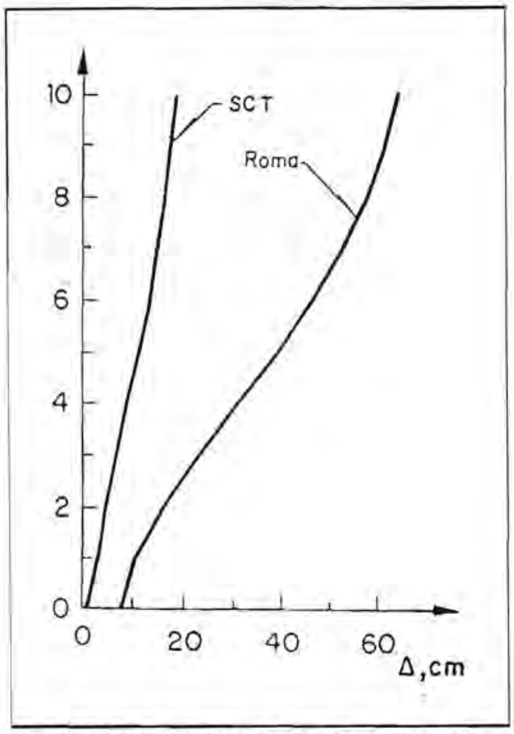

Fig. 10. Maximum lateral displacements using linear analysis for the QRO Building.

cracks were found in the main structural members. The building showed a significant tilting in its short direction (corresponding to about 2 percent of the building height).

It cannot be excluded that some tilting already existed before the earthquake, although it had not been noticed by the tenants. It is assumed that overturning moments, caused by the lateral forces, produced compressive forces on the piles in excess of those that could be transmitted through friction to the soil. Settlement and nonsymmetrical vibration of the building were then generated.

The design of the building was checked against the requirements of the building code enforced at the time of the construction. The strength for gravity and lateral forces was found to be adequate; nevertheless, lateral displacements in the transverse direction significantly exceeded allowable limits. The current code includes much stricter requirements which are not satisfied by this building.

For the computation of the linear response of the building to the 1985 earthquake, two ground motions were used. First, the SCT record was chosen because it is the only actual record obtained for the most damaged area where the building is located. After the earthquake, the network of seismic instruments was greatly enhanced in Mexico City. In particular, one instrument was placed at a distance of about 1000 m (3500 ft) from this building. Records of two moderate earthquakes have been obtained at this station, called ROMA.

For the same event, the amplitudes of the ground motion recorded at ROMA were consistently greater than that at the SCT site. There is convincing evidence that the response of the soil is linear in this area, even for very large earthquakes such as that of 1985, and that the shape of the linear elastic response spectrum remains essentially the same, except for a scale factor which depends on the earthquake intensity.[10] Therefore, an estimate of the ground motion experienced at the ROMA site in 1985 can be made by multiplying the spectrum obtained in 1989 at this site by a scale factor obtained from other stations where both motions had been recorded.

In Fig. 9, the scaled-up ROMA 1989 spectrum is shown. This is significantly more severe than the SCT 1985 spectrum. This finding is consistent with the greater damage observed around the ROMA site with respect to that at the SCT area. The factored (scaled-up) ROMA record was, therefore, also used to estimate the response of the QRO Building.

Only the response of the building to the ground motion in the transverse direction will be discussed. The effects in the other direction were significantly smaller, the response remaining essentially in the elastic range. The maximum lateral displacements at each floor level are shown in Fig. 10 for both ground motions. Fig. 11 compares the maximum bending moments induced at the beam ends with the resisting moments computed considering expected (average) material properties. At the left end of the beams, the comparison is made for the SCT record, and at the right end, for the factored ROMA record.

From the analytical results, it can be observed, firstly, that the factored ROMA motion produced a response exceeding that of the SCT motion by a factor of approximately 3. The difference is explained by the observation of the response spectra of the two records. For a period of 1.3 seconds, corresponding to the fundamental period of the QRO Building, the ratio of the spectral ordinates of the two records is 3.7. The fact that the ratio of

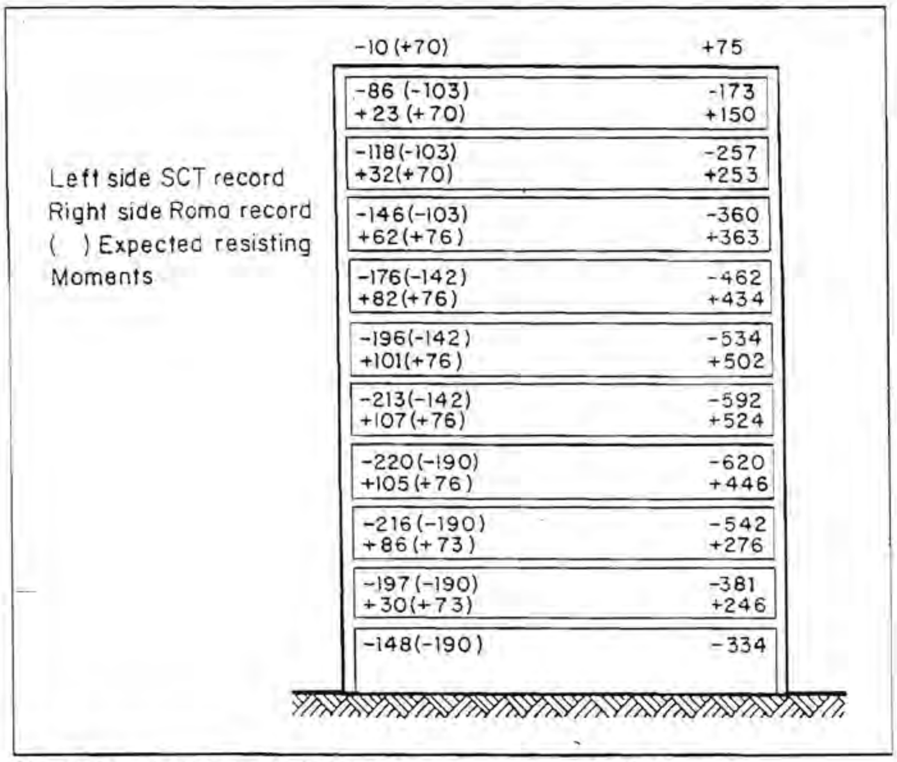

Fig. 11. Bending moments (ton-m) at beam end of the QRO Building.

the responses is similar to the ratio of spectral ordinates means that the structure responded essentially in the first vibration mode.

Secondly, the computed lateral displacements were extremely large. The maximum interstory drift was 0.7 percent for the SCT record and 2.5 percent for the ROMA record. Lateral displacements of this order of magnitude should have caused significant structural damage.

Finally, the bending moments that should have been induced by the ground motions in the building, if the response remained linear elastic during the earthquake, exceeded the flexural strength of the beams by a factor up to 15 percent for the SCT record and up to 240 percent for the ROMA record. The base shear force corresponding to a linear response is 0.30 and 0.55 of the total building weight, for the SCT and ROMA records, respectively.

The results of the nonlinear analysis for the SCT record indicated the formation of plastic hinges for positive and negative moments, as shown in Fig. 12, with a maximum ductility demand of 3. Under the effect of the factored ROMA record, the plastification was more widespread and the ductility demand much larger.

From the analysis, it is apparent that the observed response did not correspond to that predicted, since the lack of structural damage indicates that the structure remained linearly elastic. It cannot be excluded that some cracks formed during the vibration and then closed, leaving no visible evidence; nevertheless, strains in the steel and concrete could not reach the values indicated by the analysis.

It is assumed that the shaking actually induced in the building was much smaller than that which corresponded to the factored ROMA record and was probably also smaller than that of the SCT record. The reason for this can be attributed to the energy dissipation, associated with the loss of friction between the piles and soil, that took place at the base of the building.

Another possible reason for the difference is that the actual strength of the structural members significantly exceeded that computed by conventional design methods. Some additional comments on this issue will be made in the final section of this paper.

TAM BUILDING

This eight-story office building is located at the boundary of the lake bed area, where the depth of the soft clay deposits is about two-thirds of that at the QRO Building. In 1985, the damage in the area was moderate, indicating that the amplitude of the shaking was significantly less than that in the most damaged zone. The building has a cast-in-place concrete waffle slab supported by reinforced concrete columns.

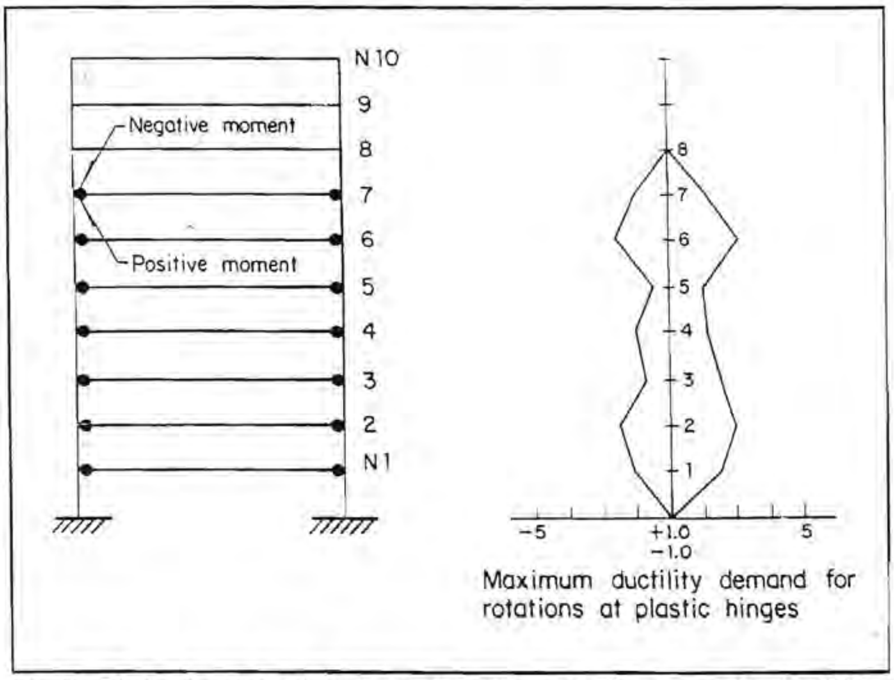

Fig. 12. Distribution of plastic hinges and maximum ductility demand for the SCT accelerogram in the QRO Building.

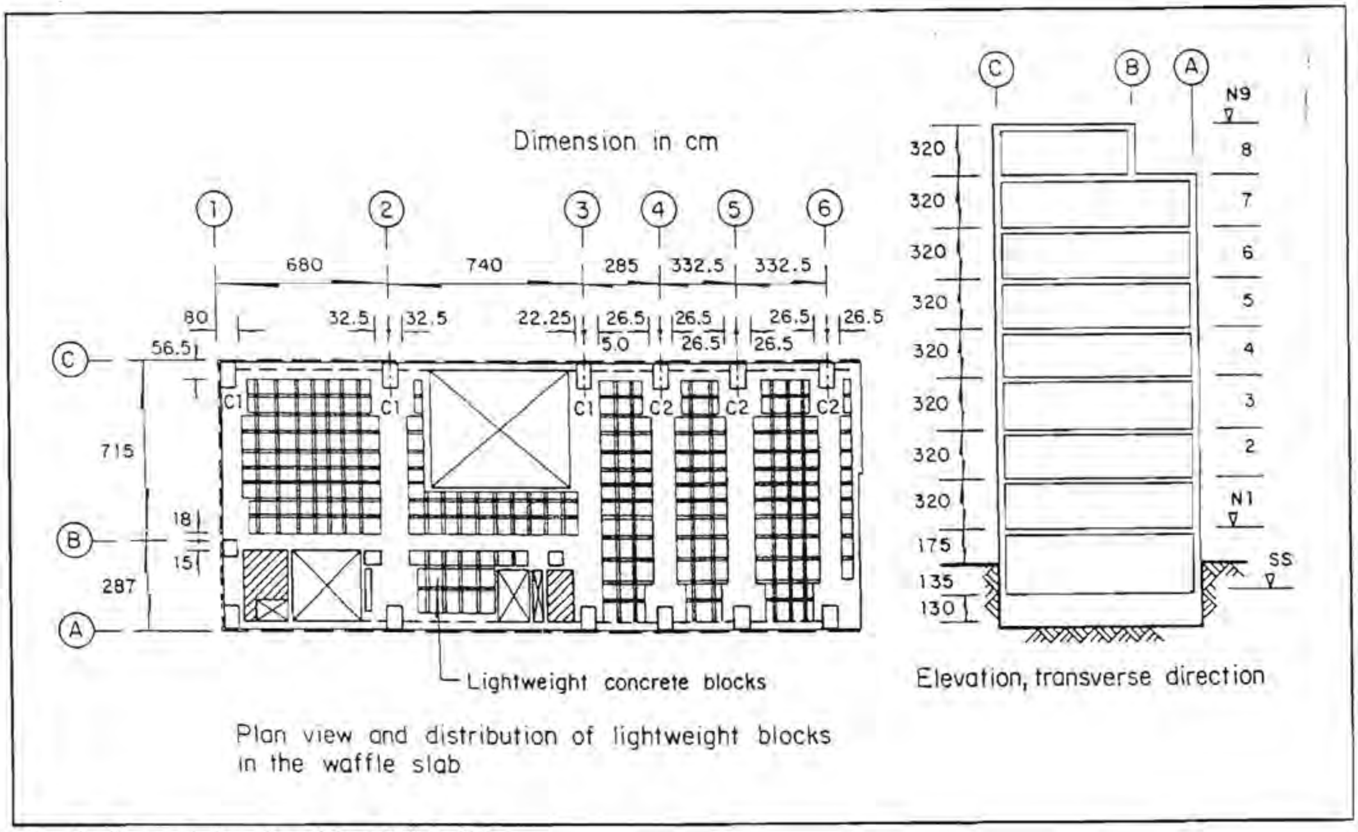

Fig. 13. Plan and elevation of the TAM Building.

The ribs of the waffle slab on the column lines are post-tensioned. Mild steel reinforcement in the ribs is very light, particularly for positive moments. Additional mild steel reinforcement is located at the solid zones around the columns, in both directions and in both faces. Thus, a sharp reduction in bending strength developed at the perimeter of the solid zone.

The end bays in the longitudinal direction are filled with masonry walls. Interior partitions are of flexible materials. The foundation is through point bearing concrete piles. A sketch of the structure is shown in Fig. 13 and the reinforcement of principal members is shown in Fig. 14. No evidence of

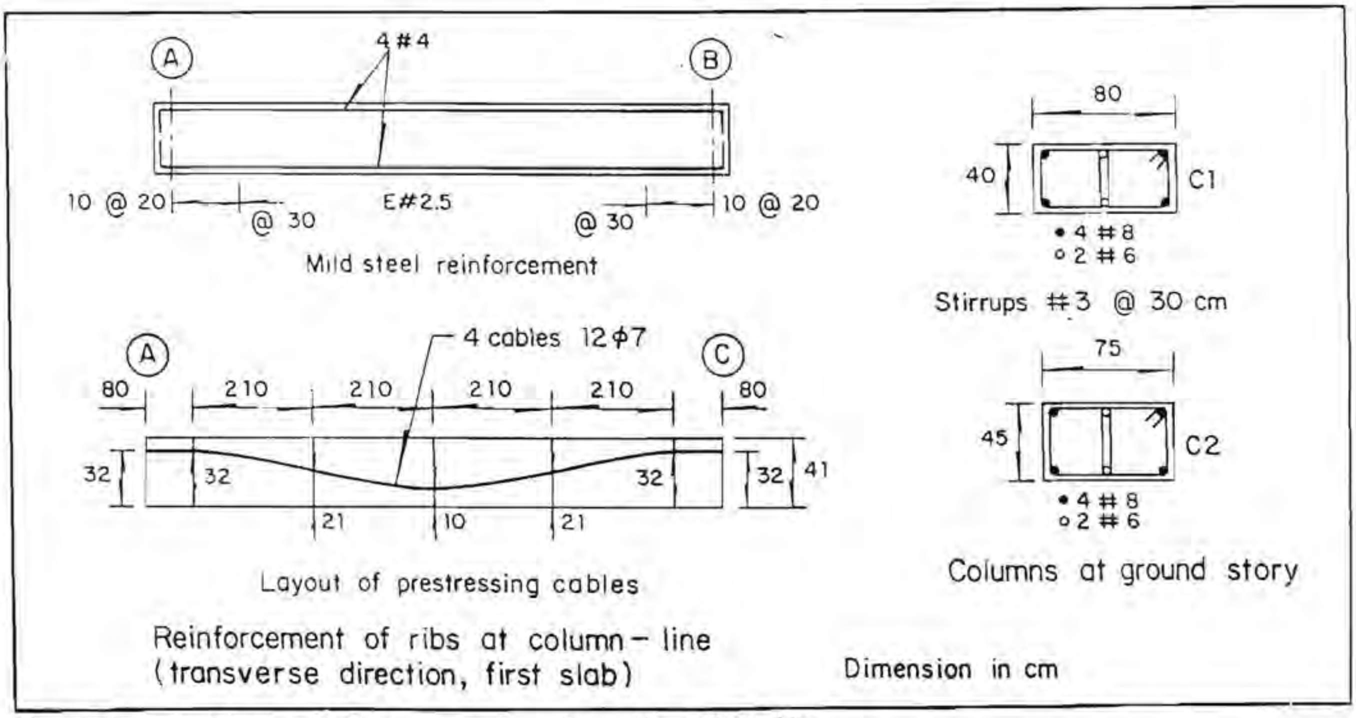

Fig. 14. Typical reinforcing details of structural members of the TAM Building.

damage was found when the building was inspected. Tenants mentioned that large lateral displacements were felt and that light non-structural damage was caused by the earthquake.

The study of the dynamic properties of the analytical model showed that the effect of the soil-structure interaction was small for this building, as can be deduced from the small difference between the fundamental periods computed for the model fixed at the foundation and the one considering deformation at the base (see Table 1).

Since the building was located at the boundary of the lake zone, it was analyzed both for the effect of the SCT and VIV ground motions. Maximum lateral displacements are shown in Fig. 15. For the SCT records, interstory drifts in some stories reached values that are commonly associated with structural damage. Bending moments in the beams at column faces exceeded resisting values.

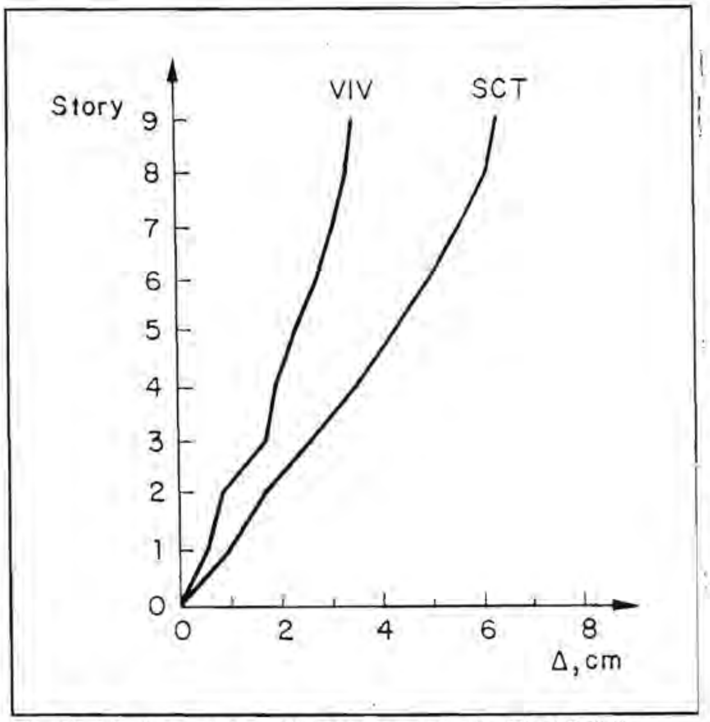

Fig. 15. Maximum lateral-displacements in the TAM Building.

The situation was particularly critical for positive moments at the perimeter of the solid zone, where, due to the very low amount of reinforcement, a very limited flexural strength was available once seismic moments exceeded the effects of vertical loads. Ductility demands at some sections exceeded 10.

Under the VIV record, the displacements were moderate and the internal forces did not exceed the resisting values. The base shear force induced by the SCT record corresponded to 21 percent of the building's weight, compared to 10 percent for the VIV record.

It is assumed that the ground motion at the site of this building was much lower than in the most affected area, being closer to the VIV record than to the SCT record. The building is rather weak and flexible in the transverse direction and would have been significantly affected by the earthquake if situated in the most critical zone.

OTHER BUILDINGS

The three other buildings under study are located in areas where the seismic effects were smaller, and none of them experienced any damage. They will be described briefly. Further details on their properties and on the analytical results can be found in Ref. 7.

- The building labeled as TAC is a four-story structure which is used for the storage of furniture. Its story height [6.8 m (22.3 ft)] is greater than usual, giving rise to a total height of 27.2 m (89.2 ft), roughly corresponding to a typical eight-story building. It is located at the foot of the hills on the west side of the city, where the area of firm soil begins. There are only a few meters of soft soil over the solid strata.

The structural system is composed of cast-in-place concrete frames with rather long spans [10 and 11.7 m (32.8 and 38.4 ft)]. Beams are post-tensioned with draped tendons. Mild steel reinforcement at the ends of beams is small. The

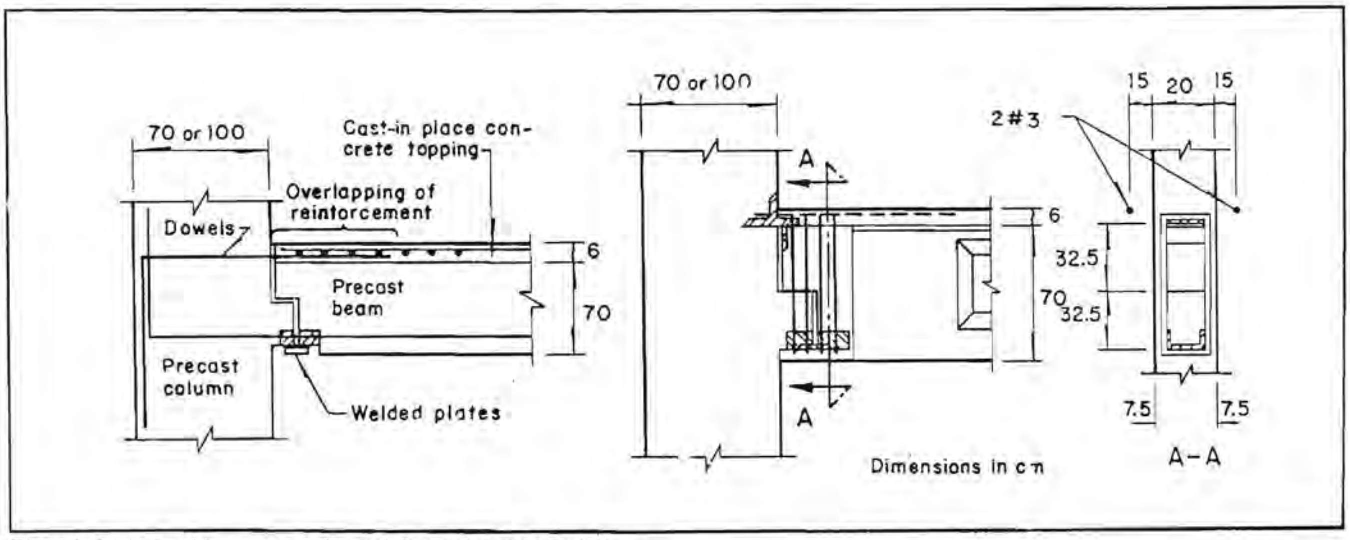

Fig. 16. Details of beam-to-column joint in the IMP Building.

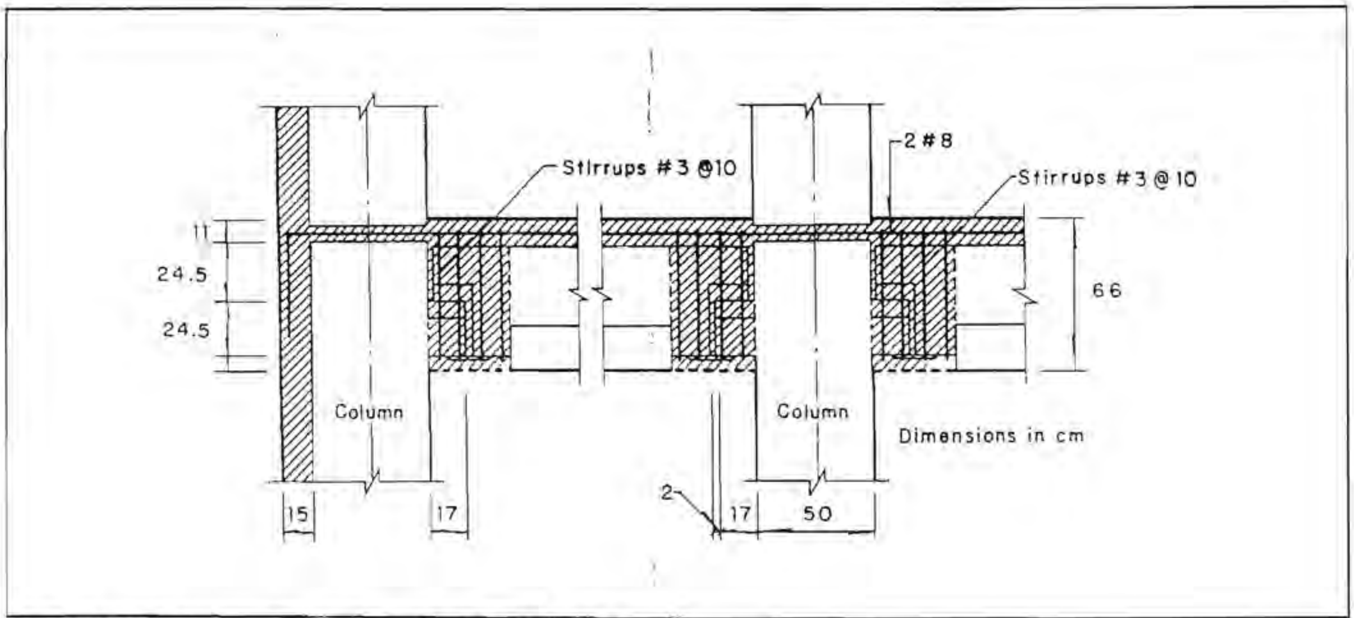

Fig. 17. Details of beam-to-column joint in the SMO Building.

perimeter frames are completely infilled with masonry walls, which contribute significantly to the lateral stiffness.

The analyses showed almost no effect of soil-structure interaction and that the building is very flexible in the transverse direction. Nevertheless, under the effect of the VIV record, the lateral displacements remained within allowable limits and the moments in the beams did not exceed the flexural strength, thus explaining the lack of damage to the building.

- The IMP Building has five stories and a basement and is located in the so-called transition zone, which has about 10 m (32.8 ft) of clay deposits over firm soil. The structure is completely precast, with only one long span in the transverse direction and 10 short spans [5 m (16.4 ft) each] in the longitudinal direction. The floor system is composed of prestressed T-beams with a compression topping reinforced with mild steel to provide continuity with the columns (see Fig. 16).

The analyses showed that the building is rather flexible in both directions and that, under the effects of the VIV ground motion, neither the allowable displacements nor the strength of the members were exceeded. This building did not suffer any damage from the earthquake.

- Finally, the SMO Building is a completely precast eight-story structure which has double T-beams overlayed with a 6 cm (2.4 in.) thick concrete topping, providing diaphragm action of the floor and continuity with the columns. A detail of the beam-to-column connection is shown in Fig. 17.

The analyses of this building were made for the CU record (obtained on firm ground) and indicated a very small response, not exceeding allowable values. Accordingly, no signs of damage were observed in this building.

DISCUSSION OF THE RESULTS

The five buildings whose seismic responses were analyzed in detail are representative of the different techniques used for prestressed concrete construction in Mexico City. They are mainly frame structures with prestressed concrete beams and nonprestressed columns. Beams were either cast-in-place and post-tensioned, or precast and prestressed.

Frame members were rather slender, giving rise to flexible structures with long vibration periods, which are particularly susceptible to the type of ground motion induced in the soft clays of Mexico City. In four of the five buildings, masonry walls filled the exterior frames adjacent to other constructions. These walls significantly increased the lateral stiffness and strength of the structure. Therefore, these buildings were much stronger in one direction, i.e., perpendicular to the street, than in the other direction.

Analytical models adequately estimated the dynamic properties of these buildings when proper consideration was made of the contribution to the stiffness by the floor system, of the stiffening effect of masonry infills and of the deformations of the base of the buildings on soft soils.

The vibration periods of the first modes computed analytically were very similar to those measured by ambient vibration tests on the structure. The average error was only 1 percent with a standard deviation of 2 percent. When the masonry infills were included in the models, the vibration periods decreased by as much as 40 percent, corresponding to a two-fold increase in the lateral stiffness. On the other hand, the displacements and rocking at the base of the buildings were significant only for slender buildings on thick strata of soft soil. The maximum increase in the period due to this effect was about 30 percent.

Damping coefficients measured by ambient vibration techniques varied between 2 and 5 percent, with an average of 3 percent. These values are very similar to those that have been determined in non-prestressed, reinforced concrete buildings. It must be emphasized that ambient vibration tests correspond to very low stress levels in the structural members, and that for larger vibrations, as those induced by severe earthquakes, both the damping and period increase. Then, the damping coefficient of 5 percent, which is generally assumed for seismic design, appears reasonable.

Since there were no instruments to measure the responses of the buildings or the motions induced at their bases, no quantitative comparison can be made between the actual and the computed responses to the 1985 earthquake. Furthermore, significant variations in ground motions originated by the same earthquake, even between nearby sites, make it difficult to estimate the shaking experienced by a particular building. For these reasons, only very general conclusions can be drawn.

Three of the five buildings were located in areas where the amplitude of the ground shaking in 1985 was moderate. Therefore, the analyses showed that the response should have remained in the linear elastic range of behavior, which is in agreement with the lack of damage experienced by the buildings.

Another building (TAM), with a post-tensioned flat plate system, was particularly flexible and weak in one direction. The analyses showed that it should have undergone large inelastic deformations if it were affected by the ground motion measured in the most damaged area. It is assumed that its good behavior is partially due to a structural capacity in excess of that computed by analytical methods, and mainly to the fact that the ground shaking at this particular site was weaker than in the critical area.

The most interesting case is that of the QRO Building, which is located in an area where the ground motion was particularly strong. The lack of severe damage in this building can probably be explained by the fact that the energy induced by the shaking was mainly dissipated by the nonlinear deformations between the foundation and the soil, rather than by inelastic behavior in the structure.

CONCLUSIONS AND RECOMMENDATIONS

It is important to emphasize that the five prestressed concrete frame structures showed good behavior under a severe earthquake, without any sign of distress in the joints or elsewhere. This suggests that, by following the much stricter code practice now accepted for this type of structure, an adequate safety level can be obtained for even the most severe seismic conditions.

On the other hand, particular care must be given to some structural design features which were common to the buildings under study, as well as to most prestressed concrete buildings in Mexico. These features must be significantly improved in order to attain an adequate seismic safety level.

The structural system must be laterally stiffer and stronger than that provided by the rather slender frame members typically used in the buildings under consideration. This can be attained by using more robust columns and beams, but, preferably, by adding stiffening members such as shear walls or braces. The advantage of having stiff structures is particularly important for buildings located on soft soil, where the long periods prevailing in the ground motion particularly affect flexible structures.

Flexural capacity at beam-to-column joints must be increased, especially with regard to positive moments. The concept of partial prestressing, in which the capacity to resist seismic effects is mainly assigned to mild steel reinforcement, must be favored because of the greater ductility and continuity that can be obtained from this method as compared with full prestressing.

Reinforcement detailing in beams, columns and joints must be improved to attain larger ductilities. In particular, the confinement of concrete and longitudinal steel in sections of possible formation of plastic hinges must be achieved using closely spaced transverse reinforcement.

When precast concrete elements are used in floor slabs, measures must be taken to ensure that they constitute a stiff horizontal diaphragm — for example, by casting a thick reinforced concrete topping. Otherwise, in-plane distortion of the slabs and a concentration of lateral forces in some frames could occur.

The importance of instrumenting prestressed concrete buildings to record their responses to seismic effects must be emphasized. Only in this way could the analyses and design methods discussed be fully validated. The frequent seismic activity in the Mexico City Valley and the large variety of types of structures existing in the area make it a particularly convenient site for seismic instrumentation of buildings.

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