Structural damage due to the 1985 Mexican earthquake

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ABSTRACT: Characteristics of the 1985 Mexican earthquake and the influence of the subsoil conditions on the strong ground motion in Mexico City are described. Some aspects of structural design practice and examples of structural damage are discussed along with the emergency changes to the building code.

1 INTRODUCTION

This paper summarizes the structural design practice in Mexico City, the dominant role of the soil conditions on the strong ground motion, the structural damage in Mexico City and the resulting emergency code changes. These observations of structural damage were made during the site visit of the five member Canadian team (Mitchell et al. 1986a&b) in October 1985 and during a subsequent visit by the author in September 1986.

2 NEAR SOURCE GROUND MOTION

The September 19, 1985 earthquake which occurred off the Pacific coast of Mexico along the Michoacán segment of the Cocos Plate had a magnitude of 8.1 followed by a magnitude 7.4 aftershock on September 20, 1985. See Fig. 1. The subduction of the Cocos Plate under the North American Plate has resulted in a number of earthquakes of magnitude 7.4 to 8.0 along the Pacific coast. Figure 2 gives the accelerograms recorded at La Villita close to the epicentre. The maximum acceleration measured was only 12% g in both the E-W and N-S directions and the ground motion was characterized by high frequencies with a second major burst of energy released after about 40 seconds.

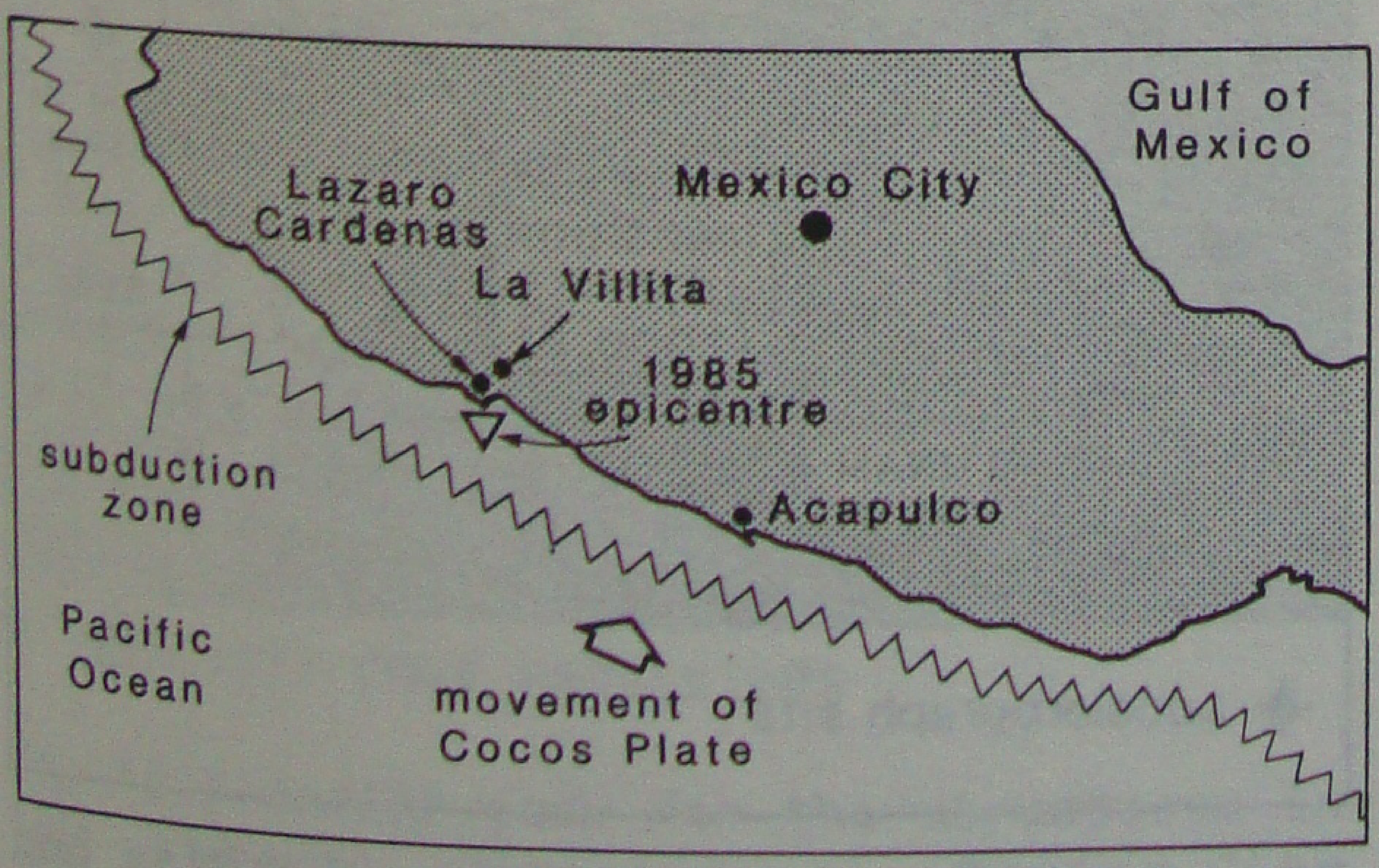

Fig. 1. Epicentral region of the 1985 Michoacan earthquake.

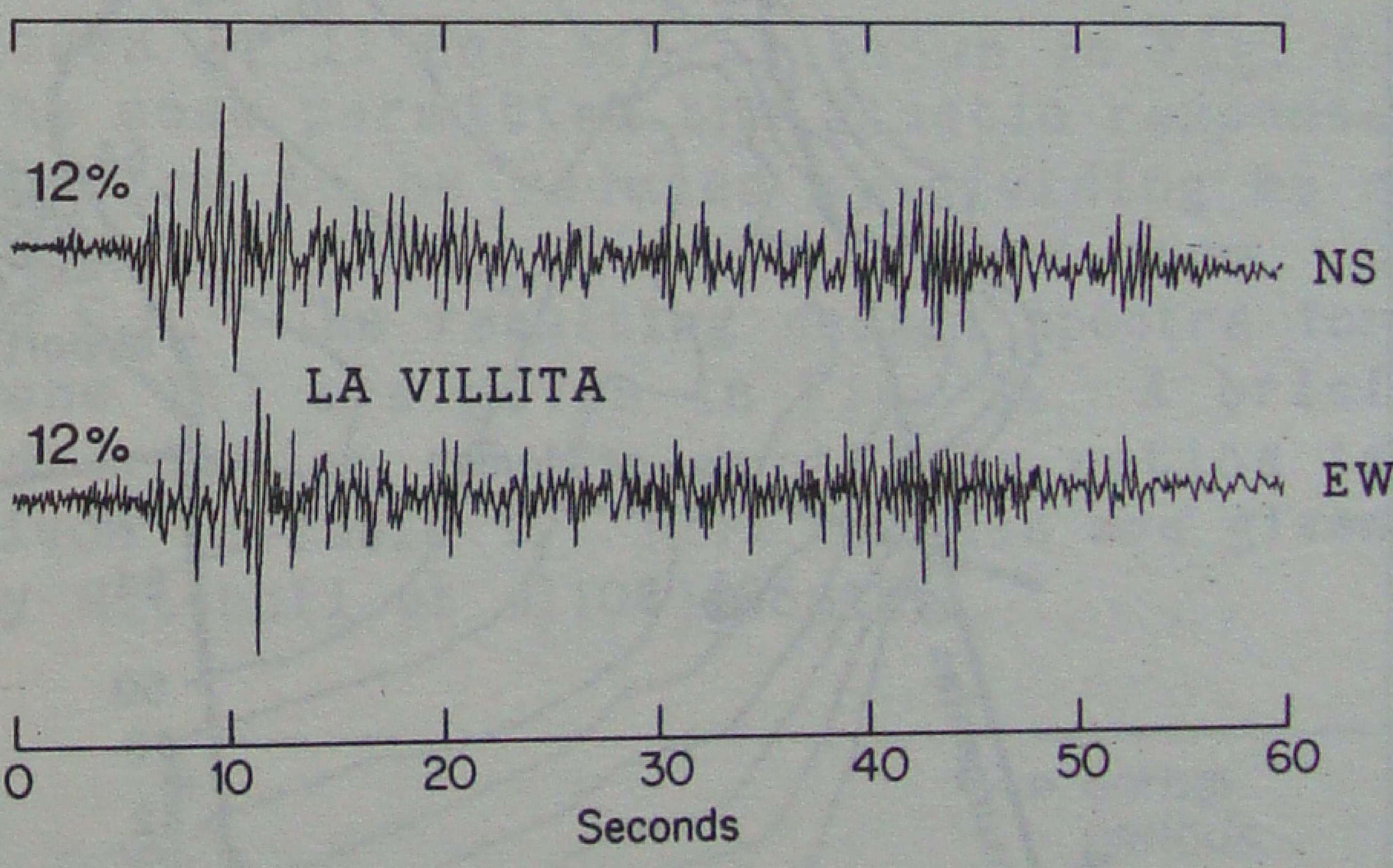

Fig. 2. The N-S and E-W accelerograms recorded at La Villita (Anderson et al. 1985).

3 THE EFFECT OF SUBSOIL CONDITIONS ON STRONG GROUND MOTION

Although the earthquake caused damage to low rise masonry buildings near the epicentral region, only minor damage occurred in the coastal resorts of Ixtapa, Zihuatanejo, and Acapulco. Considerably greater damage was reported near the port of Lázaro Cárdenas at the mouth of the Balsas River (see Fig. 1). This damage included the collapse of a concrete conveyor structure, failure of the base of a silo penthouse structure, and spalling of circular concrete bridge piers (Mitchell et al. 1986a&b). The ground motion was probably amplified by the soft sediments in the delta region.

Although Mexico City is about 350 km from the epicentre unprecedented structural damage occurred in this heavily populated city. The damage occurred in one confined region of the city due to the dominant role played by the subsoil conditions. The soil stratigraphy in the lake zone consists of an upper layer of compressible clay to a depth of 20 to 40 m followed by a clayey sand layer which about 2 to 5 m thick over a second layer of clay about 5 to 10 m in thickness. The upper clay layer has a water content which varies from 200 to 400% (Marsal and Mazari 1969) and is highly compressible. Figure 3 shows the depths to the lower sand layer and Fig. 4 gives the soil stratigraphy along a N-S line between points A and B.

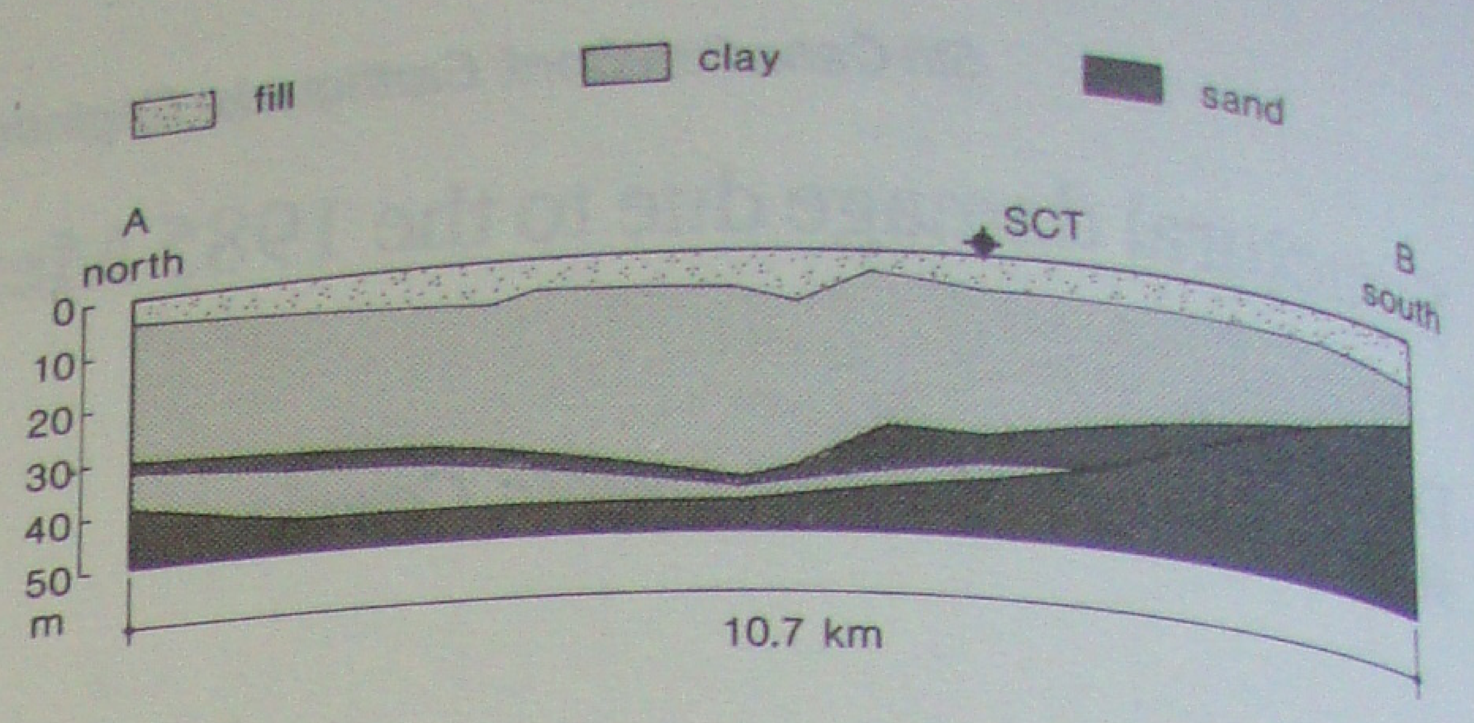

Fig. 4. N-S soil stratigraphy (Instituto de Ingeniería 1985).

For seismic design purposes the 1977 design code (Instituto de Ingeniería 1977a) divided the city into three subsoil zones: Zone I (firm ground) has less than 3 m of compressible soil, Zone II (transition zone) has between 3 and 20 m of compressible soil and Zone III (lake zone) which has greater than 20 m of compressible soil. See Fig. 5. The dominant influence of the depth of compressible soil on the ground motion is evident from Fig. 6. The UNAM (National Autonomous University of Mexico) site, on firm ground had a peak acceleration

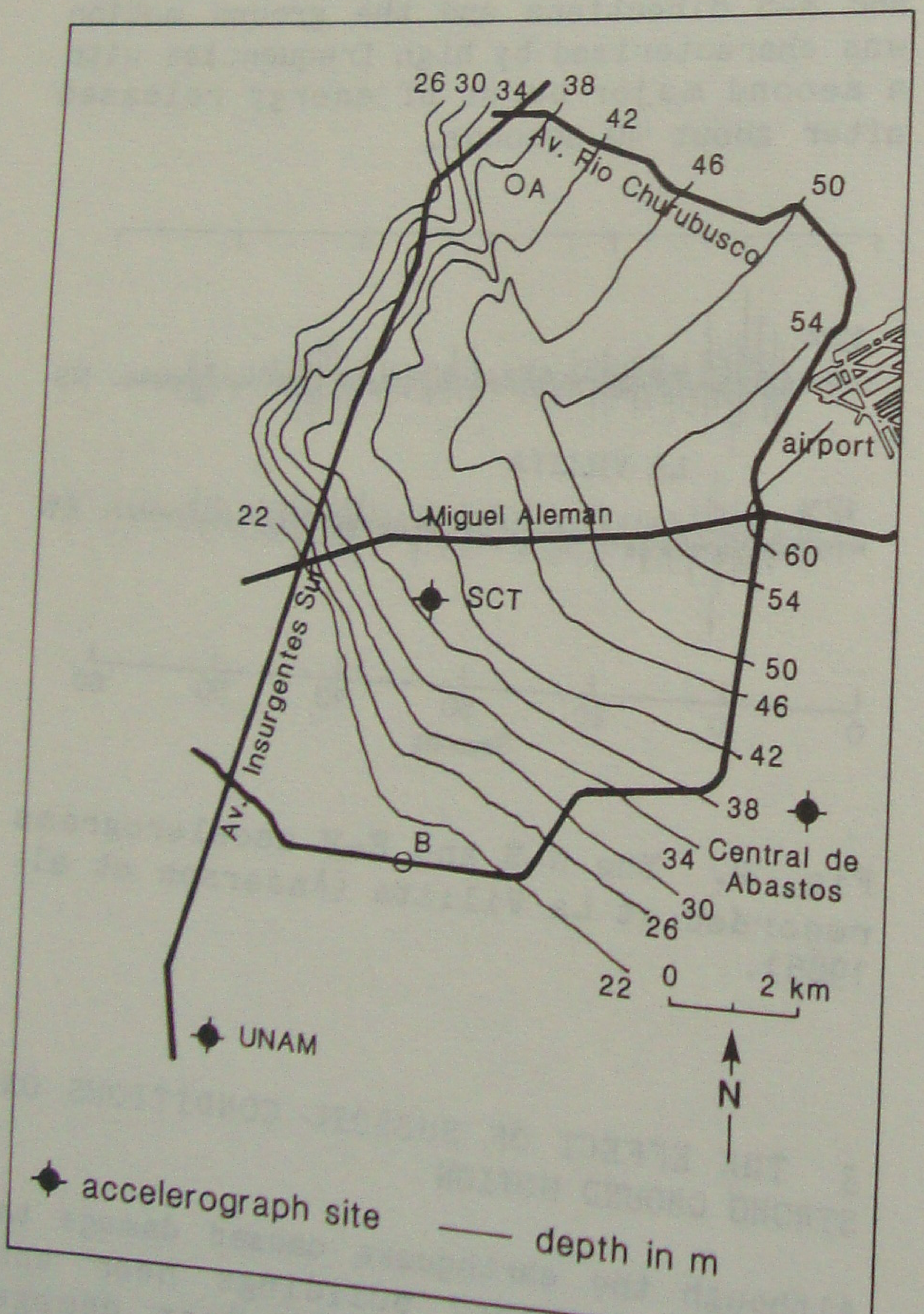

Fig. 3. Contours of lower sand layer depths (Instituto de Ingeniería 1985).

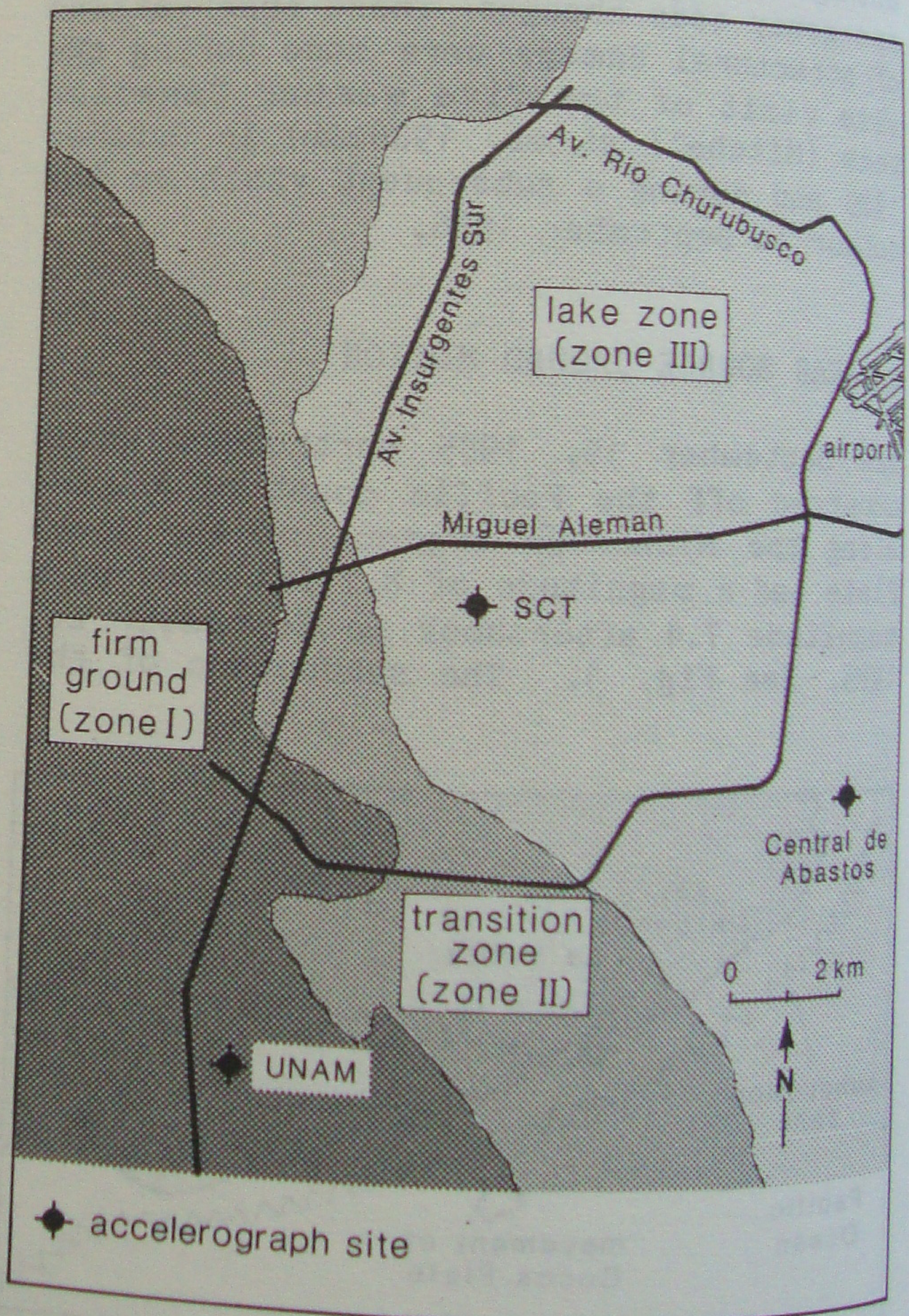

Fig. 5. Subsoil zones of Mexico City and accelerograph sites (Instituto de Ingenieria 1985).

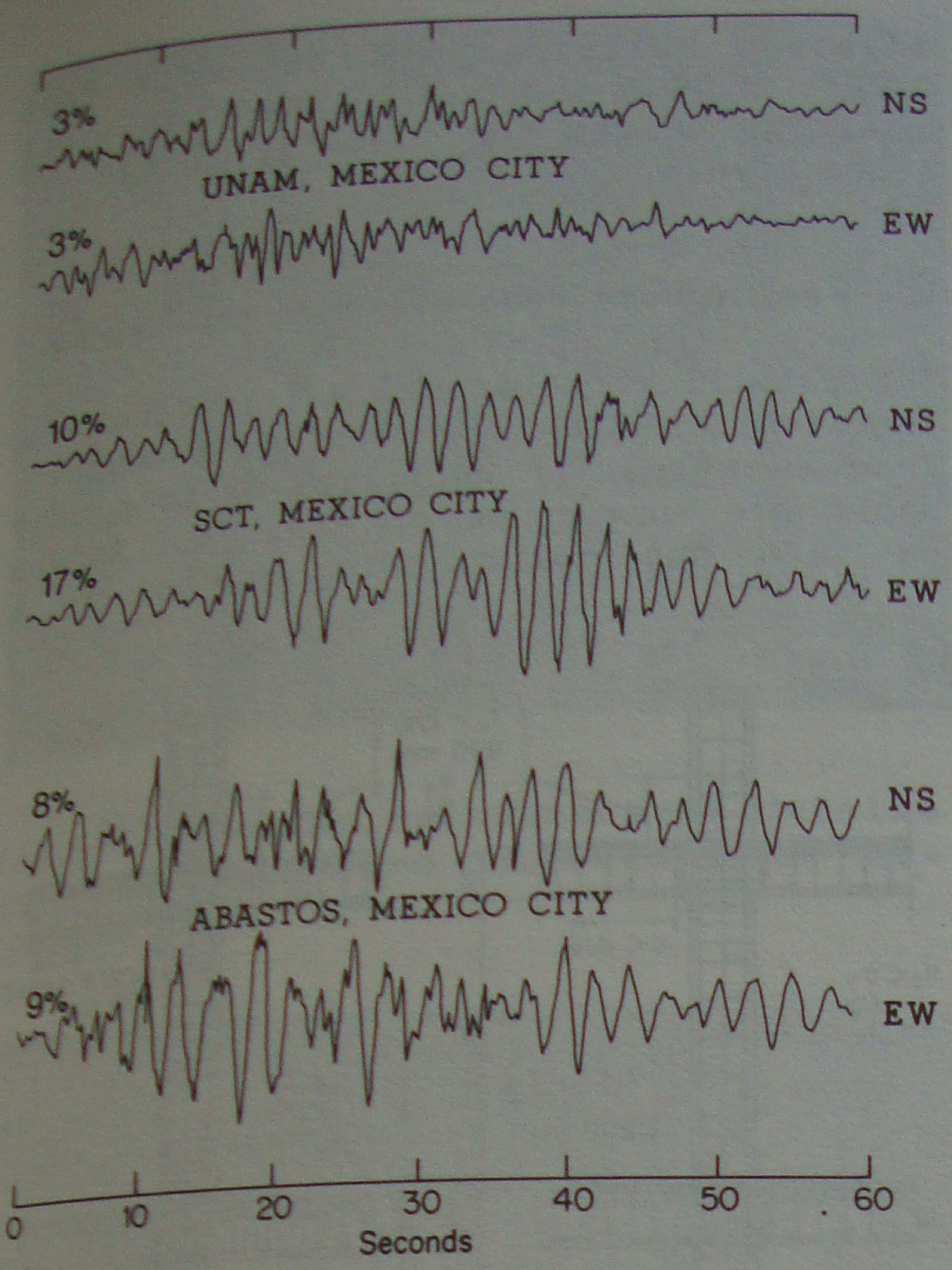

Fig. 6. N-S and E-W accelerograms recorded at UNAM, SCT, and Abastos (Prince et al. 1985a, Mena et al. 1985 and Quaas et al. 1985).

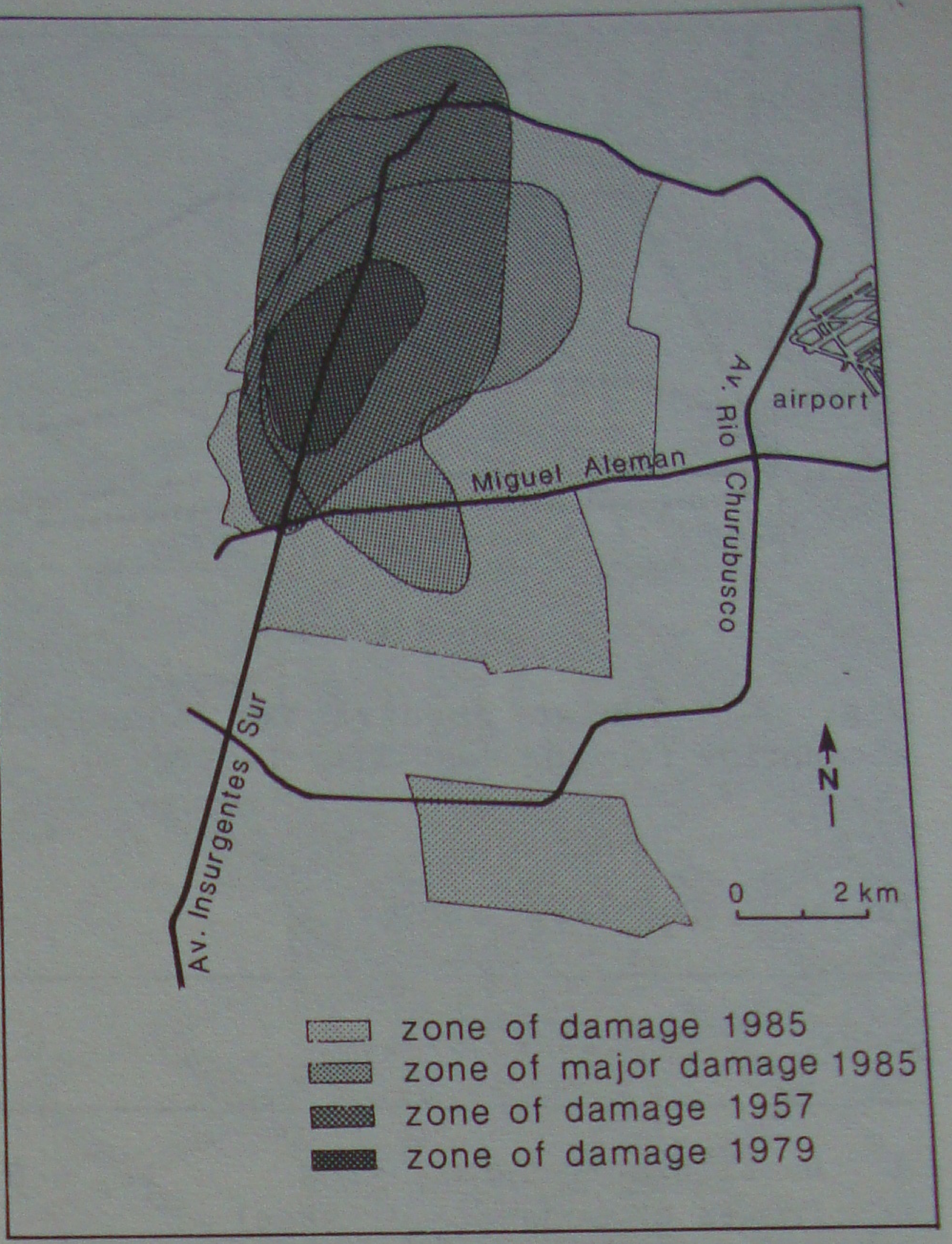

Fig. 7. Zones of damage in the 1957, 1979 and 1985 earthquakes (Instituto de Ingeniería 1985).

resultant of about 4% g but lacked the high frequency content that was experienced at La Villita (See Fig. 2). At the Secretary of Communications and Transportation (SCT) complex the 30 m of compressible soil resulted in a peak acceleration resultant of about 20% g with a predominant period of vibration of about 2 sec. This represents an amplification of about 5 times the peak acceleration recorded on firm ground. At the Central de Abastos site the thicker layer of compressible soil resulted in a peak acceleration of about 12% g with a predominant period of about 3 sec.

It can be seen from Fig. 7 that the zones of structural damage have repeatedly coincided with regions having compressible soil depths of 26 m to 40 m in the lake zone.

4 THE 1977 CODE PROVISIONS

The 1977 design code for the structures in the Federal District in Mexico City (Instituto de Ingeniería 1977a) recognized the importance of sub-zonation and gave three different elastic design spectra for zones I, II and III as shown in Fig. 8. The code permitted the elastic response spectra to be reduced by dividing by a ductility factor Q which ranged from 1.0 to 6.0. The resulting design spectra for zone III are given in Fig. 9. A brief description of the ductility ratios is given in Table 1. More details are given by Mitchell et al. (1986a&b).

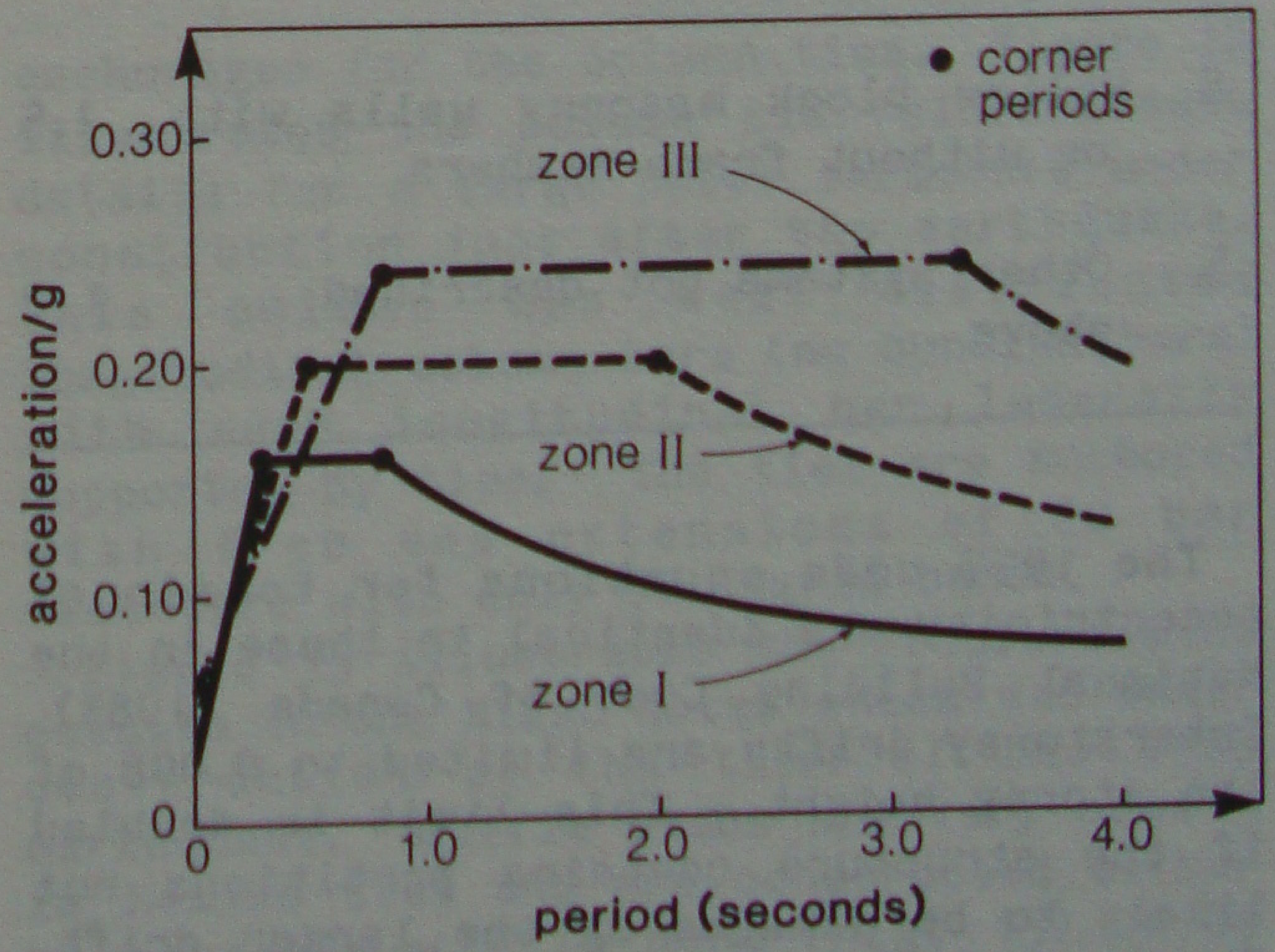

Fig. 8. Elastic design spectra (Instituto de Ingeniería 1977a).

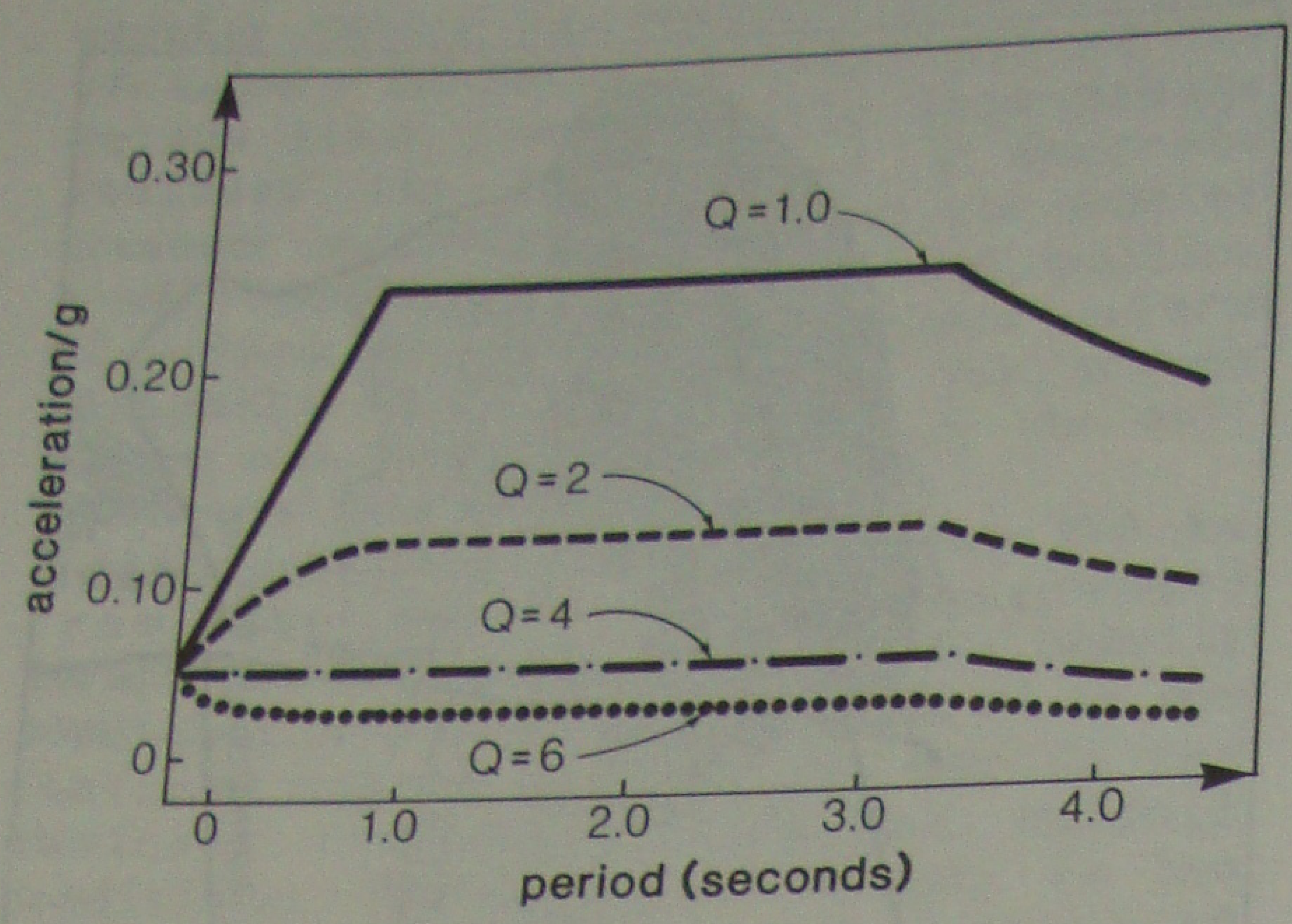

Fig. 9. Design spectra for zone III accounting for the ductility factor, Q.

Table 1. Ductility factor, Q

Case	Structural system	Q
1	Ductile moment-resisting frames of concrete or steel meeting special design and detailing requirements	6.0
2	Dual systems of moment-resisting frames together with either braced frames or concrete shear walls meeting special design and detailing requirements	4.0
3	Includes frames (braced or unbraced) with beams and columns not satisfying cases 1 or 2 and includes solid masonry walls meeting special confinement requirements	2.0
4	Hollow block masonry walls with or without frame members	1.5
5	Other systems not described above	1.0

The 1977 code equations for torsional eccentricity are identical to those in the National Building Code of Canada (1985). Interstorey drifts are limited to 0.008 of the storey height. This limit is doubled if the structure contains partitions not likely to be damaged by the larger drift. The code gives a minimum separation between adjacent buildings equal to the sum of the calculated deflections of the adjacent buildings but not less than 0.001, 0.0015 and 0.002 times the total building height for zones I, II and III respectively.

4.1 Reinforced concrete code provisions

The detailing requirements (Instituto de Ingeniería 1977b) for reinforced concrete frame members for ductility factors of 6 and 4 are summarized in Fig. 10. As can be seen the column tie spacing is a

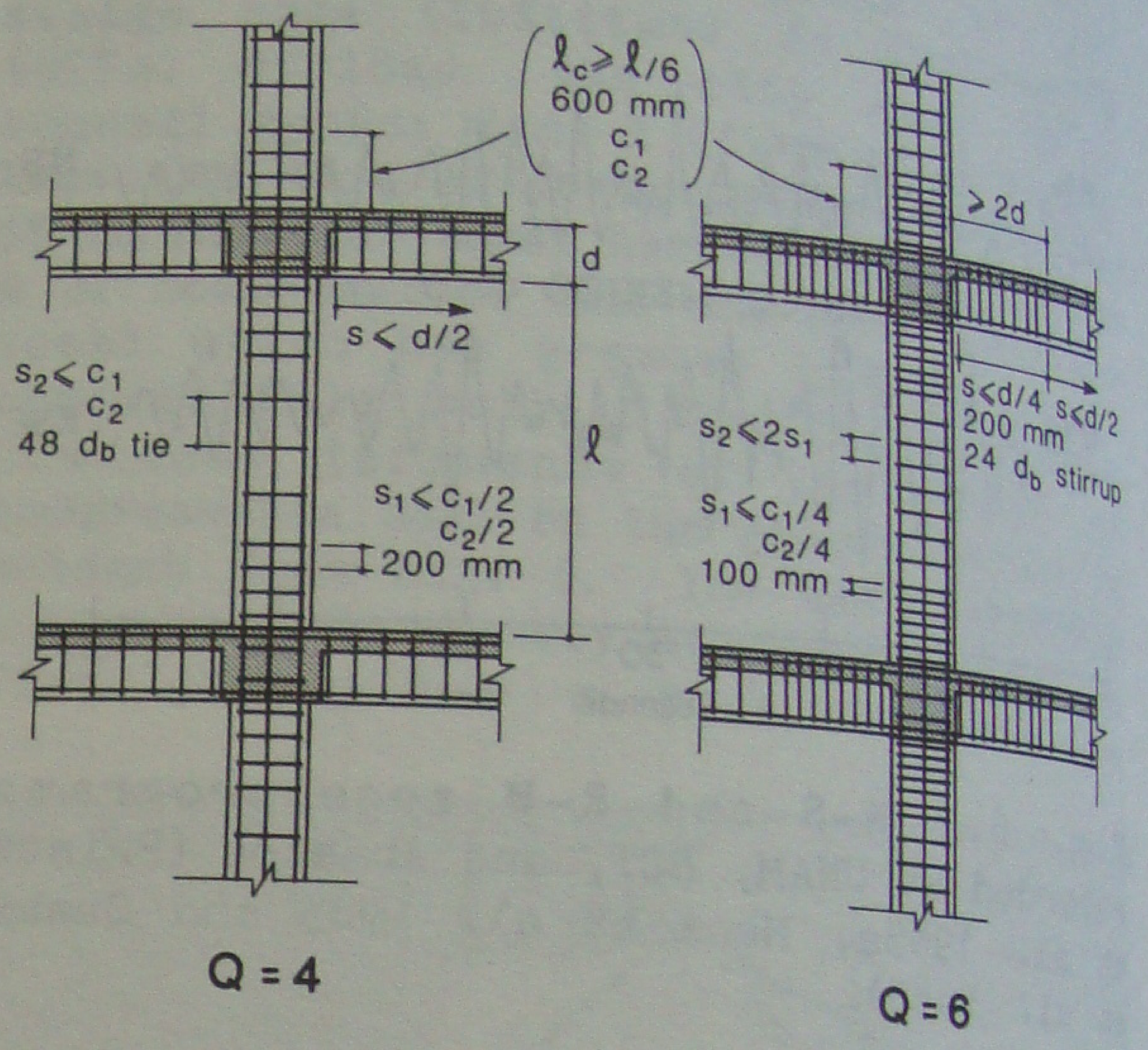

Fig. 10. Details for reinforced concrete frame members.

function of the column dimensions (c_1 and c_2) and the tie diameter (d_b tie). The 1977 code required that the ends of column ties be anchored either with 135 degree bends with a straight bar extension of 10 bar diameters or with 90 degree bends with a straight bar extension of 20 bar diameters. It is noted that after spalling of the concrete cover these 90 degree bend anchorages would be ineffective and therefore are not permitted by many codes (e.g. CSA 1984) for structures designed to exhibit significant levels of ductility and energy absorption. In addition to the tie spacing requirements shown in Fig. 10 the ties must have a yield force of at least 1/200 times the yield force of the largest longitudinal bar or bundle of longitudinal bars. In order to prevent buckling of the longitudinal bars the tie spacing is limited to 13 times the longitudinal bar diameter for the usual grade of steel (412 MPa). In addition minimum confinement reinforcement is required in the columns.

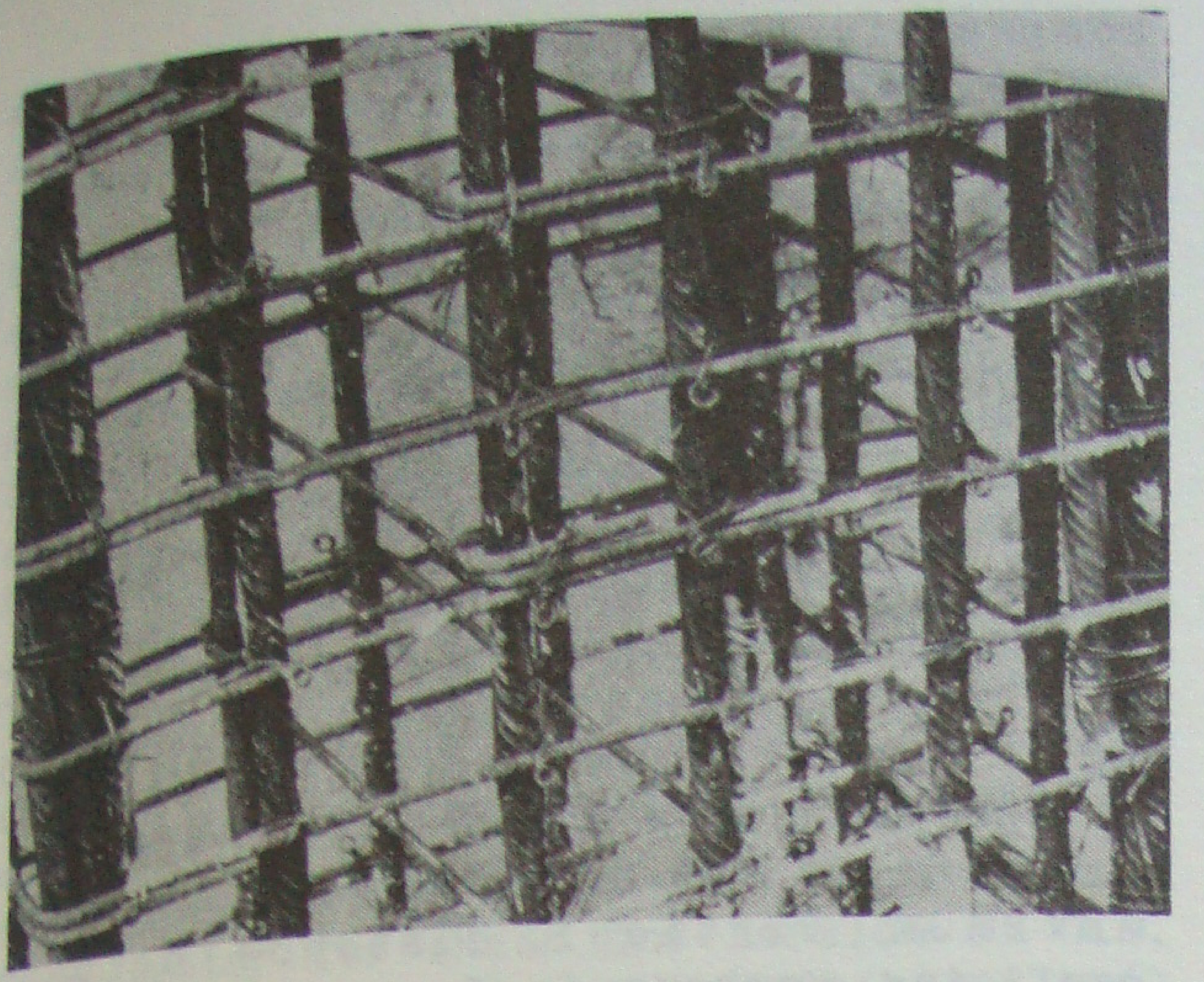

Fig. 11. Column reinforcement details with bundled bars (Mitchell et al. 1986a&b).

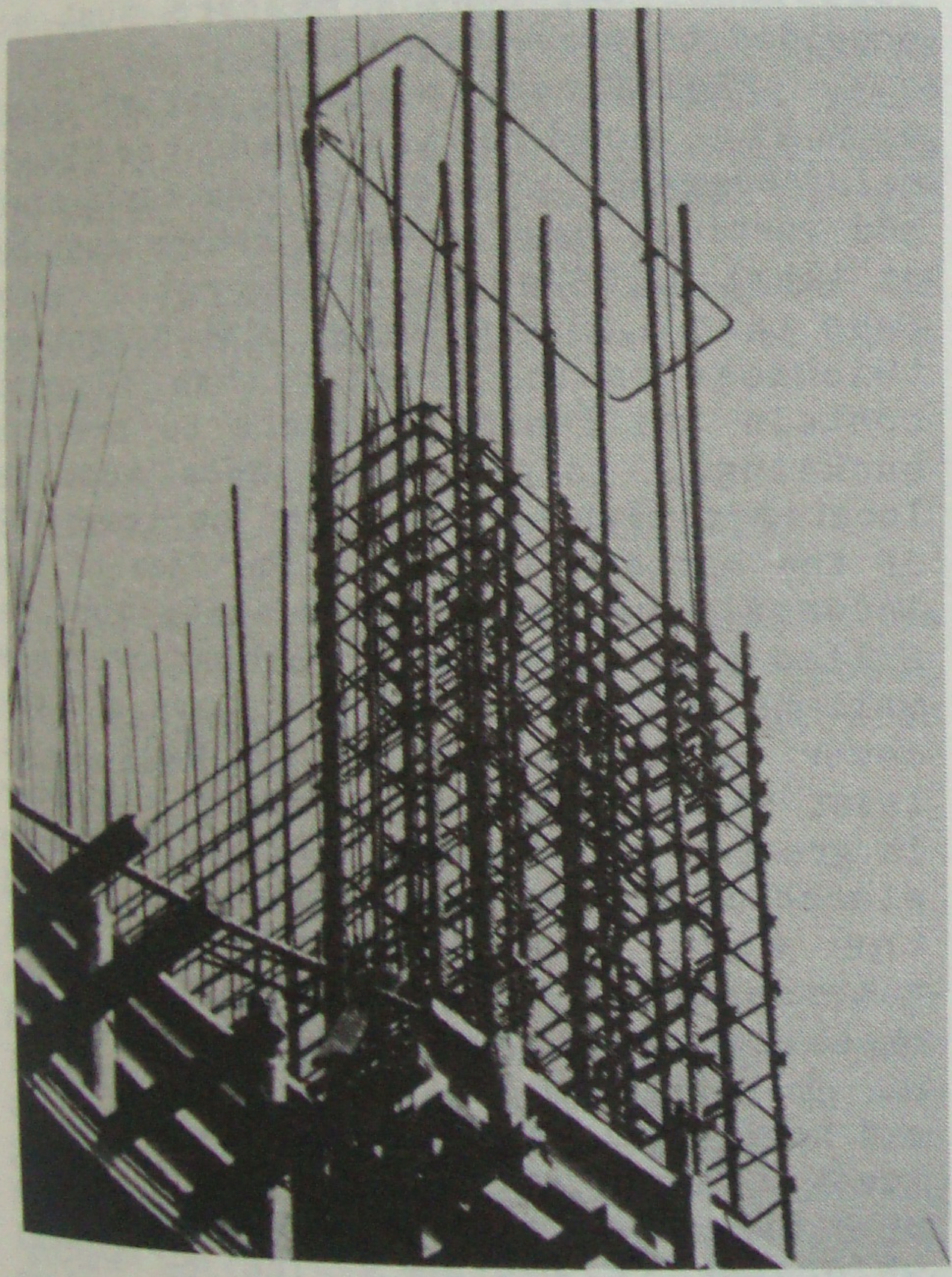

Fig. 12. Column reinforcement details.

Figure 11 shows the reinforcement details at the end of a column under construction in Mexico City in October 1985, just after the earthquake. This column has a good distribution of longitudinal bars around the perimeter of the cross-section and every alternate longitudinal bar is tied with closely spaced ties. Figure 11 also illustrates the common use of bundled longitudinal bars in the corners of columns and the use of only 90 degree bend

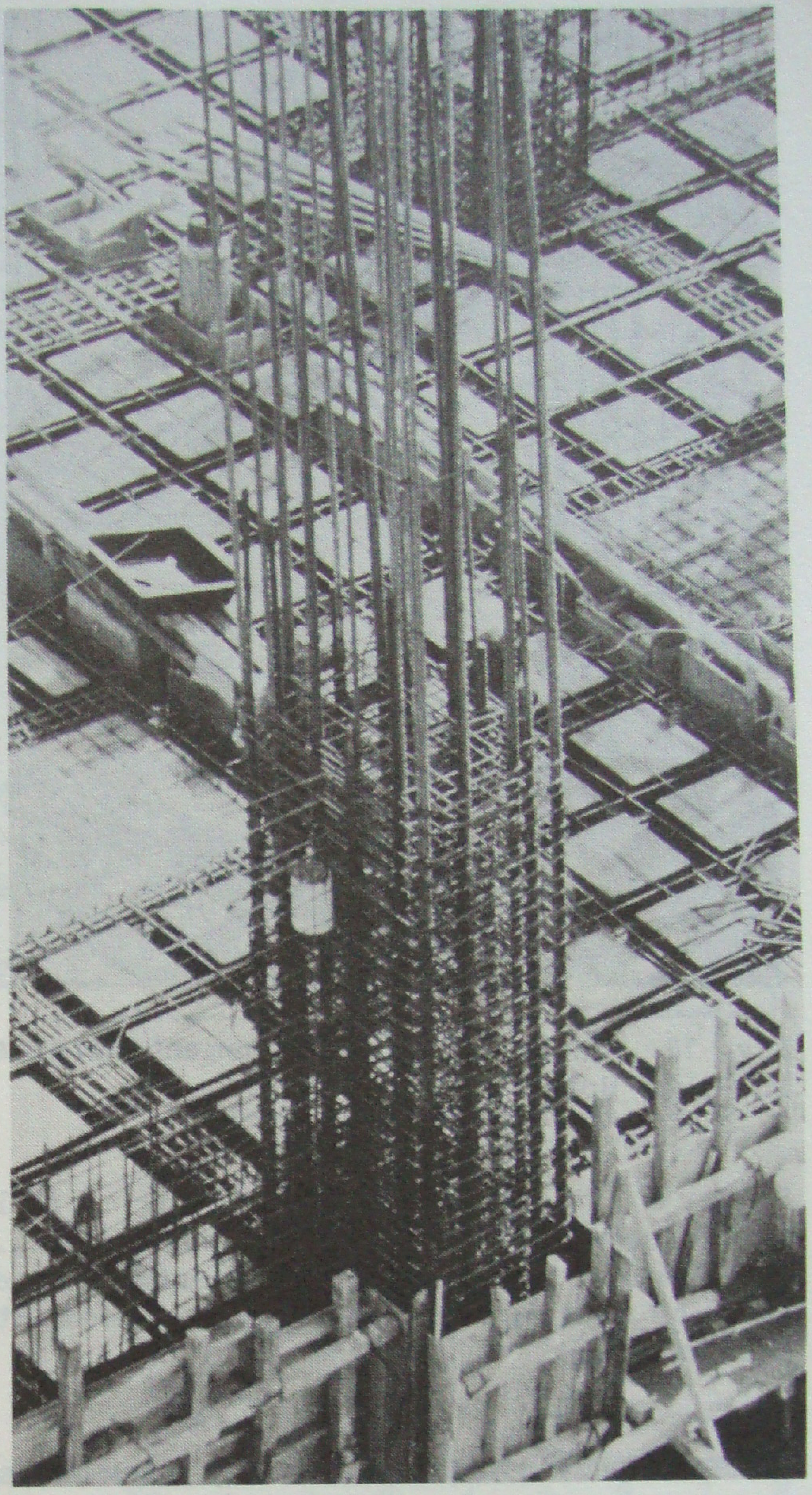

Fig. 13. Waffle slab construction in Nikko Hotel.

anchorages for the column ties. Figure 12 illustrates the column tie reinforcement details for a large hotel structure under construction just after the earthquake. This column has well distributed longitudinal column bars (no bundled bars) with each longitudinal bar laterally supported by ties. The ties are anchored with free end extensions of 20 bar diameters with only 90 degree bends.

Figure 13 illustrates the reinforcement details in the Nikko Hotel which was under construction at the time of the 1985 earthquake. Waffle slabs are very common in Mexico City and are constructed either by reusable waffle pans (Fig. 13) or by embedding hollow masonry blocks in the slab. Note the presence of reinforced beams between the columns and walls.

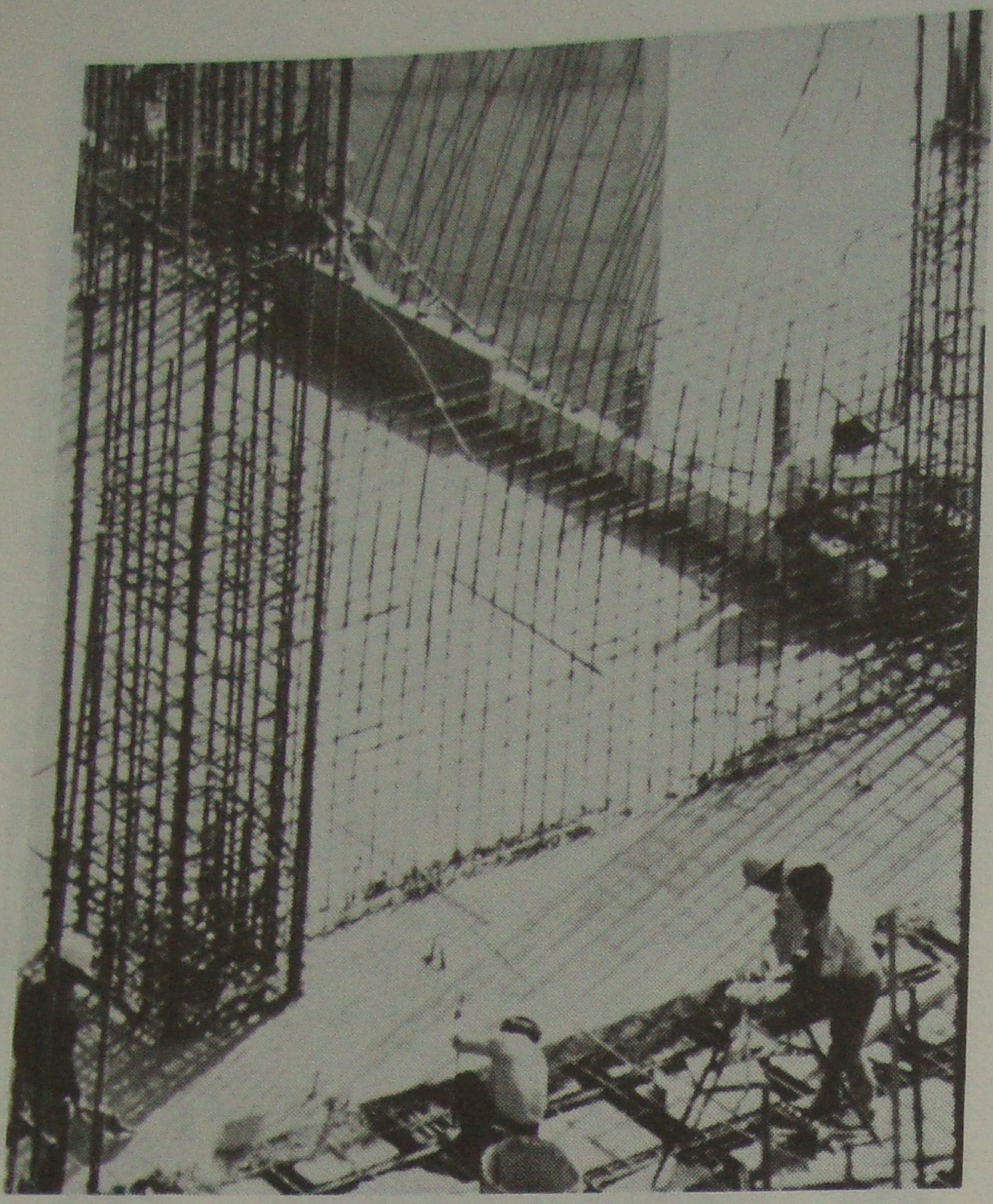

Fig. 14. Shear wall reinforcement details in Nikko Hotel.

Figure 14 shows the shear wall reinforcement details for the Nikko Hotel. These walls have boundary elements in the form of columns containing confinement reinforcement.

4.2 Code provisions for masonry structures

The 1977 masonry code (Instituto de Ingeniería 1977c) gives methods for determining the compressive strength of masonry as a function of the nominal compressive strength of the bricks or blocks as well as the type of mortar. The average nominal compressive strength of blocks is taken as 0.67 times the average measured compressive strength for plant produced and 0.53 times the average measured strength for blocks not factory produced. Masonry walls are classified as follows:

1. "Diaphragm walls" are defined as being surrounded by beams and columns of a structural frame.
2. "Confined walls" consist of infilled masonry walls confined by reinforced concrete frame members or secondary beams and pilasters with thicknesses at least equal to the wall thickness. These confining members must have a concrete compressive strength of at least 15 MPa and contain at least 3 longitudinal bars with a minimum percentage of steel equal to 0.2 f'_c/f_y. These bars must be anchored to the boundary elements such that the yield strengths of this steel can be developed. These members must contain a minimum amount of transverse reinforcement equal to 1000 $s/f_y d_c$ where s is the distance between the ties or stirrups and d_c is the depth of the confining member. The maximum spacing between the ties or stirrups must not exceed 1.5 d_c nor 200 mm. The maximum horizontal distance between confining members is 1.5 times the wall height but cannot exceed 4 m. The maximum vertical distance between confining members is 3 m. All openings greater than 1/4 of the wall dimensions must have confining members around their perimeter. If the height to thickness ratio of the wall is greater than 30 additional stiffening members must be provided to prevent buckling.
3. "Reinforced masonry walls" must contain horizontal and vertical reinforcement. The sum of the horizontal and vertical reinforcement ratios must be at least 0.002 with neither ratio being less than 0.0007. Walls with height to thickness ratios greater than 30 must contain stiffening members to prevent buckling. Longitudinal bars must be located a clear distance of at least 1/2 of the bar diameter from the face of the internal cavity and the cavity must be filled with concrete or mortar over the wall height. The bars must have a clear cover to the outside of the wall of at least 15 mm or the bar diameter whichever is greater. At least one #3 bar should be placed in 2 consecutive cavities around the wall boundary, at all wall intersections and the spacing between these groups of bars should not exceed 3 m. In addition the individual vertical and horizontal bars in the wall must be at least #3 and must have a maximum spacing of 6 times the wall thickness or 900 mm whichever is smaller. The horizontal steel shall be continuous and anchored at the ends of the wall.
4. "Unreinforced walls" are those not meeting the above requirements.

For ordinary structures the code (Instituto de Ingeniería 1977a) gave a simplified analysis procedure for low-rise (less than 13 m in height) structures primarily consisting of walls. For example for a structure with brick walls in Zone III the lateral force coefficient for a structure between 4 and 7 m in height is 0.09 (0.10 for hollow block walls). These coefficients must be multiplied by 1.3 for important structures.

4.3 Code provisions for steel structures

The requirements of the 1977 steel design code (Instituto de Ingeniería 1978) and the recommendations given by Bazán and Meli (1985) for steel structures designed with a ductility factor, Q = 6 are:

1. Only compact sections may be used to achieve the desired level of ductility.
2. Beams are designed with an increased load factor for shear and torsion of 1.4. For the usual grade of steel (ASTM A36) Bazán and Meli (1985) recommend that the ends of beams be laterally supported (over a length equal to twice the beam depth) by bracing with a maximum spacing of 63.2 times the radius of gyration of the beam.
3. The factored axial load on columns must not exceed 0.6 times the axial yield load, P_y, of the column. For column loads in excess of 0.15 P_y a load factor of 1.4 instead of 1.1 must be used.
4. Joints must be capable of transmitting the yield forces of beam flanges.

5 STRUCTURAL DAMAGE IN MEXICO CITY

The September 19, 1985 earthquake and its aftershock on September 20 caused unprecedented damage in Mexico City as discussed below.

5.1 Severity of ground motion

The peak acceleration on firm ground at UNAM was 4% g with a slightly higher peak acceleration of 4.5% g measured in the transition zone (See Fig. 5 and 6). The peak acceleration at the SCT site in the lake zone region was 20% g. This strong ground motion had 11 cycles greater than 10% g with a dominant period of about 2 sec. The unusual ground motion characteristics, that is , an almost sinusoidal vibration at a period of 2 sec, a large peak acceleration together with the relatively long duration of this motion was a major cause of the structural damage in Mexico City.

The response spectra calculated from the E-W accelerograms recorded at the three seismograph stations shown in Fig. 5 are given in Fig. 15. Also shown are the elastic response spectra from the 1977 code. As can be seen the response

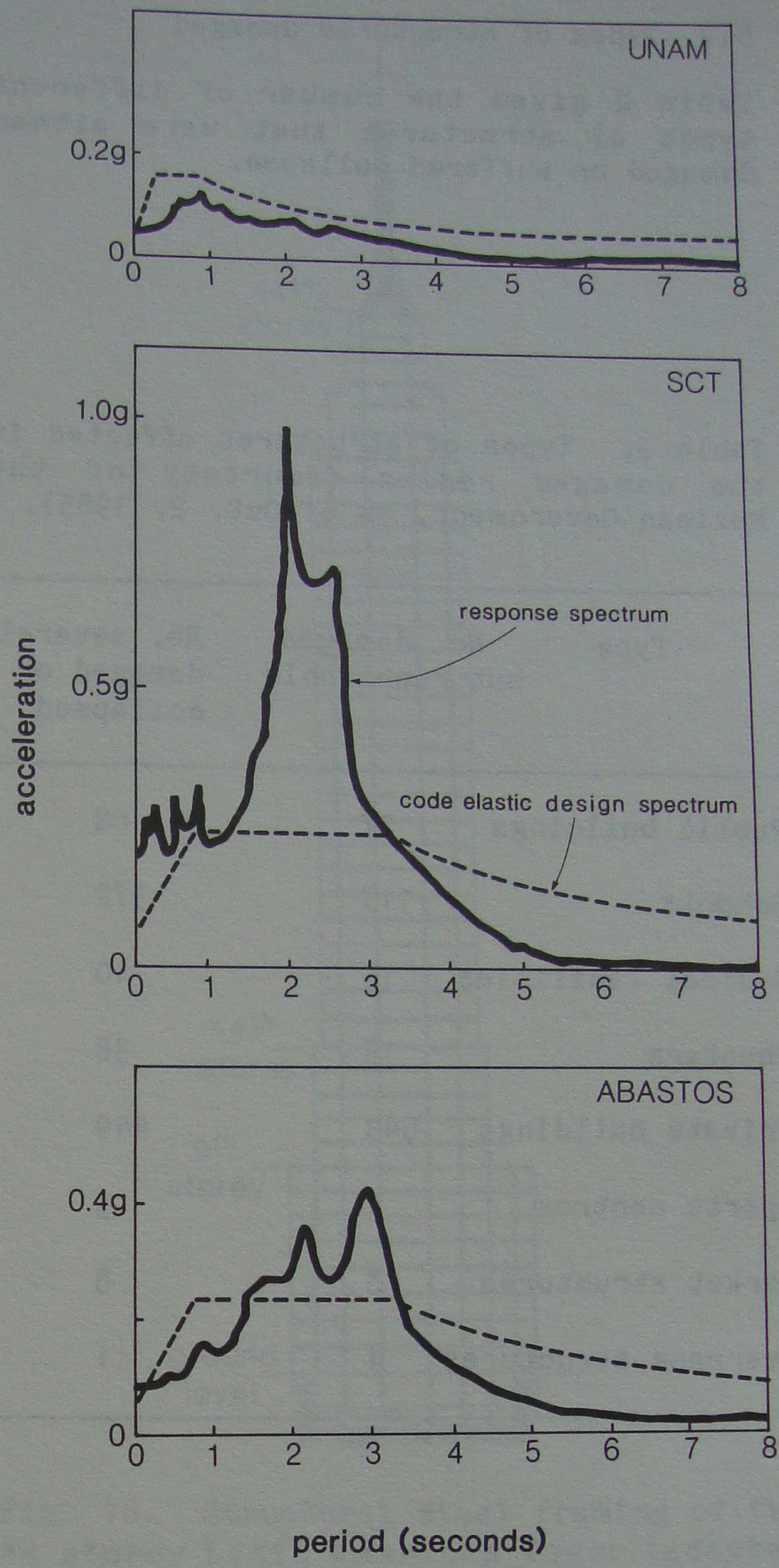

Fig. 15. Response spectra calculated for E-W components recorded at UNAM, SCT and Abastos for 5% damping (Prince et al. 1985a).

spectrum determined from the records at UNAM is below the design spectrum for this zone I site. The response spectrum for the SCT site in the lake bed region (zone III) indicates a peak response of 1.0 g at a period of about 2 sec. This response is clearly much greater than the design spectrum and is certainly one of the principal causes of the severe damage in this lake bed region. The response spectrum for the Central de Abastos site, which has a thicker layer of compressible clay, gives a maximum response of about 0.4 g at a period of about 3 sec.

5.2 Types of structures damaged

Table 2 gives the number of different types of structures that were either damaged or suffered collapse.

Table 2. Types of structures affected in the damaged region (courtesy of the Mexican Government, as of Oct. 2, 1985).

Type	No. damaged but repairable	No. severely damaged or collapsed
Public buildings	27	68
Schools	115	372
Medical facilities	15	10
Theaters	18	38
Private buildings	643	449
Sports centres	1	3
Market structures	5	8
Overpass structures	1	1

Figure 16 classifies 263 damaged or collapsed structures into type of structural framing and year of construction. Out of the 263 structures surveyed 142 were concrete frame structures, 10 were steel frame structures, 86 were reinforced concrete flat plate structures, 17 were masonry structures and 8 had other structural framing systems. As can be seen from Fig. 16 the majority of the structures were built after 1957.

A survey of damage reported by the Instituto de Ingeniería (1985) indicated that buildings in the damage zone between 6 and 15 storeys were most affected by the 1985 earthquake as shown in Table 3. In all, 3% of the structures suffered damage in the zone of damage outlined in Fig. 7.

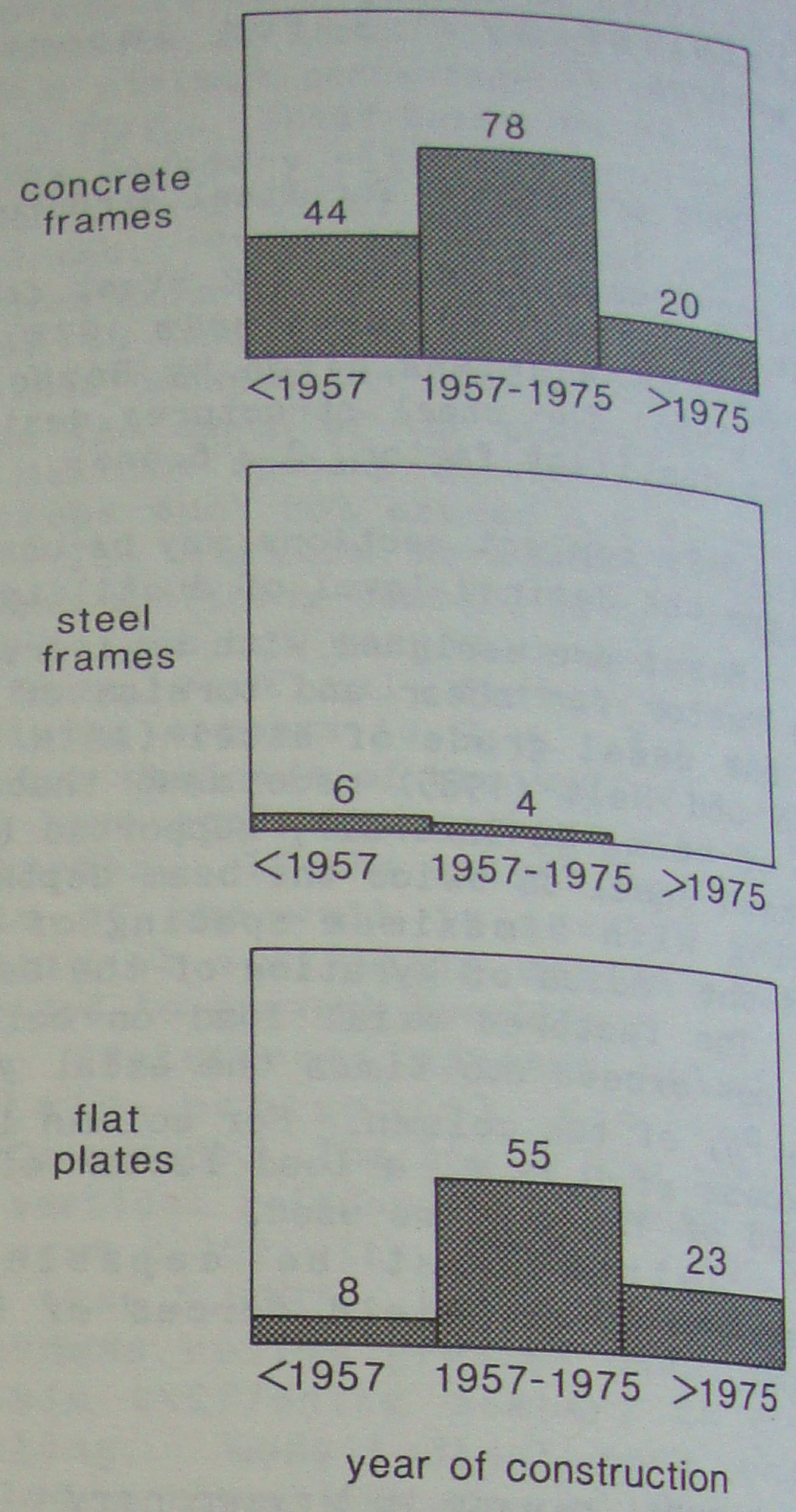

Fig. 16. Damage survey of 263 structures according to type of framing and year of construction.

Table 3. Percentage of buildings in each height category that suffered severe damage or collapse.

No. of storeys	Percentage collapsed or severely damaged
≤ 2	2%
3 to 5	3%
6 to 8	16%
9 to 12	23%
> 12	22%

5.3 Effect of foundation rocking and cumulative damage

Figure 17 illustrates the reasons for the susceptibility of the 6 to 15 storey structures to severe damage. The

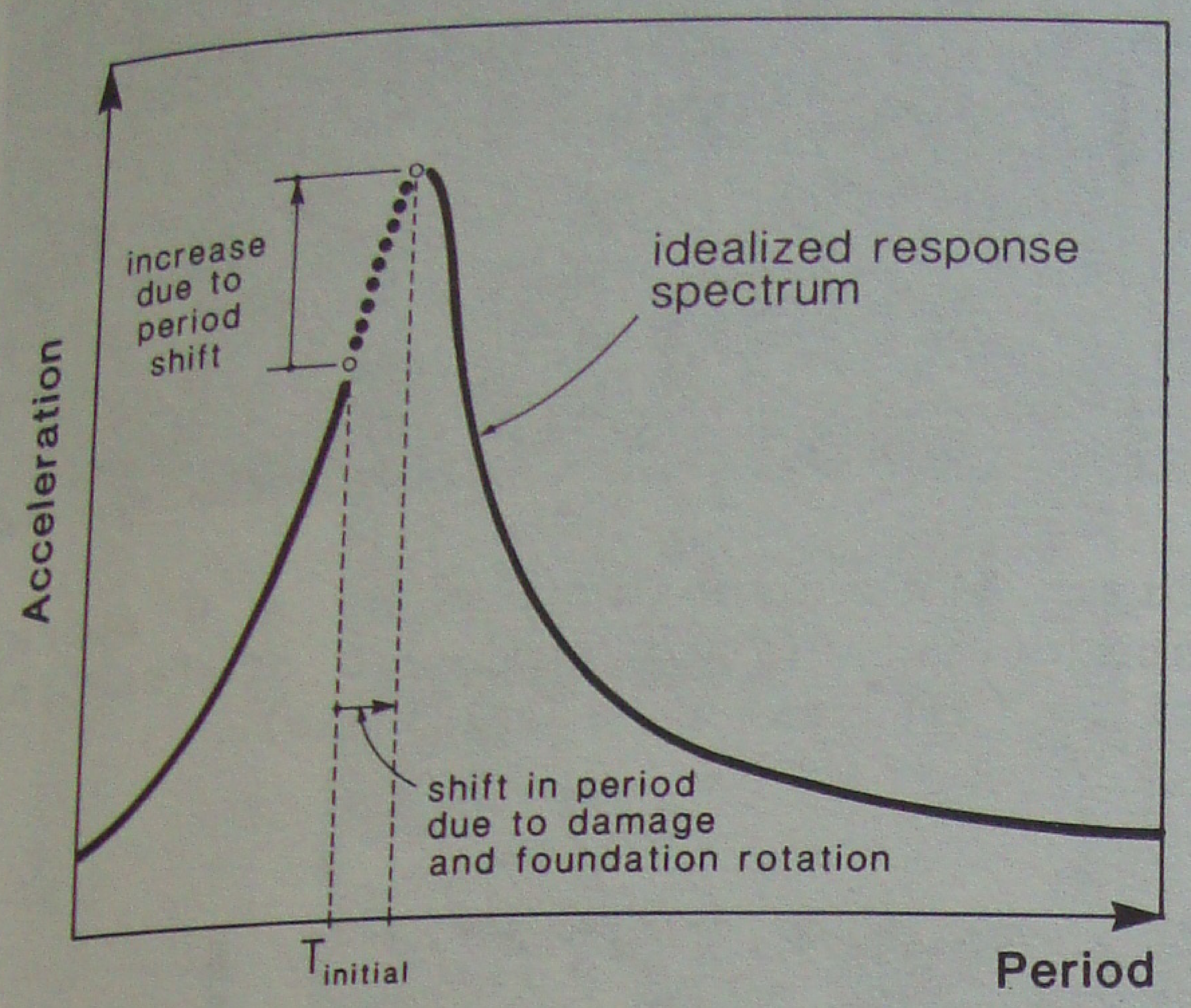

Fig. 17. Accumulated damage of 6 to 15 storey structures (Mitchell et al. 1986a&b).

idealized response spectrum represents the response spectrum for the lake zone region with compressible soil depths of about 30 m. If the 6 to 15 storey structures had rigid foundations and responded elastically they would have fundamental periods of vibration below the 2 sec critical period. These structures had their periods of vibration lengthened due to both foundation rocking and due to structural damage. With each successive cycle of strong ground motion the structures suffered cumulative damage, the period of vibration increased and as a result the structure attracted greater inertial forces! This resulted in severe damage or collapse in many cases.

It is interesting to note that a taller structure (greater than about 20 storeys) would attract smaller inertial forces as its period lengthens due to foundation rocking and due to inelastic response. For example the steel-framed 44 storey Latin American Tower (see Fig. 18) has calculated periods of vibration of 3.66, 1.54 and 0.98 sec for the first, second and third modes (Zeevaert and Newmark 1956). This moment resisting frame structure is founded on end bearing piles and is situated in the compressible soil region of Mexico City. The period of rotation of the foundation is 1.31 sec resulting in combined structure-foundation periods of 3.88, 2.02 and 1.64 sec for the first, second and third modes (Zeevaert 1972). The effect of foundation rotation is to increase the periods of vibration, particularly the higher modes of vibration. The increase in the periods of higher modes of vibration results in a greater participation of these higher modes resulting in greater overall inertial forces and in particular larger inertial forces in the upper storeys. This phenomenon was an important factor in the damage to the upper storeys of many structures.

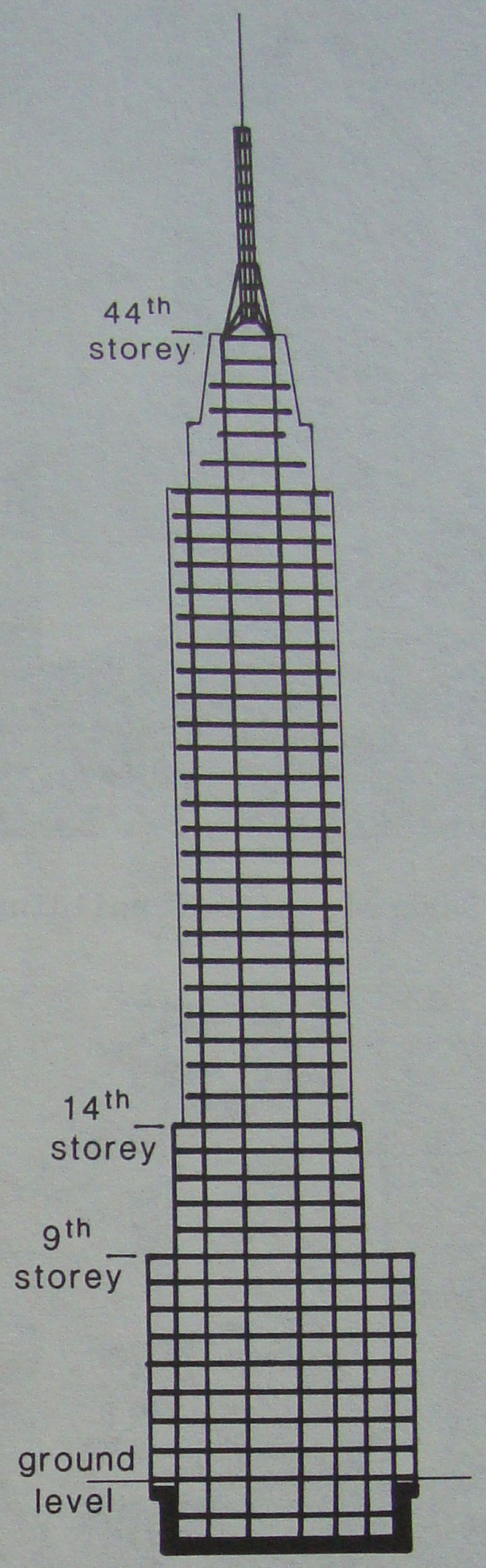

Fig. 18. Structural steel framing of the 44 storey Latin American Tower (adapted from Zeevaert and Cuevas 1983).

The Latin American Tower which was completed in 1956 has been subjected to three major earthquakes (1957, 1979 and 1985) without experiencing structural damage.

Fig. 19. Severely tilted building.

Fig. 20. Foundation settlement of about 1 m relative to road.

5.4 Foundation problems

The main influence of the subsoil was to amplify the strong ground motion and to filter out the high frequencies causing severe damage and collapse of many structures. In addition there were many examples of foundation failures. Figures 19, 20 and 21 give examples of foundation tilting, foundation settlement and overturning due to pullout of friction piles. A close-up of the pull-out of a friction pile is given in Fig. 22.

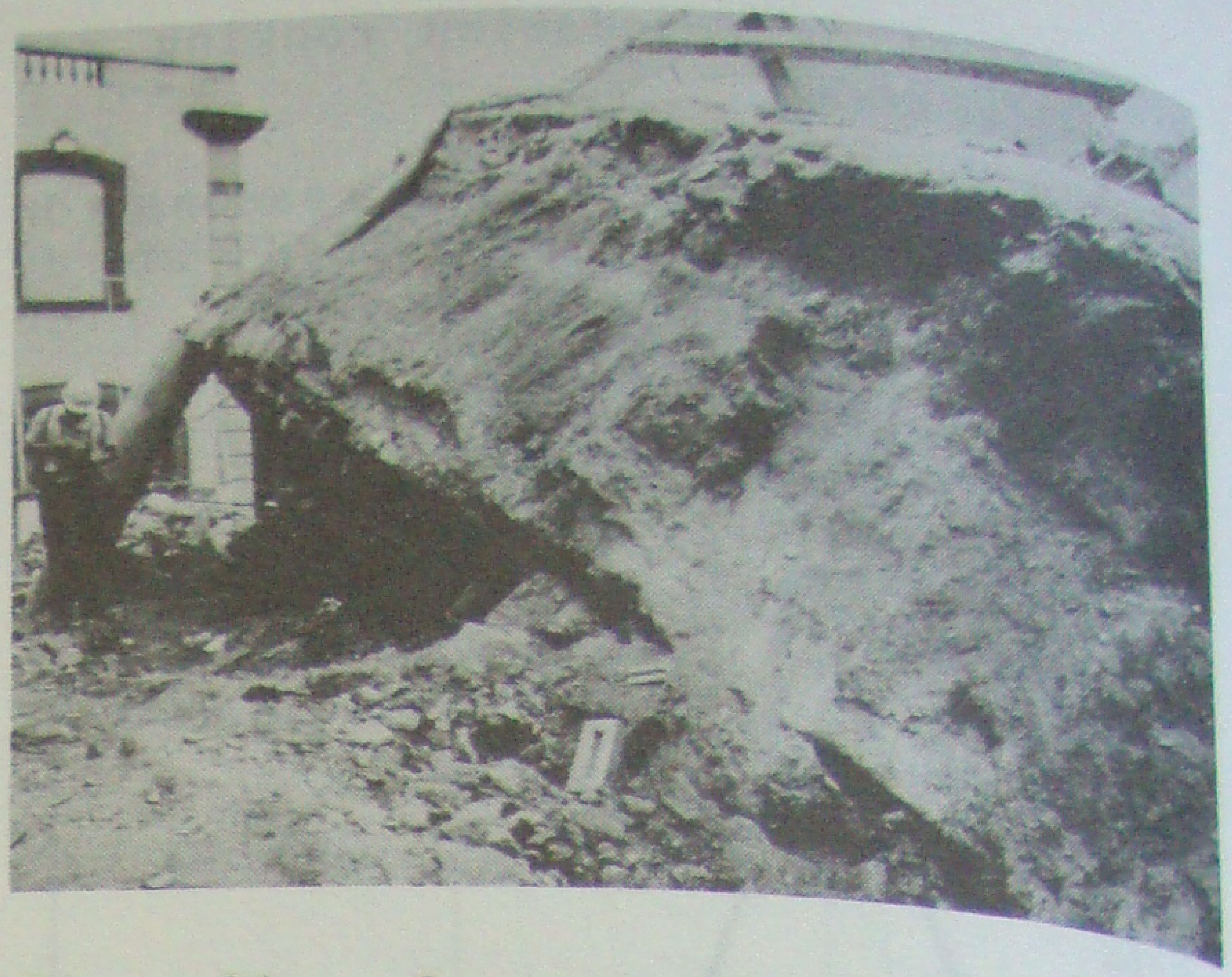

Fig. 21. Overturning of a 9 storey structure due to pull-out of friction piles (Mitchell et al. 1986a&b).

Fig. 22. Close-up of pile pull-out.

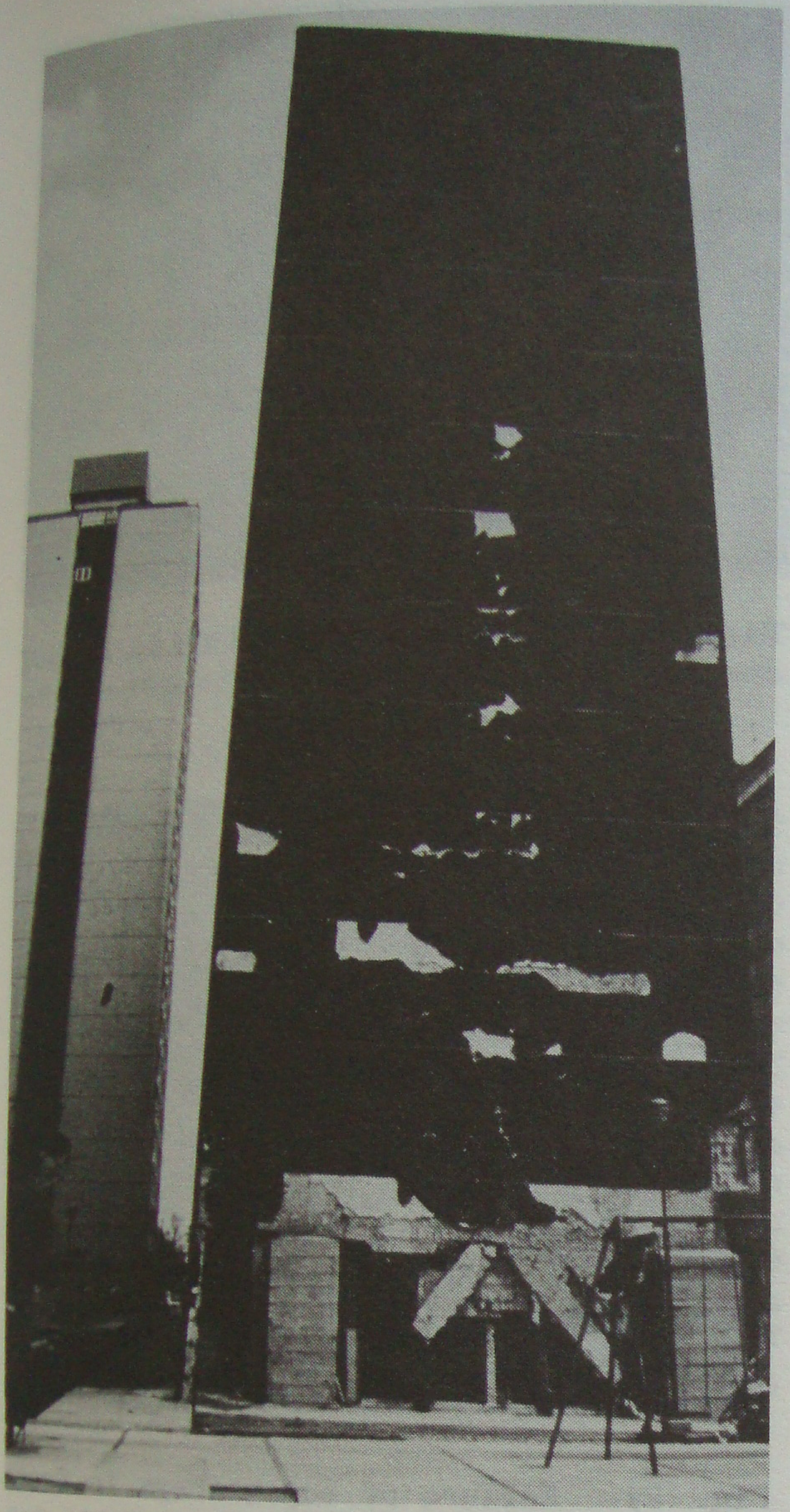

Fig. 23. Failure of reinforced concrete bracing and infilled masonry wall of 13 storey apartment structure.

5.5 Damage due to torsional eccentricity

Although structures with symmetrically placed masonry walls performed well during the earthquake many structures with eccentrically placed walls suffered severe damage. Figure 23 shows the damage to an end wall at the first storey level of a long, narrow apartment building. This figure also illustrates the use of reinforced concrete diagonal bracing members with infilled masonry which is common in Mexico City. The bottom storey is shored with timber columns and two large concrete piers have been cast at the corners of the wall to facilitate evacuation of this structure. A close-up view of the diagonal bracing members is given in Fig. 24. Figure 25 shows the failure of the joint region of a reinforced concrete structure with diagonal braces. This structure was triangular in plan and was subjected to large torsional loading due to the eccentrically located infilled masonry walls.

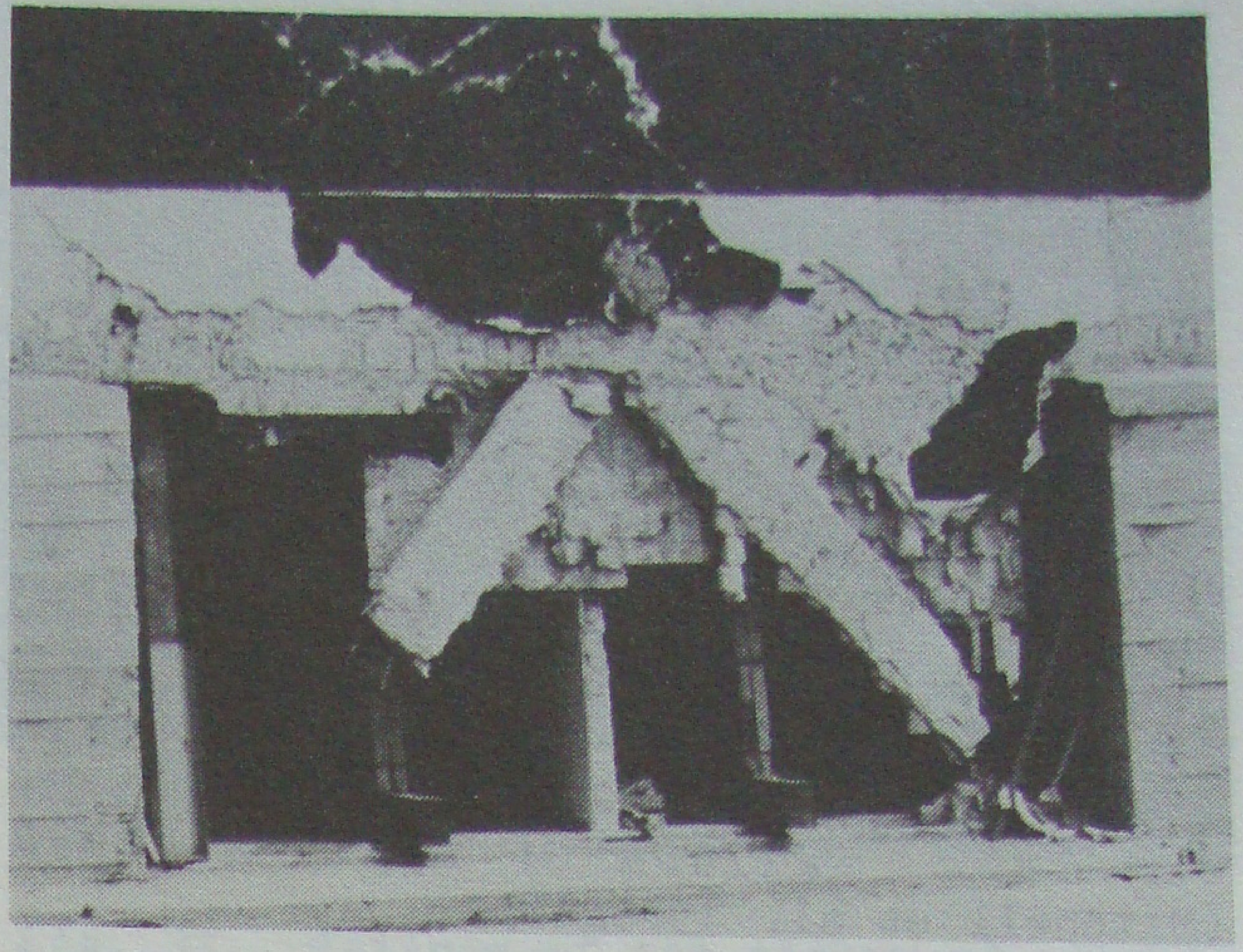

Fig. 24. Close-up of bracing failure.

Fig. 25. Failure of joint of reinforced concrete braced structure with masonry wall infills.

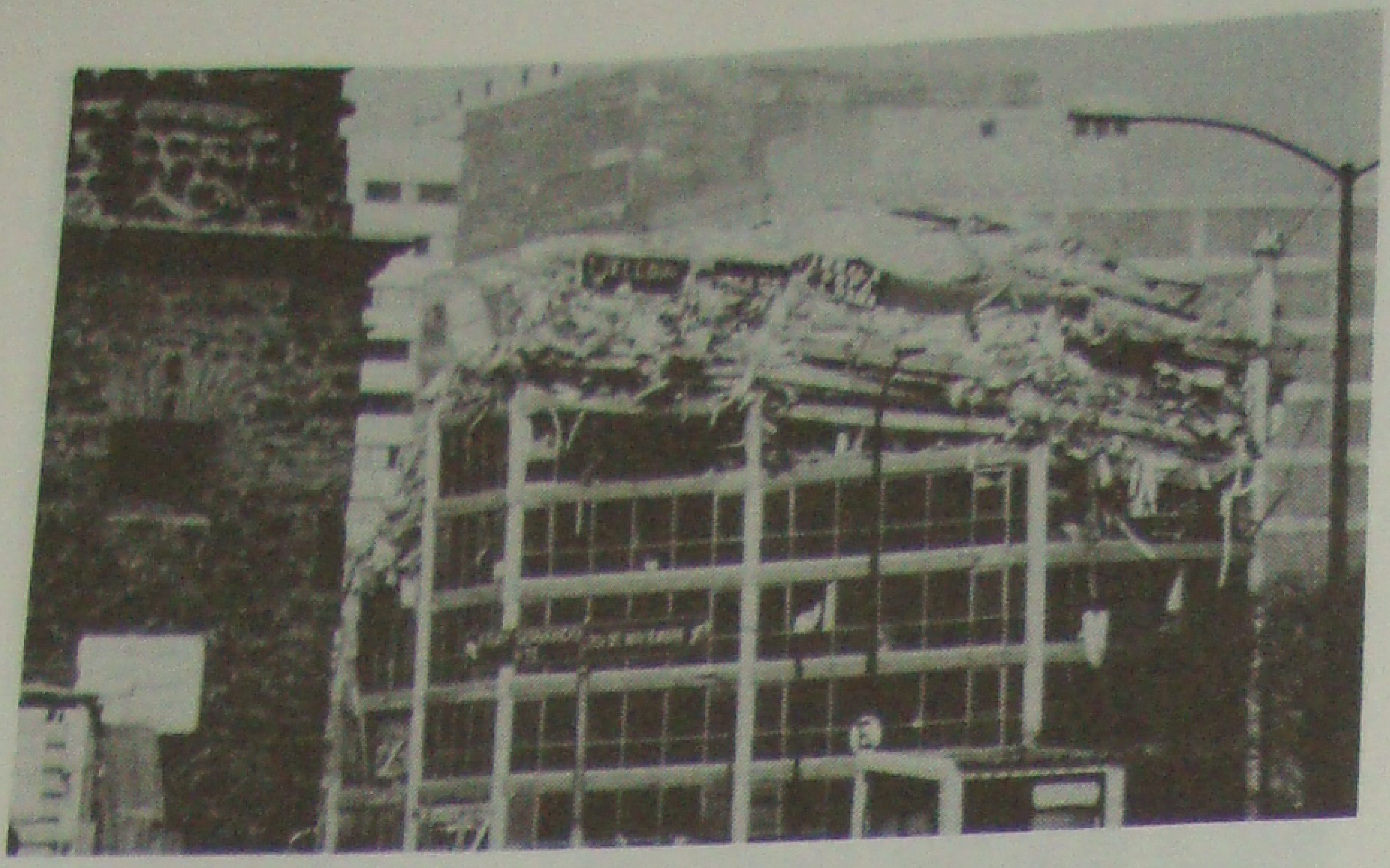

Fig. 26. Failure of upper storeys (Mitchell et al. 1986a&b).

Fig. 27. Failure of upper storeys of government building with heavy storage loads.

5.6 Failures of upper storeys

There were numerous examples of failures in the intermediate and upper storeys of structures. Factors which contributed to this phenomenon were: (a) increased participation of higher modes due to foundation rocking, (b) the practice of significantly reducing column sizes of

Fig. 28. Failure of top stories of SCT complex.

Fig. 29. Failure of top storeys in front wing of structure.

Fig. 30. Example of pounding failures in adjacent parking structures (Mitchell et al. 1986a&b).

Fig. 31. Collapse of the top 6 storeys of the Continental Hotel (Mitchell et al. 1986a&b).

upper storey columns, and (c) the use of upper storeys for storage especially in government buildings. Figure 26 illustrates upper storey failures of an isolated building. Figure 27 shows the effect of large storage loads in a government building which suffered collapse starting in the second storey from the top. The large SCT government complex is located in the compressible soil region where the largest accelerations were recorded. The collapse of the upper floors of this structure is shown in Fig. 28. Another example of failure of upper storeys is given in Fig. 29.

5.7 Pounding of adjacent structures

Due to inadequate separation between buildings and due to the flexibility of some structures there were many examples of collisions or pounding between adjacent structures. Figure 30 shows the failure due to pounding of two adjacent parking structures (a large span precast structure and a reinforced concrete frame structure with infilled masonry walls). The Continental Hotel suffered severe collapse of the top 6 storeys of a 12 storey wing as shown in Fig. 31. This 12 storey wing impacted against the 15 storey wing introducing large lateral forces at the top of the 12 storey structure. Figure 32 illustrates the top storey damage to a large waffle slab structure due to pounding of adjacent wings. There were several cases of pounding between adjacent structures whose floors were at different levels. This resulted in severe damage to the columns (see Fig. 33).

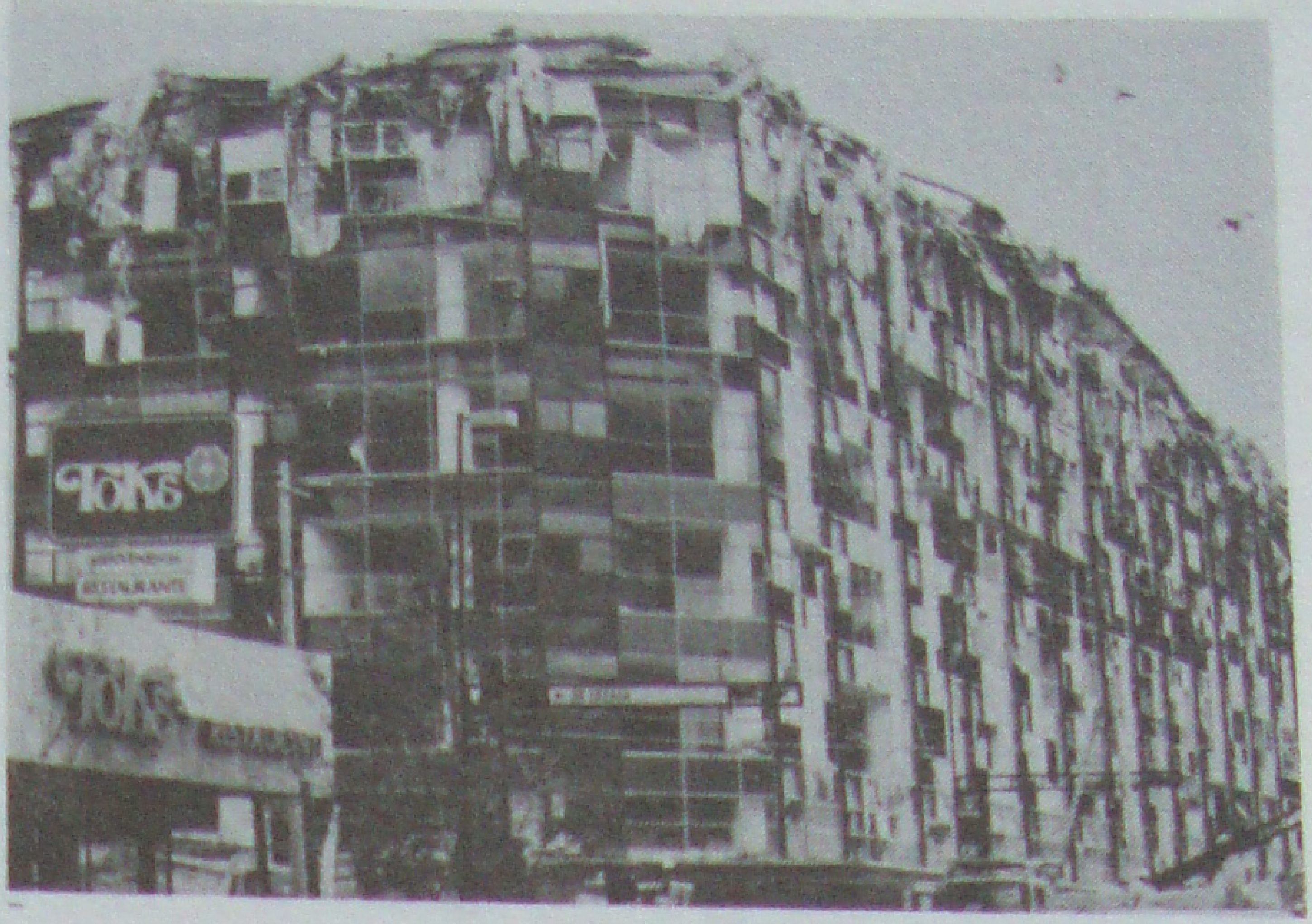

Fig. 32. Failure of top storey of a large structure due to pounding of adjacent wings.

Fig. 33. Shear failure of column due to pounding between adjacent structures.

Fig. 34. Upper storey failures of flat plate structure.

Fig. 35. Punching shear failures in 4 storey school structure with embedded hollow block masonry in floor slabs.

5.8 Flat plate and waffle slab structures

Structural systems consisting of waffle slabs and columns are very common in Mexico City. This type of structural system is very flexible and is susceptible to P-delta effects. A large number of failures occurred due to punching shear

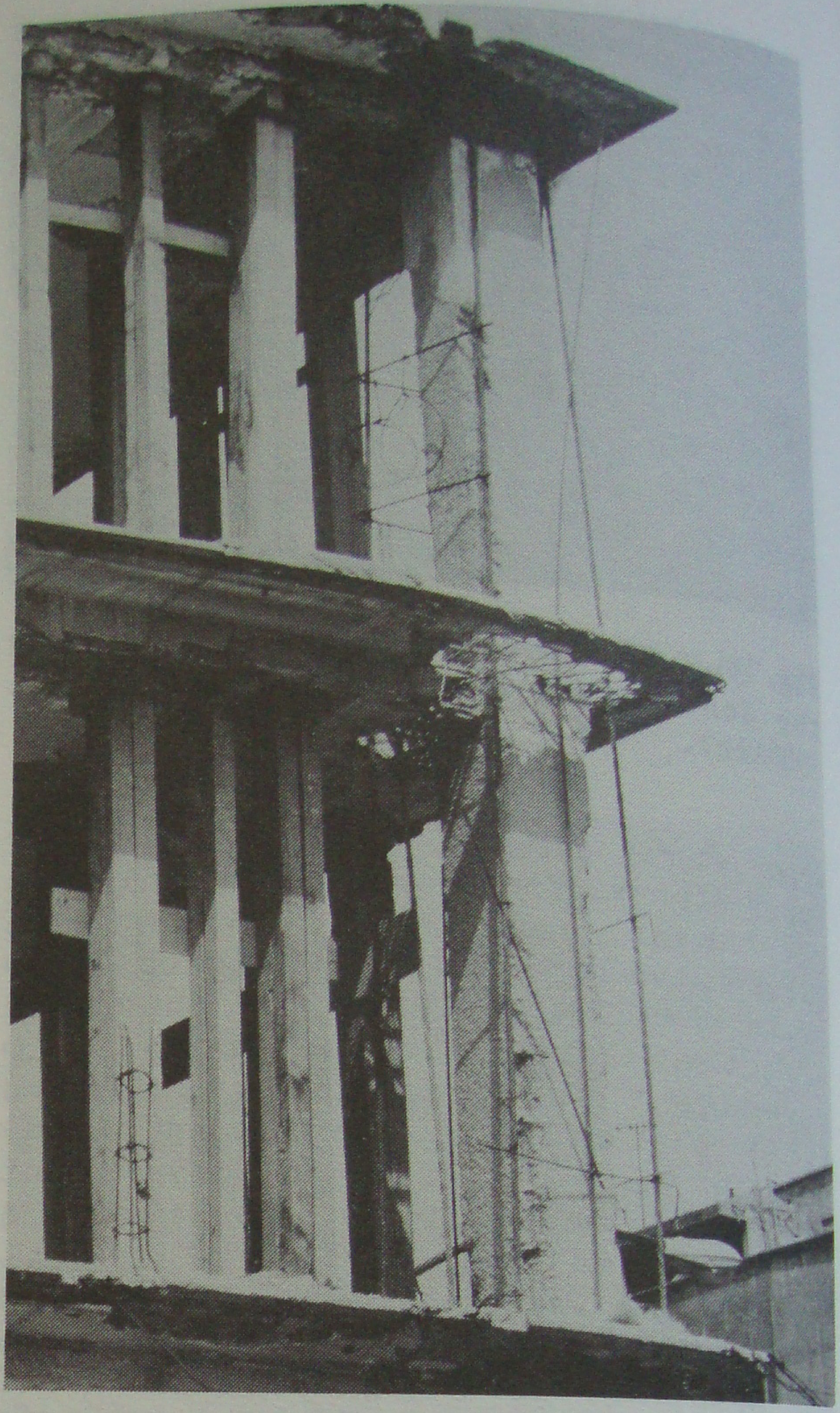

Fig. 36. Punching shear failure at corner column.

failures in the slabs. Figures 34, 35 and 36 illustrate failures in these types of structures.

5.9 Reinforced concrete column failures

There were many examples of failures of "short" columns. Such situations were caused by openings in the infilled walls of concrete frame-wall structures exposing a short length of column (see Fig. 37). Figure 38 illustrates an example of "short" columns created by relatively deep spandrel beams. The short column in Fig. 39 was caused by a masonry filler panel. These short columns are susceptible to shear failures as can be seen from these three figures.

Fig. 37. Shear failure of short exposed column due to partially infilled masonry wall.

Fig. 38. Shear failure of short exposed column due to relatively deep spandrel beams.

Figure 40 shows a shear failure in a reinforced concrete circular column. Examples of failures of columns due to inadequate confinement details are given in Fig. 41, 42 and 43. These columns which failed had wide tie spacings, 90 degree bend anchorages for the ties and large spaces between the longitudinal bars. The 90 degree bend anchorages became ineffective after the concrete cover spalled off. In addition the longitudinal steel in the column shown in Fig. 41 had ruptured without displaying any "necking down" of the cross-section. This indicates that the reinforcement was brittle.

5.10 Reinforced concrete beam failures

Figure 44 shows the buckling of the bottom longitudinal bars of a beam due to the wide spacing between the stirrups.

Figure 45 shows the shear failure of a large reinforced concrete beam that was supporting a masonry infilled frame-wall. The stirrups across the shear failure plane have ruptured.

5.11 Reinforced concrete joint and anchorage failures

Figure 46 illustrates the failure of a corner joint. Note the congestion of the hooked anchorages for the beam bars, the placement of the beam bars outside of the column bars and the absence of any transverse reinforcement in the joint region. Figure 47 also illustrates an

Fig. 39. Short column failure due to presence of partial storey height masonry wall (Mitchell et al. 1986a&b).

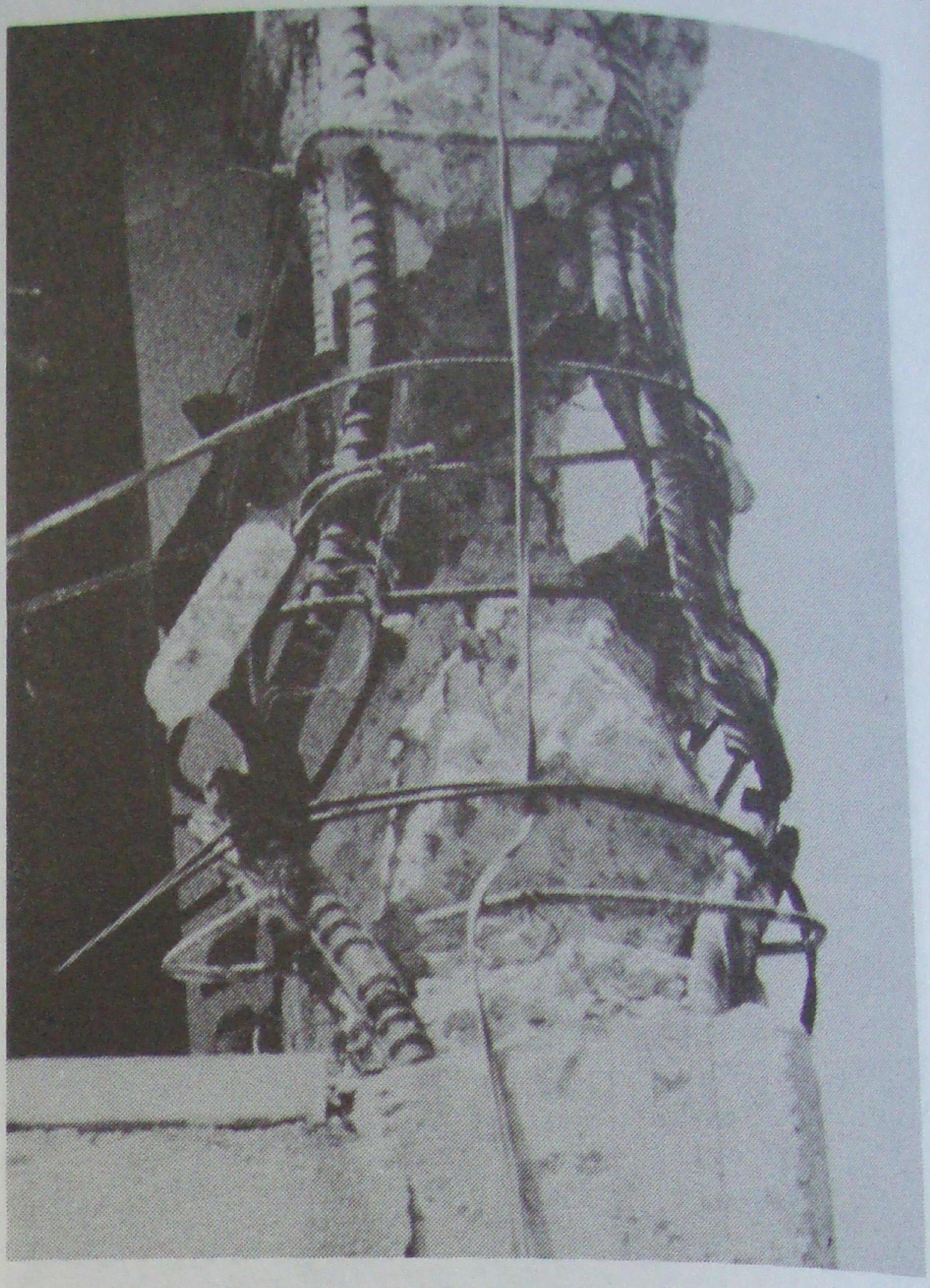

Fig. 41. Failure of column in second storey of 14 storey Nuevo León apartment building (Mitchell et al. 1986a&b).

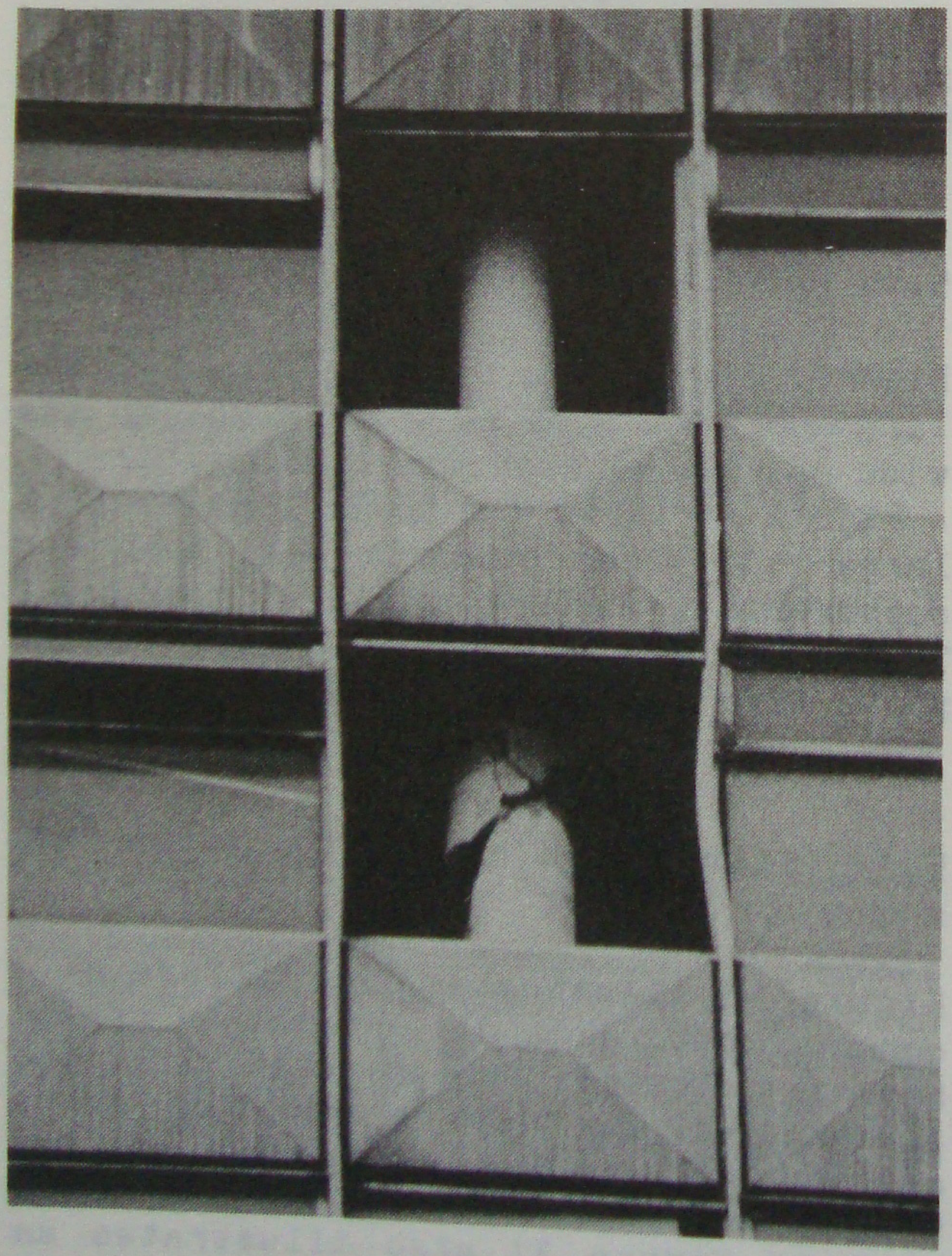

Fig. 40. Shear failure of circular column.

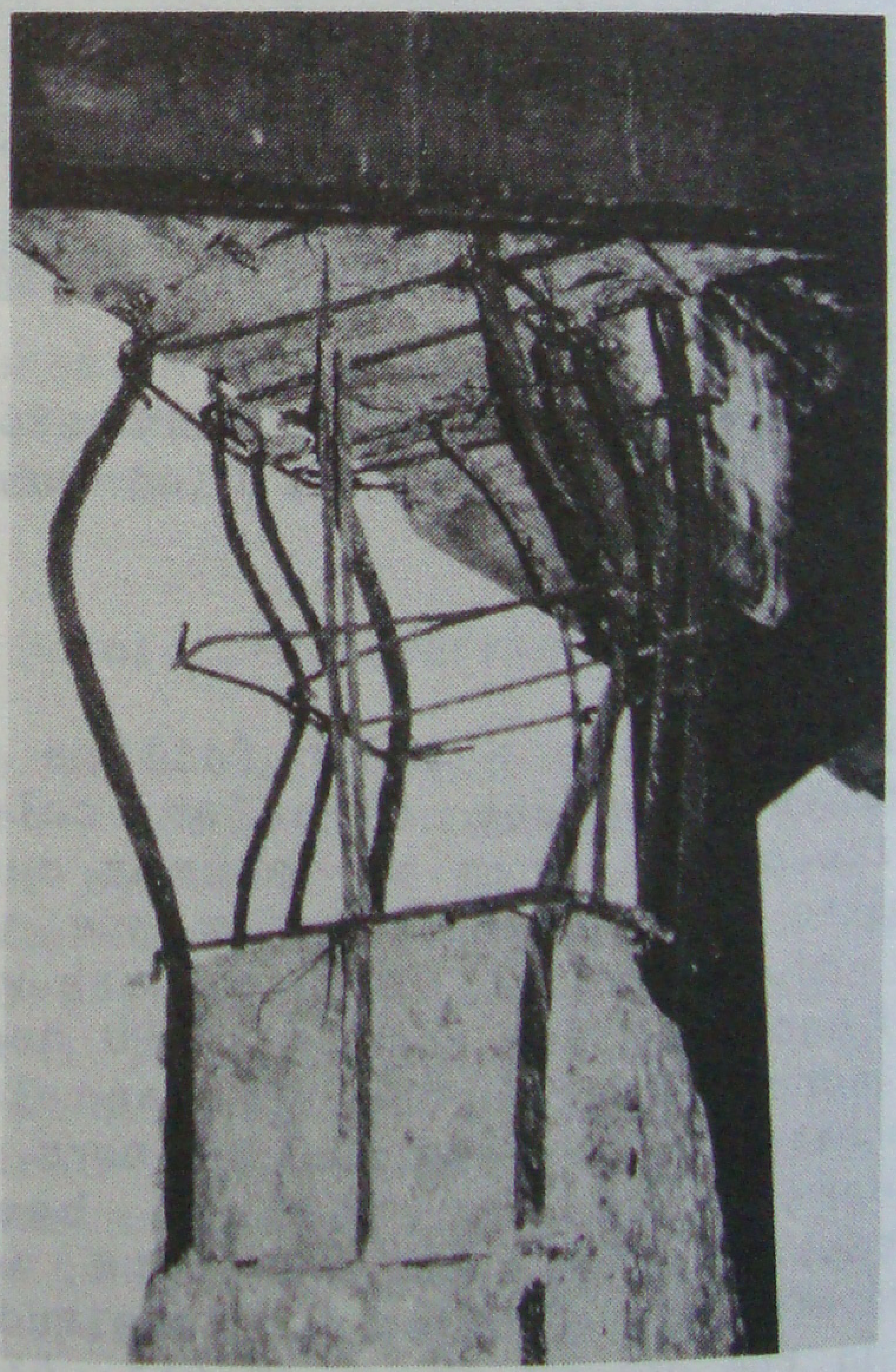

Fig. 42. Failure of upper storey column of 9 storey office building.

Fig. 43. Failure of ground storey column of school.

Fig. 44. Buckling of bottom bars in beam of 9 storey office building.

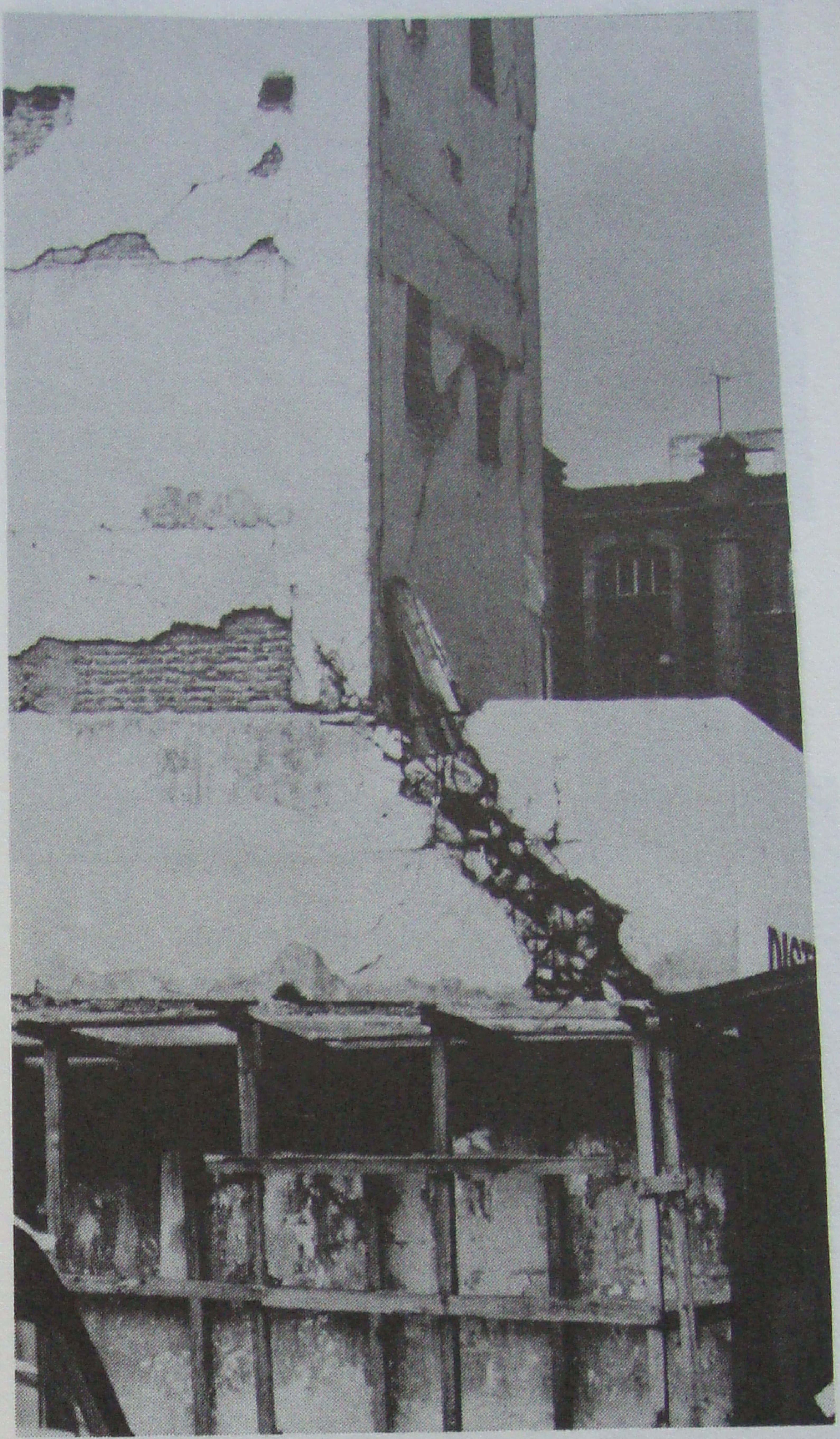

Fig. 45. Shear failure of transfer girder in 13 storey structure.

example of the loss of anchorage of the beam reinforcing bars which were placed outside of the longitudinal column bars. Figure 48 shows a ten storey reinforced concrete frame structure with masonry infilled slabs which experienced significant shear distortions in the joints. A close-up of the joint distress and loss of anchorage of the slab bars is given in Fig. 49.

5.12 Collapse of 21 storey steel structure

The Pino-Suarez government office complex contained three 21 storey structures and two 14 storey structures. One of the 21 storey office towers collapsed onto the adjacent 14 storey structure causing it to collapse as shown in Fig. 50. The framing of these structures consists of built-up tubular steel columns, tubular steel trusses and K-bracing as shown in Fig. 51. Figure 52 shows a buckled corner column of one of the 21 storey structures that is still standing. The tubular columns were fabricated from 4-20 mm thick steel plates which had fillet welds at the corners to form a 400 mm x 600 mm box section. A close-up view of one of the buckled corner columns (see Fig. 53) reveals the local buckling of the plates and the failure of the welds. This structure was severely damaged with a permanent displacement at the top, in one direction, of 1200 mm. In addition permanent torsional deformations of the structure were evident.

Fig. 46. Failure of corner joint.

Fig. 47. Improper anchorage of beam steel in joint region.

6 EMERGENCY CODE CHANGES

On October 18, 1985 a Presidential Decree was published in the "Diario Oficial" (de la Madrid 1985) giving emergency code changes for construction in the Federal District in Mexico City. The articles of this decree are summarized below along with comments on the requirements.

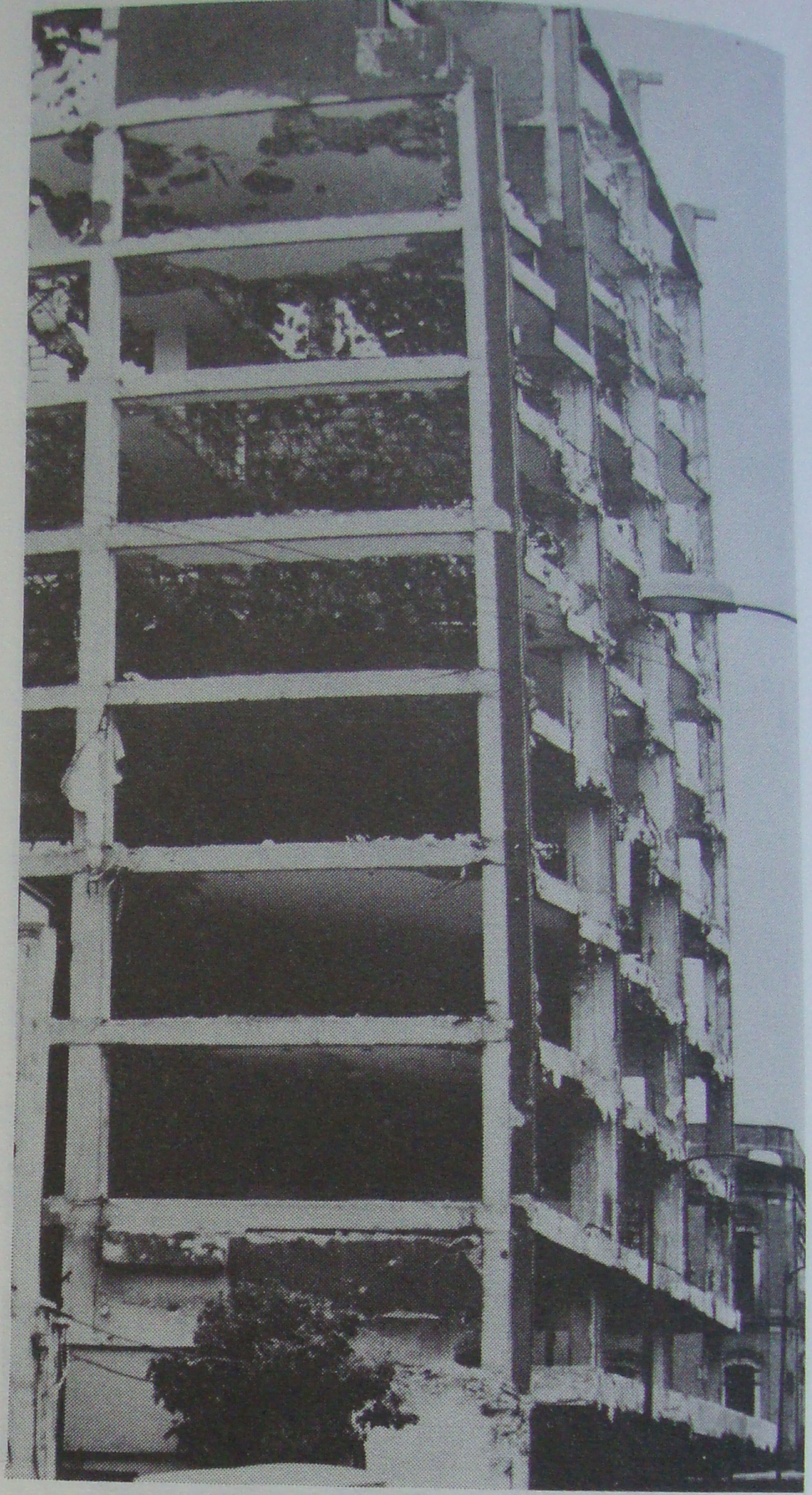

Fig. 48. Ten storey structure with large joint shear distortions.

ARTICLE 1 All buildings which were either damaged, are currently under construction, or will be built in the future must comply to the emergency changes.

ARTICLE 2 These changes apply to all the building codes including the material codes.

ARTICLE 3 All structural damage must be reported to the authorities.

ARTICLE 4 The owners must submit a technical report of the damage to the authorities and authorization must be obtained for any repair or new construction.

ARTICLE 5 Structures under construction on September 19, 1985 that were located in Zones I and II, which did not suffer damage, must satisfy only Article 17 concerning separation between adjacent

Fig. 49. Close-up of joint distress at corner column.

Fig. 50. Collapse of the 21 storey Pino-Suarez steel structure (Mitchell et al. 1986a&b).

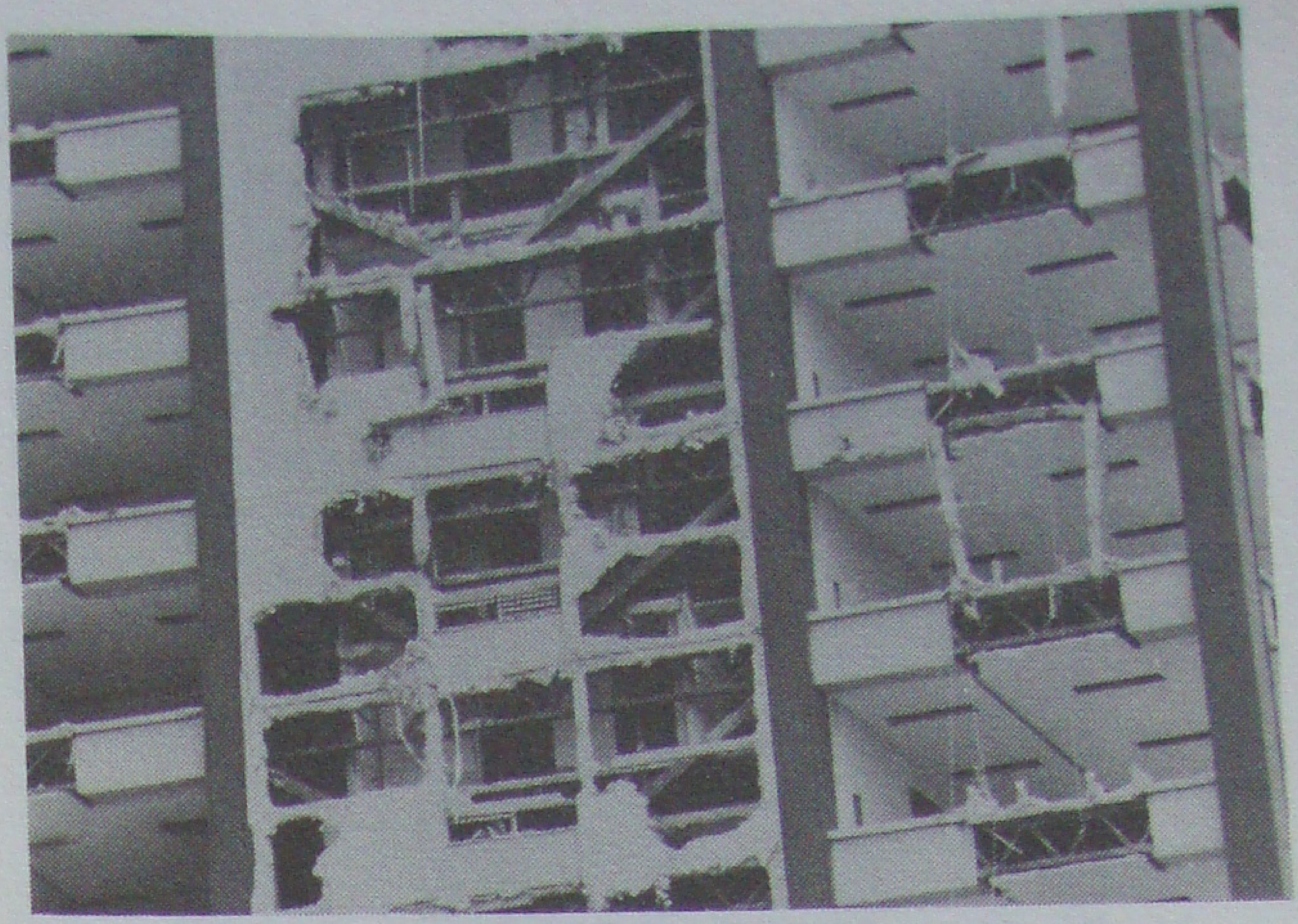

Fig. 51. Structural framing of the Pino-Suarez office towers.

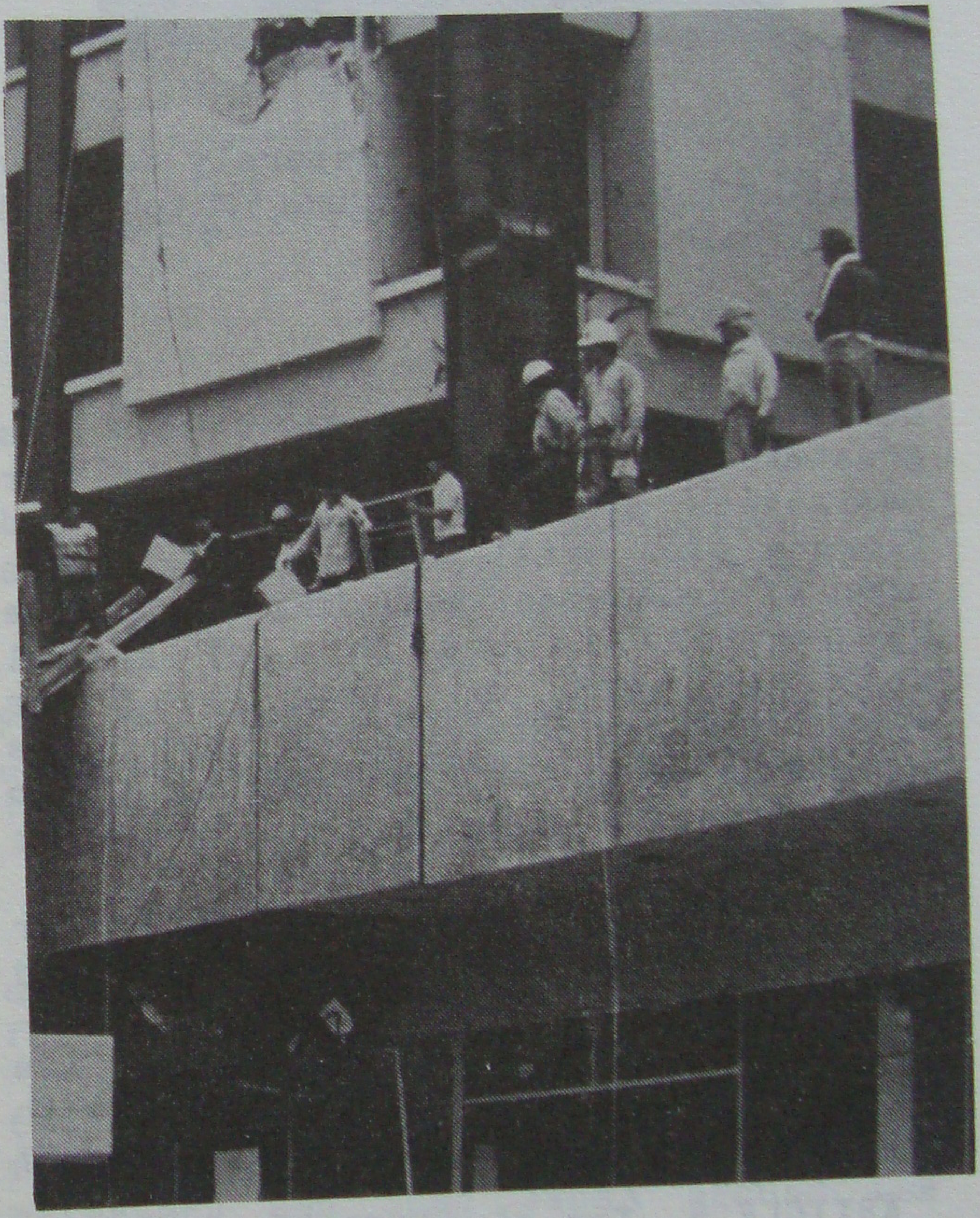

Fig. 52. Buckled column at second storey level of 21 storey structure.

buildings. All structures currently being constructed classified as "important" must be revised according to the emergency changes.

ARTICLE 6 The resistance factor for

Fig. 53. Close-up of severely damaged built-up tubular steel column (Mitchell et al. 1986a&b).

columns where the ductility factor Q is greater than 2 is lowered to 0.5 for resistance under shear, torsion and flexure plus axial compression. If a level of confinement according to the code is provided in the form of spiral reinforcement or with ties or supplementary cross-ties then this resistance factor may be taken as 0.6 in calculating the resistance under flexure and compression.

The resistance factor of 0.35 used for the frictional forces between soil and caissons or piles is emphasized.

<u>ARTICLE 7</u> The live load for office buildings to be used in conjunction with earthquake loading is increased from 1.47 kPa to 1.77 kPa.

<u>ARTICLE 8</u> The simplified method for determining lateral forces may only be applied to buildings less than 8.5 m in height (13 m in 1977 code).

<u>ARTICLE 9</u> The seismic lateral force coefficient for "ordinary" structures is increased from 0.20 to 0.27 for Zone II (transition zone) and from 0.24 to 0.40 for Zone III (compressible soil zone). The vertical intersepts given in Fig. 7 are increased from 0.045 to 0.054 for Zone II and from 0.06 to 0.10 for Zone III.

The seismic lateral force coefficients to be used in the simplified method for "ordinary" low rise structures are increased. The importance factor is increased from 1.3 to 1.5.

<u>ARTICLE 10</u> The ductility factor Q is to be modified as follows:

(a) <u>Q = 6</u> This case is eliminated.
(b) <u>Q = 4</u> In order to use Q = 4 the following requirements must be met:
(i) The resistance in all levels is supplied by unbraced frames together with braced frames or with concrete walls in which the capacity of the frames alone is at least 50 percent of the total.
(ii) The minimum ratio of the resisting capacity of one storey to the design force level should not differ by more than 30 percent of the average of these ratios for all storeys.
(iii) <u>Columns with tie Reinforcement</u> - The minimum dimension of the column must be at least 300 mm, the maximum spacing between longitudinal bars shall not exceed 300 mm and closed ties must anchor at least every alternate bar and all of the corner bars. Also no unrestrained longitudinal bar shall be more than 150 mm from a restrained bar.

Closed ties of at least 9.5 mm in diameter at spacings that do not exceed 200 mm nor 10.8 longitudinal bar diameters (for f_y = 412 MPa) must be provided. These limits are reduced by one-half at the ends of the column over a length equal to the larger column dimension or 600 mm whichever is larger. This results in a decrease in the spacing of the ties near the ends of the column to prevent buckling of the longitudinal steel.

The sum of the tie areas, A_v in each direction of the section of the column shall not be less than 0.4 $p'd_c s_h$ where p' is the volumetric ratio specified by the code, d_c is the core dimension confined by the ties in the direction considered and s_h is the tie spacing.
(iv) The ends of the beams must be designed and detailed to allow the formation of plastic hinges.
(v) The ends of reinforced concrete walls must be reinforced to resist axial compression and moment. If the area of steel exceeds 0.0075 times the area of the wall then the ends must be detailed as columns.
(vi) <u>Steel Frame Structures</u> - The beams and the columns must comply with the requirements for compact sections. If the frame consists of beams made of trusses then the compressive members must be

designed with a resistance factor of 0.7. The beam-column connections must permit large rotations and special attention must be given to the transmission of the horizontal forces through the column.

(c) Q = 3 In order to use a ductility factor, Q, equal to 3 the following conditions must be met:

(i) The resistance at each level is provided by concrete columns with flat plates, or rigid frames of steel with beams made of trusses, or concrete walls or combinations of these in which the contribution of the walls to the lateral load resistance exceeds 50 percent.

(ii) Parts (ii), (iii) and (v) of Article 10(b) must be satisifed.

(iii) The flat plates must comply with Article 12 of these emergency provisions.

(d) Q = 2 A ductility factor, Q, equal to 2 may be used for structures in which the resistance to lateral force is provided by frames of reinforced concrete, wood or steel, braced or unbraced, or concrete walls that do not comply with some of the requirements of Article 10b and 10c. Also solid brick masonry walls confined by pilasters, bond beams, columns or beams of reinforced concrete or steel may be used.

(e) Q = 1.5 A ductility factor of Q = 1.5 may be used if the resistance to lateral loads is provided at all levels by hollow block masonry walls, confined or internally reinforced or a combination of these walls with elements described in cases (b) to (d) above.

(f) Q = 1 A ductility factor of Q = 1 is to be used for structures in which the lateral load resistance is provided at least in part by elements or different materials not specified above, unless it can be demonstrated to the authorities that a higher ductility may be used.

ARTICLE 11 It must be verified that the code requirements for service limit state pertaining to seismic action are satisfied.

ARTICLE 12 In using the equivalent frame analysis for regular slab structures subjected to vertical loads the column stiffnesses should be reduced by one-half. For lateral load analysis an equivalent slab beam width equal to c_2 + 3h is used (where c_2 = column dimension perpendicular to the direction in which moments are being determined and h = slab thickness). At least 75 percent of the longitudinal slab reinforcement necessary to resist seismic loads must pass through the column and the rest of the reinforcement must be within a distance of 1.5 h from the column face. The waffle slabs must contain a solid slab region around the column over a distance of at least 2h from each column face. In the analysis of slabs it is necessary to account for the variation of the moment of inertia of the equivalent beam. The reinforcement of the equivalent beam in the solid portion of the slab around the column must have stirrups having a spacing not exceeding 1/3 of the effective slab depth. (The resulting details of a waffle slab are illustrated in Fig. 54).

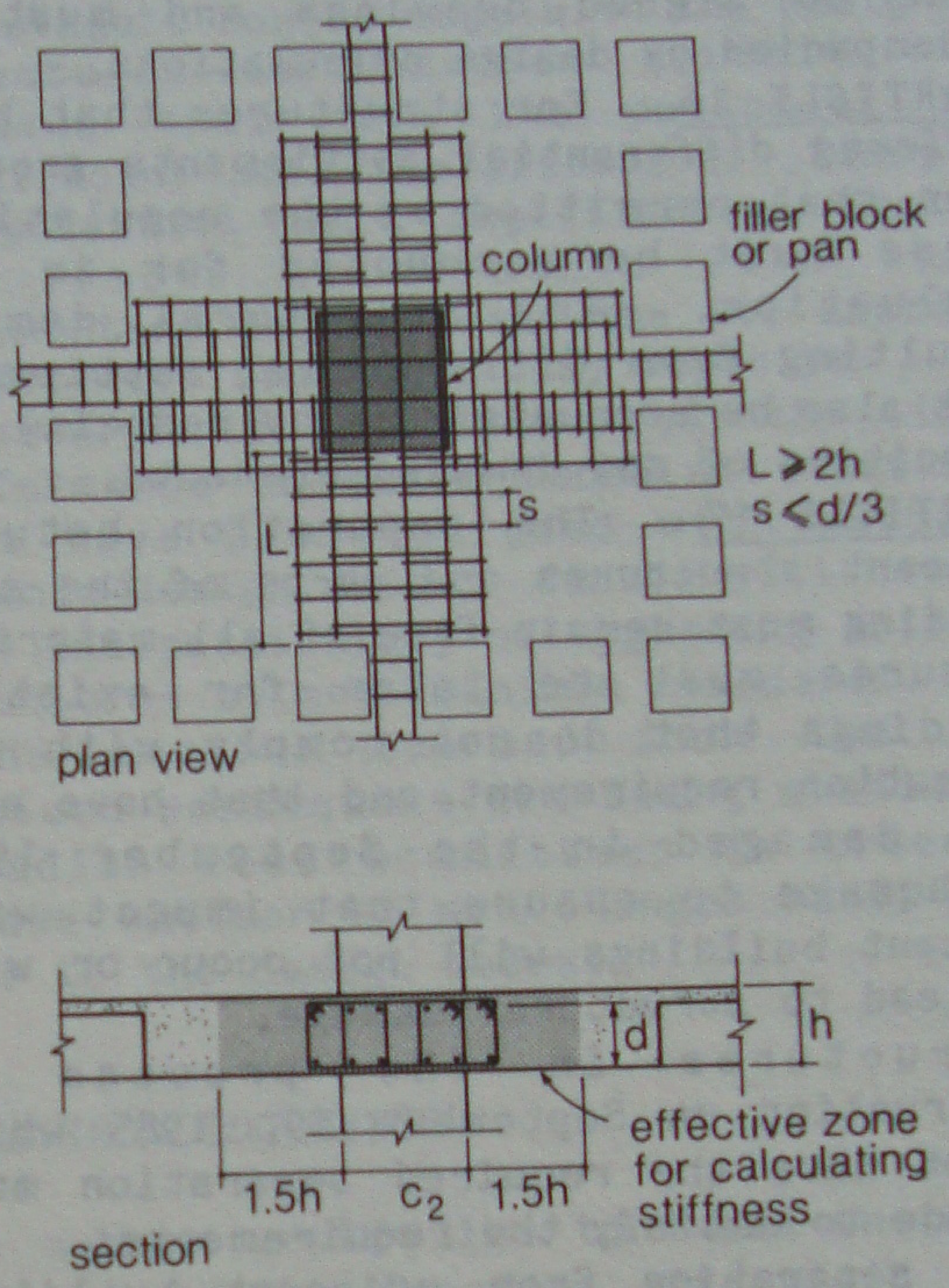

Fig. 54. Waffle slab details required by emergency code changes (Mitchell et al. 1986a&b).

ARTICLE 13 The bearing walls or dividing partitions must be of the following types:

Type I consists of walls contributing to the lateral load resistance and attached firmly to the structural frame or to pilasters and bond beams around the perimeter of the wall. The pilasters and bond beams must in turn be attached to the frame. It must be verified that the beams or slabs and columns are capable of resisting the shear forces, the flexural moments, the axial loads and in some cases the torsions that are induced by the walls. It must also be verified that the connections between these elements are capable of resisting the seismic actions.

Type II consists of walls which do not contribute in resisting lateral loads (partitions and bearing panels). These types of walls must be isolated such that no damage is done from the deformations of the structure.

ARTICLE 14 The calculated torsional eccentricity at any level is not permitted to exceed 20 percent of the plan dimension of this level measured in the direction of the eccentricity.

ARTICLE 15 In inspecting and evaluating the resistance of existing structural elements needing repair the dead and live loads must also be evaluated and the results of the evaluation must be made on signed drawings and must be accompanied by design calculations.

ARTICLE 16 For structures that have suffered differential settlements greater than that permitted by the regulations these must be accounted for in the evaluation. Any structural damage resulting from differential settlement must also be accounted for by reducing the capacities of any damaged elements.

ARTICLE 17 The separation between adjacent structures and parts of the same building must remain free of all material. Measures must be taken for existing buildings that do not comply with the separation requirement and that have also been damaged in the September 1985 earthquake to ensure that impact with adjacent buildings will not occur or will not lead to structural damage.

Structures in the process of construction on September 19, 1985 which do not have the required separation must be made to satisfy the requirements.

The separation from adjacent buildings must be clearly evident on the architectural and structural drawings.

ARTICLE 18 Structures requiring strengthening or repair must be propped up such that the safety is guaranteed under the effects of the estimated dead load and 25 percent of the lateral load required by the present provisions.

ARTICLE 19 The placement and anchorage details for reinforcement and for connections between concrete structural members must be shown to scale on the drawings. When rivets or bolts are used the diameter, the number and the location must be indicated. When the connections are welded all the details must be shown using appropriate symbols and if necessary scale drawings of the details.

The fabrication and erection drawings must give the necessary information so that the fabrication and erection of the structure comply with requirements given in the structural drawings. The drawings of the erection details must be approved by the structural designer in all relevant matters of safety.

ARTICLE 20 Group B (ordinary) structures having a total height of more than 15 m or having a total floor area of more than 3000 m^2 as well as all Group A (important) structures must have the construction supervised by a resident supervisor authorized by the authorities. The supervisor must submit written reports to the authorites on the execution of the construction. Any deviation from the structural drawings must have a previous written approval from the structural designer.

ARTICLE 21 In order to change the use of a structure written documentation must first be filed with the authorities. It must be shown that the proposed change of use does not lead to unfavourable conditions.

7 SUMMARY AND CONCLUSIONS

7.1 Severity of ground motion in Mexico City

The compressible soil in the lake zone of Mexico City resulted in an amplification of the strong ground motion (up to 20% g acceleration) with dominant periods from 2 to 3 sec. This coupled with the long duration (up to 3 minutes at the Central de Abastos) resulted in cumulative structure damage and collapse of many structures.

7.2 Types of structural failures

Most of the structures that suffered severe damage had between 6 to 15 storeys. Structures in this height range were particularly susceptible to cumulative structural damage. As their periods increased with each successive cycle of motion they attracted larger inertial forces due to the shape of the response spectrum in the lake zone.

There were a large number of failures in upper storeys of structures due to the participation of higher modes of vibration, the reduction of column sizes in upper storeys and due to the presence of significant storage loads near the top of some structures. The presence of large torsional eccentricities played an important role particularly due to the presence of eccentrically located masonry walls.

There were many examples of failures due to pounding between adjacent structures due to inadequate separation between the structures. There were a large number of reinforced concrete column failures due to inadequate detailing of the reinforcement, lack of confinement and due to shear failures caused by exposed short column lengths.

Punching shear failures in reinforced concrete slabs caused many structural collapses. Other types of failures of reinforced concrete structures include shear failures in beams, failures of joints between beams and columns, loss of anchorage and failures of joints of diagonally braced concrete frames.

There were several examples of large settlements and tilting of foundations and one example of pull-out of friction piles resulting in the overturning of a structure.

The 21 storey Pino Suarez steel structure collapsed, probably due to local buckling of the second storey columns.

7.3 Emergency code changes

The emergency changes to the 1977 building code addressed many structural design aspects. Design lateral load levels were increased by 35% in Zone II and by 67% in Zone III. Large adjustments were made to the ductility factors, Q. The case of Q = 6 was eliminated and the categories for other levels of ductility were redefined. Office live loads were increased to account for the large number of cases (particularly government buildings) that suffered damage or collapse due to large storage loads.

Some changes were made in the reinforcement details of reinforced concrete columns. The spacing between column ties were reduced, particularly at the ends in order to prevent longitudinal bars from buckling and the amount of transverse confinement reinforcement was increased. Unfortunately the emergency code changes did not address the practice of using 90 degree bend anchorages for column tie reinforcement.

Due to the large number of punching shear failures of waffle slabs a number of design and detailing changes were made. These include reducing the stiffness in design (to more properly account for the flexibility), increased amount of longitudinal slab steel passing directly through the column and the provision of a minimum amount of stirrups in the slabs around columns.

The torsional eccentricity is limited to 20% of the plan dimension of the structure and all walls contributing to the lateral load resistance must be included in the analysis. The minimum separation between buildings must be respected. Both torsional eccentricity and pounding between adjacent structures contributed to major damage and many collapses.

7.4 Structural design practice in Canada

Design codes in Mexico City have adopted microzonation to account for the dominant influence of the compressible soil on the ground motion. Studies are needed to investigate how to cope with the design of structures in Canadian seismic risk regions that have the potential for significant soil amplification. Examples of potential soil amplification include the Fraser River delta with thick sedimentary deposits and the St. Lawrence Valley with sensitive clay deposits.

Research is needed to study the likely performance of older Canadian structures that have been designed with outdated seismic design codes. Canada will have to face the possibility of upgrading existing structures to meet minimum standards.

The lessons learned from the 1985 Mexican earthquake are being studied by Canadian code committees in order to assess whether or not changes need to be made to the design codes.

8 ACKNOWLEDGEMENTS

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