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THE 1985 MEXICAN EARTHQUAKE - A SITE-VISIT REPORT*

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ABSTRACT

Severe damage during the September 19, 1985 Mexican earthquake prompted a site visit by three engineers and two seismologists representing the Canadian National Committee on Earthquake Engineering. This report contains background information on earthquake history, subsoil conditions, past structural damage and building codes, together with details of the 1985 earthquake and its strong ground motion. The team's observations of moderate damage in the epicentral area are consistent with the relatively low near-field accelerations (15% g) and high frequencies recorded. In the damaged parts of Mexico City, soft soil conditions amplified the ground motion and resulted in almost pure harmonic motion with a period of about two seconds. These characteristics, together with the long duration of strong ground motion, caused severe damage to many 6-20 storey buildings (both steel and reinforced concrete), as is illustrated in the report. Lessons learned from the earthquake together with the Mexican emergency code changes are discussed.

RÉSUMÉ

Trois ingénieurs et deux séismologues mandatés par le Comité national Canadien de génie paraséismique sont allés examiner sur place les dommages importants dus au tremblement de terre mexicain du 19 septembre 1985. Ce rapport présente des informations de base sur l'histoire des séismes, sur les conditions des dépôts meubles, sur les dommages structuraux précédents et sur les codes du bâtiment, ainsi que des détails sur le séisme de 1985 et ses vibrations fortes du sol. Les dommages modérés tels qu'observés par l'équipe dans la région de l'épicentre sont en accord avec les accélérations relativement faibles (15% g) et les hautes fréquences enregistrées en champ proche. Dans les quartiers endommagés de Mexico, les sols mous ont amplifié les vibrations du sol en un mouvement harmonique quasi parfait avec une période d'environ 2 secondes. Ces caractéristiques, couplées à la longue durée des vibrations fortes de sol ont causé des dommages importants à plusieurs édifices de 6 à 20 étages (en acier ou en béton armé), tel qu'illustre ce rapport. Les leçons tirées de ce tremblement de terre ainsi que les changements au code d'urgence mexicain du bâtiment sont discutés.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to the many individuals in Canada and in Mexico who assisted the team during the site visit.

We are grateful to Professor S.M. Uzumeri, the Chairman of the Canadian National Committee on Earthquake Engineering, to Mr. D.A. Lutes of the National Research Council of Canada and to Dr. M.J. Berry, the Director of the Division of Seismology and Geomagnetism for arranging the site visit to Mexico City.

We would like to thank the Department of External Affairs for obtaining approval for the trip from the Mexican authorities. Special thanks are due to Monsieur J.-P. Petit, the First Secretary of Cultural Affairs and Information and to Senor G. Alvarez, the Coordinator of Information and Press in the Canadian Embassy in Mexico City for providing all the local arrangements during the site visit.

We are very grateful to Dr. Cinna Lomnitz of the National Seismological Service of Mexico for helping to orient the team at the start of the site visit and for explaining details of the seismicity of the region.

We would like to thank Dr. Jorge Prince, President of the Seismic Engineering Society of Mexico for providing and explaining the seismic records of the earthquakes.

We are especially grateful to Dr. Luis Esteva, Director of the Institute of Engineering at the National Autonomous University of Mexico for providing information on the extent of the structural damage and for explaining the emergency codes changes that were being prepared.

The team would also like to thank Dr. Raul J. Marsal of the

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In all our site visits we were greatly assisted by local residents and by many individuals at universities, government agencies and industrial installations.

The site-visit team is grateful to Eduardo Turcott for providing translation of the many Mexican documents.

CHAPTER 1

INTRODUCTION

On September 19, 1985 an earthquake of magnitude 8.1 followed the next day by a second major shock of magnitude 7.5 occurred on the Pacific coast of Mexico between Jalisco and Oaxaca. This earthquake referred to by the Mexicans as "El Grande" ("The Big One") caused some damage in the coastal resort towns and resulted in unprecedented loss of life and destruction in Mexico City.

The nature of the ground motion together with amplification of this motion in Mexico City due to the soft underlying soil resulted in unusually severe ground motion characteristics. It is estimated that about 10,000 people died as a result of the earthquake, with about 200,000 people displaced from their homes.

The Canadian National Committee on Earthquake Engineering under the sponsorship of the National Research Council of Canada has participated in site visits by sending representatives of the Committee to earthquake-damaged areas in several countries. However, this is the first visit by an all-Canadian team. It was decided by the Chairman of the Canadian National Committee on Earthquake Engineering (CANCEE) that the site visit to Mexico City be delayed until the emergency-rescue work was completed. The Director of the Division of Seismology and Geomagnetism arranged for permission with the Mexican authorities through the Canadian Department of External Affairs and through the Canadian Embassy in Mexico City. Funding of the trip was provided by the National Research Council of Canada and Energy, Mines and Resources Canada.

The Canadian site-visit team consisted of five members; two seismologists, one geotechnical engineer and two structural engineers. The team arrived in Mexico City on Oct. 7, 1985, 17 days after the earthquake.

The First Secretary of the Canadian Embassy made initial arrangements for the team to meet with Mexican experts. In addition the Canadian Embassy provided each team member with a document addressed to the civil and military authorities in order that the team be granted access to the damaged sites. This document proved essential in gaining access to the many damaged areas which were heavily guarded by armed military personnel. A copy of one of these documents is shown in Fig. 1.1.

The objectives of the site visit were to gather information by direct observation and through contact with Mexican experts in order to interpret the earthquake damage and report on lessons for earthquake engineering in Canada.

The members of the site-visit team and their areas of expertise are:

Dr. Denis Mitchell - structural engineering

Dr. John Adams - seismology and geology

Dr. Ronald H. DeVall - structural engineering

Dr. Robert C. Lo - geotechnical engineering

Dr. Dieter Weichert - seismology (strong ground motion)

The team spent three days together in Mexico City surveying the damage, and comparing notes. DeVall and Mitchell remained in Mexico City for three additional days to gather more detailed information on the structural damage. Adams, Lo and Weichert flew to the Pacific coast to examine damage in the epicentral area and were later joined for one day by DeVall to examine the structural damage. All team members departed by Oct. 17, 1985.

Canadian Embassy — Ambassade du Canada

MEXICO, D. F.
OCTUBRE 8 DE 1985

A LAS AUTORIDADES CIVILES Y MILITARES:

LA EMBAJADA DE CANADA SOLICITA ATENTAMENTE A LAS AUTORIDADES CIVILES Y MILITARES, PRESTEN TODA LA COLABORACION NECESARIA Y PERMITAN EL ACCESO A LOS SITIOS QUE ASI LO REQUIERA EL SEÑOR:

D. MITCHELL

EXPERTO CANADIENSE EN SISMOLOGIA, PARA QUE PUEDA REALIZAR SU LABOR ADECUADAMENTE.

LA EMBAJADA DE CANADA AGRACEDE SU COLABORACION.

JEAN-PIERRE PETIT
PRIMER SECRETARIO

Canadian Embassy - Ambassade du Canada
MEXICO

Fig. 1.1 Document Issued by the Canadian Embassy in Order to Gain Access to Damaged Areas

CHAPTER 2

EARTHQUAKE SOURCE CHARACTERISTICS

2.1 Tectonic Background

Subduction of the oceanic crust underlying the Pacific Ocean beneath the North American and Caribbean plates causes large earthquakes along the Pacific coast of Mexico. The seismic zone extends from about 8°N to 20°N, and 83°W to 105°W (Fig. 2.1) and is marked along most of its length by a deep offshore trench, where the subducting plate bends down. From its eastern end to the Guatemala triple junction near the Gulf of Tehuantepec it is paralleled by a chain of volcanoes (Ref. 2.1, 2.2). Further west, the volcanoes lie much further inland, along a west-east line from approximately Guadalarjara past Mexico City to Vera Cruz on the Atlantic coast, (stars on Fig. 2.2). In other subduction zones it is found that the volcanoes mark the 100-km depth contour of the subducting plate and therefore the volcanoes show that the subduction west of the triple junction is more complicated than to the east. In fact, in this area two different plates are subducting, the Rivera plate is now subducting independently but earlier was probably part of the Cocos plate (Ref. 2.3).

The oceanic crust now being subducted was formed a few million years ago by spreading at the East Pacific Rise. In the case of the Rivera plate, subduction occurs at about 20 mm/year and the crust now being subducted is 9 million years old. The Cocos plate subducts towards the northeast at about 50 mm/yr near its western end (near its border with the Rivera plate), increasing to about 80 mm/yr near the Guatemala triple junction. The age of the crust being subducted varies similarly from about 3 to 10 million years (Ref. 2.4).

Plate subduction rates are time averages, derived from global plate tectonic considerations. Comparison with the rate derived from

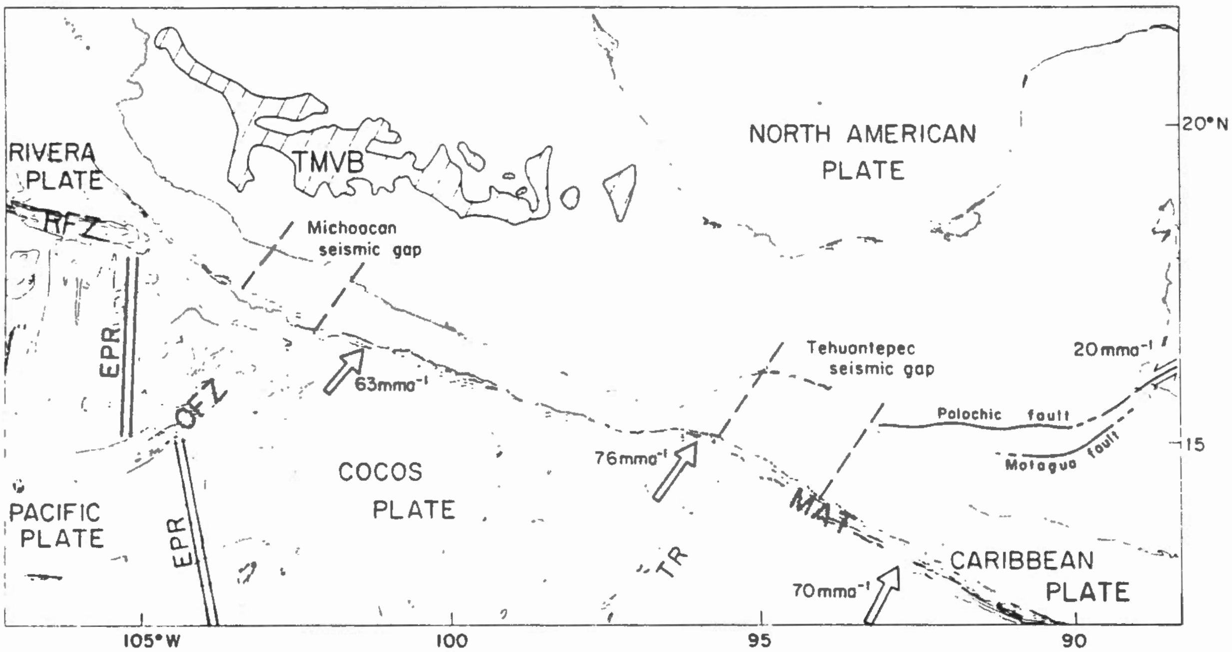

Fig. 1. Plate geometry and bathymetry of the study area (Lambert projection). Bathymetry is from *Chase et al.* [1970]. Contours of depth greater than 4000 m are indicated for the trench. Depths less than 3300 m are indicated for the ridges and fracture zones. The contour interval is 200 m. Features of the seafloor shown are Rivera Fracture Zone (RFZ), East Pacific Rise (EPR), Orozco Fracture Zone (OFZ), Tehuantepec Ridge (TR), and Middle America Trench (MAT). The Trans-Mexican Volcanic Belt (TMVB) is also shown. Plate rates are calculated from *Minster and Jordan* [1978]. The North American/Caribbean plate boundary on land is along the Polochic-Motagua fault system; offshore its location is uncertain but probably extends westward to roughly the area of intersection of the Tehuantepec Ridge with the trench. Both the Orozco Fracture Zone and the Tehuantepec Ridge are subducting in areas of seismic gaps.

Fig. 2.1 (Taken from Ref. 2.1)

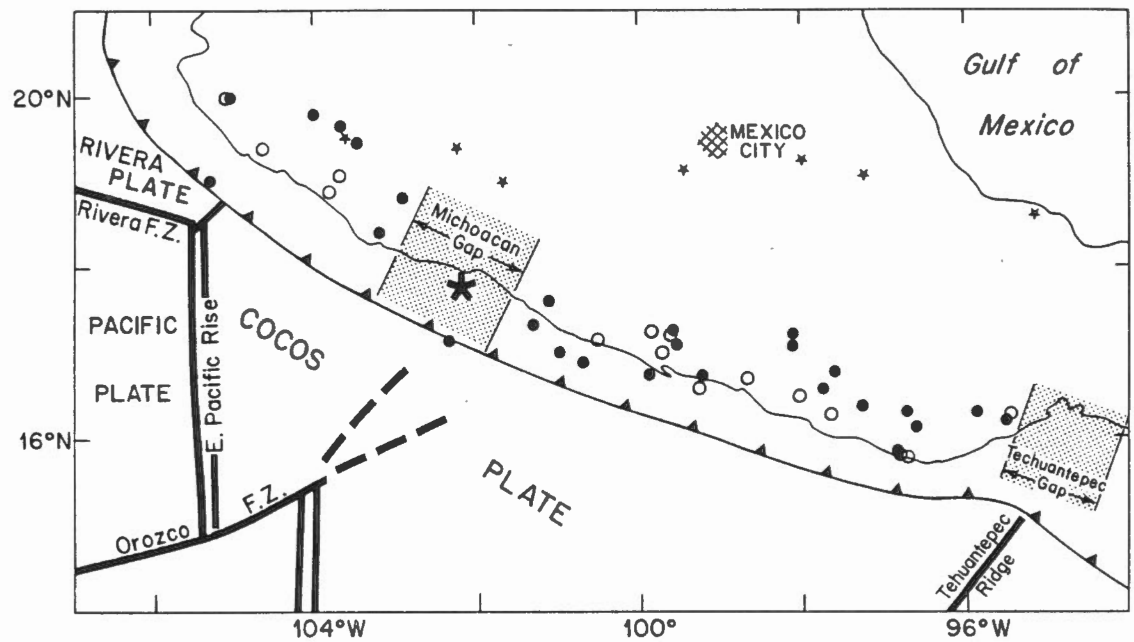

Fig. 2.2 Map of Mexico showing plate boundaries off the Mexican coast, the Middle America subduction zone (barbed line), volcanoes (stars), subduction earthquakes 1800-1899 (open circles) and 1900-1979 (filled circles), the Michoacan and Tehuantepec gaps, and the 1985 earthquake (asterisk). (Modified from Ref. 2.2, Figs. 1 and 2.)

historical seismicity shows general agreement along sections of the Mexican coast. A catalog of large Mexican earthquakes is given in Table 2.1, and those considered to be subduction zone events are shown on Fig. 2.2.

The observed pattern is one of rupture along 100 km segments every 30 to 60 years, generating earthquakes of magnitude 7.4 to 8.0. One of the two exceptions is the section of the subduction zone near its intersection with the Orozco fracture zone, a zone of weakness which is marked by a bathymetric high intercepting the coast near Zihuatanejo. In fact, near the trench, the Orozco fracture zone widens to a broad region of shallower seafloor that encompasses almost the whole length of the 1985 rupture, from about Puerto Maruata in Michoacan state to Petatlan in Guerrero state.

McNally and Minster (Ref. 2.4) report coastal terraces, up to 100 m high and extending for 80 km, onshore of the Orozco fracture zone (101.6°W to 102.6°W), but not to the southeast. One could conclude that in the Michoacan-Guerrero region subduction of the young and still warm Cocos plate is opposed by the thermal buoyancy of the bathymetrically-high Orozco fracture zone. Our own observations suggest only moderate uplift. Offshore from Ixtapa at 101.6°W, two flat-topped islands about 30 m high could represent remnants of a marine terrace. Inland, river gravels were seen to about 30 m above present river level (about 70 m above sea level) and could have been deposited by rivers graded to this terrace. The lack of well-defined, higher terraces suggests the 30 m terrace correlates with the many worldwide terraces cut during the last interglacial age (about 100,000 years ago), and so indicates a long-term uplift rate of 0.3 mm/year. This rate is a factor of ten slower than found above many subduction zones, but is similar to the long-term rate determined from the Pacific coast of Washington and Oregon, above the Juan de Fuca subduction

A CATALOG OF LARGE 19TH CENTURY EARTHQUAKES OF MEXICO (ADOPTED FROM SINGH ET AL., UNPUBLISHED DATA)

Event No.	Date	Region	Epicenter		M_s
			Lat. (°N)	Long. (°W)	
1	25 Mar. 1806	Coast of Colima-Michoacán	18.9	103.8	7.5
2	31 May 1818	Coast of Colima-Michoacán	19.1	103.6	7.7
3	4 May 1820	Coast of Guerrero	17.2	99.6	7.6
4	22 Nov. 1837	Jalisco	20.0	105.0	7.7
5	9 Mar. 1845	Oaxaca	16.6	97.0	7.5
6	7 Apr. 1845	Coast of Guerrero	16.6	99.2	7.9
7	5 May 1854	Coast of Oaxaca	16.3	97.6	7.7
8	19 Jun. 1858	North Michoacán	19.6	101.6	7.5
9	3 Oct. 1864	Puebla-Veracruz	18.7	97.4	7.3
10	11 May 1870	Coast of Oaxaca	15.8	96.7	7.9
11	27 Mar. 1872	Coast of Oaxaca	15.7	96.6	7.4
12	16 Mar. 1874	Guerrero	17.7	99.1	7.3
13	11 Feb. 1875	Jalisco	21.0	103.8	7.5
14	9 Mar. 1875	Coast of Jalisco-Colima	19.4	104.6	7.4
15	17 May 1879	Puebla	18.6	98.0	7.0
16	19 Jul. 1882	Guerrero-Oaxaca	17.7	98.2	7.5
17	3 May 1887	Bavispe, Sonora	31.0	109.2	7.3
18	29 May 1887	Guerrero	17.2	99.8	7.2
19	6 Sep. 1889	Coast of Guerrero	17.0	99.7	7.0
20	2 Dec. 1890	Coast of Guerrero	16.7	98.6	7.2
21	2 Nov. 1894	Coast of Oaxaca-Guerrero	16.5	98.0	7.4
22	5 Jun. 1897	Coast of Oaxaca	16.3	95.4	7.4
23	24 Jan. 1899	Coast of Guerrero	17.1	100.5	7.9

CATALOG OF LARGE EARTHQUAKES ($M_S \geqq 7.0$) OF MEXICO (1900 TO 1979). ONLY EVENTS BETWEEN 15° TO 20°N AND 94.5° TO 105.5°W ARE GIVEN

Event No.	Date	Epicenter		M_s	Depth (km) [S = shallow (≦ 60 km)]
		Lat. (°N)	Long. (°W)		
1	20 Jan. 1900	20.0	105.0[1]	7.9[2]	S[1]
2	16 May 1900	20.0	105.0[1]	7.4[2]	S[1]
3	14 Jan. 1903	15.0	98.0[1]	8.1[2]	S[1]
4	15 Apr. 1907	16.7	99.2[3]	8.0[4]	S[1]
5	26 Mar. 1908	16.7	99.2[3]	8.1[1]	80[1]
6	27 Mar. 1908	17.0	101.0[1]	7.5[1]	S[1]
7	30 Jul. 1909	16.8	99.9[3]	7.4[4]	S[1]
8	7 Jun. 1911	19.7	103.7[3,5]	7.7[4]	S[1]
9	16 Dec. 1911	16.9	100.7[3]	7.5[6]	50[6]
10	19 Nov. 1912	19.9	99.8[3]	7.0[1]	80[1]
11	2 Jun. 1916	17.5	95.0[1]	7.1[1]	150±[6]
12	29 Dec. 1917	15.0	97.0[1]	7.7[1]	S[1]
13	22 Mar. 1928	16.23	95.45[7]	7.5[1]	S[1]
14	17 Jun. 1928	16.33	96.70[7]	7.8[4]	S[1]
15	4 Aug. 1928	16.83	97.61[7]	7.4[1]	S[1]
16	9 Oct. 1928	16.34	97.29[7]	7.6[1]	S[1]
17	15 Jan. 1931	16.10	96.64[7]	7.8[4]	S[1]
18	3 Jun. 1932	19.84	103.99[7]	8.2[4]	S[1]
19	18 Jun. 1932	19.5	103.5[1]	7.8[4]	S[1]
20	30 Nov. 1934	19.00	105.31[7]	7.0[1]	S[1]
21	26 Jul. 1937	18.45	96.44[8]	7.3[1]	85[8]
22	23 Dec. 1937	17.10	98.07[7]	7.5[1]	S[1]
23	15 Apr. 1941	18.85	102.94[7]	7.7[1]	S[1]
24	22 Feb. 1943	17.62	101.15[7]	7.5[1]	S[1]
25	6 Jan. 1948	17.0	98.0[1]	7.0[1]	80±[1]
26	6 Jan. 1948	17.0	98.0[1]	7.0[1]	80±[1]
27	14 Dec. 1950	17.22	98.12[7]	7.3[1]	S[1]
28	28 Jul. 1957	17.11	99.10[7]	7.5[9]	S[1]
29	11 May 1962	17.25	99.58[7]	7.0[1]	40[1]
30	19 May 1962	17.12	99.57[7]	7.2[1]	33[1]
31	6 Jul. 1964	18.3	100.4[1]	7.4[1]	100[1]
32	23 Aug. 1965	16.3	95.8[10]	7.6[9]	28[10]
33	2 Aug. 1968	16.6	97.7[10]	7.4[10-12]	40[10]
34	30 Jan. 1973	18.39	103.21[13,14]	7.5[10]	32[13,14]
35	28 Aug. 1973	18.30	96.54[8]	7.1[11,12]	82[8]
36	29 Nov. 1978	15.77	96.80[15]	7.8[10]	20[15]
37	14 Mar. 1979	17.31	101.36[16]	7.6[10]	30[17]

Table 2.1 Large Mexican earthquakes 1800-1979 (taken from Ref. 2.2).

zone (Ref. 2.5).

2.2 Previous Mexican Subduction Earthquakes

A space-time diagram, provides a convenient representation of the earthquake activity. In Fig. 2.3 (taken from Ref. 2.2) the x-axis represents distance along the subduction zone from northwest to southeast, and the y-axis represents time. The figure shows the rupture lengths of recent Mexican subduction earthquakes and the seismic gaps identified. The Jalisco segment was last ruptured in 1932 by a M 8.2 and the Colima segment in 1973 by a M 7.5 (Ref. 2.3 and 2.6), while the segment immediately to the east of the 1985 Michoacan earthquake broke in 1979 (Petatlan, M 7.6, Ref. 2.7). Further to the east, the Oaxaca segment was ruptured by an earthquake (M 7.8) in 1978 which had been predicted on the basis of the seismic gap hypothesis.

The Michoacan segment had not ruptured at least during the past 70 years, and probably not since 1800 or even longer (Ref. 2.2 and 2.4), while most other segments of the Mexican subduction zone had ruptured regularly during historical times. Another similar segment ("seismic gap") has been identified (Ref. 2.2) and together they are labelled the Michoacan and Tehuantepec gaps. The authors of Ref. 2.2 and 2.4 recognized that the lack of gap-filling earthquakes for such a long time implied either an anomalously long return period (perhaps 200 years against the more normal 30 to 70 years), or that subduction was occurring smoothly and continuously without the strain buildup required for earthquakes to occur.

The earthquakes of 19 and 20 September proved conclusively that subduction at the Michoacan gap occurs seismically, though with a longer interval between the earthquakes than is generally the case.

Of considerable concern to the understanding of the Michoacan gap is a M 7.9 earthquake in 1911 which had a similar intensity distribution as

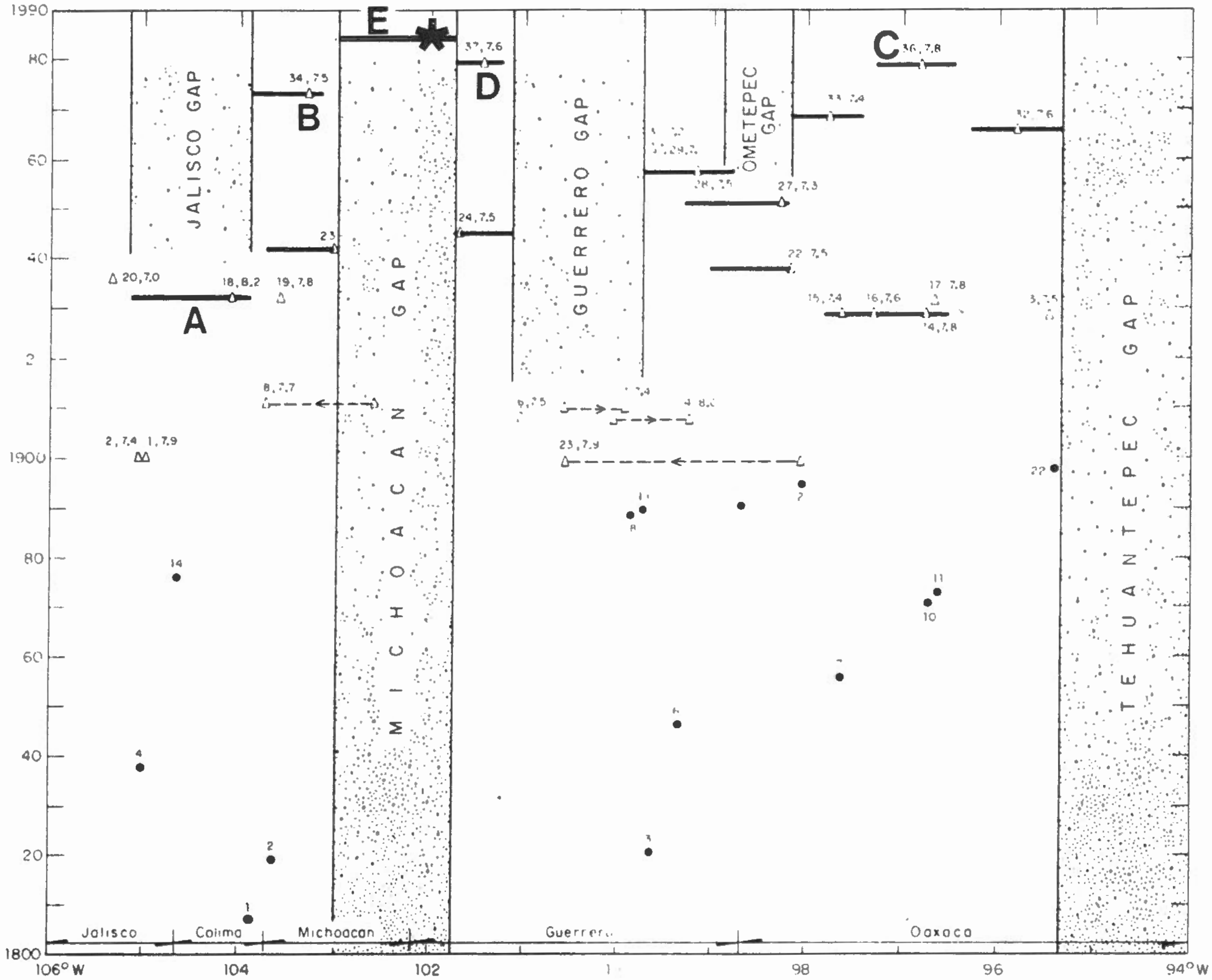

FIG. 3. Space-time plot of large, shallow interplate earthquakes along the Mexican subduction zone. Regions which have not experienced large events in 30 yr or more are shown as gaps. Events with instrumental magnitudes are shown by open triangles. Triangles connected by a dashed line show Gutenberg and Richter's (1954) location and location given in Tables 1 and 2 (arrows point toward more reliable location). Numbers to events refer to their numbers in Tables 1 (1800 to 1899) and 2 (1900 to 1979). Horizontal bars indicate rupture dimensions (from 1928 to 1972 after Kelleher *et al.*, 1973; from 1973 onward from sources given in Table 4).

Fig. 2.3 Space-time plot of large Mexican subduction earthquakes. Letters indicate (A) 1932 Jalisco, (B) 1973 Colima, (C) 1978 Oaxaca, (D) 1979 Petatlan and (E) 1985 Michoacan earthquakes (Modified after Ref. 2.2, Fig. 3).

the 1985 earthquake (Ref. 2.8). This earthquake was relocated outside the Michoacan gap by Singh et al. (Ref. 2.2) but future evaluations may prove it to be a 1985-style event, in which case the return period for the gap is about 75 yr instead of more than 180 yr.

2.3 September 19 and 20th Rupture Areas

The rupture of the M 8.1 earthquake of 19 September started at 17.6°N, 102.5°W (Ref. 2.8) and propagated mainly to the northwest to about 103.5°W, with a short segment breaking to the southeast, for a total length of about 150 km. The width of the rupture area (normal to the coast) was about 80 km (C. Lomnitz, pers. comm.).

The largest aftershock (M 7.5) occurred on the 20 September at the east end of the gap (about 17.3°N, 102°W), and ruptured a length of only 50 km. Other aftershocks seem to have ruptured somewhat beyond the rupture area of the main shocks, completing the filling of the Michoacan gap from Petatlan to Puerto Maruata. Together the two main events ruptured 190 km of the plate boundary down an 80 km width of the subduction interface.

In 1981 a M 7.3 earthquake occurred at Plaza Azul within the Michoacan seismic gap; (Ref. 2.1, Fig. 2.4), but was not large enough to fill the gap. The epicentre for the M 8.1 earthquake was west of, and that of the M 7.5 east of, the Plaza Azul aftershock area (Ref. 2.9). This suggests the region that ruptured in 1981 controlled the 1985 earthquake process.

It is clear that the mainshock and principal aftershock filled the Michoacan gap, which is seen to lie between two recently ruptured smaller gaps, the Colima to the west and the Petatlan to the east (Fig. 2.3 and Fig. 2.4). These gaps, having recently ruptured (1973 and 1979) had not yet stored enough strain energy to rupture again in 1985. However a

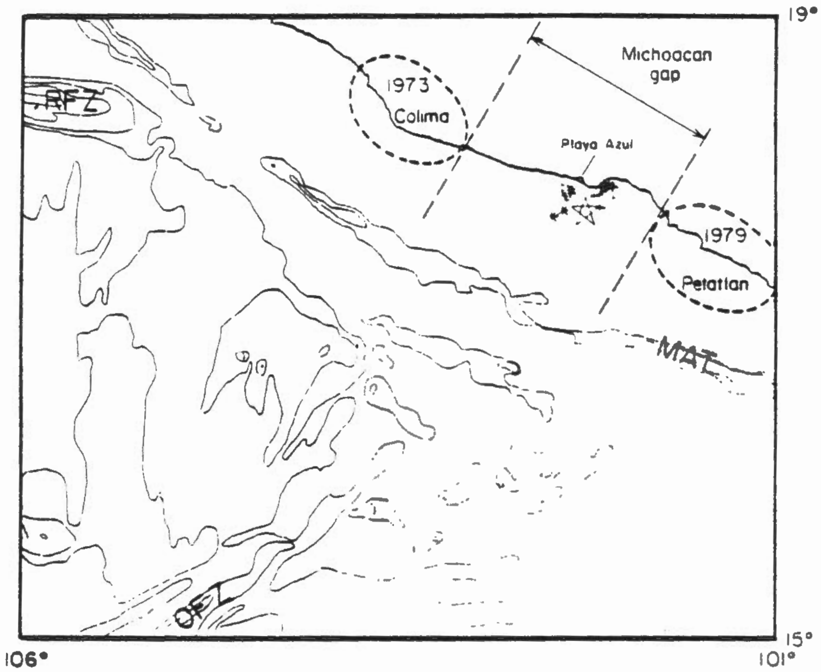

Fig. 8. The Playa Azul earthquake sequence and its tectonic setting. The Orozco Feature Zone (OFZ) spreads out into a broad feature as it nears the trench (MAT) in this area. The aftershock areas of the 1973 Colima and 1979 Petatlan events are indicated. The area between them is the Michoacan gap. The Playa Azul main shock (star) occurred in this area on October 25, 1981. Asterisks indicate two foreshocks, 3 and 4 months before the main shock. Crosses indicate aftershocks which occurred during 6 days after the main shock. The aftershocks form two distinct clusters. Locations are taken from *Havskov et al.* [1983].

Fig. 2.4 Aftershock areas of the Colima, Petatlan and Playa Azul earthquakes in relation to Michoacan gap(taken from Ref. 2.1).

few aftershocks of the 1985 earthquake have spilled over the Petatlan gap (D on Fig. 2.3) into the Guerrero gap (which includes the city of Acapulco). The Guerrero gap last ruptured in 1899 and 1907-1909. As previous series of Mexican subduction earthquakes have tended to migrate southeastwards along the boundary (see Fig. 2.3), the Guerrero gap must presently be considered of high seismic potential.

2.4 Magnitudes

The earthquake of the 19 September was originally estimated to be magnitude 7.5. Later estimates increased the magnitude to 7.8 and finally 8.1. These successive revisions are unavoidable for great earthquakes and are due to saturation of all nearby seismographs. Therefore, it is necessary to analyse worldwide data, involving a time lag, before a definitive magnitude can be calculated. The pressure to inform the public results in the release of preliminary values. The aftershock magnitude was similarly revised upwards to 7.5.

Another reason for the successive magnitude revision is the complexity of magnitude calculations; "Richter" magnitude is just a convenient term for the public media. Earthquake magnitudes can be defined in several frequency bands. For great earthquakes, magnitudes have in the past usually been given as M_s (surface wave magnitude), which are calculated from 20-second period surface waves. However, in very large earthquake such as the 1985 Michoacan events, even longer period waves are generated with sufficient power to be observed and that are truly representative of the overall dimensions and energy release of the earthquake. The magnitudes denoted M in this paper are mostly the surface wave magnitude M_s and may still be revised upwards.

The 1985 mainshock was substantially larger than the immediately previous Mexican subduction earthquakes. Earthquake size is crudely proportional to the product of the rupture area (i.e. fault length x fault width - here 190 x 80 km^2) and the displacement on the plane. Given the similar nature of the Cocos plate along the subduction zone, the fault width (down the subduction zone) is likely to be similar for all the earthquakes (Ref. 2.6 p. 1310), so that one variable is the length (along the subduction zone) of the segment.

The Michoacan segment is about twice as long as those that have ruptured during M 7.5 earthquakes in this century. The other parameter, displacement, is a function of the long-term subduction rate (about 50 mm/yr) and the time since the last gap-filling earthquake. If the time is 200 years (say) the expected displacement is about 10 m, compared to about 3 m for a more typical segment rupture occurring every 60 years. A factor of 2 for length and 3 for displacement makes the Michoacan earthquake about 6 times larger than the M 7.5 earthquakes or about M 8.3. This is the approximate equivalent of the combined events of September 1985.

Hence the size and return period of the Michoacan earthquake can be reconciled with the smaller previous Mexican subduction earthquakes. What is not yet understood is why the Michoacan segment fails differently from the other segments. The explanation of this phenomenon is likely to have considerable impact on Canadian studies of subduction earthquakes on the Juan de Fuca and Explorer plates. We hypothesise that the uplift of the Orozco fracture zone indicates an even more buoyant material than the rest of the Cocos plate, which in turn affects the way the plates are coupled.

2.5 Strong Ground Motions

A few days after the earthquake, the Institute of Engineering of the National University of Mexico (UNAM) distributed the first four volumes of a report describing the digital strong ground motion records of the September

19, 1985 earthquake obtained from Mexico City (Ref. 2.10 - 2.13). These were followed by a series of joint reports (in English) by UNAM and University of California at San Diego describing the strong ground motion records obtained from the Guerrero accelerograph array in the epicentral region (Refs. 2.14 - 2.16).

The Guerrero accelerograph array (Fig. 2.5) was designed with 30 stations, twenty of which had been installed by August, 1985, and which operated during the September 19 and 20 earthquakes. The array is a joint project of the Institute of Geophysics and Planetary Physics, University of California, San Diego and the Instituto de Ingenieria of the Universidad Nacional Autonoma de Mexico (UNAM) in order to record near-source, "free field" strong ground motions on bedrock from large subduction earthquakes.

Excellent records were obtained from the Guerrero Array making this earthquake by far the best ever recorded in a strong-ground-motion sense.

Figures 2.6 - 2.7 show accelerograms from the stations at Caleta de Campos, La Villita, La Union and Zihuatanejo which are located directly above the rupture plane of the September earthquake. Figure 2.8 presents the accelerograms from the more distant Teacalco, and the UNAM, Mexico City, sites. The other records from Mexico City are described in Chapter 4.

The rupture plane is thought to be 20-30 km below the coastal accelerograph sites, and remarkably the Caleta de Campos record includes both P and S phases which place the epicentre only 25 km from the site (presumably straight down). As shown on Table 2.2, the peak accelerations above the rupture zone are in the range 12% to 15% g, while that at Teacalco is about 5%, similar but somewhat higher than that recorded on the bedrock at UNAM. It is thought the relatively moderate values of the peak accelerations observed immediately above the rupture zone in Mexico are not

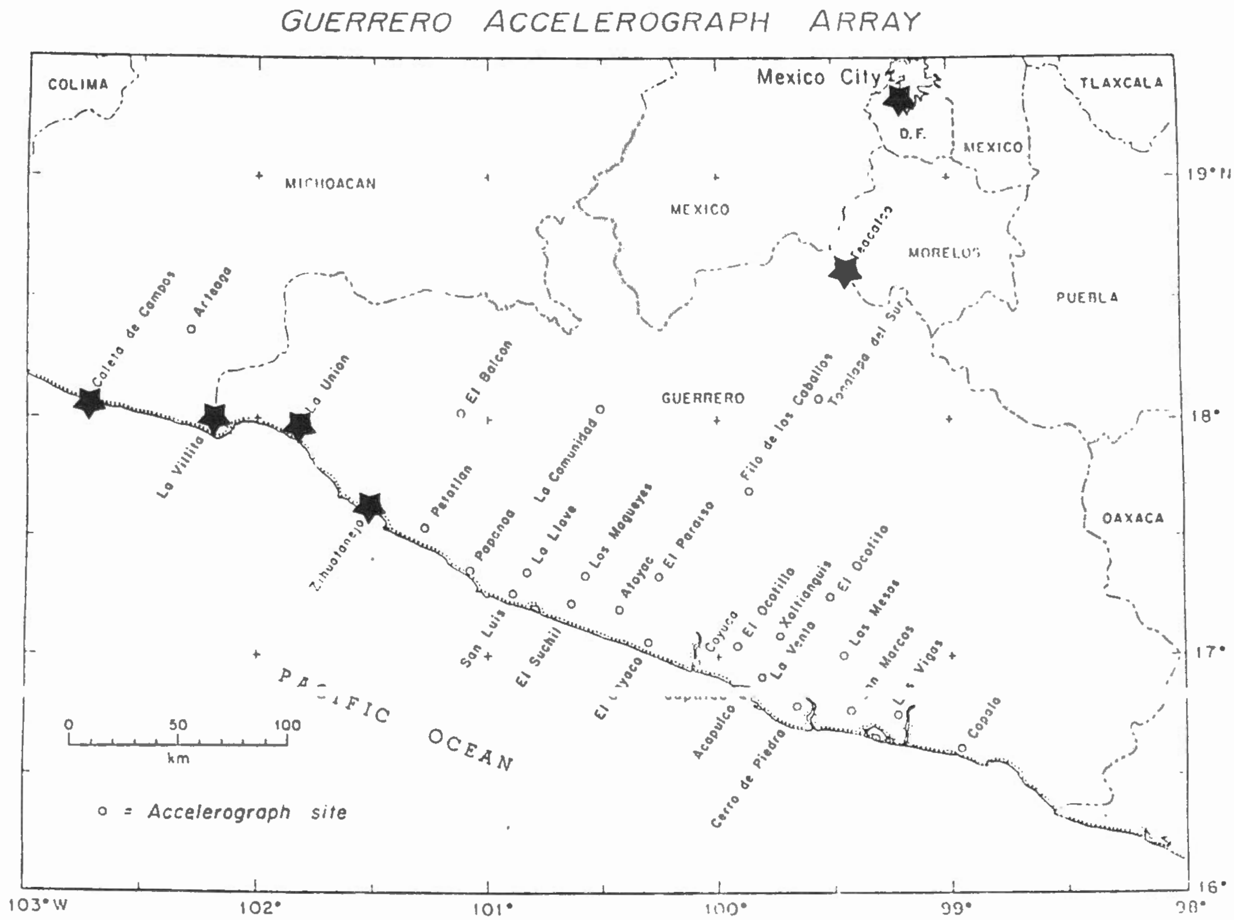

Fig. 2.5 Map of accelerograph sites in Guerrero Accelerograph Array.
Stars mark records figured in this report.

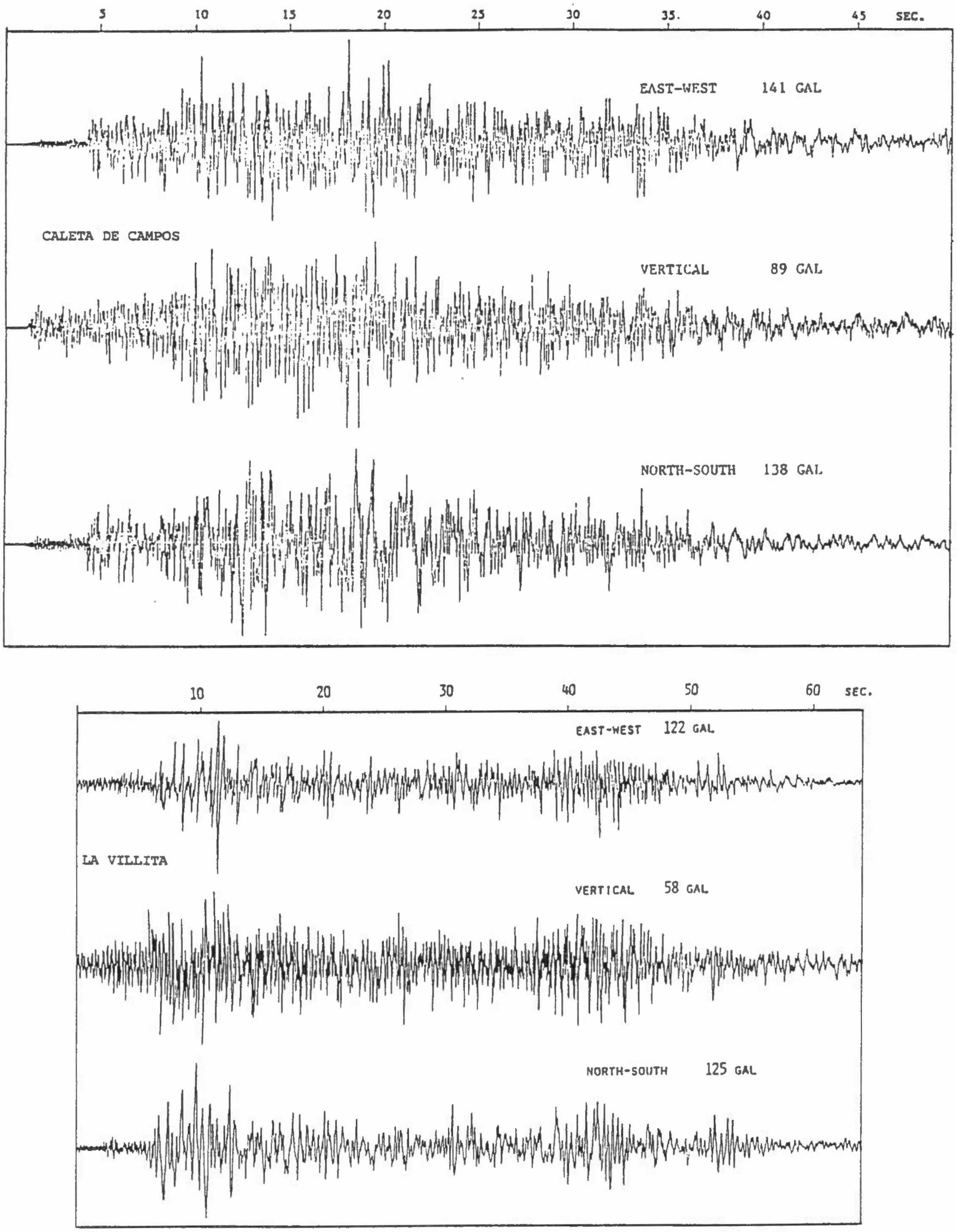

Fig. 2.6 Accelerograms of September 19 earthquake recorded at Caleta de Campos (top) and La Villita (bottom).

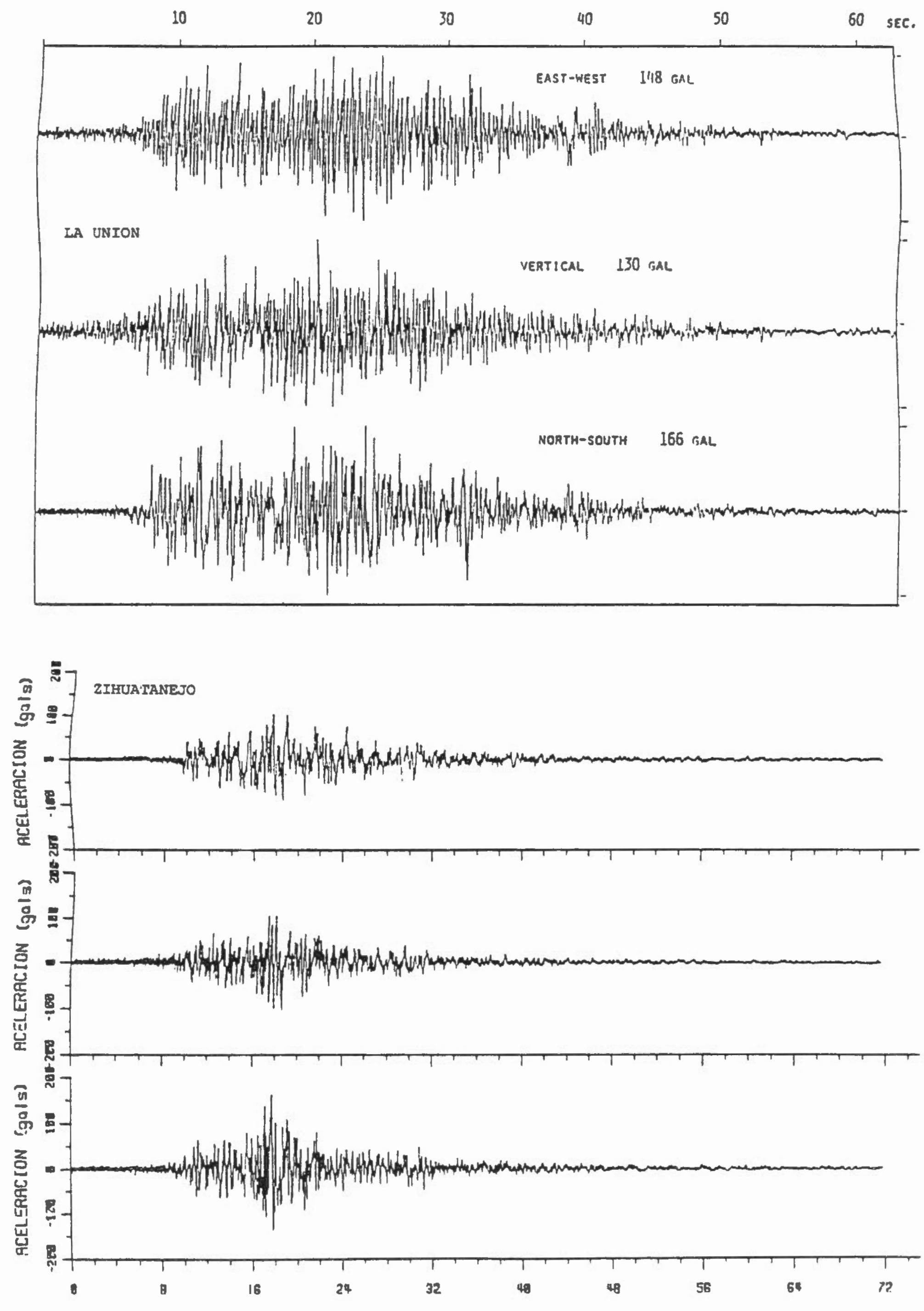

Fig. 2.7 Accelerograms of the September 19 earthquake recorded at La Union (top) and Zihuatanejo (bottom).

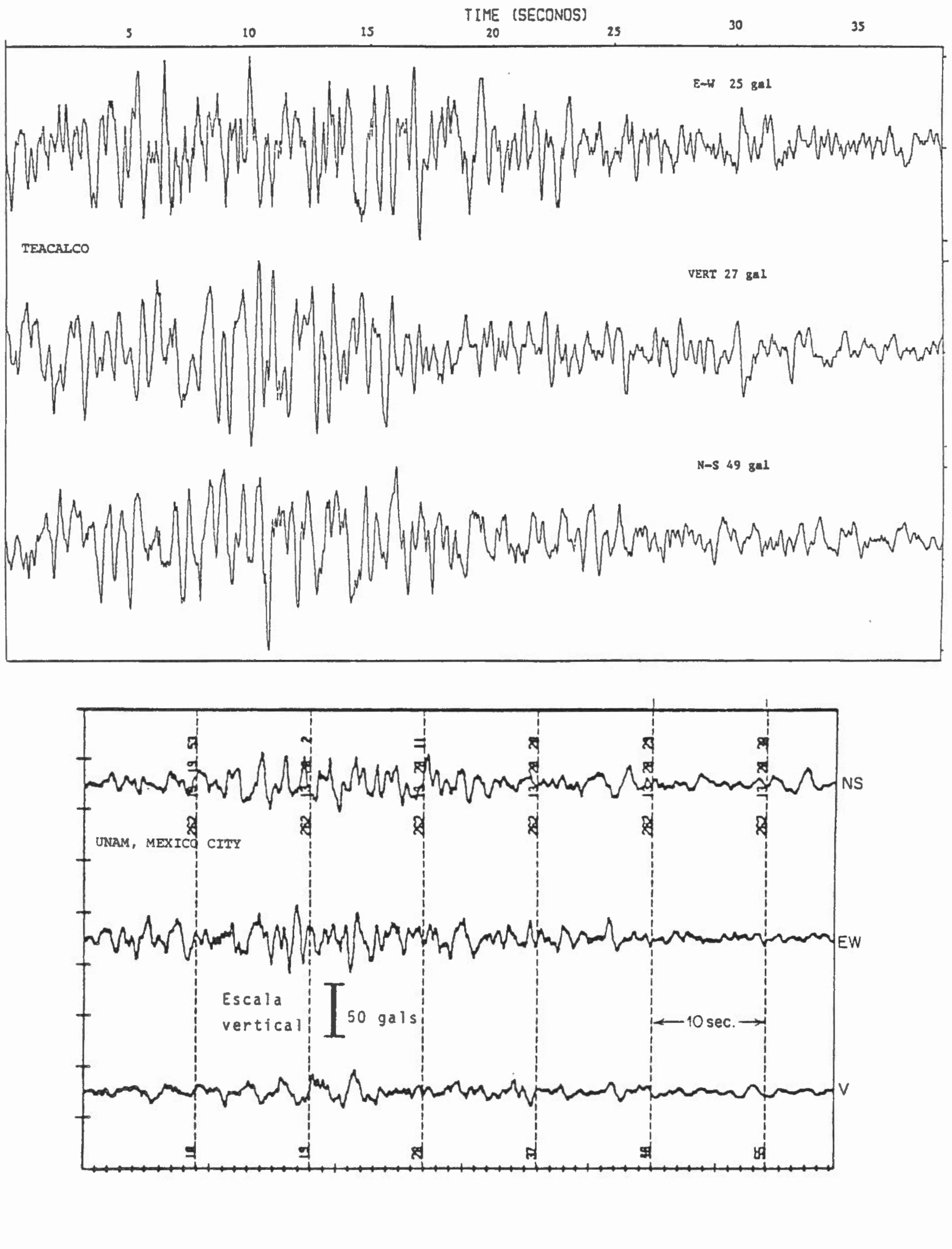

Fig. 2.8 Accelerograms of the September 19 earthquake recorded at Teacalco (top) and UNAM, Mexico City (bottom).

necessarily representative of earthquakes occurring in other subduction zones.

Direction	Caleta de Campos	La Villita	La Union	Zihuatanejo	Teacalco	Mexico City UNAM
NS	138	125	166	103	49	32
EW	141	122	148	161	24	35
V	89	58	129	104	27	22

Table 2.2 Peak ground accelerations (cm/sec^2 or 10^{-3}g) from September 19, 1985 earthquake.

Immediately obvious from the four coastal records is the rich content of high frequency energy; however, these high frequencies which characterize the source are strongly attenuated on the UNAM record and almost absent on the other Mexico City records that are figured later.

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2.16 Quaas, R. et al. "The Michoacan - Guerrero, Mexico Earthquakes of September 1985: Preliminary Description of the Strong Motion Accelerographic Array," Preliminary Report GAA-1B, October 11, Instituto de Ingenieria, UNAM and Institute of Geophysics and Planetary Physics, University of California, San Diego, 1985.

CHAPTER 3

DAMAGE OBSERVATIONS IN THE EPICENTRAL AREA

3.1 Damage Observations

Of necessity the team's observations in the epicentral area along the coast were limited to a few localities and no attempt was made to survey damage systematically. The main coastal road was undamaged, even bridges tens of meters high were undamaged. Although the topography was steep, only a pair of landslides near La Union and some rockslides along the Morales reservoir were seen. Other natural wooded slopes and steep unvegetated roadcuts had not failed.

In the towns of Zihuatanejo, Ciudad Lazaro Cardenas, Playa Azul, La Mira and Guacamayas we saw both damaged and undamaged common structures. We were able to arrange a guided tour of industrial structures in the Fertimex plant in Lazaro Cardenas, the Morelos dam and powerhouse, and the Infiernillo dam; we also inspected damage to bridge piers, a grain silo and loading facility at the Port of Lazaro Cardenas, and nine Ixtapa hotels.

3.1.1 Urban Damage on the Coast

We were surprised by the apparent lack of damage to many common structures, in particular to some incredibly fragile-looking masonry structures (e.g. Fig. 3.1). Our tour took place three weeks after the destructive earthquakes -- more than enough time to clear the rubble from lesser damage around ordinary structures. However, we could still discern that there was a higher proportion of damaged high-frequency structures than had been seen in Mexico City. In Lazaro Cardenas, earlier investigators estimated that 80% of residences had suffered medium to heavy damage and placed the intensity at VIII to IX (Ref. 3.1). In the same city, all 10 modern medium-rise (5 to 8 storey) hotels were heavily damaged

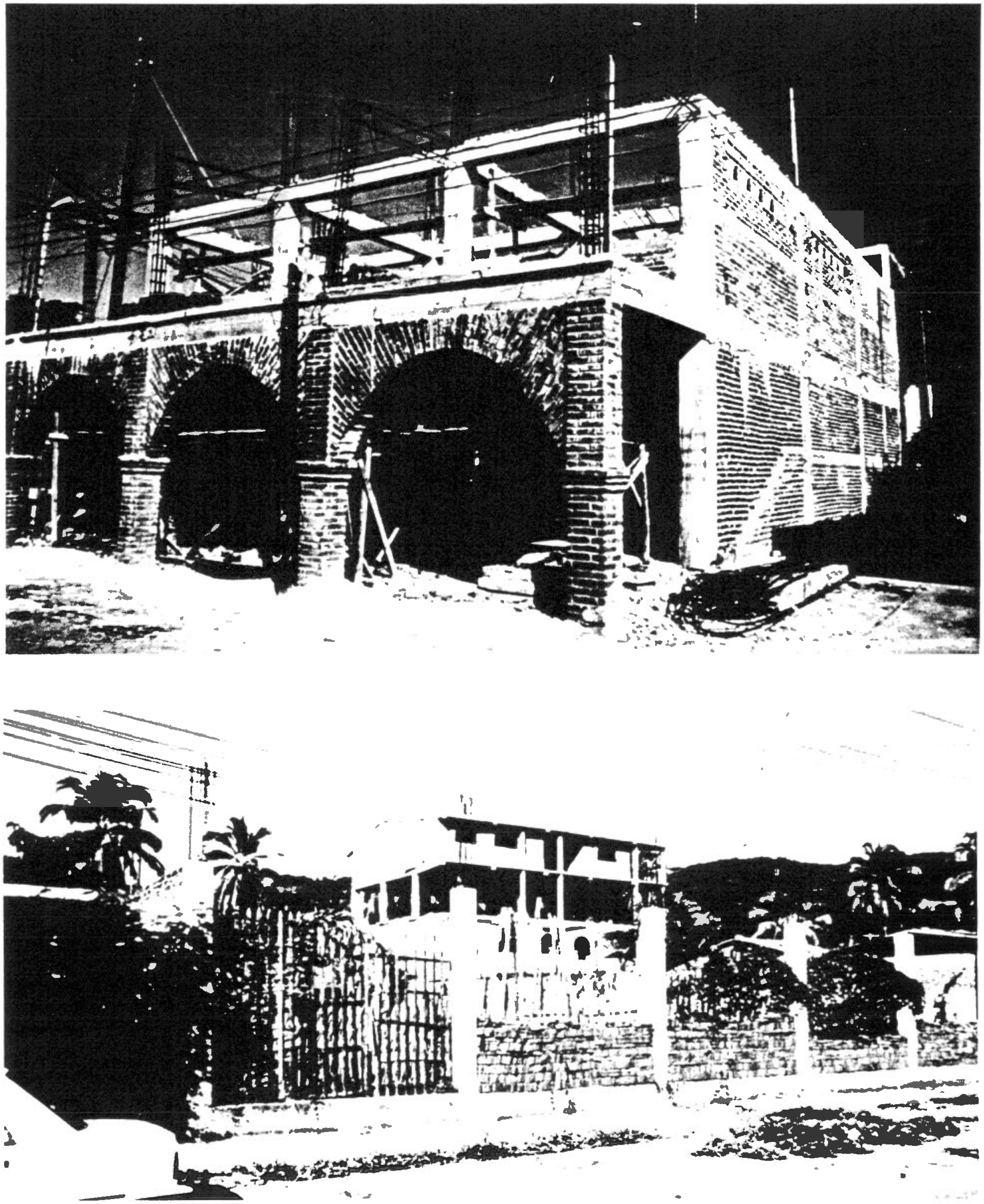

Fig. 3.1a, b Two examples of residential construction in Zihuatanejo:
(a) typical, (b) unusual structure with "soft storey".

and closed (intensity VIII). Only a few kilometres away, on the Balsas Delta, we saw damage that can only be described by a MM intensity of IX or X, ("bent railroad rails, tore apart, or crushed endwise, buried pipelines").

The high intensities observed here appear inconsistant with the strong ground motions of up to 17% g observed on hard rock nearby, and may be due to amplification on soft sediments.

3.1.2 Observations on the Balsas River Delta

In the Fertimex plant, located on the south tip of the Balsas River delta, evidence for strong ground deformation included: torn or ruptured EW oriented rails (Fig. 3.2); buckled rails, shortened in the NS direction (Fig. 3.3); about half a dozen sand boils, up to 3 m across (Fig. 3.4); buckled flange on 60 cm deep H-columns (Fig. 3.5); and finally a 20 cm thick, 70 m wide concrete slab on grade had buckled resulting in two waves 3/4 m high. The plant was nearing completion, but was not yet in production. Most of the structures were on piles and had survived well, although the ground around them had subsided up to 0.3 m. The plant was designed to California zone 3 (SEAOC zone 3). Plant engineers anticipated that the repair of the damage would only delay opening the plant by a few months.

On the road to the Port of Lazaro Cardenas, a pair of modern concrete bridges had suffered heavy spalling to the pier tops (Fig. 3.6). Nearby, a 1.5 m diameter concrete water pipe had buckled underground, indicating compression in the NS direction (Fig. 3.7). On top of a 10-storey reinforced concrete silo building, a 5-storey superstructure had sheared off its base (Fig. 3.8, 3.9). On the adjacent wharf, the east-west part of a reinforced concrete conveyor loader failed at the base of the reinforced concrete supporting piers and toppled to the north, but the

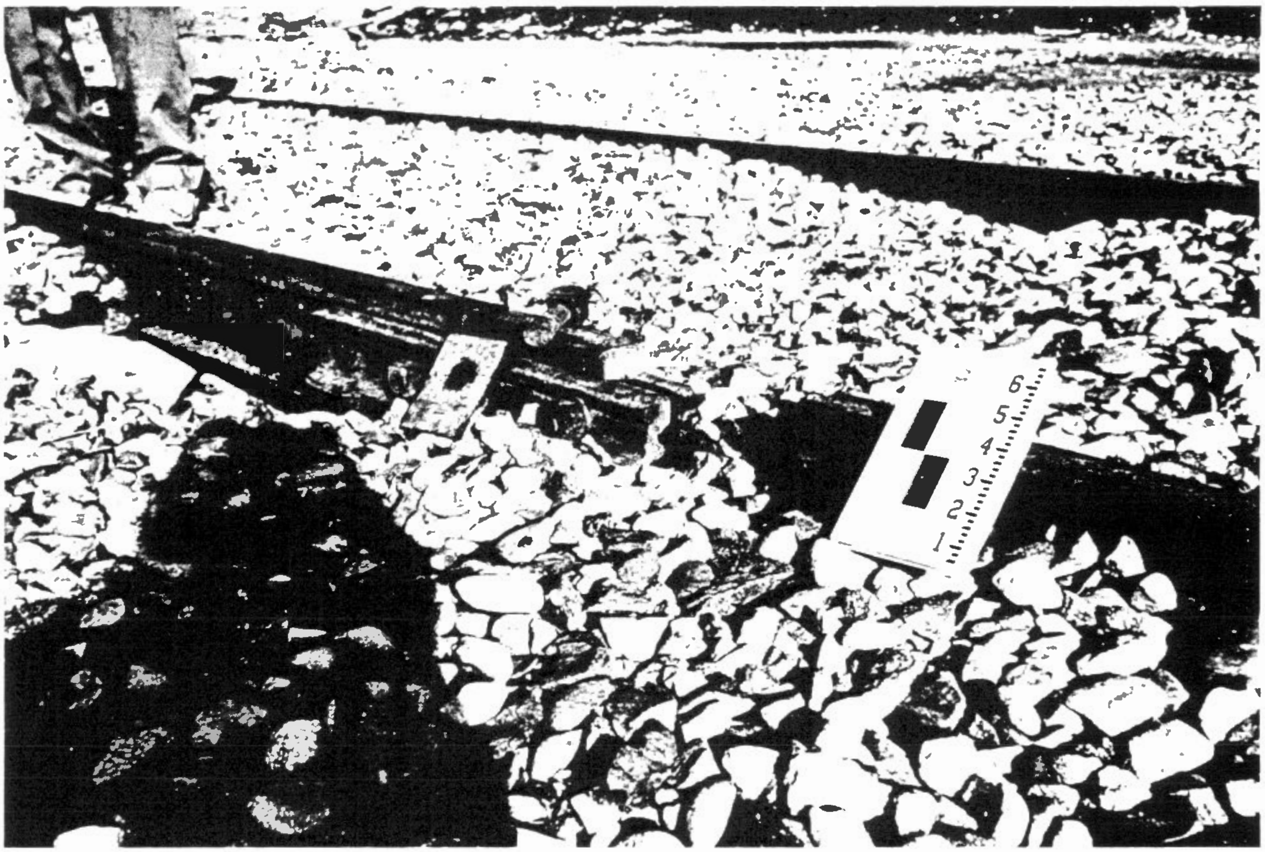

Fig. 3.2 Ruptured E-W rails on periphery of the Fertimex plant, Lazaro Cardenas. Both rails had ruptured at the same place, this one at a connector hole, and the other through the entire rail.

Fig. 3.3 Distorted N-S rails on periphery of Fertimex plant.

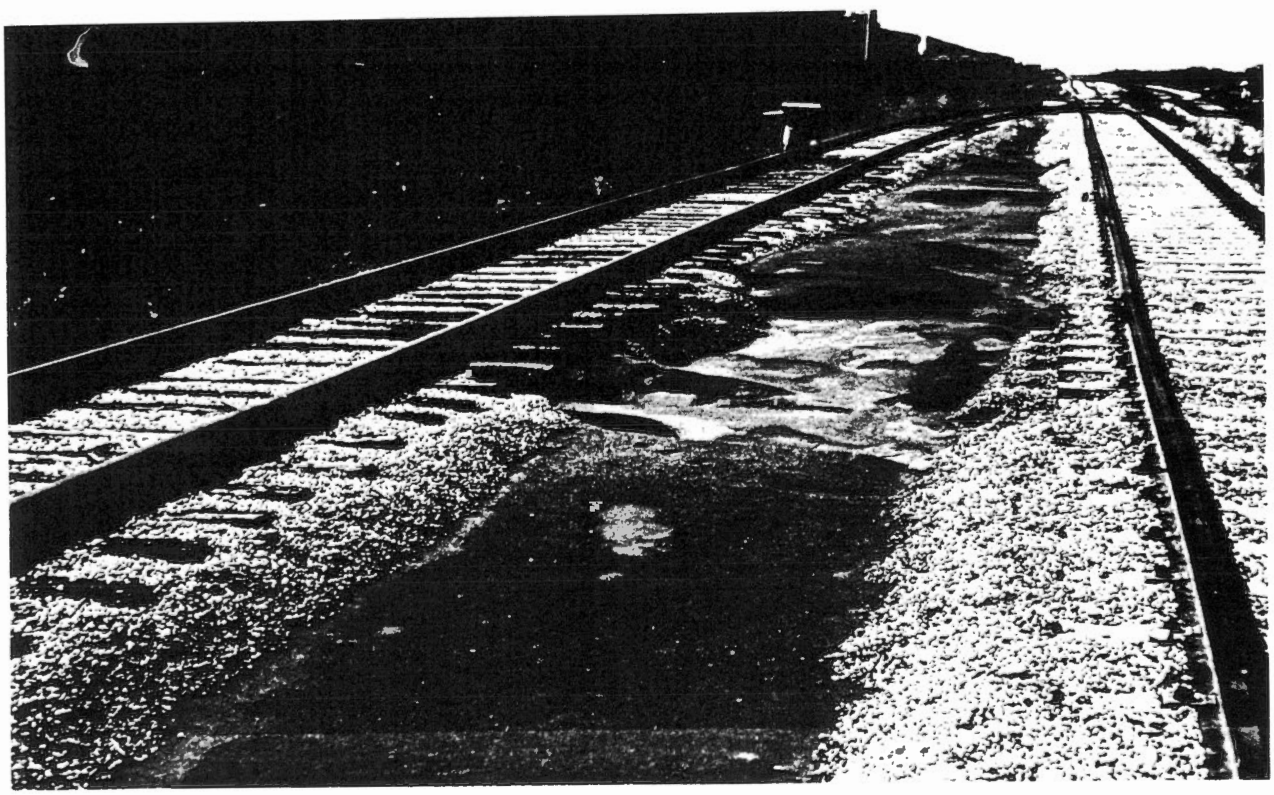

Fig. 3.4 Sand boil under railway line, in middle distance of Fig. 3.3.

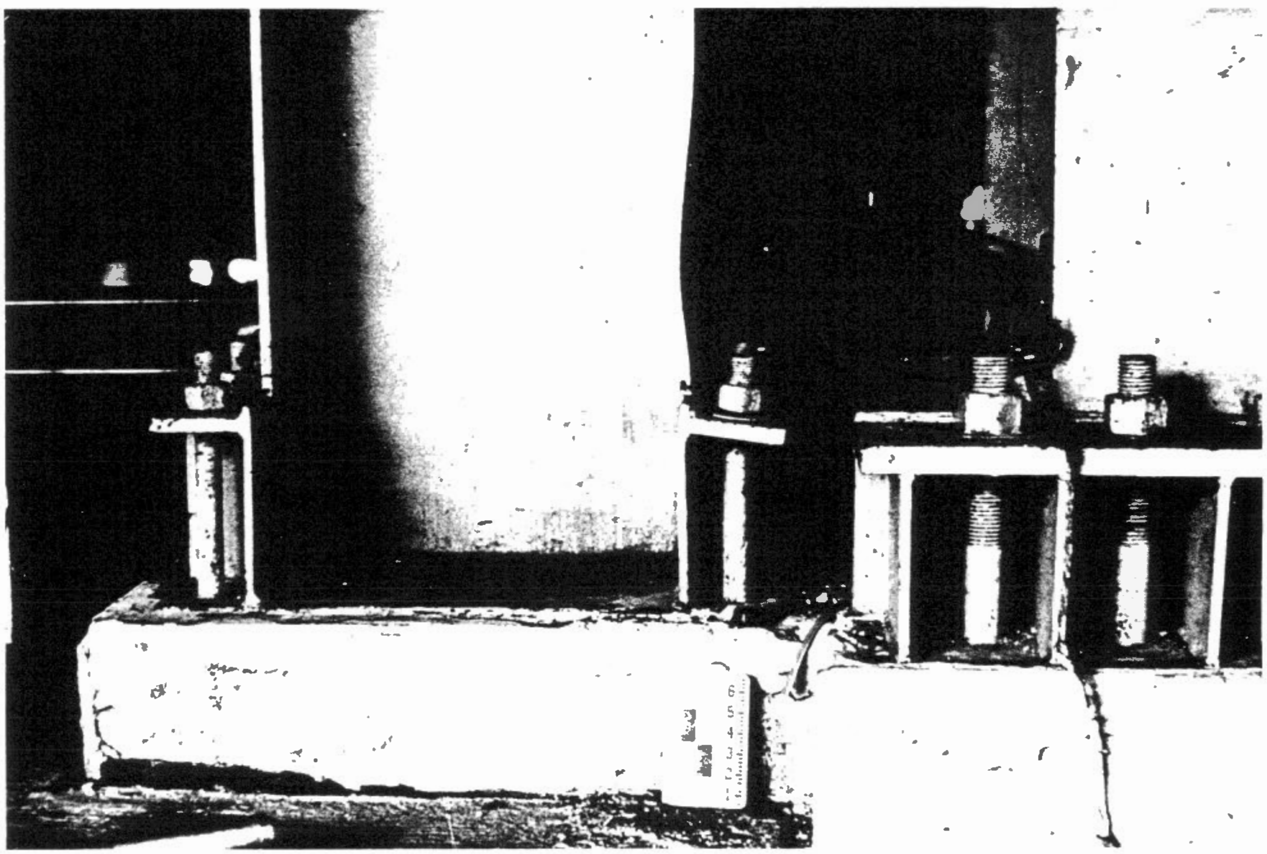

Fig. 3.5 Buckled flange of 60-cm-deep H-column at base of 6-storey steel frame, Fertimex plant. All three columns on this side of the building buckled in the same manner, as did the corresponding three on an identical building a few hundred metres away.

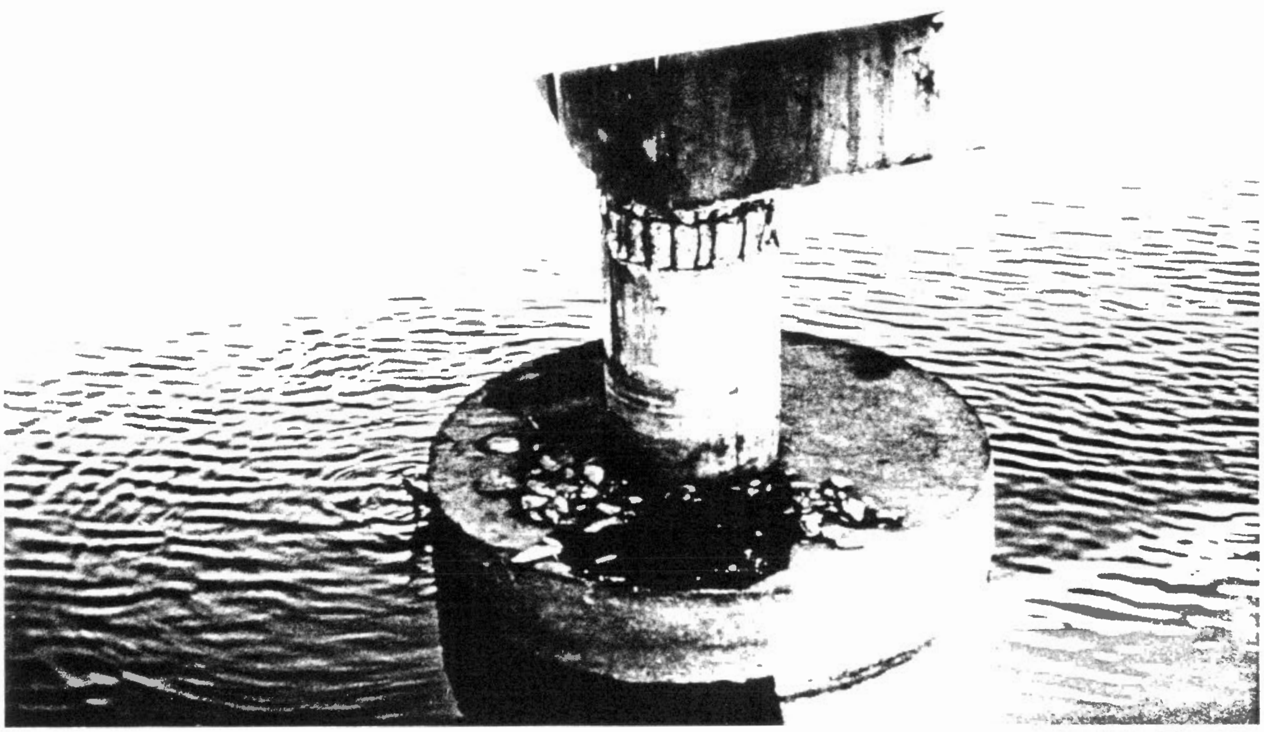

Fig. 3.6 Spalling at top of reinforced concrete bridge pier, port of Lazaro Cardenas access road. All 5 piers on each of the two parallel bridges showed the same damage. Some hammering damage was noticable at the expansion joints in the bridge superstructure. There was also minor settlement of the bridge approaches, which were fill embankments.

Fig. 3.7 Damage to 1.5 m diameter water pipe near bridges in Figure 3.6. The pipe had been dug out to examine the damage.

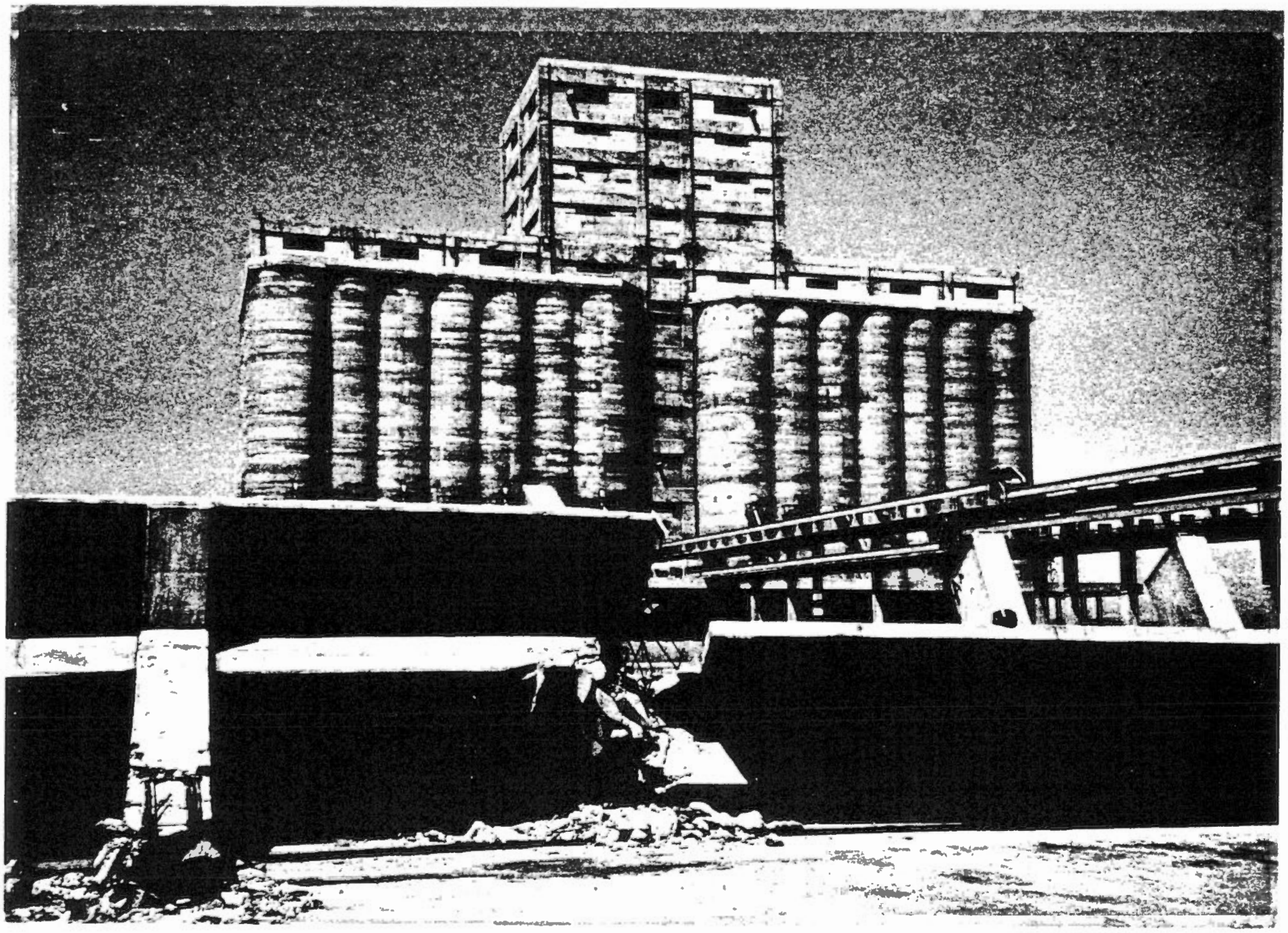

Fig. 3.8 Silo building and conveyor loader structures, port of Lazaro Cardenas. The east-west part of the reinforced concrete conveyor structure (foreground) which ran along the wharf has collapsed, falling to the north, the north-south part was twice as high but was not damaged (centre right). Both of these top-heavy structures were supported by central piers. The east-west structure may have failed because of its orientation or different natural period. Note the 5-storey penthouse, on the 10-storey silo structure, which failed at its base (see detail in Fig. 3.9).

Fig. 3.9 Detail of failure at base of silo penthouse. The penthouse block has dropped and rotated slightly as shown by the column offsets and the protruding reinforcing steel. The structure was only a few years old.

north-south part suffered only slight spalling on the north and south sides of the bases of the reinforced concrete piers.

In summary, ground deformation in this area showed an overall consistency with compression in the north-south direction, and extension east-west. The buckling of the concrete slab on grade suggests a ground strain of 0.001, which appears consistent with the other observations. The consistency leads us to conclude that they represent large scale ground deformation of the delta and not a local response to foundation failure. However, this deformation need not represent deep crustal strain changes, but may have been caused by a transient strain wave at the surface.

3.1.3 Hydroelectric Installations

The Morelos dam, a 60 m high earth and rockfill dam (Fig. 3.10), founded on 70 m thick alluvial deposits had minor subsidence at the crest and a few of the concrete barrier panels tipped over (Fig. 3.11). The 304-MW powerhouse and transformer yard showed no damage and we were assured by the operating engineer that none had occurred. At the Infiernillo dam, a 146 m high earth and rockfill dam founded on bedrock and 920 MW powerhouse 55 km upstream from the Morelos received little damage, though 120 mm of settlement occurred on the dam crest together with 100 mm wide longitudinal cracks. Both dams were well-engineered structures and highly instrumented, and we were provided with the latest inspection report containing post-earthquake survey data (Ref. 3.3).

3.1.4 Damage to Ixtapa Hotels

Ixtapa is a well-known tourist resort with 9 major beach hotels. Most of them are high-rises of 10 to 15 storeys. All of them suffered damage in the main earthquake, and only 4 remained open. Most of the damage was only architectural and was already being "repaired" and plastered over. The oldest hotel, the Aristos, was being repaired for the third time

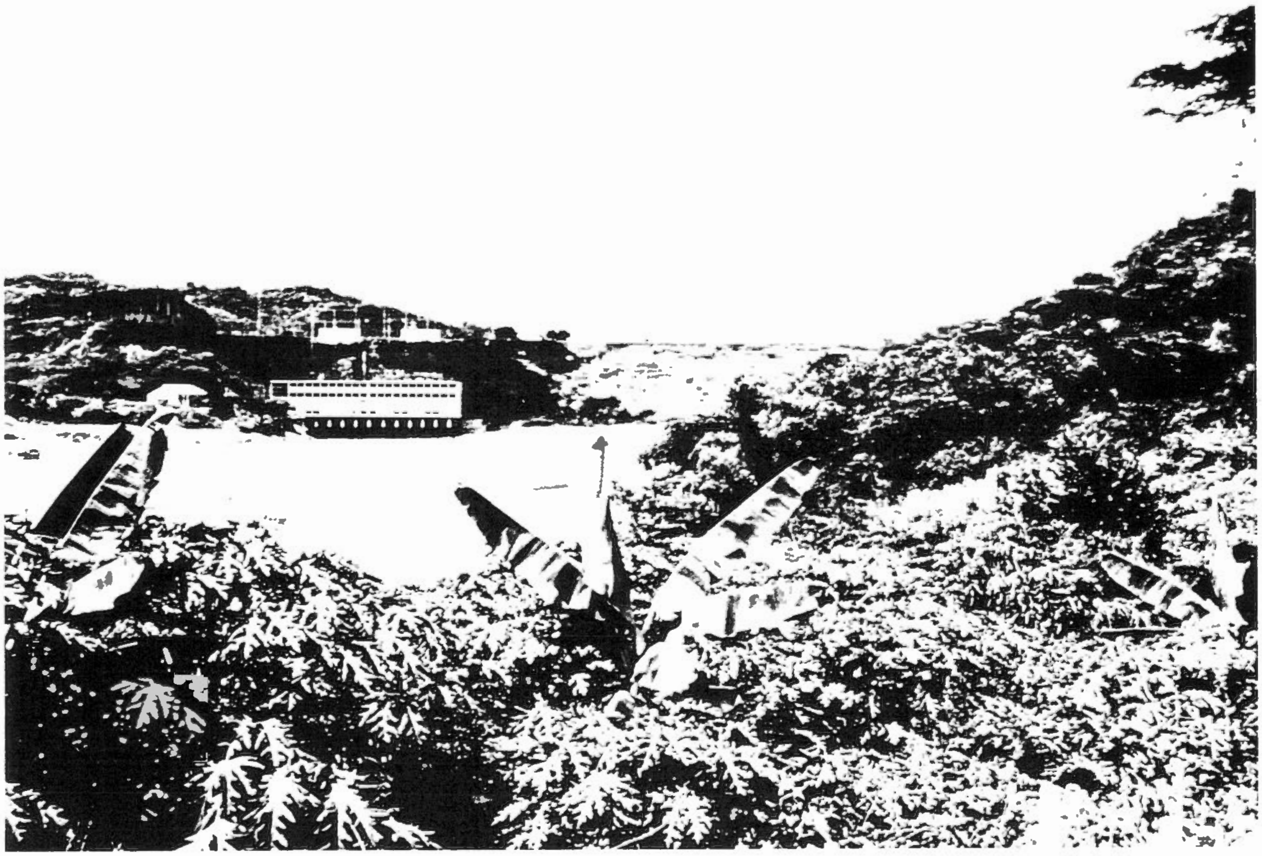

Fig. 3.10 Morelos Dam and La Villita powerhouse, located near the mouth of the Rio Balsas. The earth-filled dam is approximately 60 m high by 300 m long. The large array of high voltage equipment in the station switchyard (adjacent to the dam crest, see Fig. 3.11) was unaffected by the earthquake, even though this type of equipment is often susceptable to seismic damage. The power station is housed in a large steel frame building and consists of four 76 megawatt units, completed in 1973. Mechanical and electrical equipment in the station were undamaged by the earthquake. Protective relays disconnected the station from the power grid during the earthquake (from Ref. 3.2).

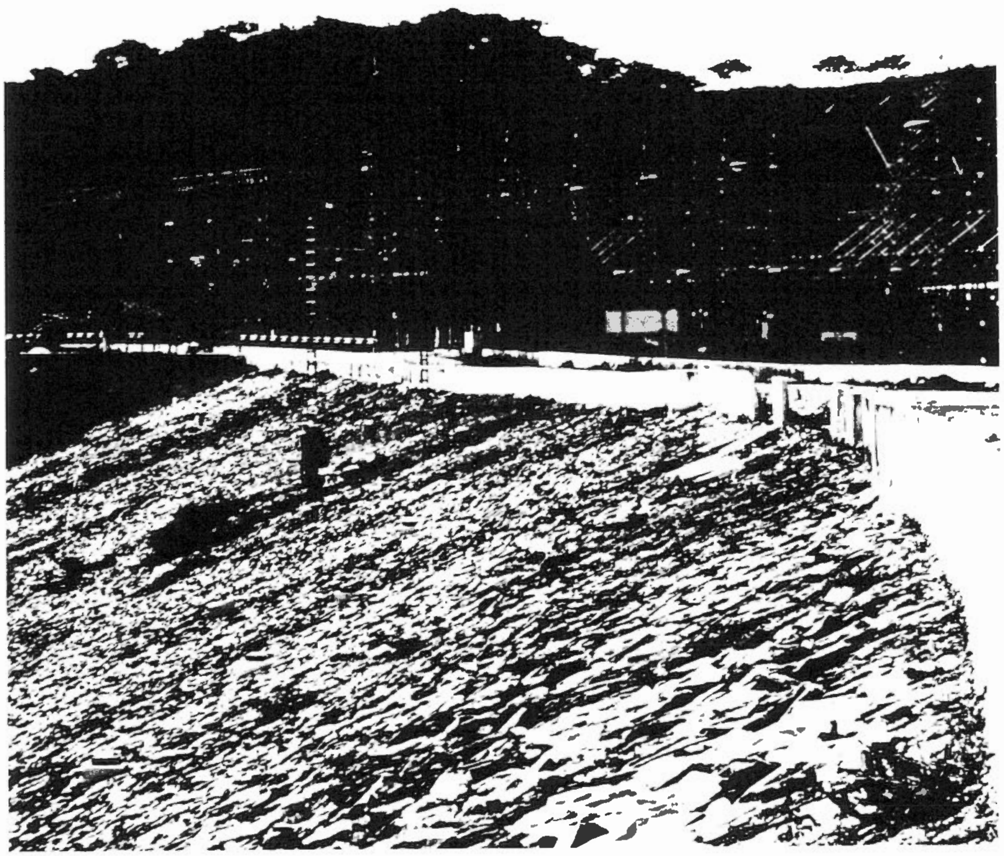

Fig. 3.11 Concrete barrier panels tipped over on crest of Morelos Dam. Dam settlement reached 320 mm on the crest together with outward displacement of 160 mm on the upstream shoulder and 100 mm on the downstream shoulder. Longitudinal cracks up to 150 mm wide formed on both shoulders. Note transformer yard in background.

Fig. 3.12 Shear cracking of columns, Holiday Inn, Ixtapa. This cracking occurred only at this intermediate storey, and only at the 2nd and 3rd columns from each corner.

in this fashion. However, there was more serious damage at the Sheraton, where spandrels crossing the atrium had partially pulled away from the 15-storey main hotel building. In the Holiday Inn, a few columns showed typical 45° shear cracks in the third floor from the top (Fig. 3.12). Otherwise, this solid-looking highly-symmetric building appeared to be in good shape. However, we were told that this hotel had suffered severe damage in the last earthquake (probably 1979) and that extensive repairs had been made by a specialist firm.

Ground settlement could be observed in many places around hotel perimeters. Around the Holiday Inn this amounted to 0.3 m. Similar settlement had evidently occurred previously and had been repaired.

A modern hotel of completely different design is the Camino Real which is built in cascade fashion against rock hill, thus being effectively only one to two storeys above ground level, except for the 6-storey lobby at the top of the slope (Fig. 3.13). This building was said to have suffered no damage during the M 8.1 main shock, but was damaged by the M 7.5 aftershock, in which none of the highrises were said to have suffered further damage. The aftershock was probably richer in high frequencies, perhaps inherent in the source but certainly because of the lower attenuation of high frequencies over the shorter epicentral distance to Ixtapa. Similarly, only the second shock caused slight new damage to a one storey restaurant.

An interesting case of damage was observed in a group of ornamental, free-standing, sandstone columns (Fig. 3.14). Monuments such as these have often been used to deduce degree and direction of ground motion, including perhaps torsional ground motion. The column failed either at the bottom or at midheight, perhaps indicating excitation of the second characteristic mode of some columns. However, we believe that

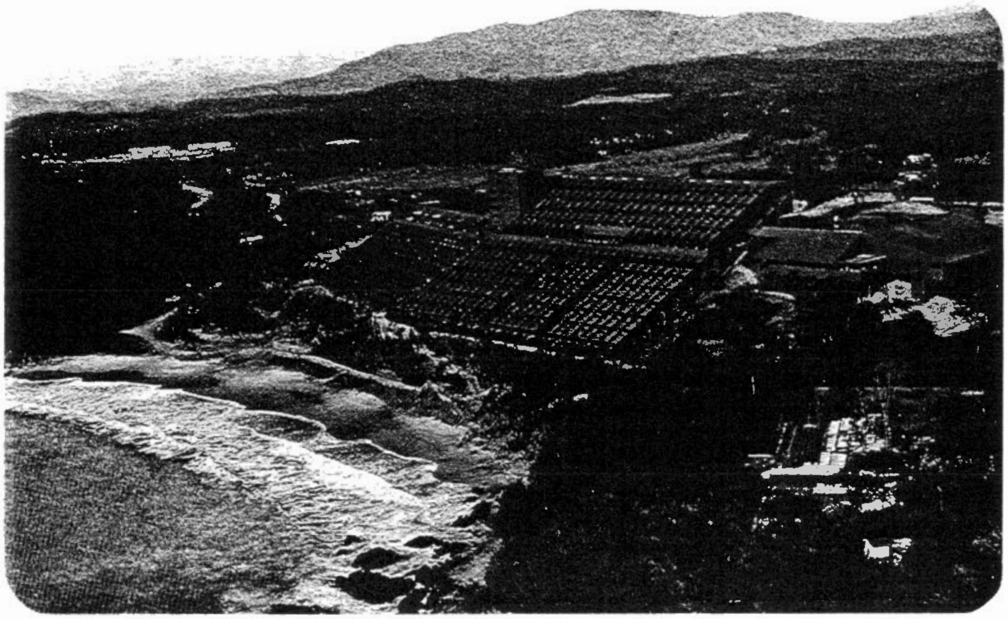

Fig. 3.13 Camino Real hotel, Ixtapa. This modern hotel is built down the hill slope to the beach, thus being effectively only one to two storeys above ground level except for the 6-storey lobby at the top of the hill. It suffered minor damage from the M 7.5 aftershock. The landscape here is typical of the coastal area. Postcard courtesy of Camino Real Hotel.

Fig. 3.14 Rotational and translational displacements of ornamental column, Ixtapa. The role of the steel anchor in tying the segments together is evident.

failure is completely controlled by the relative strength of the embedded anchors and that the column simply rotated about the strongest anchor.

3.2 Intensities in the Epicentral Area

As already indicated in the appropriate places, the team experienced some difficulty in assigning intensities. At the lower intensities (VI and less) intensities are derived from personal experiences. The higher intensities are assigned by observation of damage which led to some uncertainty because during the three weeks that had passed since the earthquakes, much cleanup work and some repair had been done. Thus it was not easy to recognize the proportion and severity of damage to ordinary building where collapse had not occurred. This led to an impression of very sporadic damage, as seen elsewhere. Where total failure was evident and could be compared with the detailed description of the Modifed Mercalli Scale, our estimates agree well with the intensities assigned by the early teams of the Geophysical and Engineering Institutes of the University of Mexico taken from Ref. 3.1 and which are summarized below:

Lazaro Cardenas (VIII-IX) Five dead, severe damage to hotels, public buildings and a hospital.

Playa Azul (IX) Several dead in partial collapse of Hotel Playa Azul. No damage from tsunami.

Zihuatanejo (VII) One dead, partial collapse of a few buildings.

Ixtapa (VII) Non-structural damage to hotels.

Acapulco (VI) No severe damage, no deaths reported.

Manzanillo (VI) No severe damage, no deaths reported.

Ciudad Guzman (VIII) Fifty dead, damage due to poor foundation conditions like those in Mexico City.

3.3 Tsunami

We heard reports of a two-metre high tsunami along the coast and our observations in the Balsas delta and on the Ixtapa beach were consistent with this. Damage was minor.

3.4 References

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CHAPTER 4

SUBSOIL CONDITIONS IN MEXICO CITY AND THEIR EFFECTS ON GROUND MOTIONS AND FOUNDATIONS

4.1 Introduction

Although Mexico City is situated at a great distance, 300 to 400 km inland from the Pacific coast where most of the major earthquakes occur, its unique subsoil conditions were the single most important reason why certain parts of the city suffered severe building damage and heavy casualties in the September earthquakes. In fact, the July 28, 1957 and March 14, 1979 earthquakes had previously revealed the dominant influence of the subsoil on the earthquake damage to the city. This chapter first describes briefly the geological formations of the Valley of Mexico and Mexico City, then outlines their effect on the ground motion characteristics and foundation design.

4.2 Geological Conditions

Marsal and Mazari (Ref. 4.1) and Marsal (Ref. 4.2) have provided excellent descriptions of the subsoil conditions in Mexico City. Much of the information presented in the following section is taken from these references.

Mexico City is located in the southwest portion of the central region of the Valley of Mexico. An east-west geological section through Mexico City is presented in Fig. 4.1, which shows that the bedrock stratigraphy consists of Tertiary volcanic formations overlying the Mesozoic folded marine rocks. Before 1789, the valley was a closed basin containing numerous lakes including Texcoco and Xaltocan lakes. The lacustrine and alluvial soils that filled the bedrock depressions were

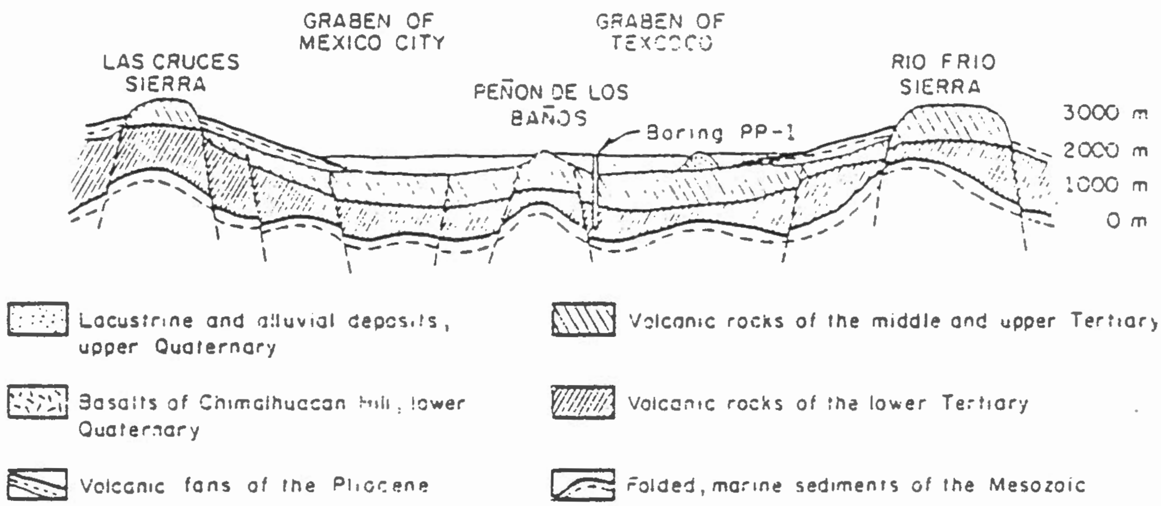

Fig. 4.1 Main geological formations of the Valley of Mexico (taken from Ref. 4.2).

deposited in the Holocene period (less than 10,000 years ago). The valley has become an open basin since the completion of the Nochistongo drainage cut in 1789. In the twentieth century, a grand drainage canal, together with two Tequixquiac tunnels and a modern 45-km long Emisor Central tunnel provide additional drainage for the valley.

Mexico City is built on the location of the old Aztec capital Tenochtitlan which was established on an island in Lake Texcoco. As Mexico City grew it expanded from the old island across the former lakebed and onto the surrounding hills of the Sierra de las Cruces. To cope with the vast differences in the subsoil conditions within the city, it was necessary for engineering purposes to divide the city into the following three subsoil zones: foothills, transition, and lake (Fig. 4.2). While the subsoil in the foothills zone comprises lava or very competent soils, that in the lake zone consists mainly of soft, compressible and sensitive lacustrine deposits interlayered with more competent alluvial deposits at various depths (Fig. 4.3). The subsoil in the transition zone contains shallower and less compressible lacustrine depoits.

4.3 Characteristics of Mexico City Clay

For foundation purposes, most soil borings extend to less than 50 m, as adequate bearing capacity for pile foundations is achieved either by friction or end-bearing piles at a shallower depth even in the lake and transition zones. Typical borehole logs showing the subsoil profiles in the lake zone and the transition zone are presented in Fig. 4.4. At borehole Pc 28, beneath a landfill of about 5 m thick, three clay layers are distinguished: the upper layer (FAS) (5 m - 32 m), the lower layer (FAI) (36 m - 50 m), and the deep layer (DP) (62 m to about 80 m). Two competent layers are sandwiched between the clay layers: a sandy clay layer (CD) about 4 m thick below the upper clay and a predominantly sand

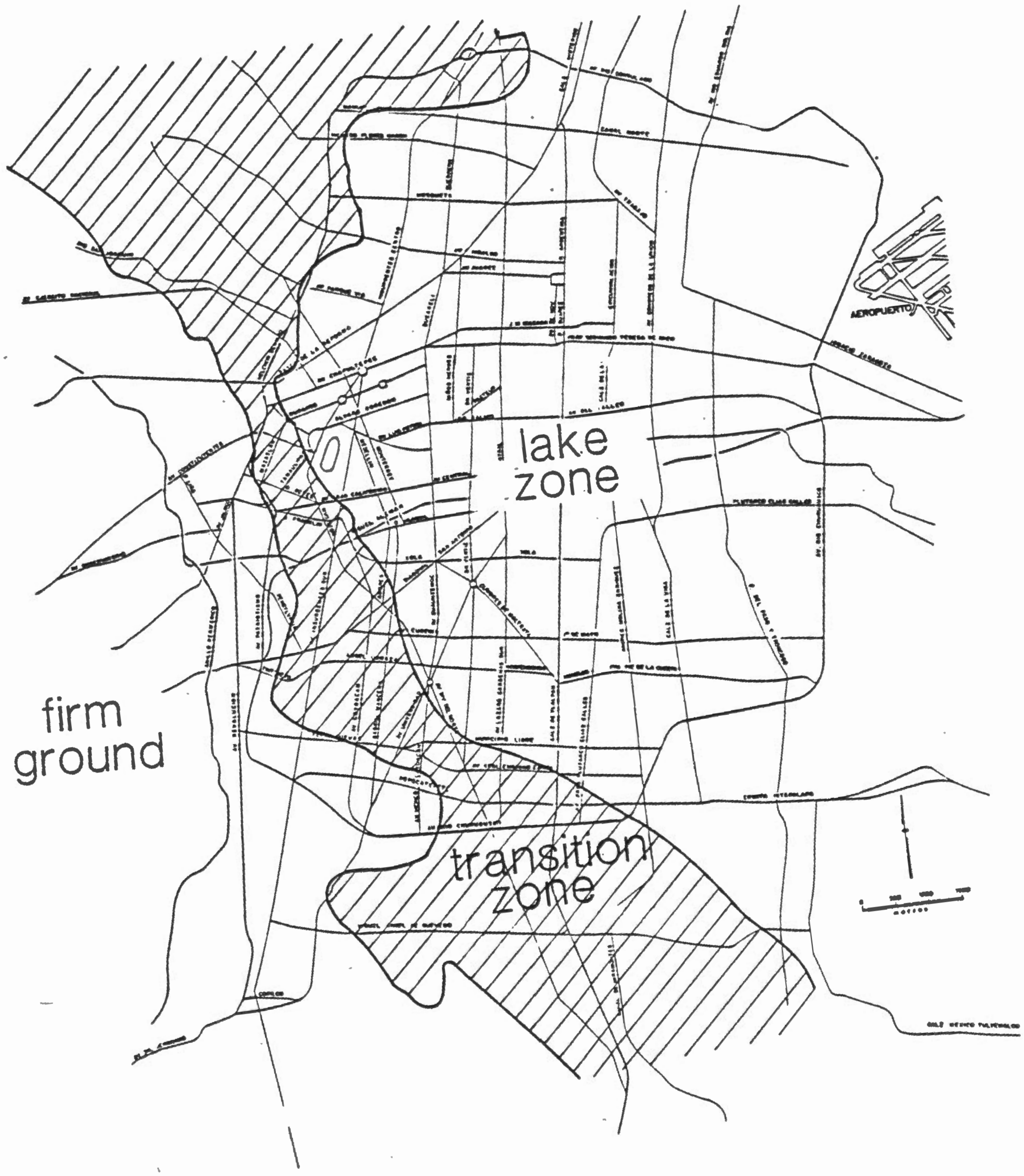

Fig. 4.2 Subsoil Zones of Mexico City (Ref. 4.3).

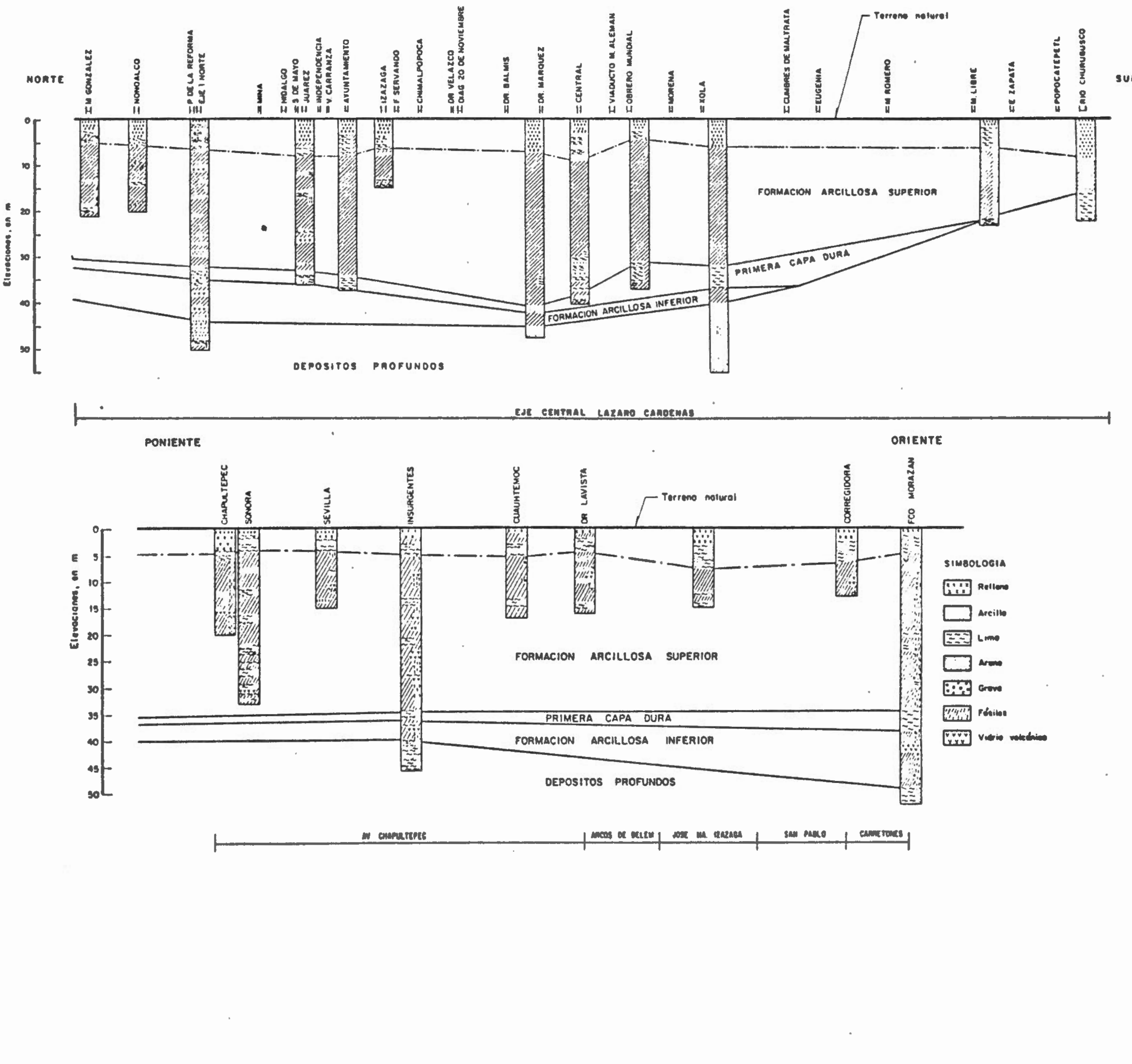

Fig. 4.3 Subsoil profiles across Mexico City showing compressible "Arcillosa" and firm ("capa dura") soil layers (taken from Ref. 4.3).

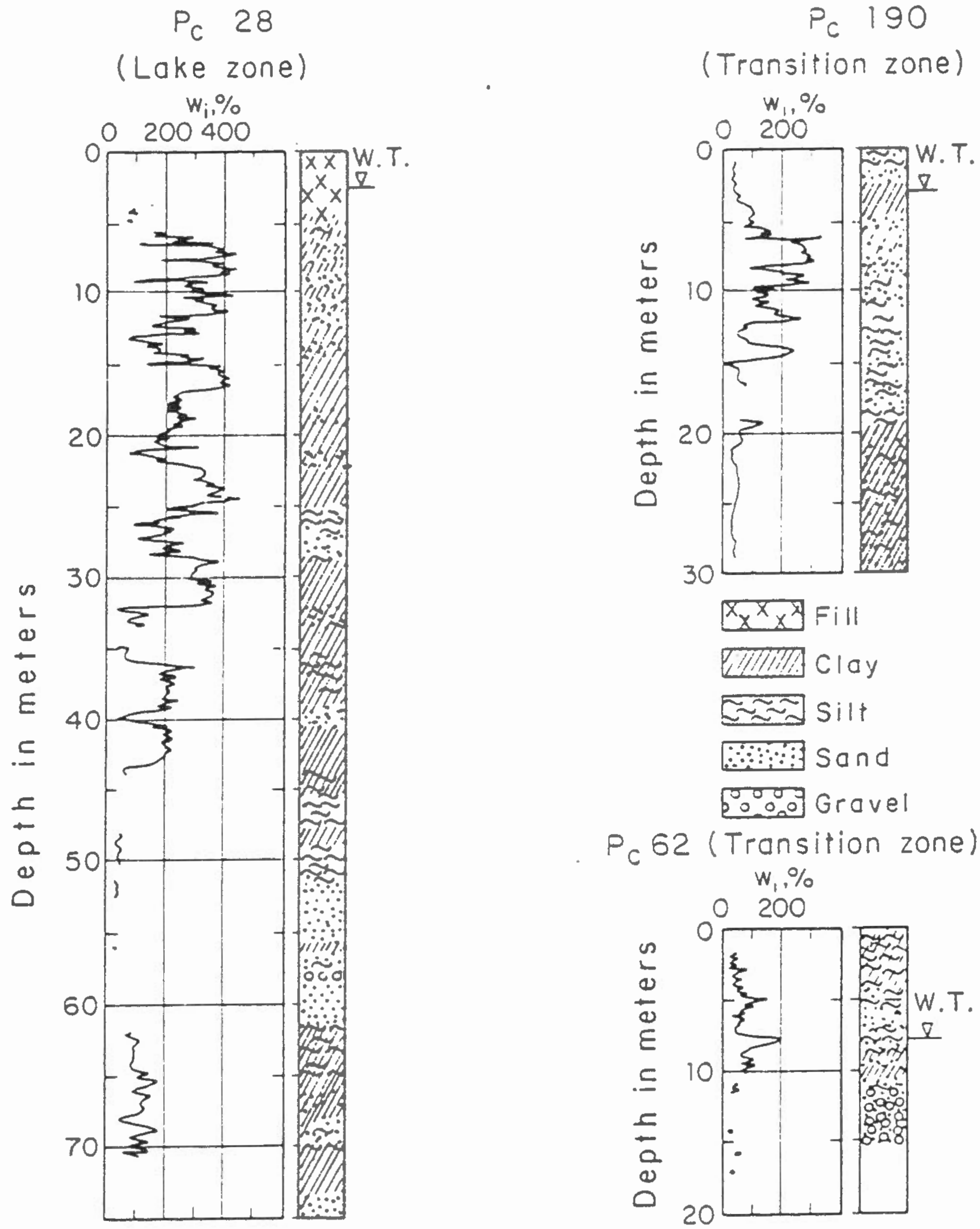

Fig. 4.4 Typical Borehole Logs in the Lake Zone (Pc 28) and Transition Zone (Pc 190 and Pc 62) of Mexico City (taken from Marsal, Ref. 4.2).

layer about 12 m thick below the lower clay. Index properties of the three upper soils (5 m - 50 m), which affect the behaviour of building foundations most, are given in Table 4.1. Logs of boreholes in the transition zone reveal shallower and less compressible lacustrine deposits than those in the lake zone.

Index Property	Soil Layer FAS	CD	FAI
Natural water content, w (%)	270	58	191
Liquid limit, w_L (%)	300	59	288
Plastic limit, w_P (%)	86	45	68
Specific gravity, s_s	2.30	2.58	2.31
Initial void ratio, e_i	6.17	1.36	4.53
Unconfined compressive strength, q_u, in kg/cm^2	0.85	2.4	1.6

Table 4.1 Average Values of Index Properties for Soils in Boring Pc-28 in the Lake Zone of Mexico City, (Marsal, Ref. 4.4).

Based on statistical analyses of a vast amount of laboratory tests, correlations of various engineering and index properties with the natural water content have been established (Ref. 4.2). They include initial void ratio, specific gravity, liquid and plastic limits, plasticity index, undisturbed and remolded unconfined compresive strength, friction angle, coefficient of compressibility, index of compressibility, and coefficient of consolidation. These correlations have become invaluable tools for the local geotechnical engineers who regularly use them to assist their foundation investigations and designs.

4.4 Ground Motions

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In summary, the "free-field" measurements from the coast indicate relatively moderate peak accelerations together with much high frequency energy. At Teacalco (Fig. 2.8) the high frequencies present at the coast have attenuated and the peak accelerations are lower.

Within Mexico City, records were obtained from the three subsoil zones as follows (see Fig. 4.5 and Table 4.2).

Variable	Direction	(Foothills Zone) UNAM	(Transition Zone) Viveros	(Lake Zone) Abastos	SCT
Acceleration gals (cm/sec^2)	NS	32	44	81	98
	EW	35	42	95	168
	V	22	18	27	36
Velocity (cm/sec)	NS	10	11	25	39
	EW	9	12	38	61
	V	8	6	9	9
Displacement (cm)	NS	6	9	15	17
	EW	8	7	19	21
	V	7	7	8	7

Table 4.2 Summary of "Free Field" Peak Ground Motions Recorded in Mexico City on September 19, 1985 (Ref. 2.13).

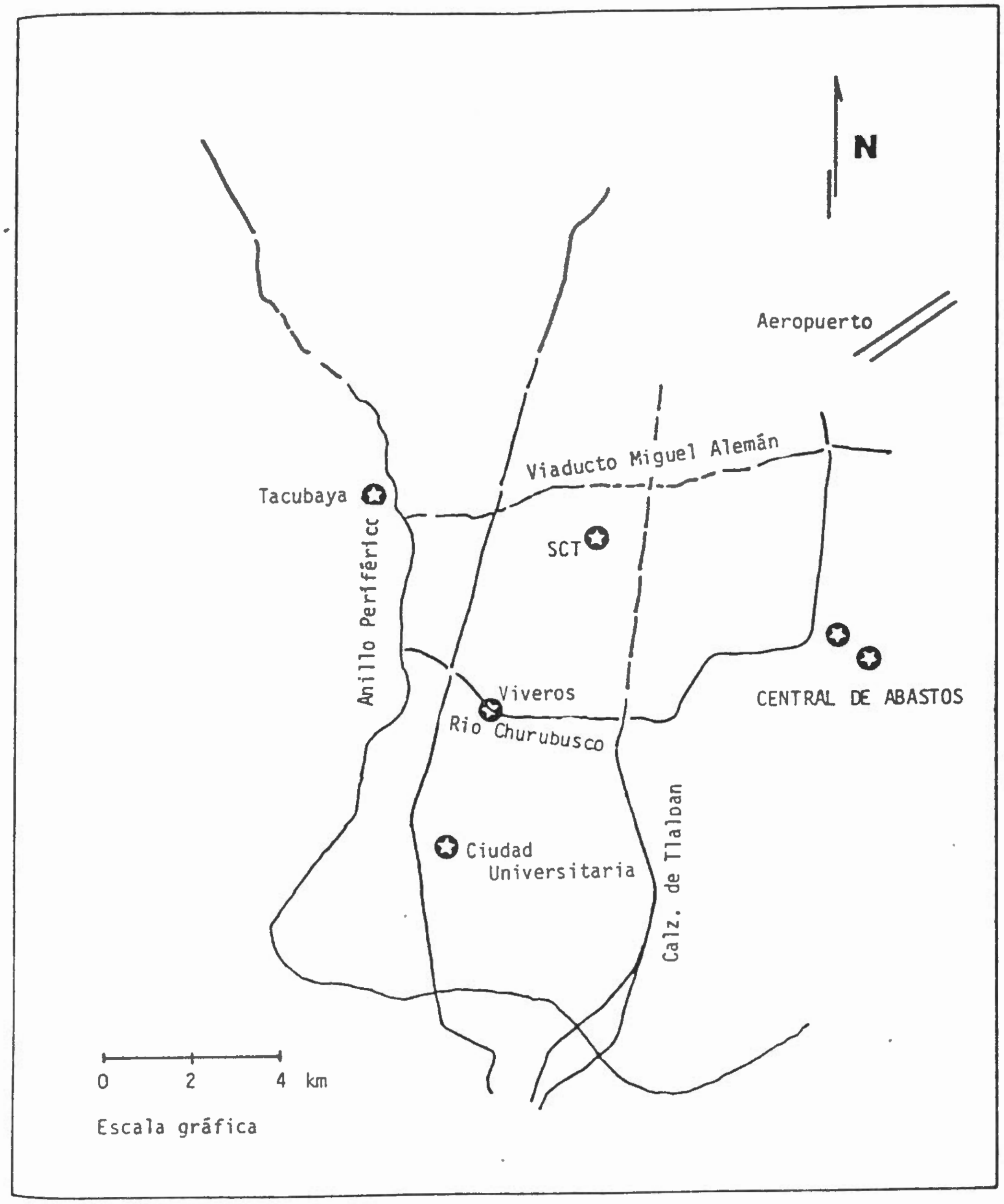

Fig. 4.5 Location of Mexico City Accelerograph Stations (taken from Ref. 2.13).

(1) Foothills Zone. The instruments on the University Campus, located on firm ground, recorded a peak acceleration of 4% g "free-field" and exceeded 0.5% g for 1 to 3 minutes (Fig. 2.8). Records from the Tacubaya observatory were also on firm soil, only about 4 km from the centre of the heavily damaged Zona Rosa. The accelerations here were slightly smaller than that at the university. Ground velocities were in the order of 10 cm/s at these sites.

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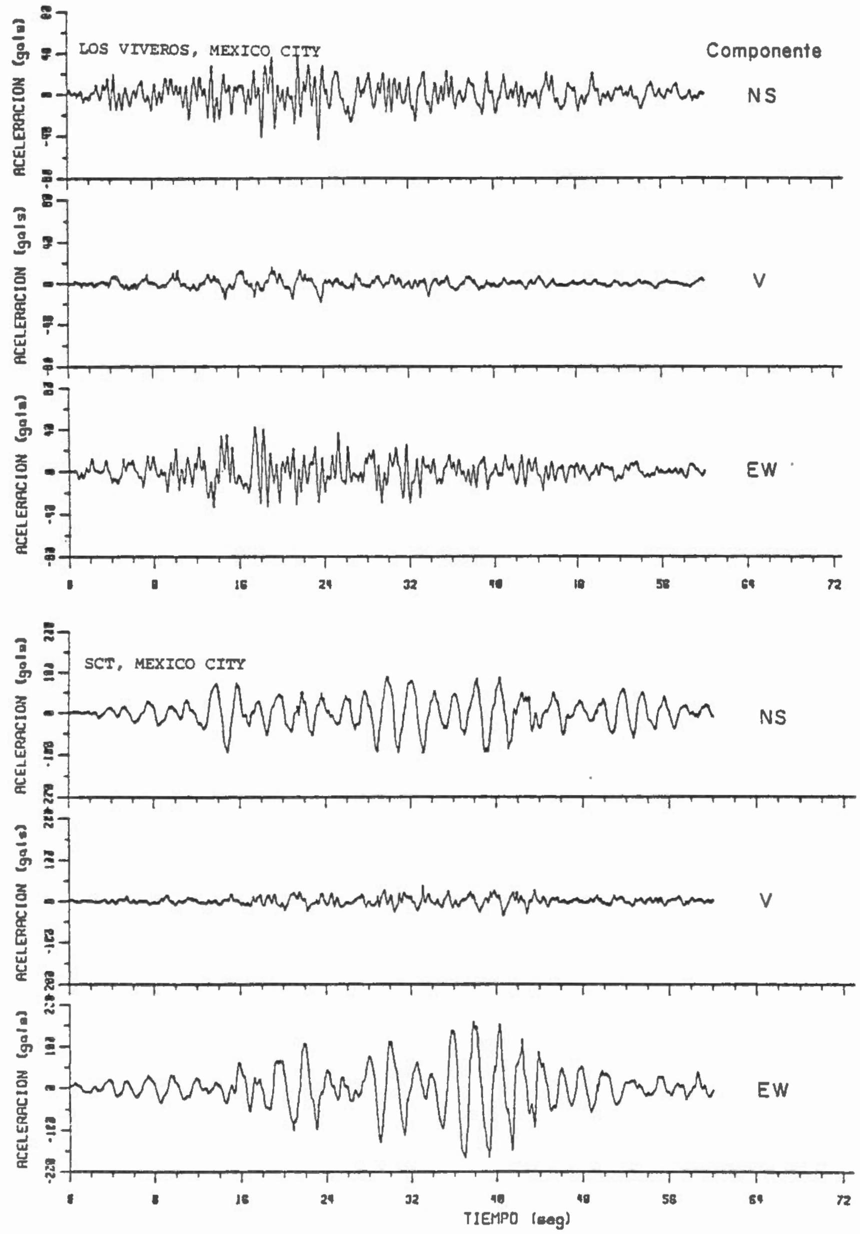

Fig. 4.6 Accelerographs of September 19, 1985 earthquake at Los Viveros (Ref. 2.13) and SCT (Ref. 2.11), Mexico City.

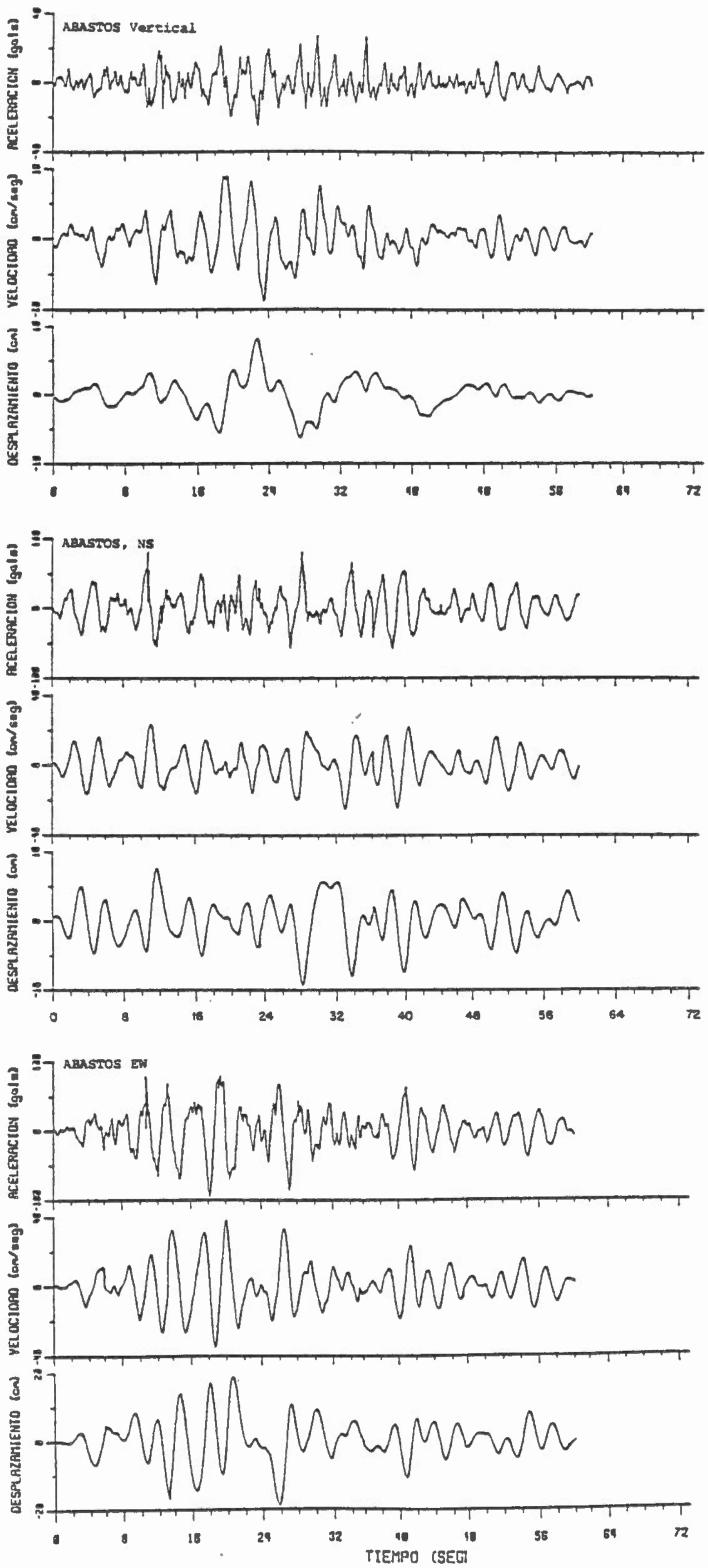

Fig. 4.7 Accelerographs and derived velocity and displacement plots for September 19, 1985 earthquake at Abastos, Mexico City (Ref. 2.12).

sites (Fig. 4.8), which show both the amplification of ground accelerations and the dominant period increase from the foothills zone through the transition zone to the lake zone as the thickness of lake deposits increases. Duration of strong ground motion also increased in this direction. As will be shown in Section 4.5, earthquakes have repeatedly damaged buildings in the parts of Mexico City lying in the western corner of the former lake bed. The corner bounds the area of more compressible soils and perhaps acts as a focus for seismic waves reverberating in the lake deposits.

4.5 Relationship Between Subsoil Conditions and Building Damage

This section summarizes the effect of the subsoil conditions on structural damage in Mexico City. More details on the structural damage can be found in Chapter 6. The material presented in this section is taken from a report (Ref. 4.3) prepared by the Institute of Engineering at the National Autonomous University of Mexico.

Figure 4.9 illustrates the distribution of damage in Mexico City for the structures that suffered total or partial collapse as well as for structures that were severely damaged.

Figure 4.10 shows the region classified as having minor but significant damage which covers an area of 65 km^2. The area classified as having major damage in the 1985 earthquake (see Fig. 4.10) covers an area of about 23 km^2. It is interesting to compare the 1985 regions of damage with the regions of damage in the July 28, 1957 and the March 14, 1979 earthquakes as shown in Fig. 4.10. The 1985 damage zones coincide in part with those of the previous earthquakes but extends to cover a much larger area towards both the east and the south.

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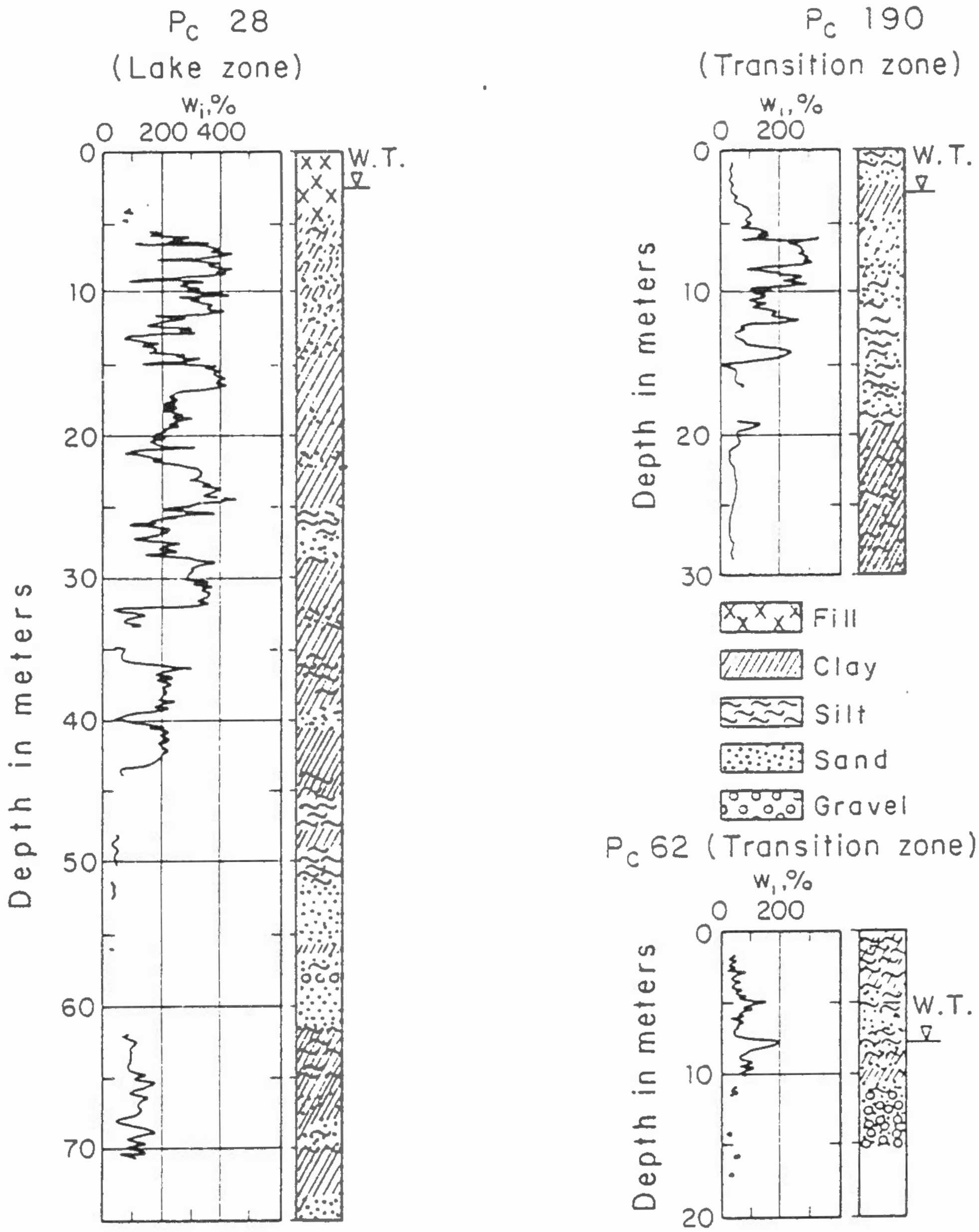

Fig. 4.4 Typical Borehole Logs in the Lake Zone (Pc 28) and Transition Zone (Pc 190 and Pc 62) of Mexico City (taken from Marsal, Ref. 4.2).

layer about 12 m thick below the lower clay. Index properties of the three upper soils (5 m - 50 m), which affect the behaviour of building foundations most, are given in Table 4.1. Logs of boreholes in the transition zone reveal shallower and less compressible lacustrine deposits than those in the lake zone.

Index Property	Soil Layer		
	FAS	CD	FAI
Natural water content, w (%)	270	58	191
Liquid limit, w_L (%)	300	59	288
Plastic limit, w_p (%)	86	45	68
Specific gravity, s_s	2.30	2.58	2.31
Initial void ratio, e_i	6.17	1.36	4.53
Unconfined compressive strength, q_u, in kg/cm^2	0.85	2.4	1.6

Table 4.1 Average Values of Index Properties for Soils in Boring Pc-28 in the Lake Zone of Mexico City, (Marsal, Ref. 4.4).

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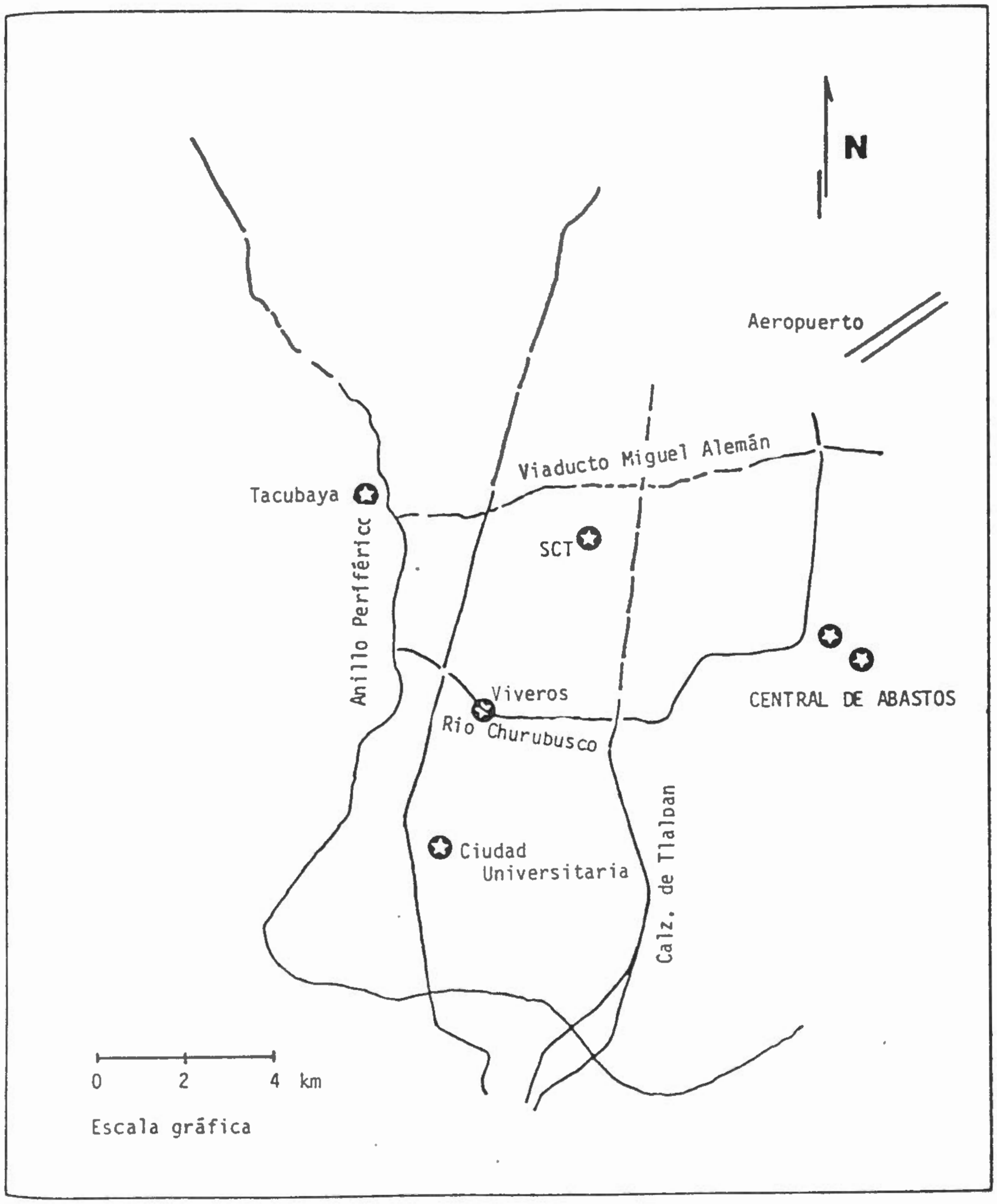

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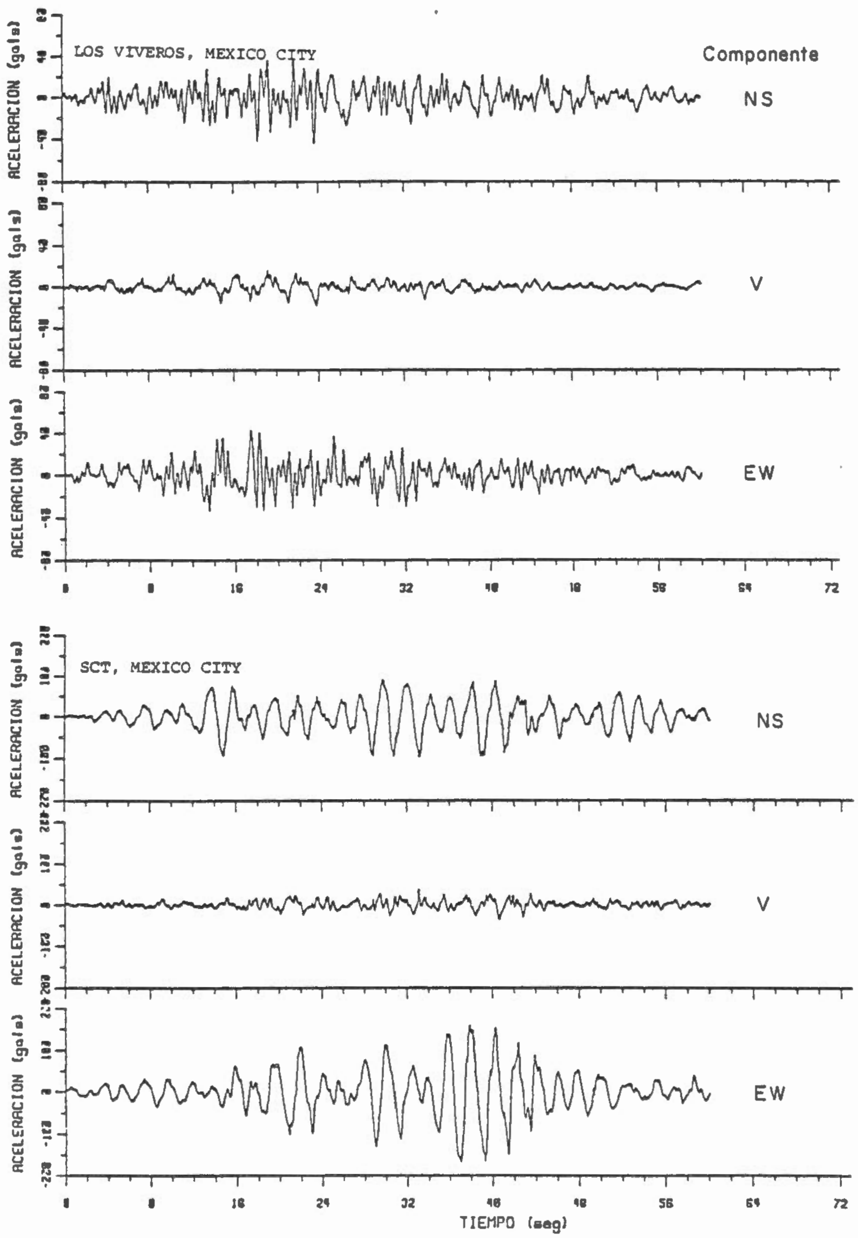

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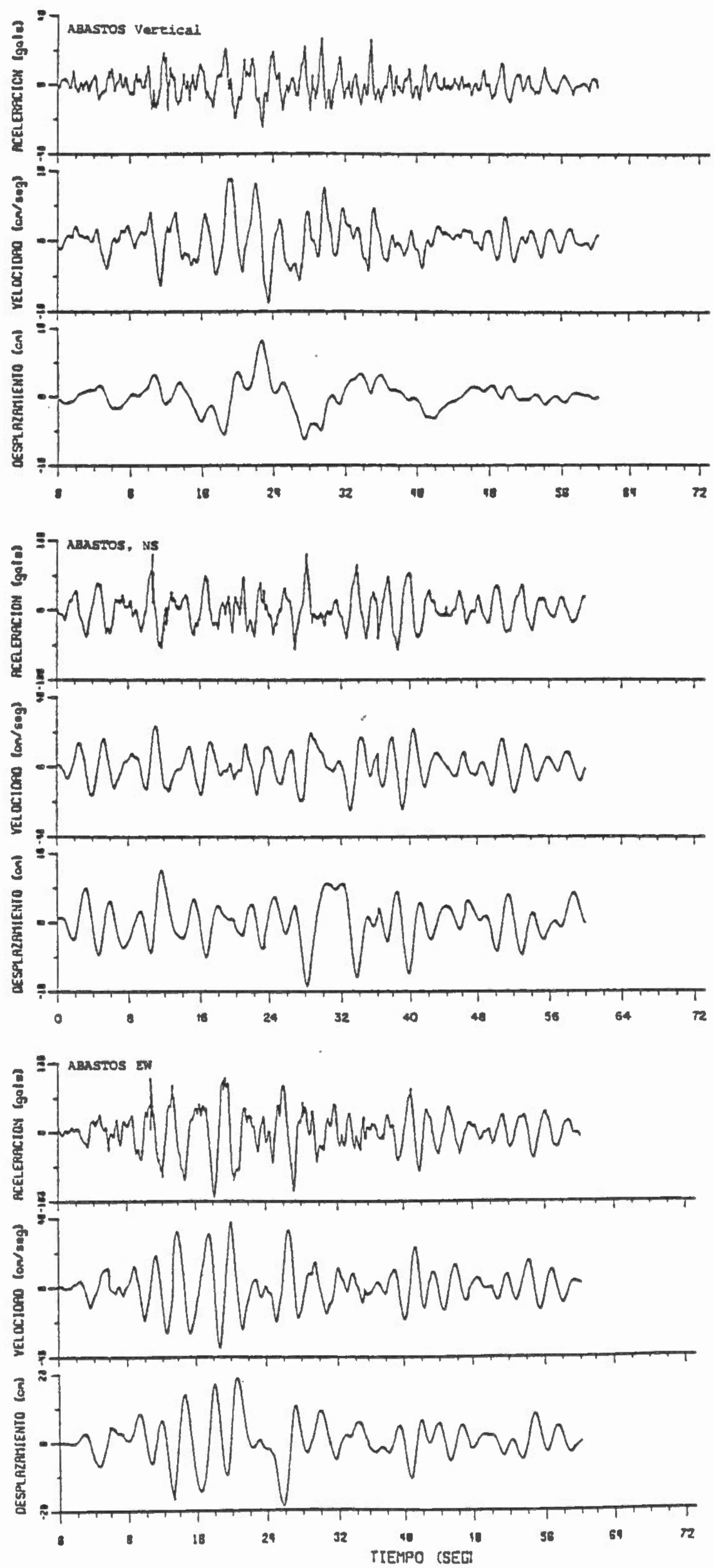

Fig. 4.7 Accelerographs and derived velocity and displacement plots for September 19, 1985 earthquake at Abastos, Mexico City (Ref. 2.12).

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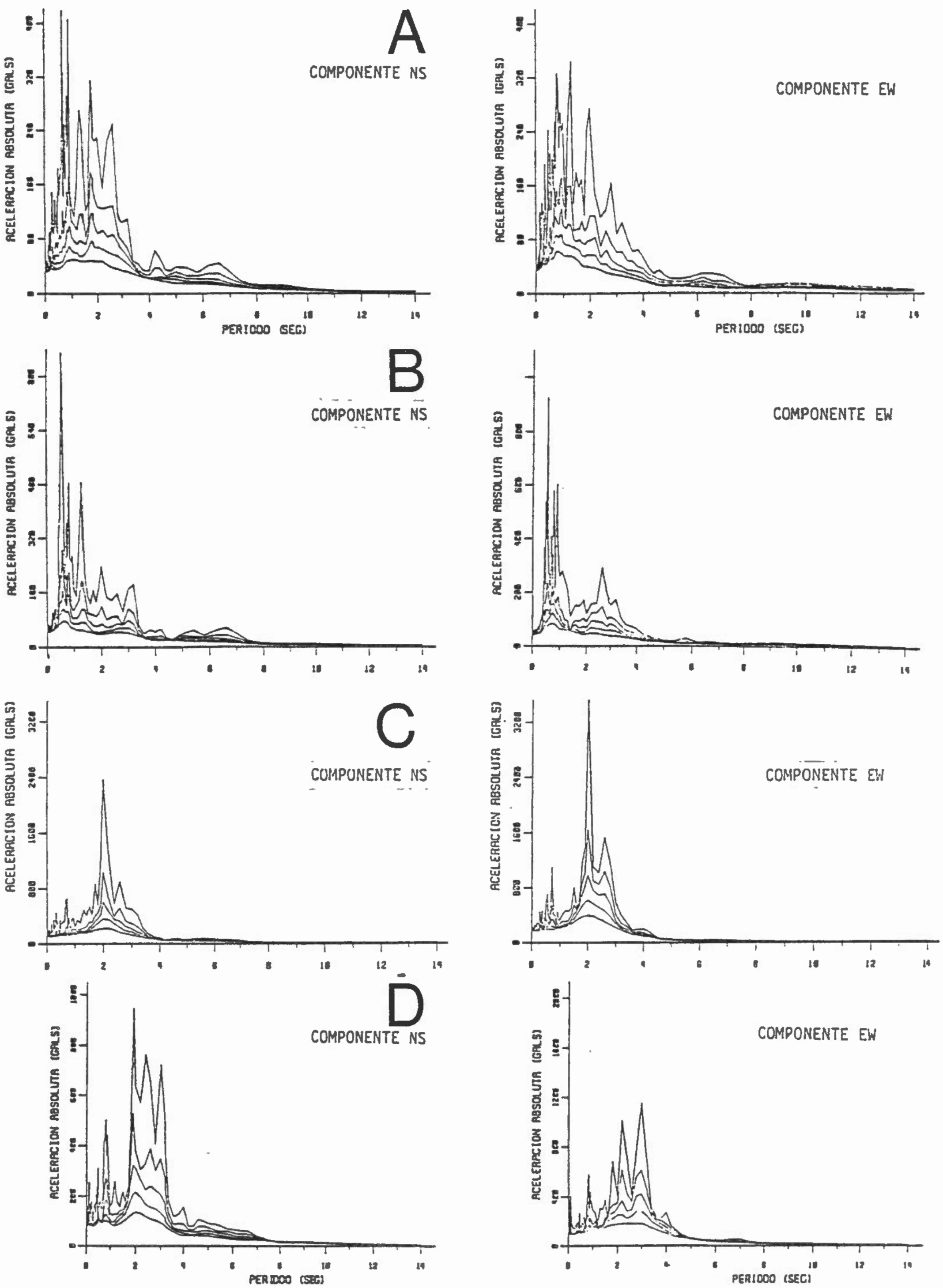

Fig. 4.8 Horizontal (NS at left, EW at right) Acceleration Response Spectra of September 19, 1985 earthquake at (A) UNAM, (B) Viveros, (C) SCT and (D) Abastos (Ref. 2.13). Care is needed in using this figure as different vertical scales are used for the spectra. The NS peak numerical values are A - 420, B - 850, C - 2400, D - 950.

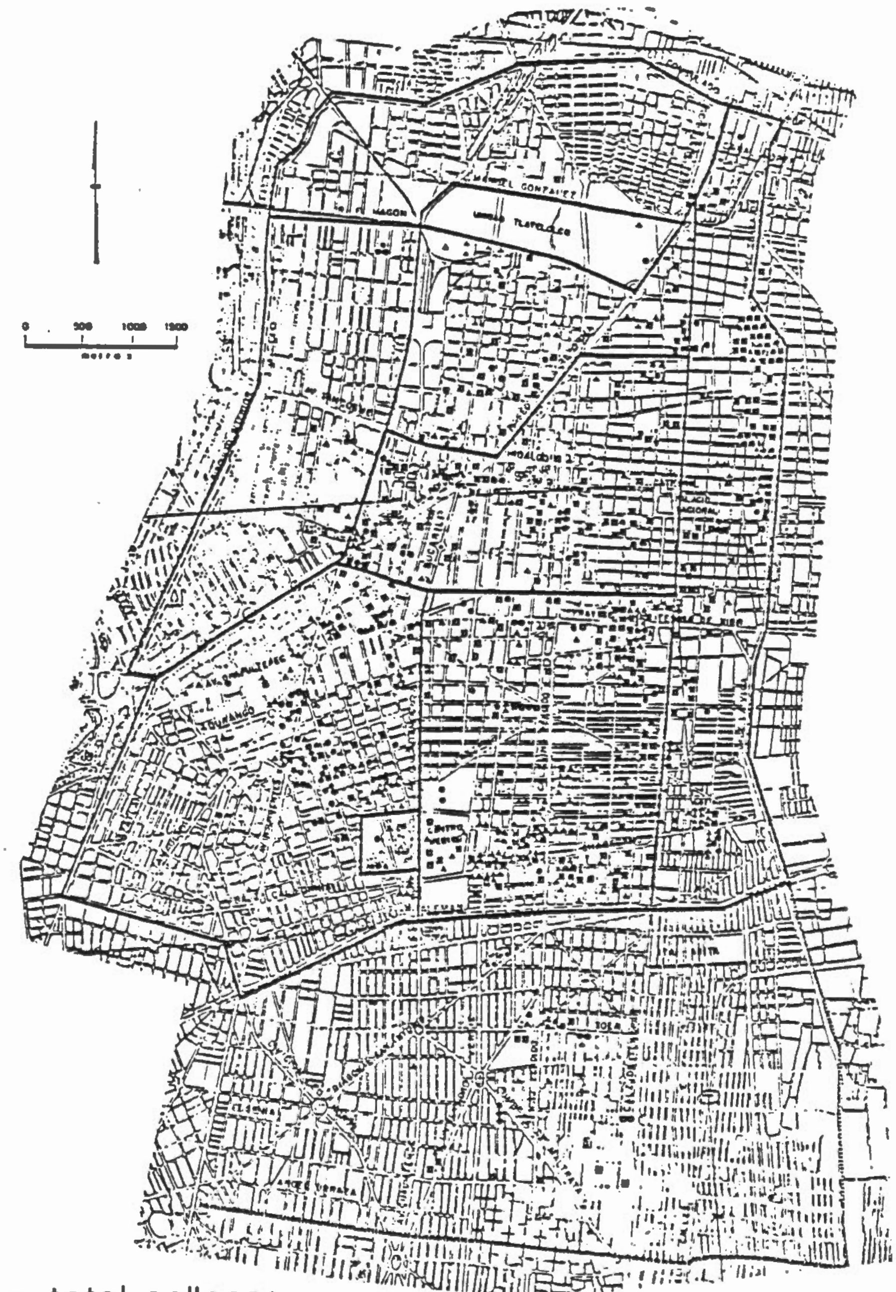

Fig. 4.9 Distribution of Collapsed or Severely Damaged Structures in Mexico City (taken from Ref. 4.3).

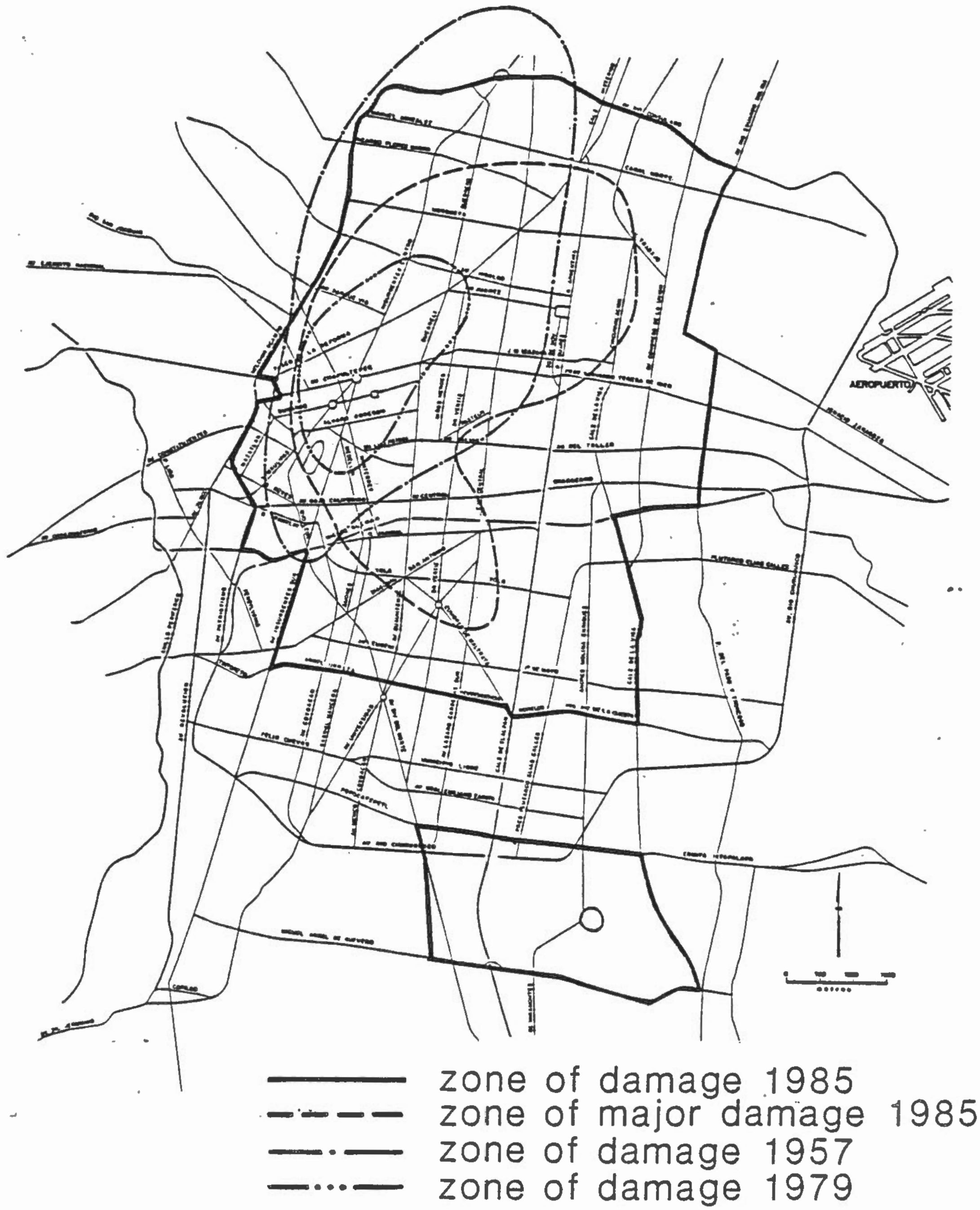

Fig. 4.10 Zones of Damage in Mexico City (taken from Ref. 4.3).

is compared with Fig. 4.10. The region with the greatest amplification and which suffered the most severe damage is located in the west lake zone where the depth to the first competent layer ranges from 26 to 32 m and the depth to the second competent layer ranges from 30 to 46 m. The ground motion amplification decreased in regions having thinner or thicker lake deposits.

4.6 Foundation Engineering Practice

Excellent descriptions of the problems associated with the design of foundations for the difficult subsoil conditions are given by Zeevaert (Ref. 4.5 and 4.6).

The withdrawal of groundwater by drainage and by pumping from sand aquifers sandwiched between highly compressible clays has been chiefly responsible for a general irregular subsidence in the lake and transition zones. Landfills and building loads also contributed to about 20% of the total settlements (Ref. 4.2). The effects of the water extraction from well pumping on the lowering of piezometric levels in the aquifers and on the observed ground subsidence were demonstrated by Nabor Carrillo in 1948 (Ref. 4.4). Since then, strict restrictions on the exploitation of aquifers within the urban area has reduced the subsidence rate signficantly. Figure 4.11 shows the progression of settlement with time at the Cathedral and at Alameda Park within the old city. As can be seen settlements of 7 m have taken place in a period of 70 years! Figure 4.12 shows the settlement contours in metres over the area of the old city.

Besides the regional subsidence discussed above, building settlements are caused by the net increase of loads from structures and/or fills and by the disturbance of the subsoil due to foundation construction and due to earthquakes. Because of the high compressibility and sensitivity of Mexico City clay, the natural heterogeneity of the deposits

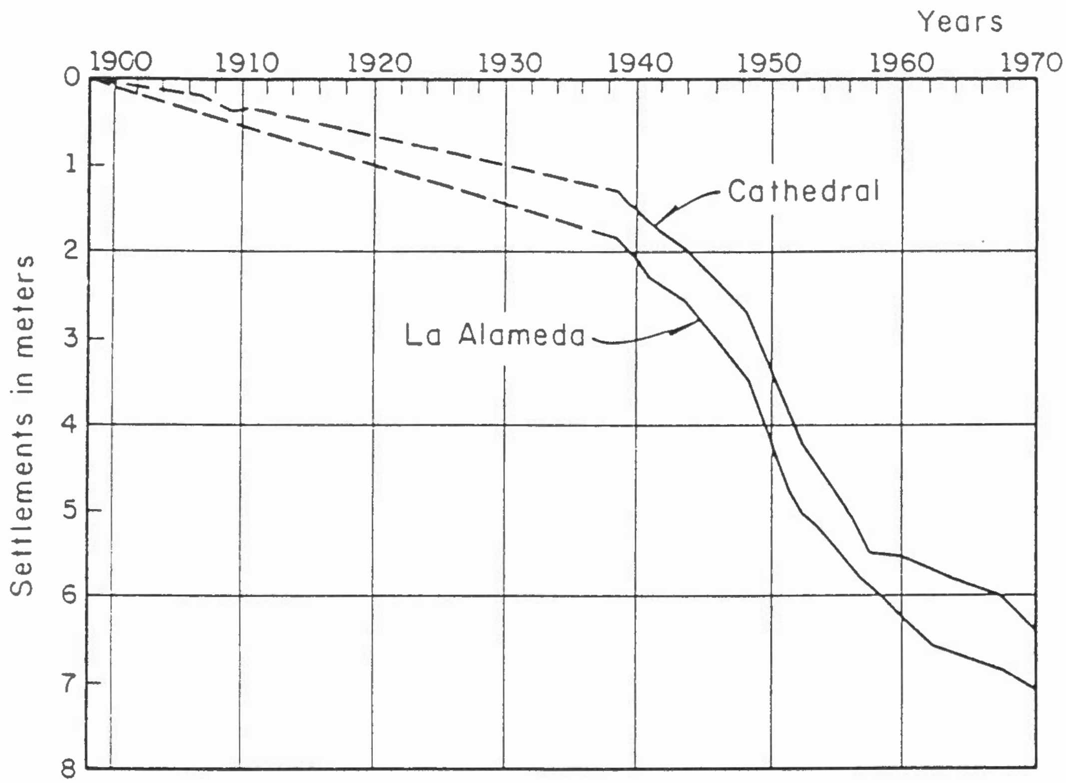

Fig. 4.11 Settlements at the Cathedral and La Alameda (taken from Ref. 4.2).

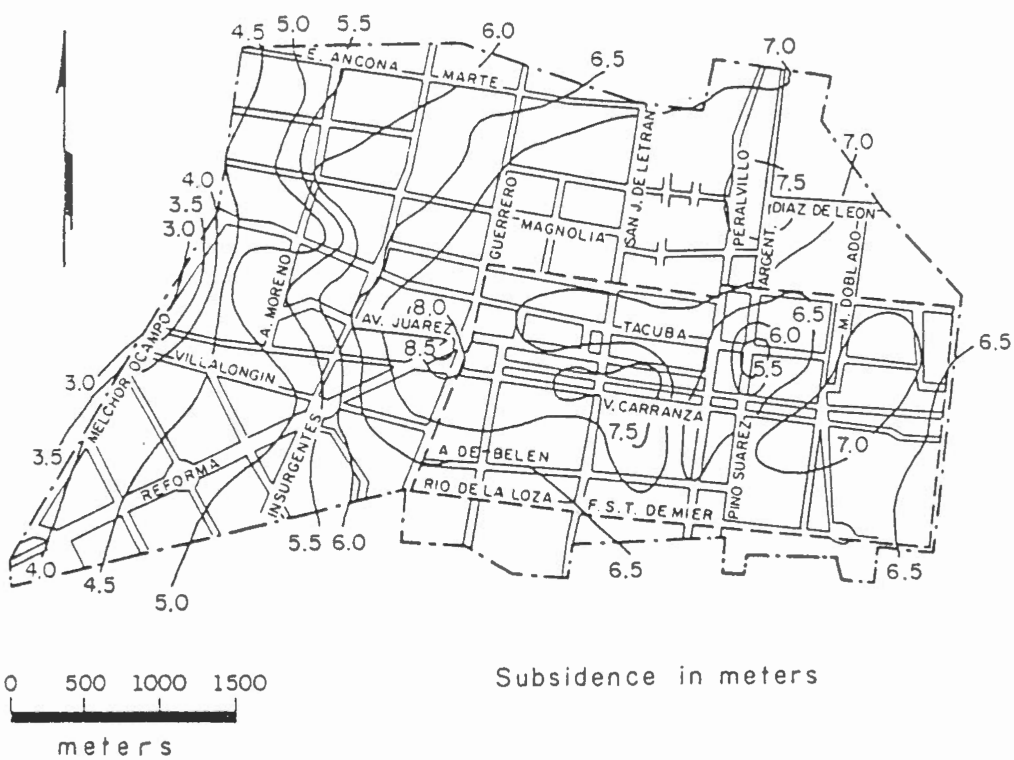

Fig. 4.12 Contours of Equal Subsidence in the Period from 1891 to 1970 (taken from Ref. 4.2).

and the varying loading history, settlements play a prominent role in the design and structural performance of buildings. The highly compressible clay deposits in the lake zone could settle in the order of 1 m with about 0.5 m differential settlements under an applied net load of only 20 kPa over a large area (Ref. 4.2).

Besides the intense shaking of ground over the lake deposits, it is believed (Ref. 4.7) that earthquakes also tend to disturb the sensitive structure of the Mexico City clay causing reduction of its shearing strength. In cases of severe disturbance, foundation failures resulting from the loss of soil strength do occur.

Light structures such as one or two storey houses are usually founded on shallow footings of masonry (see Fig. 4.13a) or concrete. These footings are constructed near the ground surface either individually or are interconnected by grade beams. Allowable bearing pressures for shallow footings founded on Mexico City clay are extremely low: about 50 kPa for areas preloaded by old buildings; and about 30 kPa for areas involving normally-consolidated clay (Ref. 4.1). Besides problems of low bearing capacity, differential settlement and disturbance of the subsoil by the preparation of foundations at deeper strata for adjacent buildings is an important concern for this type of foundation.

To mitigate the settlement problem for taller and heavier structures, "floating" or "compensating" foundations are adopted to reduce or eliminate the net load increase due to the erection of buildings by excavations for basement floors. Fig. 4.13b illustrates a concrete box floating foundation in which the bottom slab is stiffened with beams. Flat slabs together with retaining walls have also been used for floating foundations. In some special cases cylindrical shells running the length of the building and stiffened transversely by walls have been used to

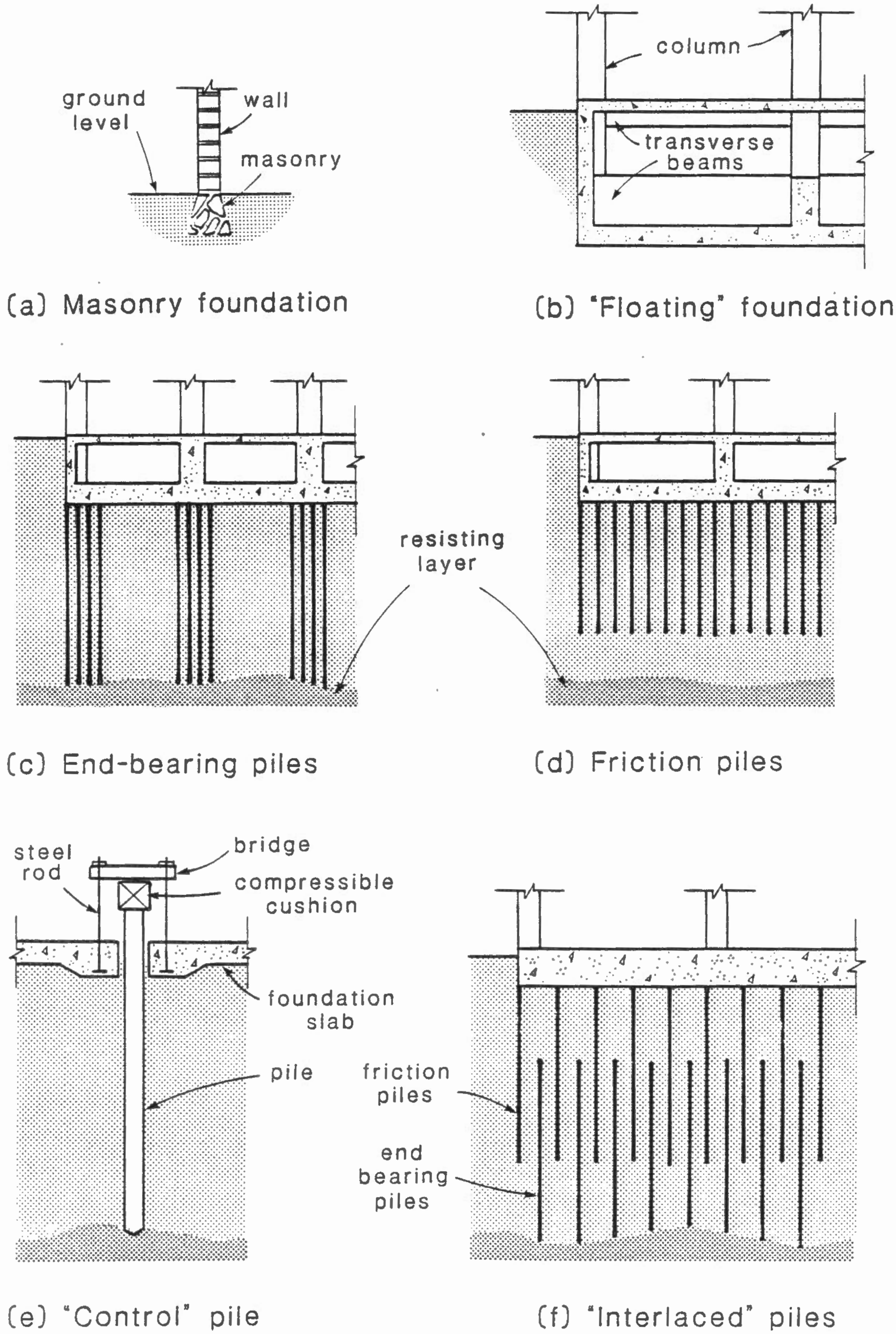

Fig. 4.13 Foundations used in Mexico City
(adapted from Ref. 4.2).

transfer loads to the subsoil. To ensure proper behaviour of a floating foundation, it is important to determine the net load increase from the structure accurately. In addition, during the excavation the control of heaving and slope stability of the excavation bottom and slopes are also critical to the subsequent satisfactory performance.

Pile foundations are also used for heavy and tall structures. If end-bearing piles, founded on the first competent layer are used (see Fig. 4.13c) then ground settlement, relative to the position of the structure, will result in the ground floor being above grade. However if friction piles are used (see Fig. 4.13d), they tend to mitigate the problem as the piles settle with the surrounding soil. In addition several special foundation systems have been employed to overcome grade changes relative to the structure.

Figure 4.13e illustrates the use of an end-bearing "control" pile. The structure hangs off of the control pile and a "cushion" of compressible material transfers the load from the structure to the pile. This system permits the structure to follow the ground settlements in a controlled way. In addition some existing structures with foundation problems have been retrofitted with control piles. These control piles must be inspected on a regular basis and adjustments made if necessary. Inspection is also necessary after an earthquake.

Figure 4.13f illustrates the use of interlaced piles. The end bearing piles are first driven to the first competent resisting layer. The top "follower" is then removed from these end bearing piles. Friction piles are then driven to "interlace" the end bearing piles. This results in a stiffening of the subsoil and a response intermediate between that of a friction and an end-bearing pile.

4.7 References

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4.2 Marsal, R.J., "The Lacustrine Clays of the Valley of Mexico," Contribution of the Instututo de Ingenieria to the International Clay Conference, July, Mexico, 1975.

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4.4 Carrillo, N., "Influence of Artesian Wells on the Sinking of Mexico City", Proc. of 2nd Int'l Conf. on Soil Mech. and Found. Engrg., Holland, 1948.

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4.6 Zeevaert, L., "Environmental Conditions in the Design of Building Foundations", 7th Nabor Carrillo Lecture, Sociedad Mexicana de Mecanica de Suelos, Mexico, 1984.

4.7 Rosenblueth, E., "The Earthquake of 28 July 1957 in Mexico City", Proceedings of the Second World Conference on Earthquake Engineering - Vol. 1", Japan 1960, pp. 359-379.

CHAPTER 5

DESIGN AND CONSTRUCTION PRACTICE

5.1 Seismic Design Provisions

This section provides a translation which summarizes the seismic design provisions contained in "The Requirements for Safety and Serviceability of Structures - Regulations for Construction in the Federal District" (Ref. 5.1).

5.1.1 Classification of Structures

Structures are classified into different groups, according to use and into different structural types as shown in Tables 5.1 and 5.2.

Group	Use
A	- important structures including electrical substations, telecommunications centres, hospitals, schools, stadia, transportation terminals, monuments, museums and industrial structures containing dangerous materials.
B	- includes ordinary buildings industrial plants, banks, apartment buildings, hotels, office buildings.
C	- includes buildings less than 2.5 m in height (one storey buildings) that do not require seismic design.

Table 5.1 Classification of Structures According to Use

Type	Structural Type
1	buildings, industrial plants, auditoria, and similar structures in which the lateral loads are resisted at each level by continuous braced or unbraced frames, by diaphragms or walls or a combination of the above systems. Also included are chimneys, towers, walls as well as inverted pendulums or structures in which over 50 percent of the mass is at the top and are supported by one main resisting element in the direction of the analysis
2	tanks
3	retaining walls
4	other structures

Table 5.2 Classification of Structural Type

5.1.2 Subsoil Zones

Fig. 5.1 and Table 5.3 gives the classification of the four different subsoil zones in the Federal District according to the thickness, H of the compressible soil.

Zone	Subsoil Classification
I	compressible soil with $H < 3$ m
II	compressible soil with $3 \text{ m} \leqq H < 20$ m
III	compressible soil with $H \geqq 20$ m
IV	region of undefined soil properties

Table 5.3 Classification of Subsoil Zones

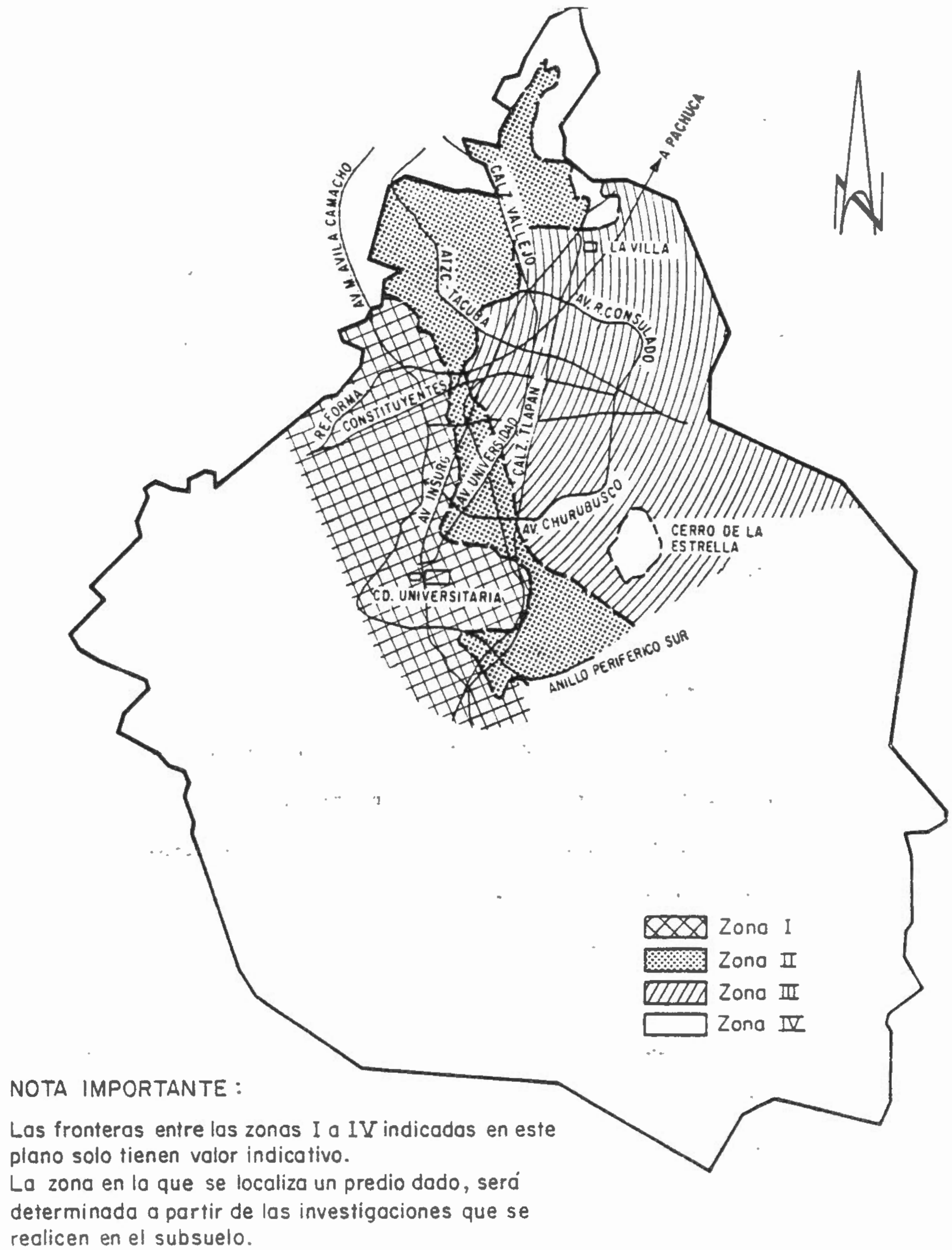


Fig. 5.1 Zonation of the Federal District into Four Types of Subsoil (From Ref. 5.1)

5.1.3 Seismic Coefficient

The non-dimensional seismic coefficient, c used for elastic analysis is given in Table 5.4.

Zone	c
I (firm ground)	0.16
II (transition soil zone)	0.20
III (compressible soil)	0.24

Note: For structures in Group A multiply the above values by 1.3.

Table 5.4 Seismic Coefficient for Structures in Group B

5.1.4 Reduction for Ductility

The seismic design forces determined by static analysis or determined by dynamic modal analysis can be accounted for by using a seismic force reduction factor, Q'. This force reduction factor, Q' is a function of the ductility factor, Q. The deformations calculated using the reduced force levels must be multiplied by the ductility factor Q. The ductility factor, Q may differ in the two orthogonal directions of the structure being analysed. Table 5.5 summarizes the requirements that must be satisfied for different levels of ductility.

Case	Type of Structure	Requirements	Ductility Factor
1	1	Resistance is provided in all the levels by unbraced frames of reinforced concrete or of steel with a defined yield plateau and satisfying the following conditions: (a) steel beams and columns must satisfy the requirements of compact sections and the joints must permit significant rotations prior to failure. (b) concrete columns must have spirals or ties such that the core has an equivalent confinement to spirals. (c) in the consideration of the failure limit state for shear force, torsion, buckling due to axial compression or other forms of brittle failure a load factor of 1.4 should be used. (d) concrete members must satisfy the requirements for plastic hinges in members. These requirements must be satisfied at each end of beams and columns or at the locations where plastic hinges are required to form a collapse mechanism at each floor level if the lateral force were sufficiently increased. (e) the minimum ratio of the resistance capacity of one storey to the design force level should not differ by more than 20 percent of the average of these ratios for all storeys.	6.0
2	1	Resistance is provided in all the levels exclusively by unbraced frames of concrete wood or steel with or without a well defined yield plateau, together with braced frames or concrete shear walls in which the capacity of the frames without the walls or braces is at least 25 percent of the total. The minimum ratio of the resistance capacity of one storey to the design force level should not differ by more than 35 percent of the average of these ratios for all storeys.	4.0
3	1	Resistance to lateral loads is provided by beams or columns of reinforced concrete, wood or steel (braced or unbraced) or concrete walls that do not meet the requirements of cases 1 and 2 of this table or by solid-block masonry walls confined by pilasters, bond beams, columns or beams of reinforced concrete or steel.	2.0
4	1	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 1 to 3 above.	1.5
5	1 to 4	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 factor may be used.	1.0

Note: All the Complementary Technical Codes must be satisfied in order to utilize the Q factors given in this table.

Table 5.5 Ductility Factor, Q

5.1.5 Dynamic Analysis

The Code (Ref. 5.1) specifies design spectra for Zones 1, II and III for use in modal dynamic analyses of structures in Group B. These spectra, shown in Fig. 5.2 reflect the amplification that takes place with softer soil conditions and also reflect the influence of the natural periods of the ground motion. These design spectra are for a ductility factor, Q = 1.0. The manner in which the design spectrum for Zone III is modified to account for the ductility factor is shown in Fig. 5.3. The Code spectra correspond to a damping ratio of approximately 5 percent (Ref. 5.2). Dynamic analysis must be used for building greater than or equal to 60 m in height.

5.1.6 Static Analysis

A static analysis method is prescribed in the code (Ref. 5.1) for those structures less than 60 m in height. The static analysis is more complex than the current Canadian Code (Ref. 5.3) and is described below.

The total base shear, V is first approximated as

$$V = c_s \Sigma W_i \qquad (5\text{-}1)$$

where c_s = factor equal to the greater of c/Q or a_o

a_o = obtained from Table 5.6 depending on zone

ΣW_i = total weight of structure = W

The horizontal force, P_i acting on the mass at level i is then estimated as:

$$P_i = \frac{W_i h_i}{\Sigma W_i h_i} V \qquad (5\text{-}2)$$

where W_i = weight at level i

h_i = the height from the base of the structure to level i

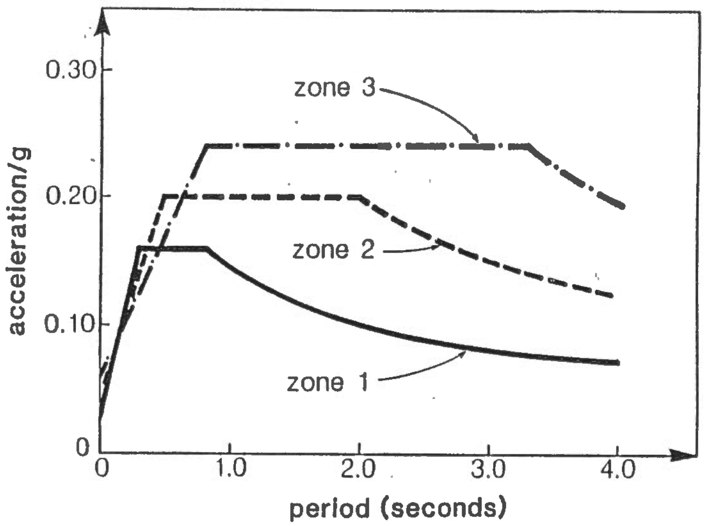


Fig. 5.2 Design Spectra for Structures in Group B

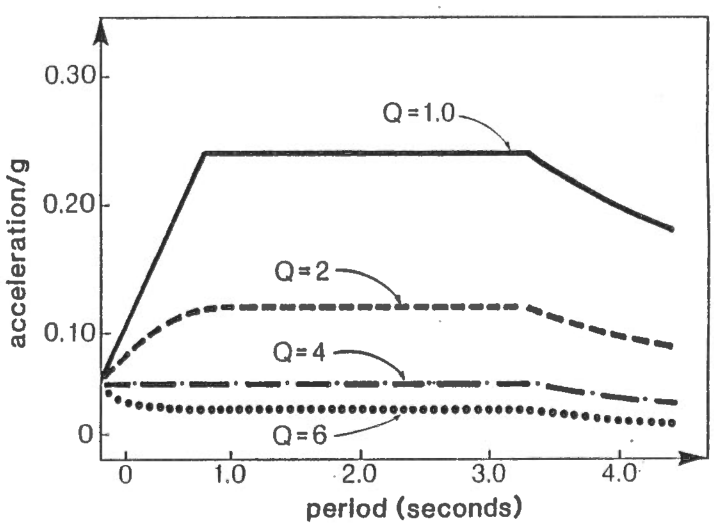


Fig. 5.3 Modification of Design Spectrum for Zone III Accounting for Ductility Factors (Group B Structures)

The spectral parameters used to determine the horizontal forces are given in Table 5.6. These parameters define the response spectra shown in Fig. 5.2 (i.e. c is the maximum acceleration ratio, a_o is the acceleration ratio when T = 0 and T_1 and T_2 are the "corner" periods).

zone	c	a_o	T_1(sec)	T_2	r
I - firm ground	0.16	0.030	0.3	0.8	1/2
II - transition zone	0.20	0.045	0.5	2.0	2/3
III - soft soil	0.24	0.060	0.8	3.3	1

Note: For Group A structures multiply c and a_o by 1.3.

Table 5.6 Factors used to Determine Lateral Forces

The forces P_i can be further reduced for some situations. First the fundamental period, T, is calculated using the Raleigh-Ritz method as follows:

$$T = 2\pi \sqrt{\frac{\Sigma W_i X_i^2}{g \, \Sigma P_i X_i}} \qquad (5\text{-}3)$$

where g = acceleration due to gravity

X_i = deflection of the mass at level i in the direction of P_i.

The lateral forces are then determined depending on the relationship between the fundamental period, T, of the structure and the "corner" periods of the response spectra, T_1 and T_2, given in Table 5.6 and shown in Fig. 5.2. The three cases are described below:

(1) If $T_1 \leqq T \leqq T_2$ no reduction from the values calculated in Equation 5-2 are permitted

(2) If $T > T_2$ then

$$P_i = W_i(k_i h_i + k_2 h_i^2)c/Q \qquad (5\text{-}4)$$

where $k_1 = q[1-r(1-q)]\, \Sigma W_i/(\Sigma W_i h_i)$

$k_2 = 1.5\, rq(1-q)\; W_i/(\Sigma W_i h_i^2)$

$q = (T_2/T)^r$

(3) If $T < T_1$ the lateral forces are proportional to those determined from Equation (5-2) with the values of P_i adjusted such that

$$V = c_{sr}\, \Sigma W_i \qquad (5\text{-}5)$$

where $c_{sr} = [a_o + (c - a_o)T/T_1]/Q'$

$Q' = 1 + (Q - 1)T/T_1$

5.1.7 Simplified Method of Analysis

The simplified method of analysis may be used for Type I structures provided all of the following conditions are satisfied:

(1) at least 75 percent of vertical loads are supported by walls either of concrete, masonry bricks or blocks

(2) There must be at least two bearing walls in each level which are parallel or form an angle between them of not larger than 20 degrees. These walls must be connected to the slabs over at least 50 percent of the building length.

(3) the longer dimension of the plan cannot exceed twice the shorter plan dimension

(4) the height of the structure must not exceed 13 m and the height must not exceed 1.5 times the minimum base dimension

If all of the above conditions are satisfied then the lateral force coefficient from Table 5.7 may be used.

	Brick Walls Height of Structure, H (m)			Hollow Block Walls Height of Structure, H (m)		
Zone	< 4	4<H<7	7<H<13	<4	4<H<7	7<H<13
I	0.06	0.08	0.08	0.07	0.11	0.11
II	0.07	0.08	0.10	0.08	0.11	0.13
III	0.07	0.09	0.10	0.08	0.10	0.12

Notes: - above values are for Group B structures
- for Group A multiply above values by 1.3

Table 5.7 Lateral Force Coefficients Reduced for Ductility for the Simplified Method of Analysis

The force levels at each level of the structure are determined using the coefficients in Table 5.7 and then converted into ultimate loads. The shear resistance of the walls at any level must equal or exceed the ultimate shear. If the storey height, h at any level exceeds 1.33 times the length of a wall, L, then the shear resistance of that wall must be reduced by multiplying by $(1.33\ L/H)^2$, where H is the height of the structure in m.

5.1.8 Torsional Eccentricity

The torsional eccentricity, e_s, is calculated at each floor level as the distance between the centre of torsion at the corresponding floor level and the position of the resultant horizontal seismic force for that floor level. The torsional moment in each is then determined by multiplying the lateral force by the most unfavourable of the following effective eccentricities:

$$e_{eff} = 1.5e_s + 0.1\ b \qquad (5\text{-}6)$$

$$e_{eff} = 1.5e_s - 0.1\ b \qquad (5\text{-}7)$$

where b = maximum plan dimension perpendicular to the lateral force direction

5.1.9 Load Factors

For combinations of permanent and variable actions (e.g. dead and live loads) the load factor is 1.4. This factor is increased to 1.5 for meeting places, schools, theaters, etc. For combinations of actions that include an accidental action (e.g. earthquake effects) as well as permanent and variable actions the load factor is 1.1. For actions or internal forces that have a favourable effect on the resistance or stability of the structure a factor of 0.9 is used.

5.1.10 Horizontal Displacement Limits

Lateral deflections of a storey relative to its adjacent storeys is limited to 0.008. This limit is increased to 0.016 if the structure contains non-structural elements that are not likely to be damaged by these larger permissible deflections.

The 1977 provisions (Ref. 5.1) require a minimum separation between adjacent buildings to prevent collision or pounding of buildings during an earthquake. The minimum separation is equal to the accumulated horizontal displacements calculated but must be at least 0.001, 0.0015 and 0.002 of the total building height for zones I, II and III respectively. If the accumulated displacements are not calculated then the minimum separation must be 0.006, 0.007 and 0.008 of the total height for zones, I, II and III respectively. In no case can the separation be less than 50 mm.

5.2 Reinforced Concrete Design and Construction

This section summarizes some of the important provisions given in "The Design and Construction of Concrete Structures - Complementary Technical Provisions to the Regulations for Construction in the Federal District" (Ref. 5.4).

In calculating the nominal resistance of a member the concrete strength to be used is 80 percent of the specified concrete strength, f'_c

whereas the unfactored specified yield stress of the steel reinforcement is used. In addition to this concrete material strength factor the nominal resistance is reduced by multiplying by the following resistance factors: 0.90 for flexure, 0.80 for shear and torsion, 0.85 for columns with special confinement reinforcement and 0.75 for columns not having special confinement reinforcement.

Figure 5.4 illustrates the detailing requirements for concrete frame members for two different ductility factors (Q = 4 and Q = 6). The column tie spacing limits are a function of the column dimensions (c_1 and c_2) and the tie diameter (d_b tie).

In addition, to prevent buckling of the longitudinal bars the tie spacing is limited to 850 $d_b/\sqrt{f_y}$ where d_b is the longitudinal bar diameter and f_y is the yield stress of the longitudinal bar in kg/cm^2. At the ends of the member the tie spacing is limited to one-half of this value.

The column ties must have at least one longitudinal bar in each corner and the ends must have 135 degree hooks with a straight extension of at least ten bar diameters. If 90° bends are used at the ends of the ties then a straight extension of at least 20 bar diameters must be used. The yield force of the tie must be at least 1/200 of the yield force of the largest longitudinal bar or bundle that it is restraining. Cross ties may be used to laterally restrain longitudinal bars that are not in the corners of the column. These crossties are anchored by 180 degree bends at each end with a straight extension of at least 10 bar diameters and must go around the longitudinal bar or bundle of bars.

In order to use a ductility factor, Q, of 6 the presence of plastic hinges in the beams must be considered in designing the beams and joints for shear and for designing the columns. A load factor of 1.4 is applied to these resulting actions for design. The provisions for minimum

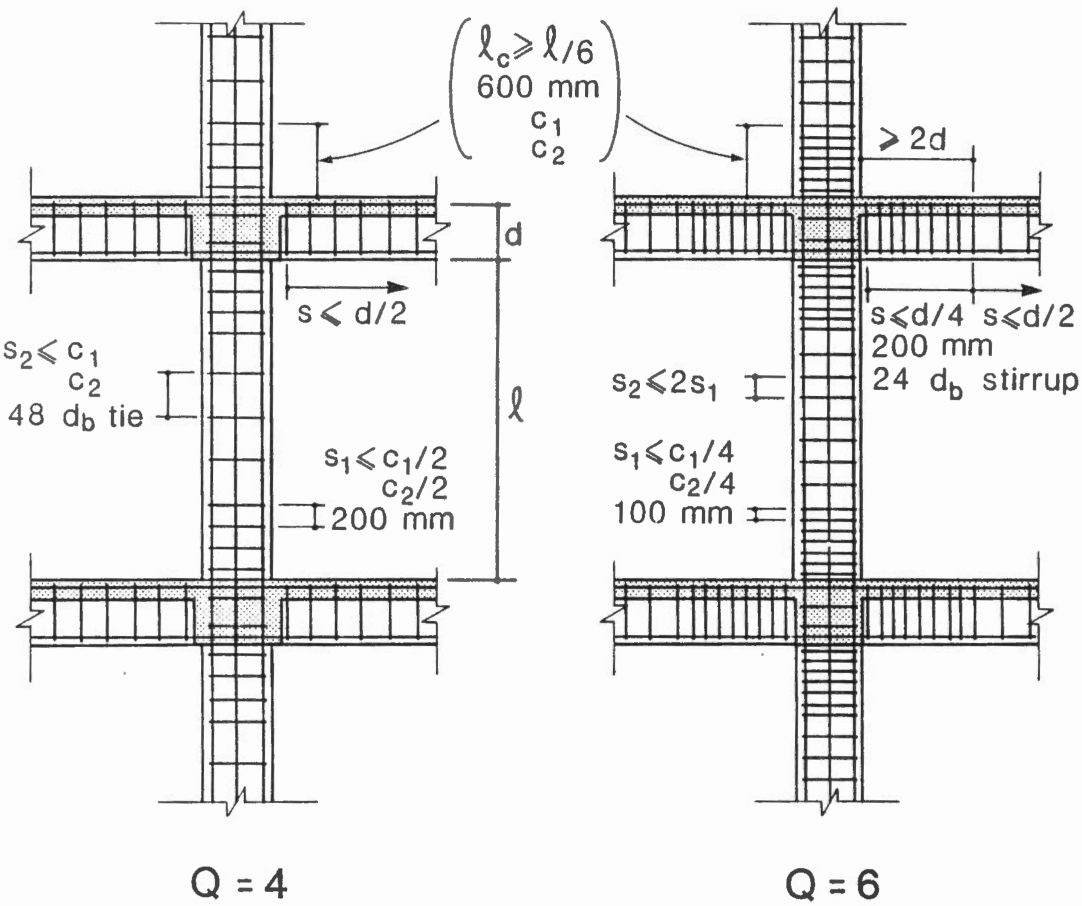


Fig. 5.4 Detailing of Beams and Columns for Different Ductility Factors, Q

confinement reinforcement for the columns in regions of plastic hinges are based on the requirements of the American Concrete Institute Code (Ref. 5.5). Figure 5.5 shows the details of column reinforcement for a 7 storey office building which was under construction in October, 1985.

Figure 5.6 illustrates the flat plate framing system that is very popular in Mexico. These waffle-type slabs are made by placing removeable waffle pans on the formwork before casting the concrete or by embedding hollow masonry blocks in the concrete.

Figure 5.7 illustrates the detailing used for a reinforced concrete shear wall in the Nikko Hotel.

A very popular form of construction is the use of reinforced concrete frames with infilled masonry walls. These structures are designed usually with Q = 4 or Q = 2. Examples of these structures are given in Chapter 6. Figure 5.8 shows a masonry wall which is not completely infilled. Although this construction detail is sometimes used for ventilation on the ground floor it results in a "short-column" over its exposed length, ℓ_o, which is susceptible to severe damage in the event of a large earthquake. It is necessary to provide extra confinement in the columns over a length equal to the exposed length of the column plus the column dimension, c, as shown in Fig. 5.8.

5.3 Design of Masonry Walls

The design provisions summarized in this section are taken from the "Design and Construction of Masonry Structures" (Ref. 5.6). The nominal compressive strength of the block varies from 0.67 times the average compressive strength for plant produced blocks to 0.53 times the average strength for those not produced in a plant. The nominal compressive of the masonry is a function of both the nominal strength of the block and the types of mortar.

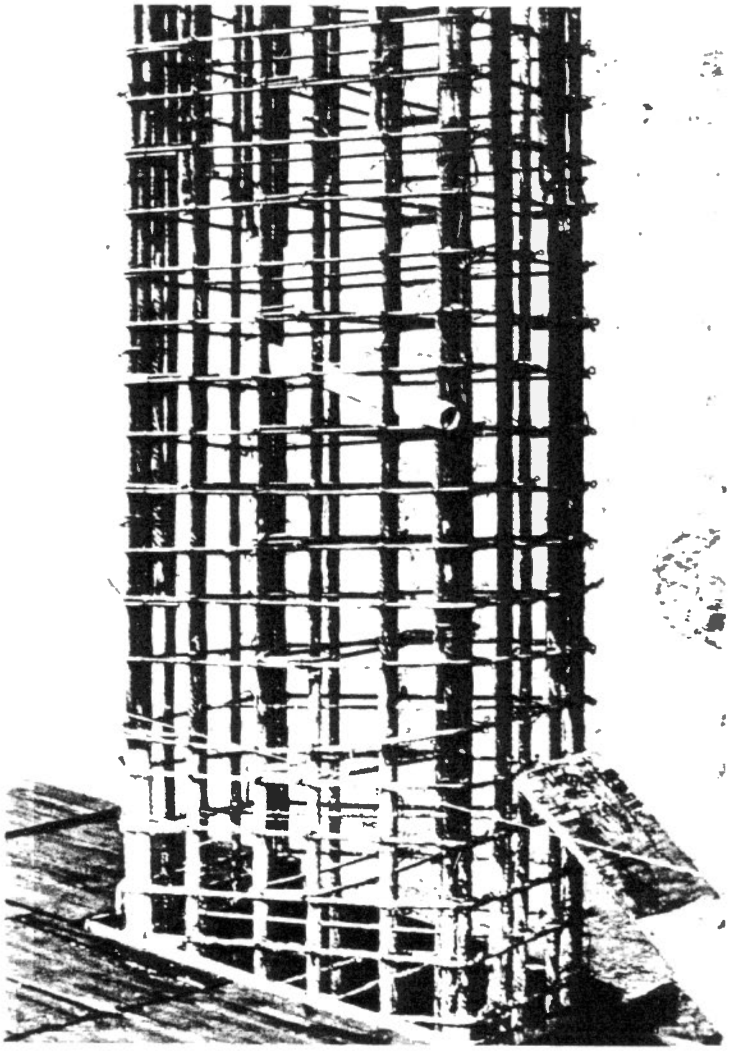
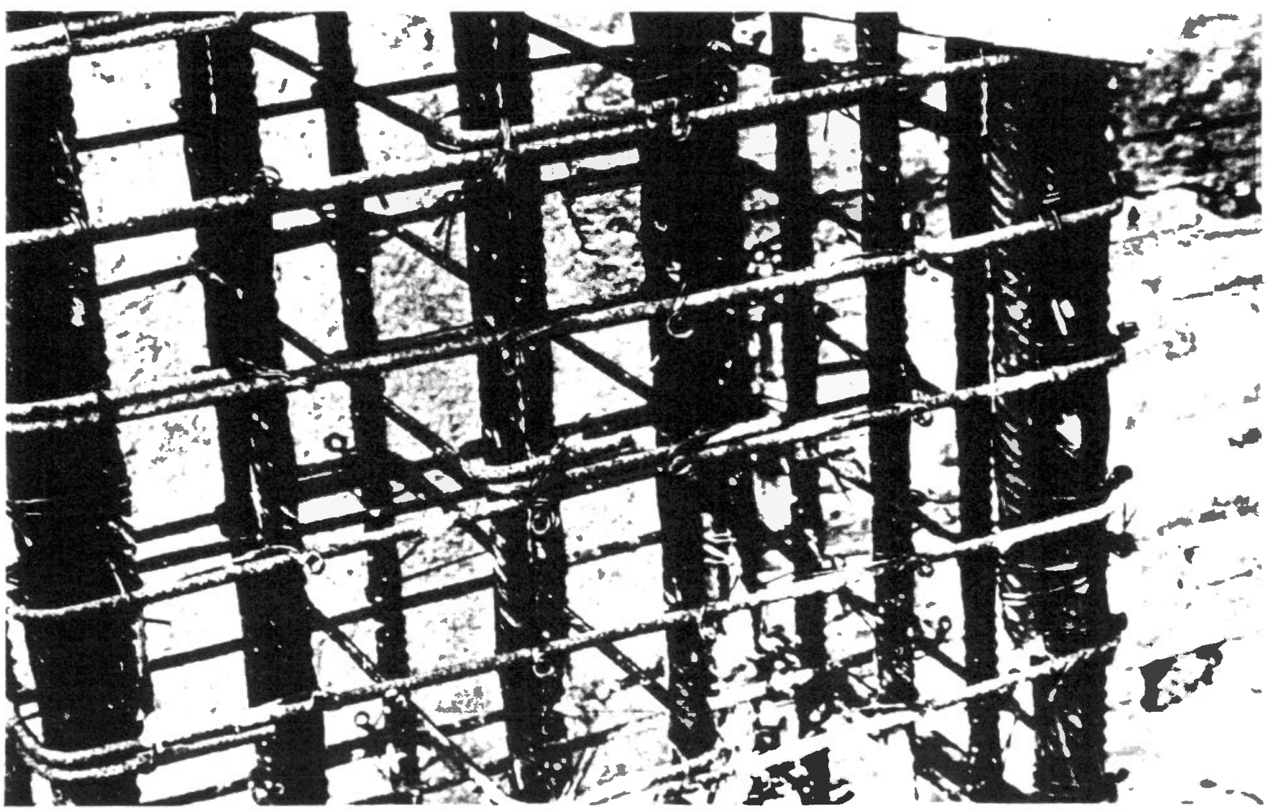


Fig. 5.5 Details of Column Reinforcement



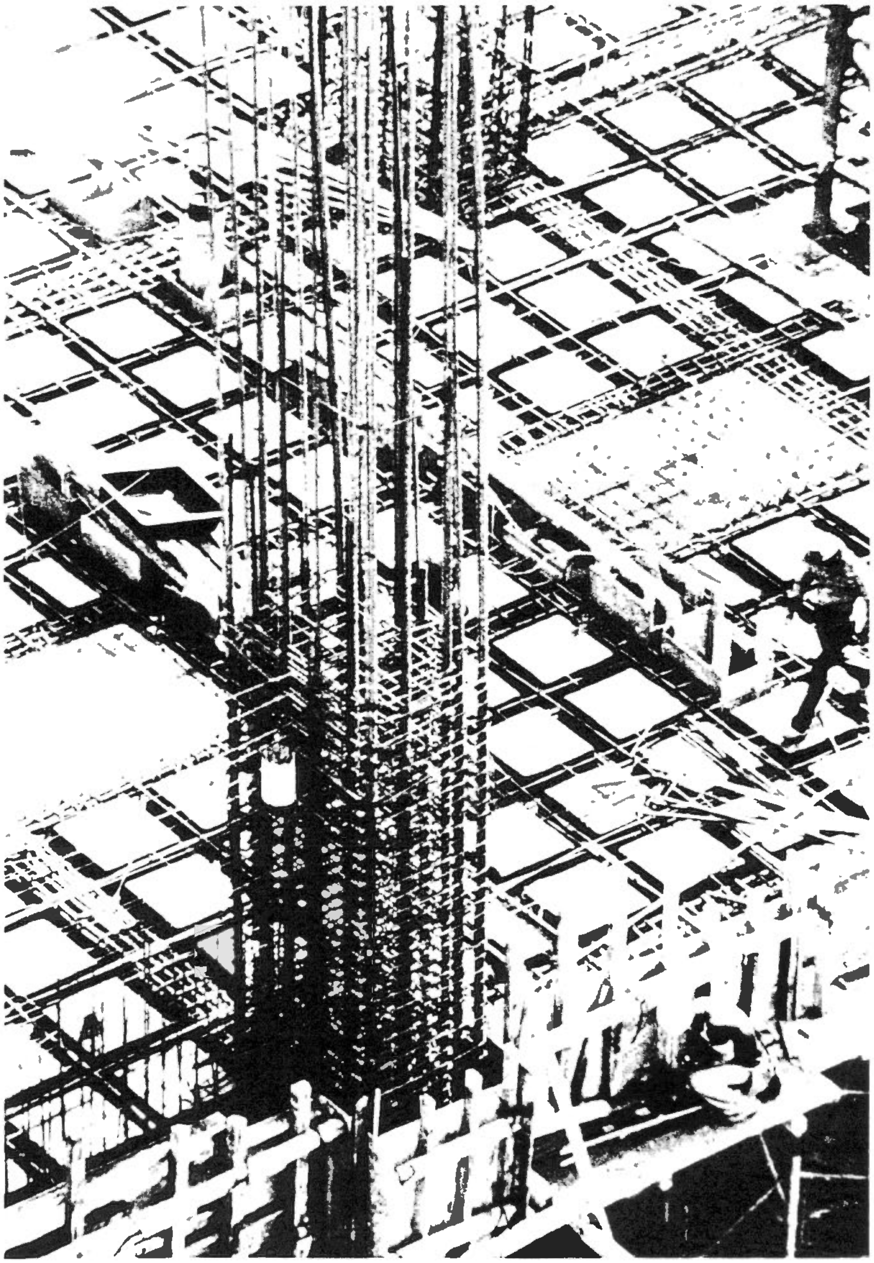


Fig. 5.6 Nikko Hotel Under Construction Showing Typical Waffle Slab Construction

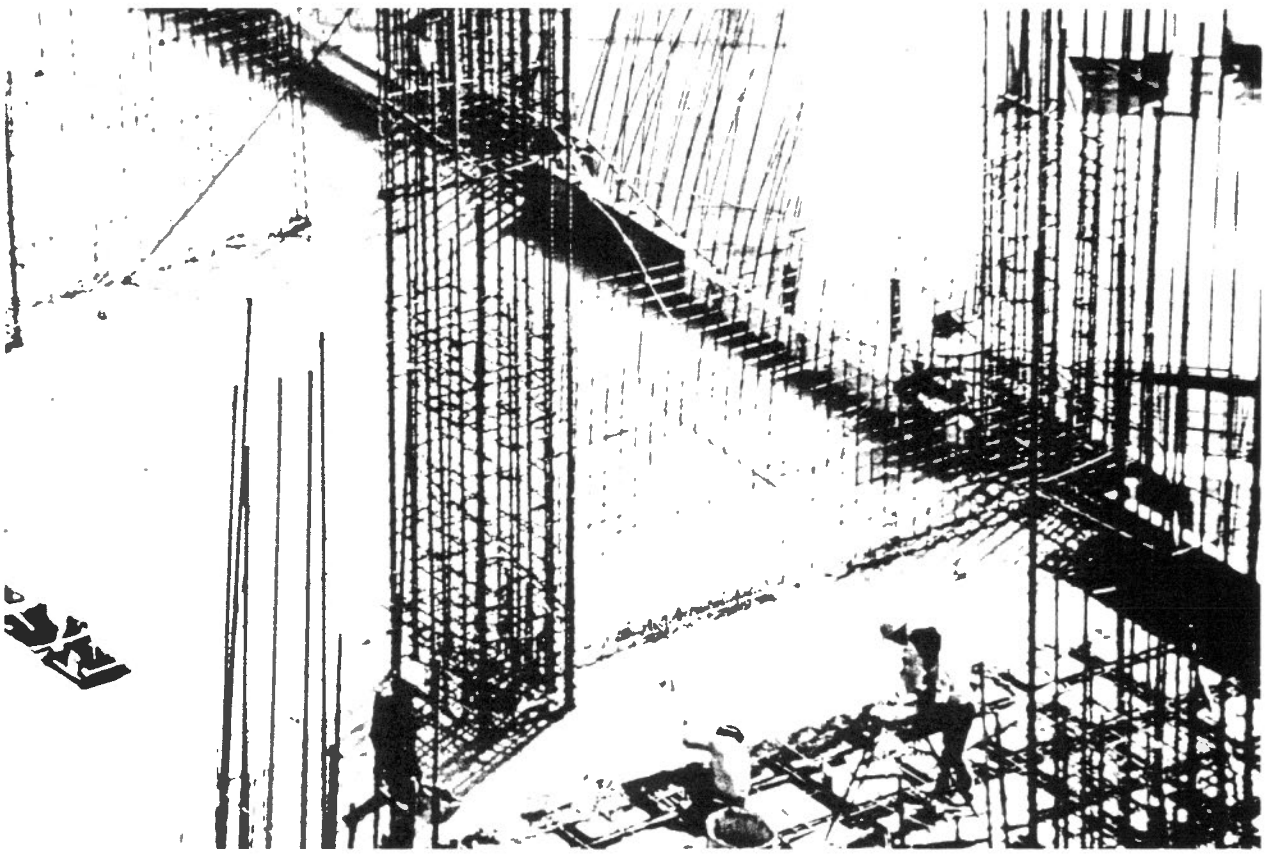


Fig. 5.7 Shear Wall Reinforcing Details for Nikko Hotel

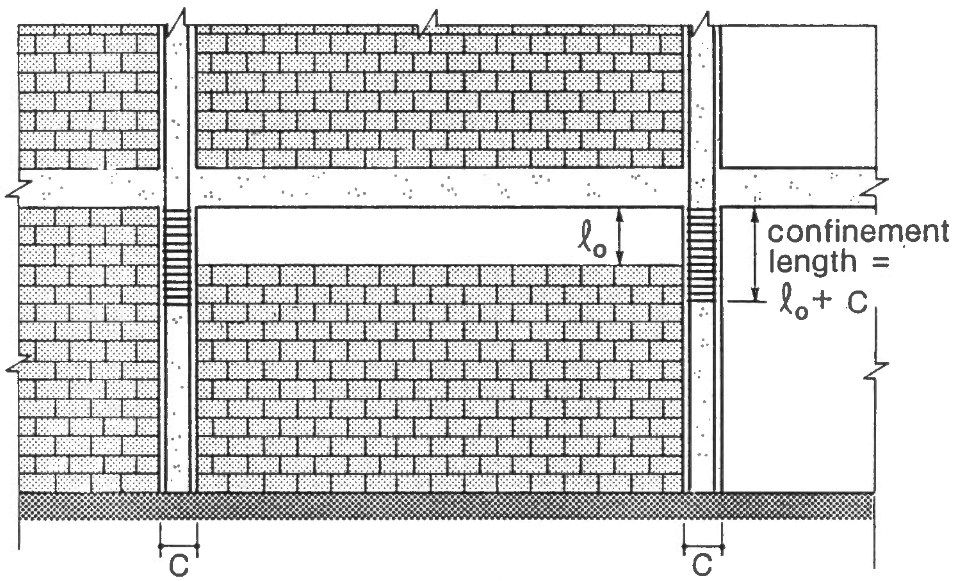


Fig. 5.8 Additional Confinement Requirement for Columns in Contact with Masonry Walls (Adapted from Ref. 5.2)

Empirical expressions are given for the axial load, moment and shear resistances of masonry walls. The walls are classifed as: (1) "diaphragm walls" (infilled walls), (2) "confined walls" (infilled walls meeting special reinforcement details) and (3) "walls with interior reinforcement". In order to qualify for this third classification the sum of the vertical and horizontal reinforcement ratios must be at least 0.002 with neither ratio being less then 0.0007.

5.4 Design of Steel Structures

This section summarizes some of the important aspects of design found in the provisions of the "Design and Construction of Metal Structures" (Ref. 5.7) and the recommendations given by Bazan and Meli (Ref. 5.2). The structural steel must conform to ASTM A36 or ASTMA7. If a ductility factor, Q equal to 6 is assumed in design then the following requirements must be satisfied:

(1) Type of Sections - Only compact sections may be used in order to achieve the desired levels of rotational capacity.

(2) Beams - The beams must be designed for shear and torsion using an increased factor of 1.4. At the ends of beams and over a distance of twice the beam depth lateral bracing shall be placed such that the spacing does not exceed 63.2 times the radius of gyration of the beam (for f_y = 2500 kg/cm^2).

(3) Columns - If the service (unfactored) axial load on the column exceeds 15 percent of the axial yield load then a load factor of 1.4 must be used instead of 1.1. The factored axial load must not exceed 0.6 times the axial yield load of the column.

(4) Beam Column Connections - The joint region must be able to transmit the yield force of beam flanges assuming that the beams have developed flexural hinges. Where necessary the web must be reinforced with

stiffeners.

5.5 References

5.1 Instituto de Ingenieria, UNAM, "Requisitos de Seguridad Y Servicio Para Las Estructuras, Titulo IV del Reglamento de Construcciones para el Distrito Federal" ("Safety and Serviceability Requirements for Structures - Regulations for Construction for the Federal District"), Series Del Instituto de Ingenieria No. 400, Julio, 1977, 159 pp.

5.2 Bazan, E. and Meli, R., "Manual de Diseno Sismico de Edificios - De acuerdo con le Reglamento de Construcciones para el Distrito Federal" ("Manual of Seismic Design of Buildings - According to the Construction Regulations for the Federal District"), Editorial Limusa, Mexico, primera edicion, 1985, 241 pp.

5.3 Associate Committee on the National Building Code, "National Building Code of Canada 1985, National Research Council of Canada, Ottawa, 1985, 454 pp.

5.4 Instituto de Ingenieria, UNAM, "Diseno y Construccion de Estructuras de Concreto, Normas Técnicas Complementaries del Reglamento de Construcciones para el Distritio Federal" ("Design and Construction of Concrete Structures - Complementary Technical Provisions to the Regulations for Construction in the Federal District"), series del Instituto de Ingenieria, No. 401, Julio, 1977, 307 pp.

5.5 ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71), "American Concrete Institute, Detroit, 1971.

5.6 Instituto de Ingenieria, UNAM, "Diseno y Construccion de Estructuras de Mamposteria - Normes Técnicas Complementarias del

Reglamento de Construcciones para el Distrito Federal" ("Design and Construction of Masonry Structures - Complementary Technical Provisions to the Regulations for Construction in the Federal District"), Series del Instituto de Ingenieria, No. 403, Julio, 1977, 98 pp.

5.7 Instituto de Ingenieria, UNAM, "Diseno y Construccion de Estructuras Metalicas - Normes Técnicas Complementarias del Reglamento de Construcciones para el Distrito Federal" ("Design and Construction of Metal Structures - Complementary Technical Provisions to the Regulations for Construction in the Federal District"), series del Instituto de Ingenieria, No. 402, Agosto, 1978, 206 pp.

CHAPTER 6

STRUCTURAL PERFORMANCE

In order to understand some of the important features of the foundation and structural design problems faced by engineers in Mexico City the important aspects of the design and construction of the Latin American Tower will first be examined.

6.1 The Latin American Tower

The Latin American Tower (La Torre Latinoamericana) stands as a symbol of earthquake-resistant design expertise in Mexico City. This structure which was completed in 1956 has survived the 1957, 1979 and 1985 earthquakes without structural damage in spite of its location in the soft soil region of Mexico City. The chief designer of the structure was Mr. Adolfo Zeevaert with Dr. Leonardo Zeevaert acting as the soils mechanics and foundations consultant and Dr. Nathan Newmark acting as a special consultant for the structure.

The main structural framing of this 44 storey building (see Fig. 6.1) consists of built-up steel columns with the beams connected to the columns with top and bottom riveted flange connections together with a bolted web connection (Ref. 6.1). The preliminary design was based on a 40 storey structure. After the steel fabrication had started the structure height was increased and a television tower was added to the top of the structure. In order to obtain a more realistic distribution of lateral shear force dynamic analyses were performed by N. Newmark. The results of the dynamic analyses indicated that the structure above the 28th floor level needed to be strengthened while the portion of the structure below this level had more than adequate strength (Ref. 6.2). The calculated periods of vibration for the structure were 3.66 sec, 1.54 sec, 0.98 sec

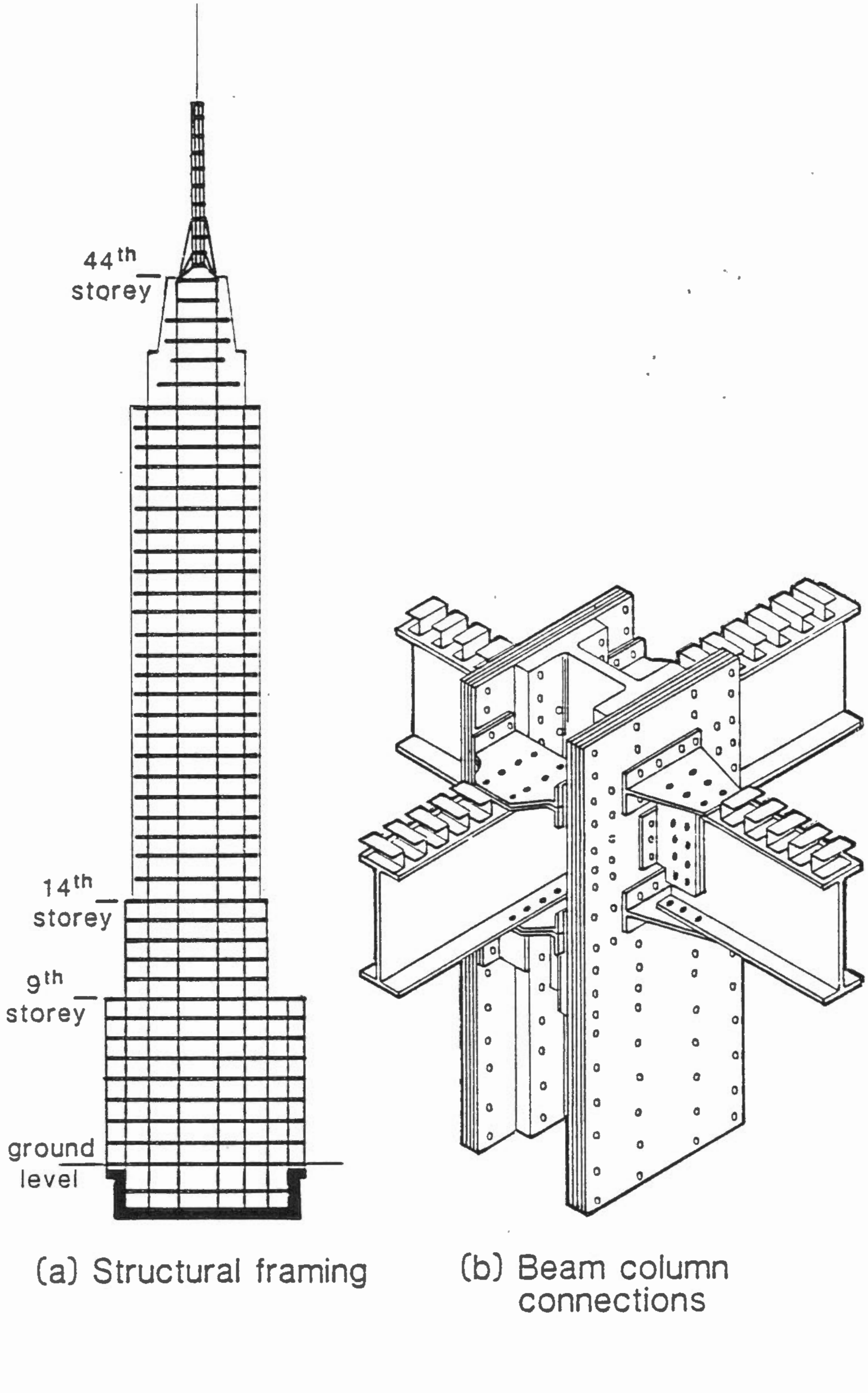


Fig. 6.1 The Latin American Tower (Adapted from Ref. 6.1)

and 0.71 sec for the first, second, third and fourth modes respectively. In order to limit the deflections of the structure the steel frame was stiffened and the concrete floor slabs were made composite with the steel floor beams by means of channel shear connectors welded to the top flange of the beams (see Fig. 6.1).

An important feature of the structure was the special "compensating foundation" (see Fig. 6.2) designed by L. Zeevaert (Ref. 6.1, 6.3 and 6.4). The impervious wooden sheet piling was first driven and then as the excavation proceeded water was pumped out and recharged into the sandy strata outside of the excavation. This technique attempts to preserve the original effective stress outside of the excavation thus minimizing soil settlement in this region (see Ref. 6.3). During the construction of the box foundation the soil was excavated from the cells of the box, one at a time, and then were filled with sand and gravel in order to compensate for the weight of the soil removed. As the building weight is increased the water level is allowed to rise gradually until the full building weight is applied and the water level is restored to its original level. The building is supported on the rigid reinforced concrete mat foundation together with 361 concrete piles bearing on the first sand layer. In order to compensate for the ground subsidence, which would result in settlement of the ground relative to the piles, the ground floor slab can be lowered to accommodate this differential movement.

An important feature of the structural response is the effect of rotation of the rigid foundation. It has been shown by L. Zeevaert (Ref. 6.4) that the effect of foundation rotation on the period of vibration can be approximated by multiplying the period of vibration, T_n, of the structural frame by $\sqrt{1 + T^2/T^2n}$ where T is the period of rotation of the foundation. Therefore the period of the structure-foundation response

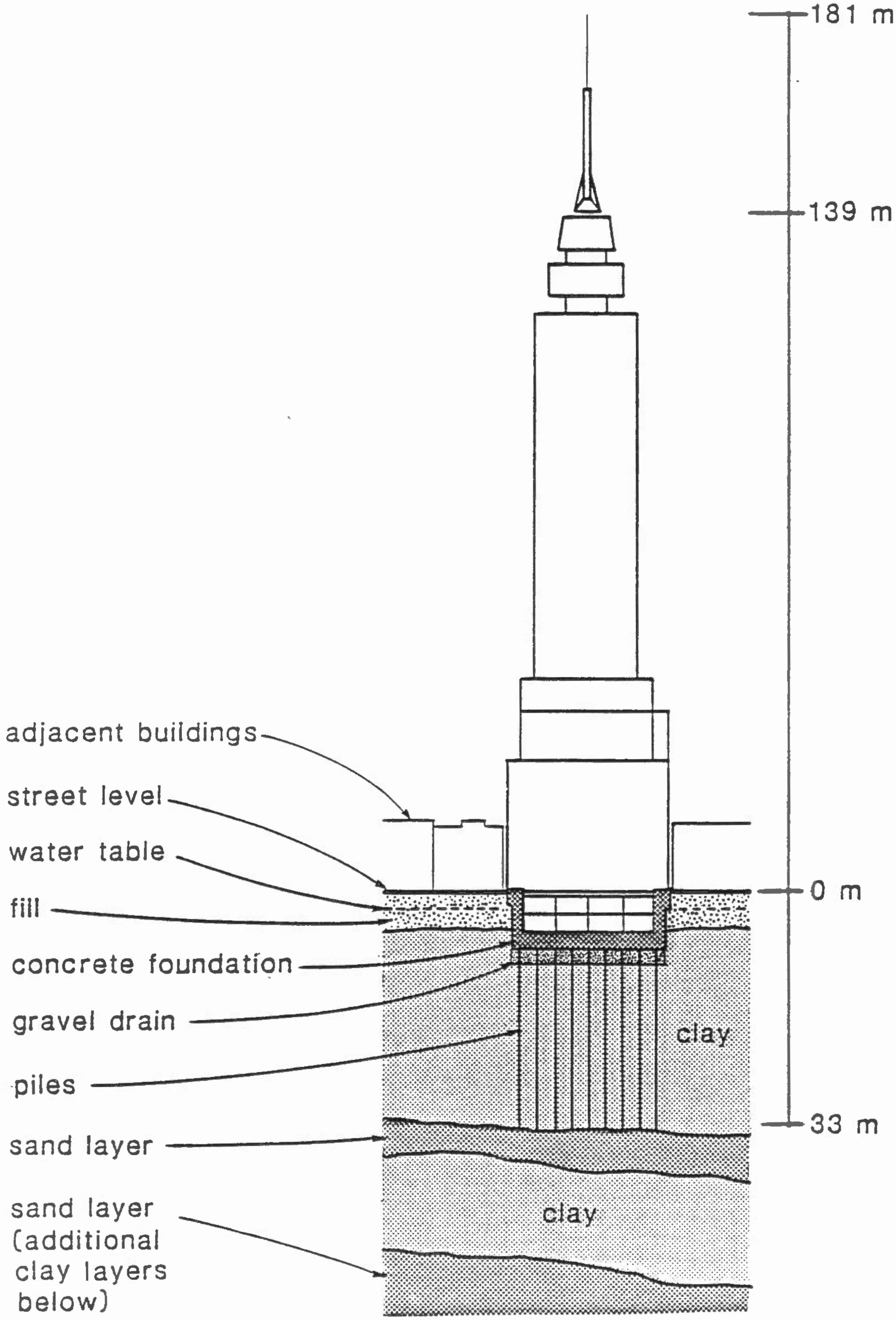


Fig. 6.2 Foundation Details for the Latin American Tower (Adapted from Ref. 6.1)

is increased due to this effect. This phenomenon will lead to an increase in the psuedo-acceleration if the period of the structure was slightly less than the peak of the response spectrum curve. The period of foundation rotation is about 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 respectively. Due to the significant participation of the second and third modes of vibration for this structure these increased periods resulted in a base shear increase of about 30 percent (Ref. 6.4). These calculations demonstrate the importance of the soil-structure interaction for structures founded on soft soils.

6.2 Structural Damage due to the 1957 Earthquake

The July 28, 1957 earthquake had a magnitude of about 7.5 and was felt in Mexico City at 2:41 a.m. and caused approximately 54 deaths.

The extent of the structural damage is described by Rosenblueth in Ref. 6.5. In total it is estimated that 1000 buildings were damaged. Of these damaged structures about 0.3 percent occurred on firm ground, 4 percent occurred in the transition zone and about 96 percent occurred in the soft soil zone (Ref. 6.5). The 10 structural collapses due to the earthquake and its aftershocks included a warehouse, two apartments, two office buildings, two theatres, part of a market place and two houses. In addition 12 buildings were condemned. All the buildings that either collapsed or were condemned were situated in the soft soil zone.

The main causes of failure given by Rosenblueth (Ref. 6.5) in decreasing order of importance are listed below:

(a) "Disregard of relative rigidities" - this includes improper account of masonry infills in the design of buildings. Many structures suffered damage due to large torsions caused by unsymmetrical layout of masonry walls.

(b) "Lack of aseismic design" - seismic design provisions were required for the first time in 1942. At this time buildings up to four storeys (or 16m in height) did not have to be designed for earthquake resistance.

(c) "Insufficiency of reinforcement away from supports" - bending or curtailment of flexural reinforcement a short distance from supports together with insufficient web reinforcement led to failure or severe damage of beams.

(d) "Pounding" - building regulations did not require separations between structures which led to pounding of the buildings during the earthquake.

(e) "Previous Differential Settlement" - differential settlement caused tilting of the bearing walls of a theatre resulting in reduced bearing support which probably contributed to the collapse during the earthquake.

(f) "Resonance" - the failure of the Angel of Liberty monument and a shell roof market structure, both inverted pendulum structures were probably caused by resonance.

(g) "Excessive Sway" - which was reported greater in steel framed buildings than in buildings with concrete frames.

(h) "Whip effect" - several failures in the upper storey of buildings thought to be due to sudden decrease in the rigidity at the top of some structures and due to poor joint framing of penthouse columns.

(i) "Foundation failures" - there was evidence of some tilting of structures on end bearing piles. There was no evidence of foundation failures although there were problems with some adjustable supports over piles. These included timber shims used to adjust the level of a building as settlement progresses. In some cases these details led to tilting due to lateral loads and needed adjustment after the earthquake.

(j) "Overturning" - not one case of overturning was observed although there were reported cases of failure of concrete columns due to the combined effects of shear, flexure and axial loads.

6.3 Surveys of Structural Damage due to the 1985 Earthquake

An excellent summary of the damage of structures in the City of Mexico is presented in a report (Ref. 6.6) prepared by the Institute of Engineering at the National Autonomous University of Mexico. Much of the material presented on the general aspects of damage are taken from this report. This report was published on Sept. 30, 1985, eleven days after the major earthquake. Although some of the data may be revised in the future, it was necessary to collect the information on damage as quickly as possible before the severely damaged buildings were torn down.

The peak acceleration on firm ground (free field measurement at the Autonomous University of Mexico) was 4% g with a slightly higher peak acceleration of 4.5% measured in the transition zone. The peak acceleration measured in the compressible soil zone (with a depth to the first sand layer of about 30 m) was 20% g having 11 cycles over 10% g at a period of 2 sec. The characteristics of the ground motion, that is, a period of two seconds, a large peak acceleration together with the large number of cycles of this nearly sinusoidal horizontal ground motion was the main reason for the degree and extent of damage suffered in the compressible soil region.

The distribution of structural damage in Mexico City is given in Fig. 4.9 and 4.10. Figure 6.3 compares the percentage of buildings of

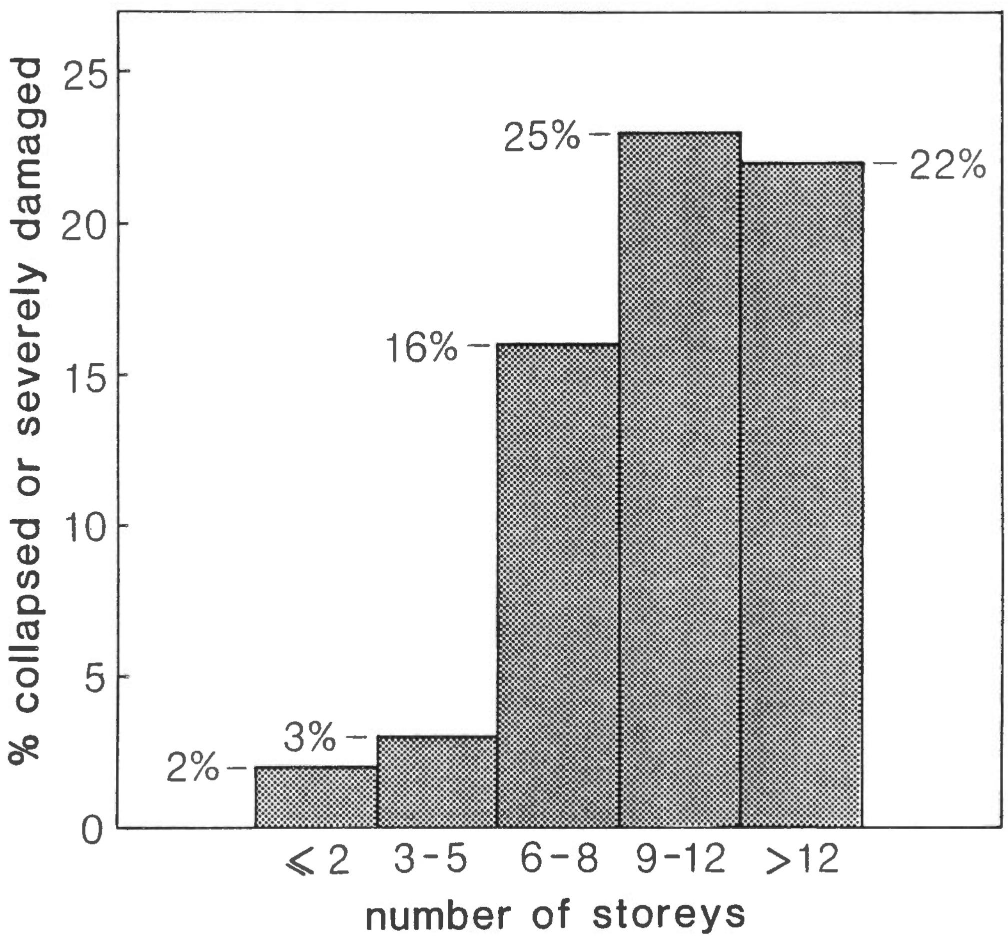


Fig. 6.3 Percentage of Buildings of Various Storey Heights that Collapsed or were Severely Damaged in the Zone of Damage

different heights in the damaged region that either collapsed or suffered severe damage. In all, 3 percent of the structures in this region were damaged with those buildings over 6 storeys in height being most affected.

Table 6.1 classifies the damage for different types of structures. In total, about 3500 buildings were damaged or suffered collapse. As can be seen, government buildings sustained significant damage with the entire facilities of some government departments being completely destroyed. The government is currently taking steps to reorganize and decentralize many of its agencies. A large number of schools were severely damaged (see Table 6.1) and many collapsed. Fortunately the earthquake occurred early in the morning (7:19 a.m.) before the schools opened, otherwise the death toll would have been much higher.

Table 6.2 gives some damage statistics for different types of structural framing and also gives information about the ages and number of storeys of structures that were severely damaged or suffered collapse.

The report (Ref. 6.6) published on September 30, 1985 by the Institute of Engineering at the University of Mexico explains that the main reason for the failure of a large number of buildings is the "exceptional intensity that the earthquake reached in one zone of the city". The ground motion transmitted by the firm ground had dominant periods which were amplified by the soft soil strata resulting in a large number of cycles of ground motion with a period of about 2 seconds.

A structure on soft soil subjected to severe ground motion will have its fundamental period lengthened due to both foundation rocking rotations and structural damage. An increased period for structures with a fundamental period less than 2 seconds could result in significant increases in inertial forces as the structure "climbs up" the spectrum with each successive cycle of response (see Fig. 6.4).

Type	No. in use	No. not in use	No. usable if repaired	Total No.
public buildings	39	68	27	134
schools	800	372	115	1287
medical facilities	28	10	15	53
cinemas and theaters	43	38	18	99
private buildings	652	449	643	1744
sports centres	7	3	1	11
city fixtures	0	1	0	1
market structures	99	8	5	112
overpass structures	1	1	1	3
cemetery	0	0	1	1
Totals	1669	950	826	3445

Table 6.1 Types of Structures Affected in the Damaged Region (Statistics as of October 2, 1985, courtesy of Mexican Government).

Type of Structure	Damage	Year of Construction			No. of Storeys				Totals
		<1957	57-76	>1975	≤5	6-10	11-15	>15	- collapsed - severe damage
Concrete Frames	- collapsed - severe damage	35 9	59 19	13 7	36 8	62 23	9 4	0 1	107 36
Steel Frames	- collapsed - severe damage	5 1	4 0	0 0	4 0	2 0	1 1	2 0	9 1
Flat Plates	- collapsed - severe damage	3 5	35 20	12 11	23 9	23 18	4 8	0 0	50 35
Masonry	- collapsed - severe damage	7 2	4 3	1 0	10 4	2 1	0 0	0 0	12 5
Others	- collapsed - severe damage	0 0	1 4	1 2	1 6	1 2	0 0	0 0	2 8
Totals	- collapsed and severly damaged	69	149	47	101	134	27	3	265

Table 6.2 Damage Classified According to Type of Structural Framing, Year of Construction and Number of Storeys (taken from Ref. 6.6).

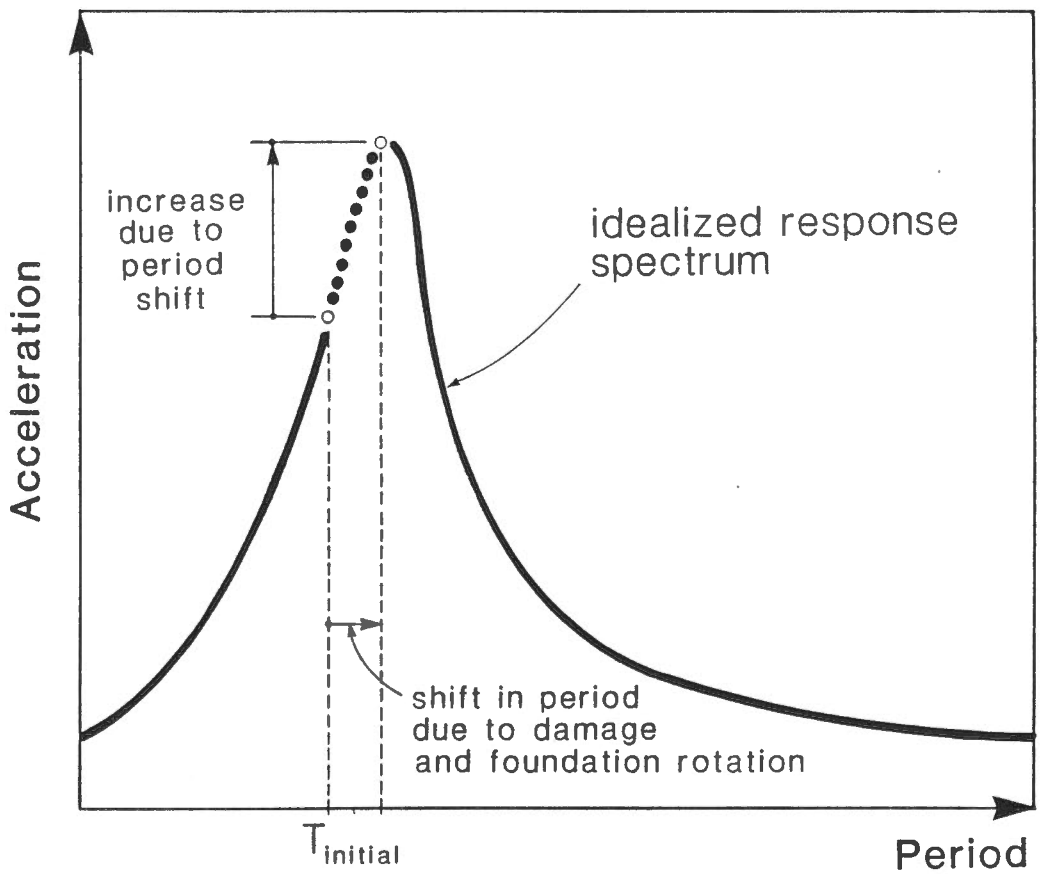


Fig. 6.4 Effect of Increasing Period on Inertial Forces

The Institute of Engineering Report (Ref. 6.6) lists the following types of structural failures:

(1) Column failures - Most of the collapses of frame structures were due to column failures. Reference 6.6 identifies "deterioration of the column concrete caused by the repetition of a large number of lateral load cycles that exceeded the resistance in axial load and bending or in shear" as the main cause of the column failures. The report also mentions that in several cases the detailing of the transverse steel and the excessive spacing of the longitudinal bars resulted in buckling of the column steel and poor concrete confinement.

(2) Effects of Masonry Walls - According to the Report masonry walls that were designed and reinforced to function as lateral load-resisting elements performed very well provided they were placed symmetrically in the plan of the building and were placed such that they were uniform over the height of the building. Many buildings failed due to the unsymmetrical layout of masonry walls. There were many examples of structural collapse caused by eliminating masonry walls in the first storey thus creating a "soft storey" effect. There were cases reported (Ref. 6.6) in which asymmetry was produced when some masonry walls failed due to weak material or due to poor anchorage. These failures led to subsequent column failures.

(3) Damage from Previous Earthquakes - In some cases of collapse the structures had suffered damage in the masonry walls from a previous earthquake and this damage, which was not

adequate`v repaired, contributed to the collapse.

(4) Short Column Failures - The presence of masonry walls or spandrel beams, which reduce the length of columns in one direction of the structure, caused shear failures in these short columns.

(5) Impact between Adjacent Buildings - Inadequate separation between buildings resulted in "pounding" of the buildings during the earthquake. Pounding is believed to be the cause of many of the numerous cases of collapses in the top stories of buildings.

(6) Excessive Overloads - According to Reference 6.6 excessive loads caused by storage was a contributing factor in several collapses.

(7) P-Delta Effect - Due to the large displacements experienced by many structures it is thought that the increased moments caused by these lateral displacements played a role in many collapses.

(8) Punching Shear Failures - At least four cases are reported in which punching shear failures have occurred in waffle slab construction due to the combined effects of shear and moments transferred through the slab-column connections.

(9) Foundation Failures - Reference 6.6 reports that although foundation failures were rare there were some cases of this type of failure in some slender buildings with friction piles or contact foundations that had large overturning moments. It is difficult to determine the role of previous foundation settlements on the behaviour of the structures during the earthquake.

6.4 Examples of Structural Damage in Mexico City

Photographs taken by the site-visit team together with brief descriptions of the structural damage are given in the following pages.

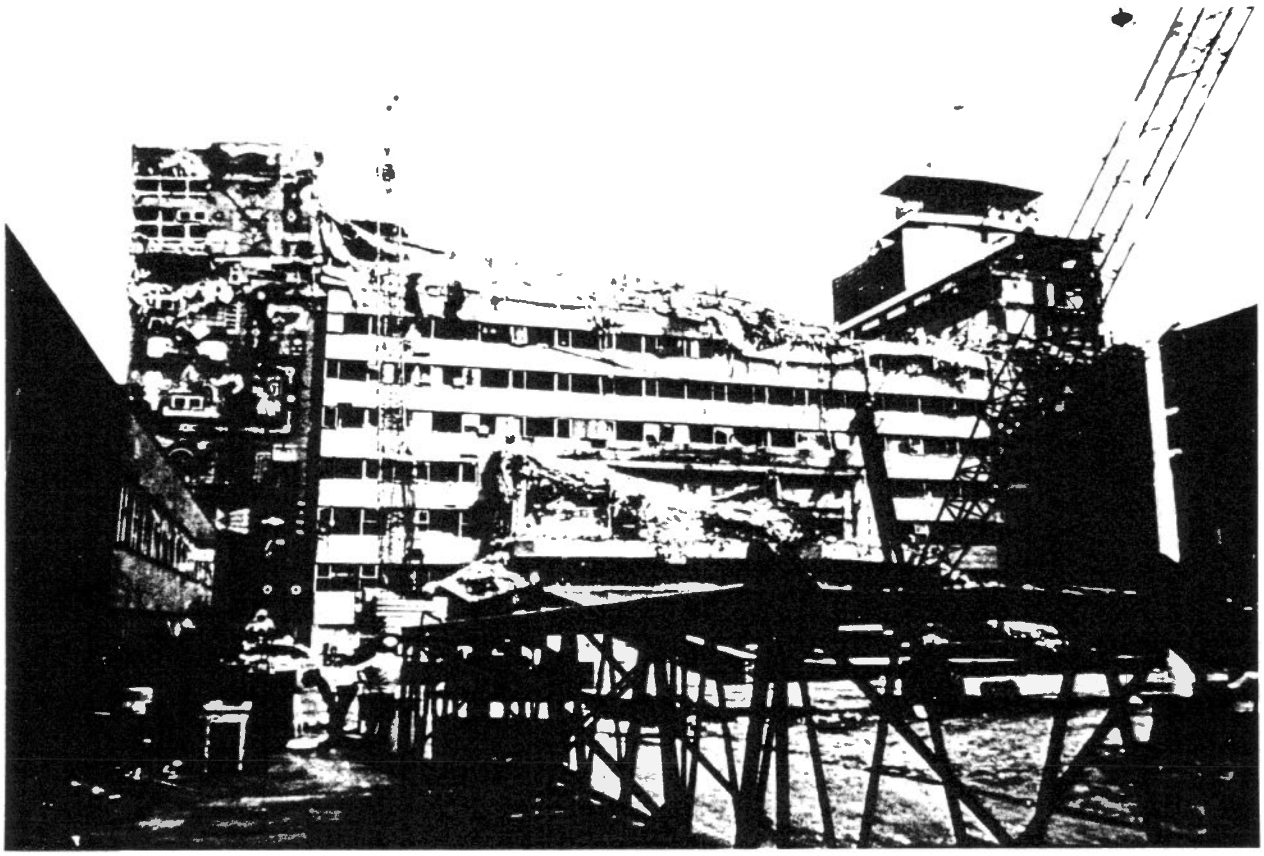


Fig. 6.5a SCT Communications Building

Due to the presence of several adjoining wings this structure is irregular in plan. This is the location of the seismograph station that recorded the largest acceleration (about 0.20 g). Close-ups of the collapsed top three storeys of this ten storey structure are shown in Fig. 6.5b and 6.5c. The period of ground motion, the large acceleration, the torsional eccentricities and possible pounding of this central wing with adjoining structures probably all contributed to the collapse.

The loss of life of the skilled telecommunications personnel and the damage to telecommunications equipment including transmission towers resulted in the loss of long distance telecommunication for several weeks.

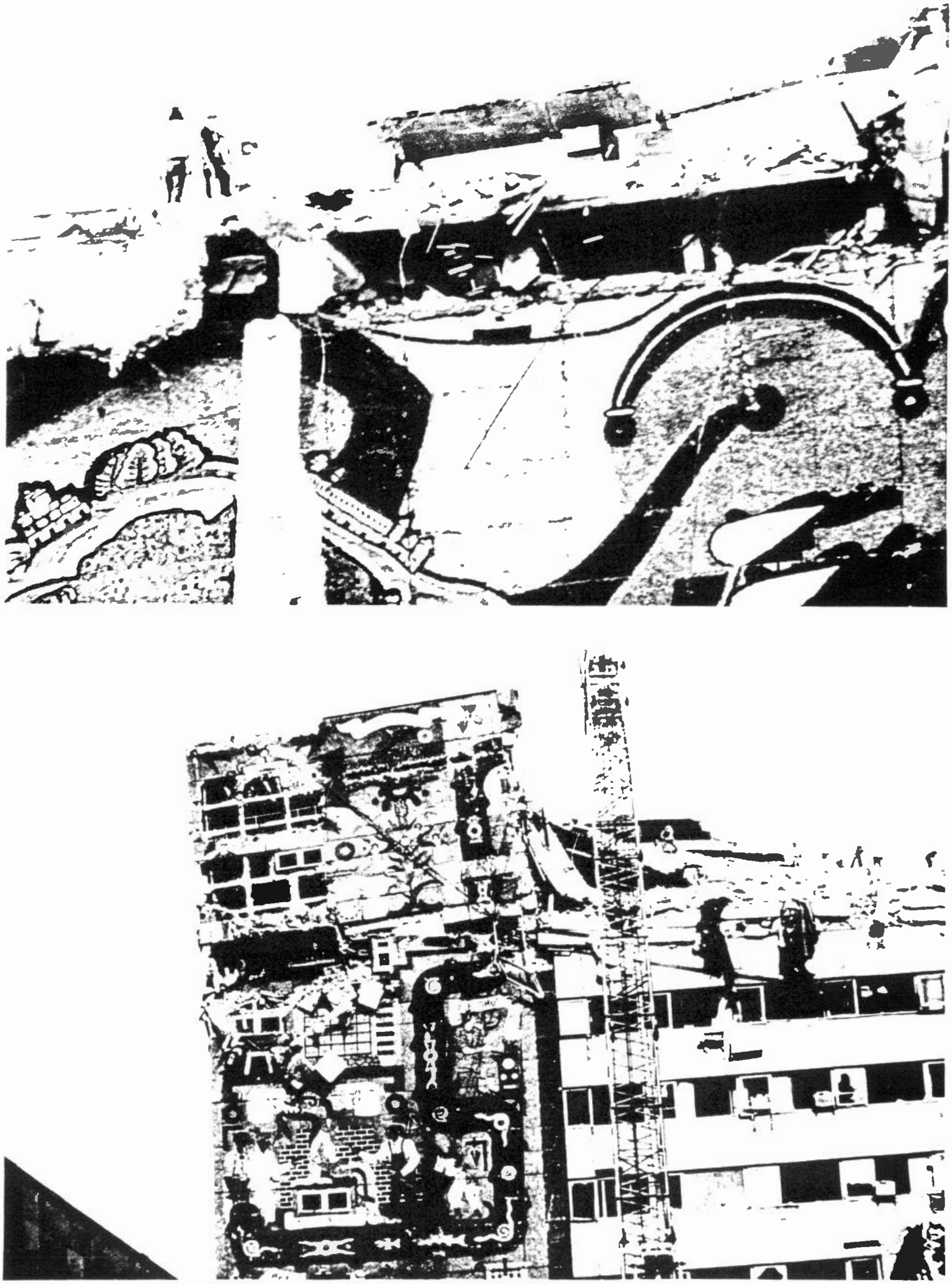


Fig. 6.5b and 6.5c SCT Communications Building

Fig. 6.6a Triangular-Shaped Reinforced Concrete Building

Due to the large number of diagonal streets there are many irregular building lots and therefore many structures with unusual shapes in plan. This triangular-shaped building is a reinforced concrete frame with infilled masonry walls. The mansonry infilled shear wall is eccentrically located. Fig. 6.6b shows the failure of an infilled wall framing into a corner column which has lost its vertical load carrying capacity together with another corner column which has completely lost its core concrete. This structure is located across from the SCT building. The severe ground motions togther with the large torsional eccentricities and poor confinement of the columns probably led to the severe damage.

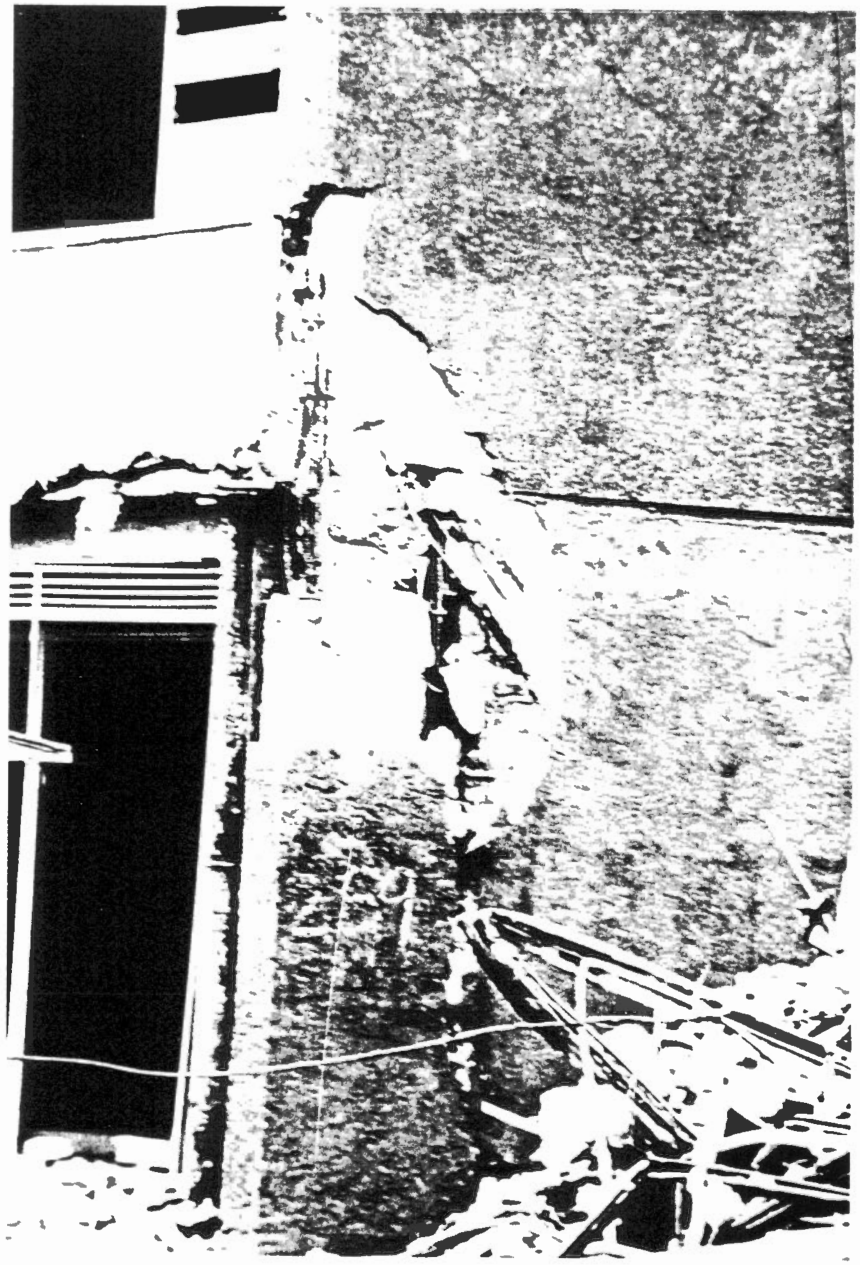


Fig. 6.6b Triangular-Shaped Reinforced Concrete Building

This close-up view shows the details of failure of a corner column and the reinforced concrete diagonal brace member in the infilled masonry wall.

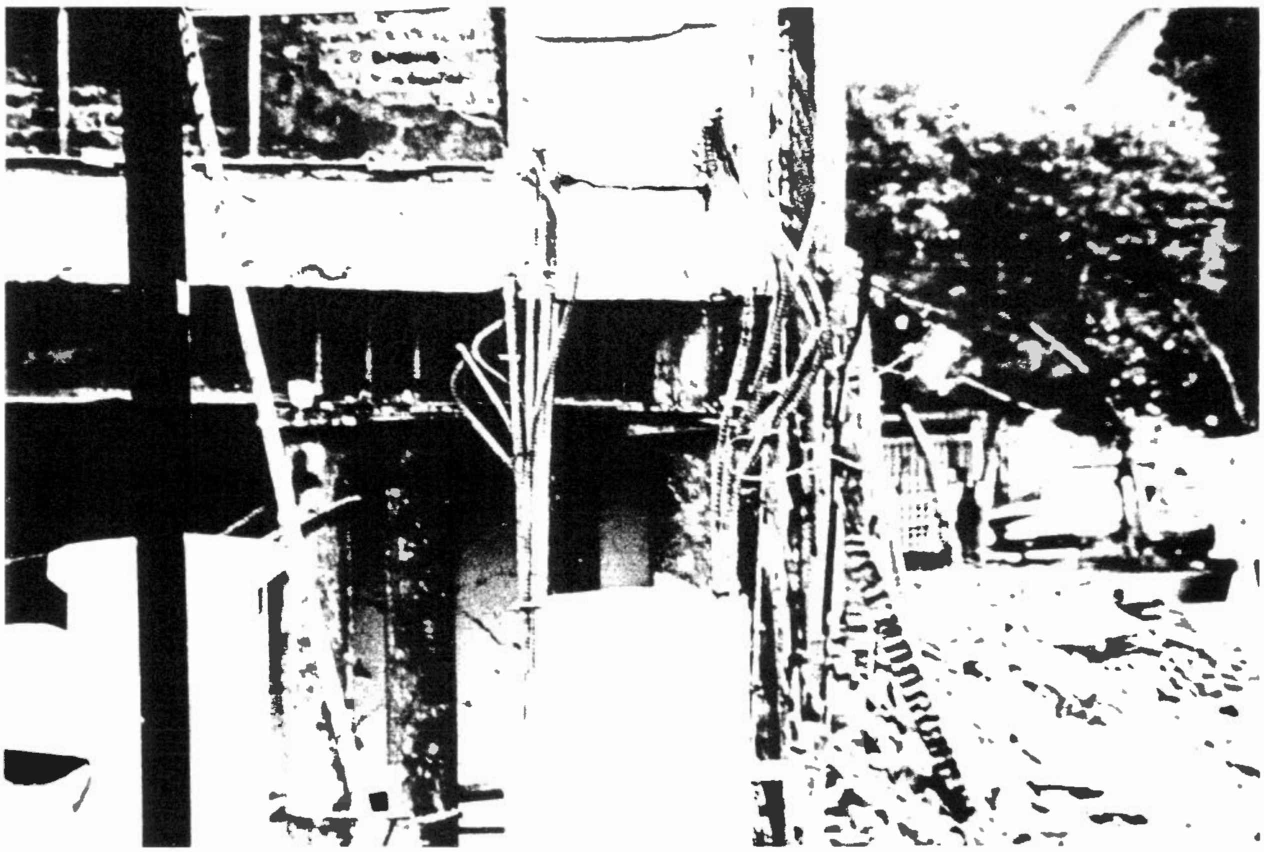


Fig. 6.6c Close-up of Corner Column Failure Showing Large Column Tie Spacing and Steel Pipe Column Temporary Supports

Fig. 6.7a Collapse of 21 Storey Steel Structure

Shown in the foreground is the collapsed Pino-Suarez government office tower (see also Fig. 6.7b). Two almost identical structures shown in the background are still standing but are severely damaged. The structures rise 20 storeys above a one storey plaza level and consist of built-up tubular columns with tubular truss floor systems having K-bracing as shown in Fig. 6.7c and 6.7d.

The structure next to the collapsed building had a permanent deformation in one direction of 1.20 m. The third structure showed signs of buckling on one of the lower storey corner columns as shown in Fig. 6.7d and 6.7e.

The 21 storey tower that collapsed fell onto a 14 storey steel structure causing it to collapse.

Fig. 6.7b Failure of Steel Building One Storey Above Plaza Level

Fig. 6.7c Structural Framing of Severely Damaged 21 Storey Steel Building

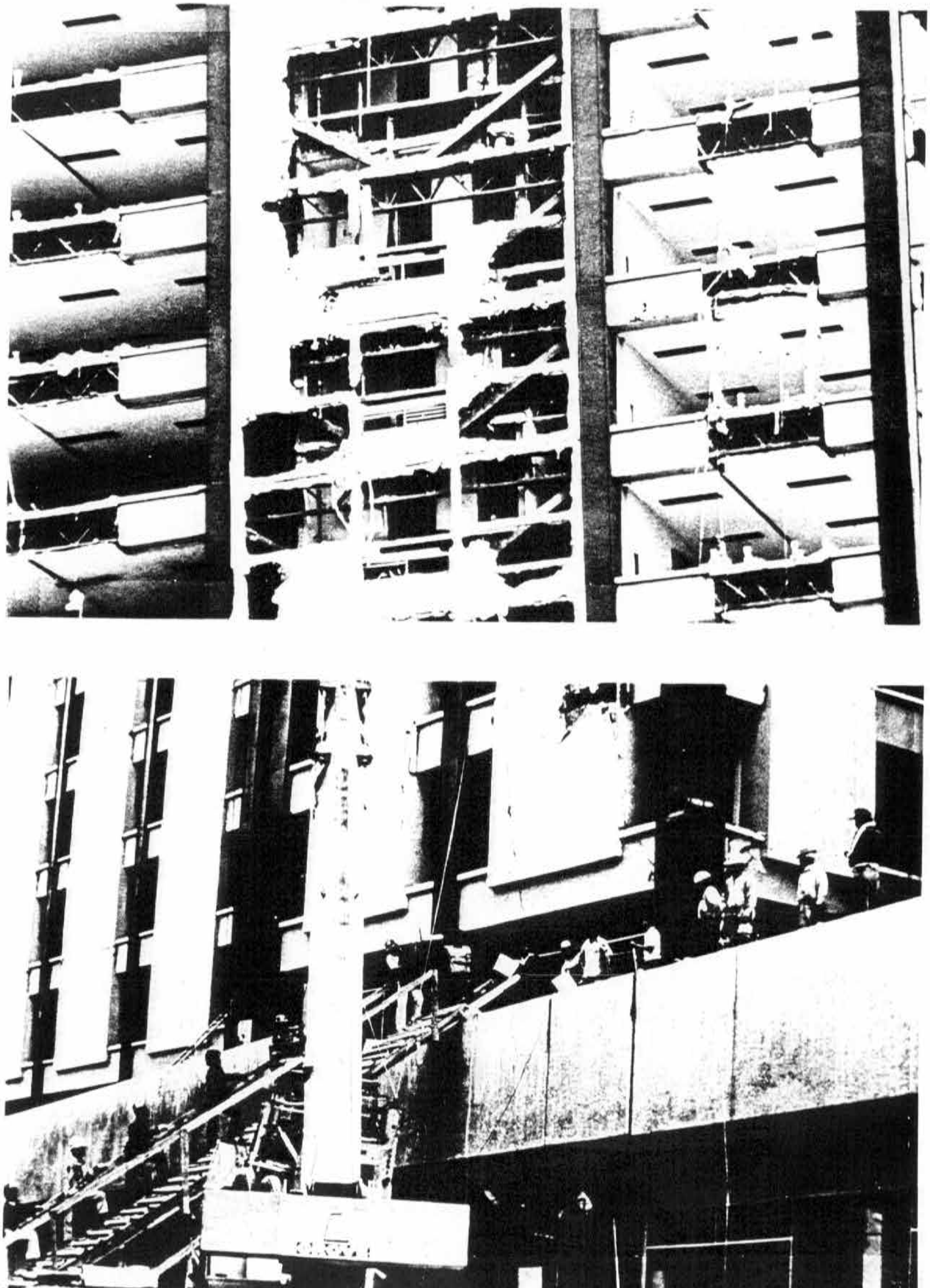


Fig. 6.7d and 6.7e Framing System and Buckled Corner Column

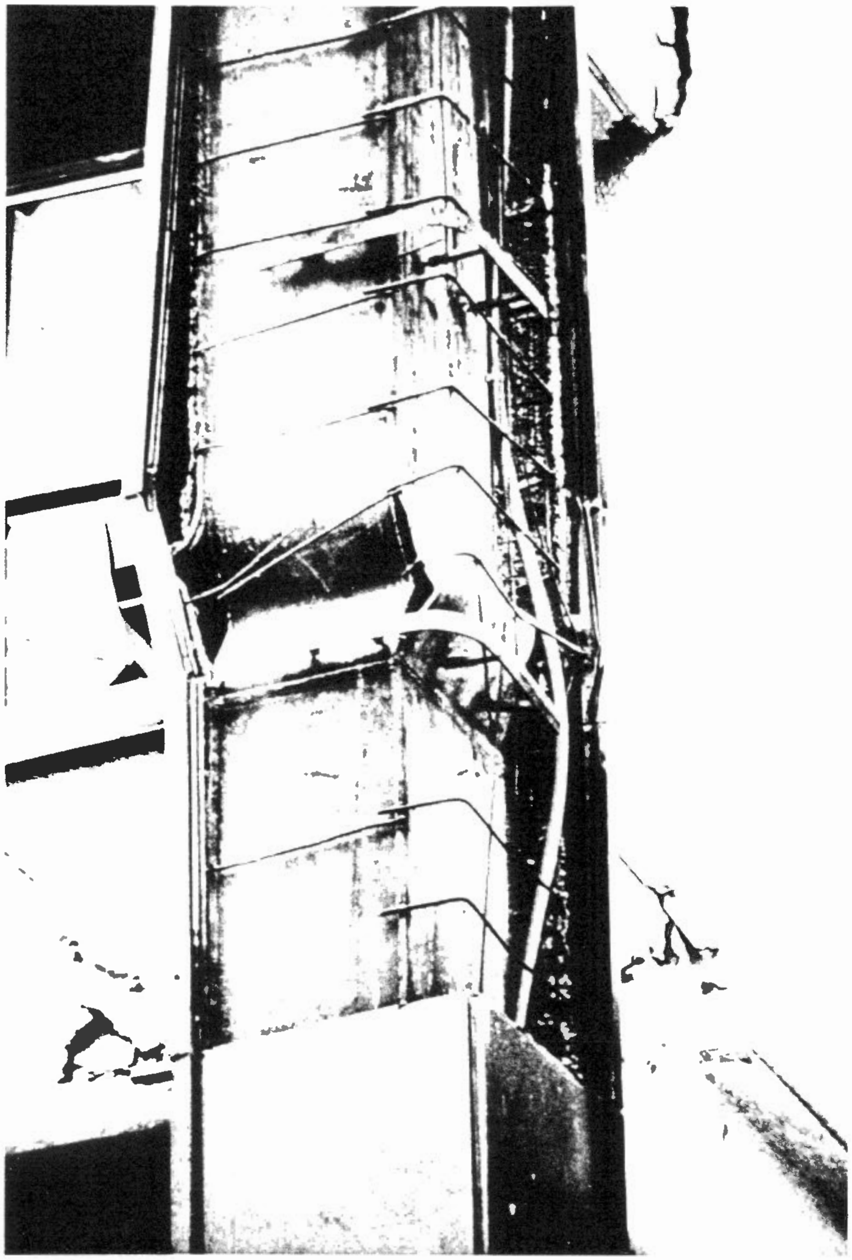


Fig. 6.7f Close-Up of Corner Column in 21 Storey Steel Office Tower

The tubular columns were fabricated from four 20 mm thick plates which were connected by 16 mm fillet welds along the outside corners only of the overlapping plates. The overall dimensions of the columns are 400 mm x 600 mm. As can be seen the welds failed and local buckling occurred in the column.

Fig. 6.8a Damaged 13 Storey Concrete Frame-Wall Structure

This structure which is beside the 21 storey steel structure that collapsed suffered damage to the columns. The structural system consists of waffle slabs, reinforced concrete frames and infilled masonry walls (see Fig. 6.8b). The relatively deep spandrel beams resulted in short external columns which were severely damaged in shear. A close-up of the "600 mm x 1200 mm columns is given in Fig. 6.8c. The mansonry walls did not appear to be severely damaged.

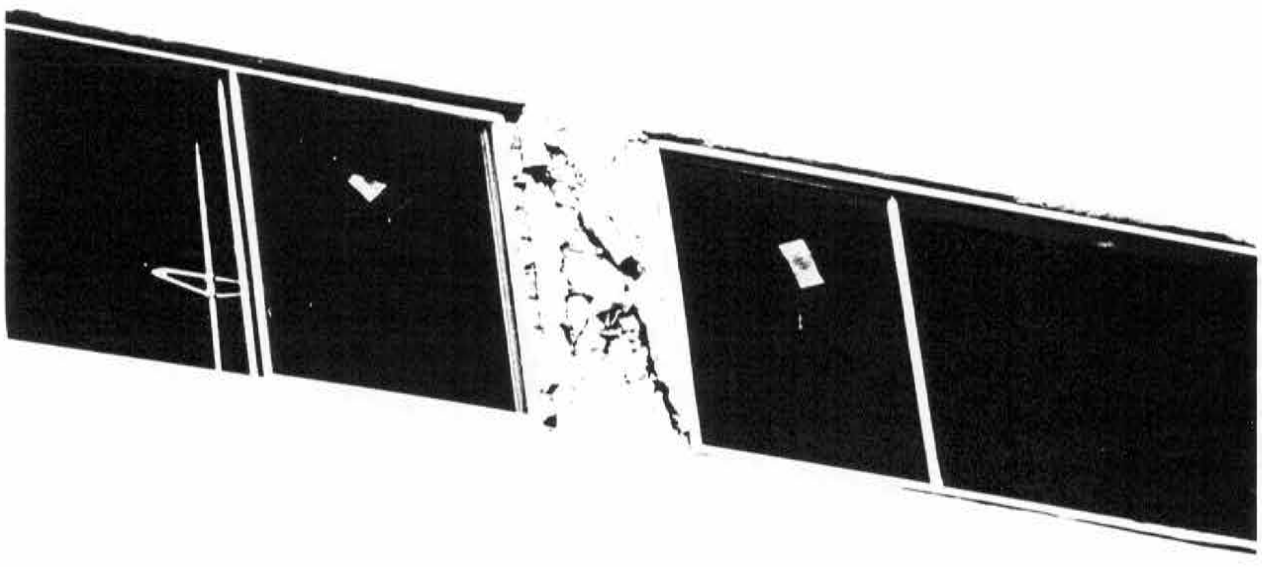


Fig. 6.8b and 6.8c Column Shear Failures in Reinforced Concrete Structure

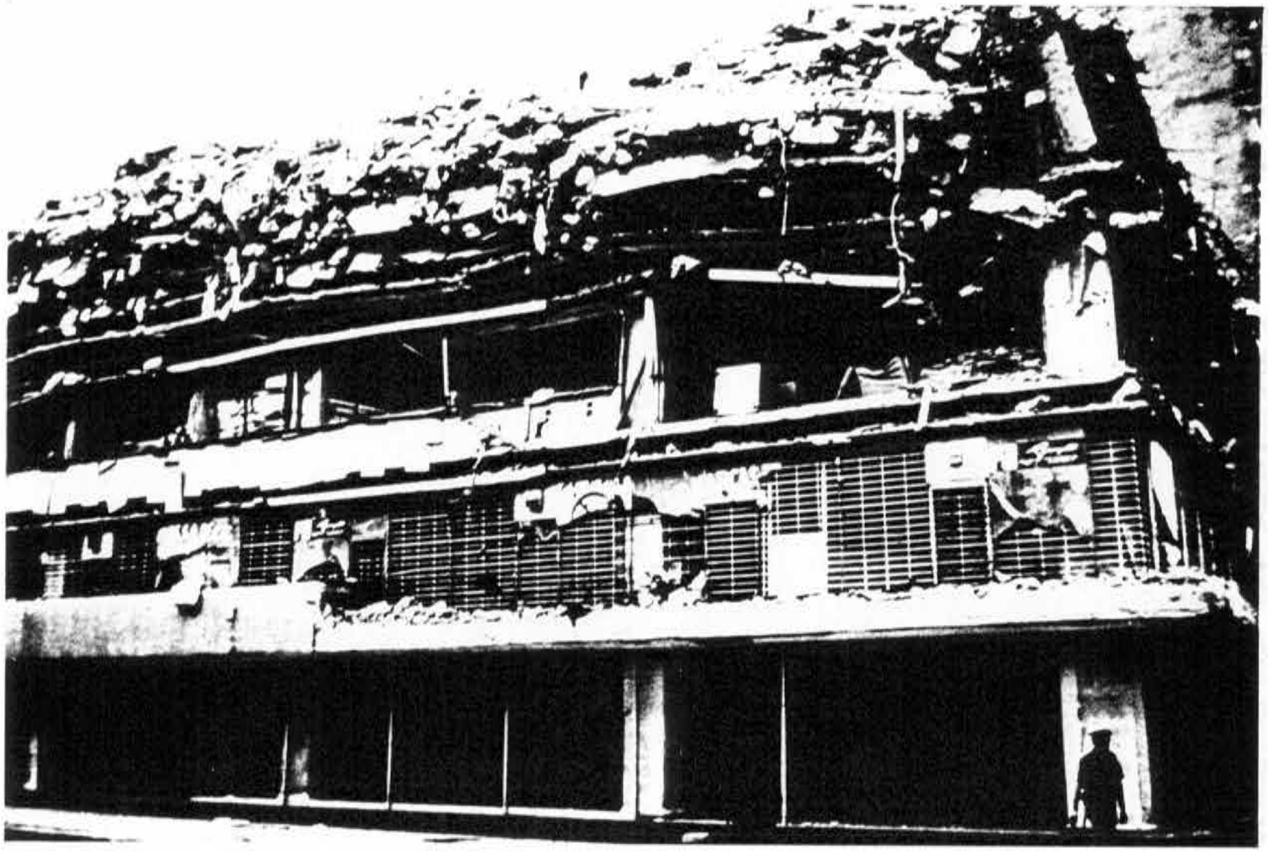


Fig. 6.9 Collapse of Top Storeys of Reinforced Concrete Flat Plate Structure

The upper storey failure of this reinforced concrete flat plate structure shows signs of columns punching through the floor slabs.

Fig. 6.10a The Undamaged 44 Storey, Steel Latin American Tower

The Latin American Tower did not show any signs of structural damage. This structure is located in a region of heavy damage. The photographs in Fig. 6.10 b and c are taken from the observation platform on top of the Latin American Tower.

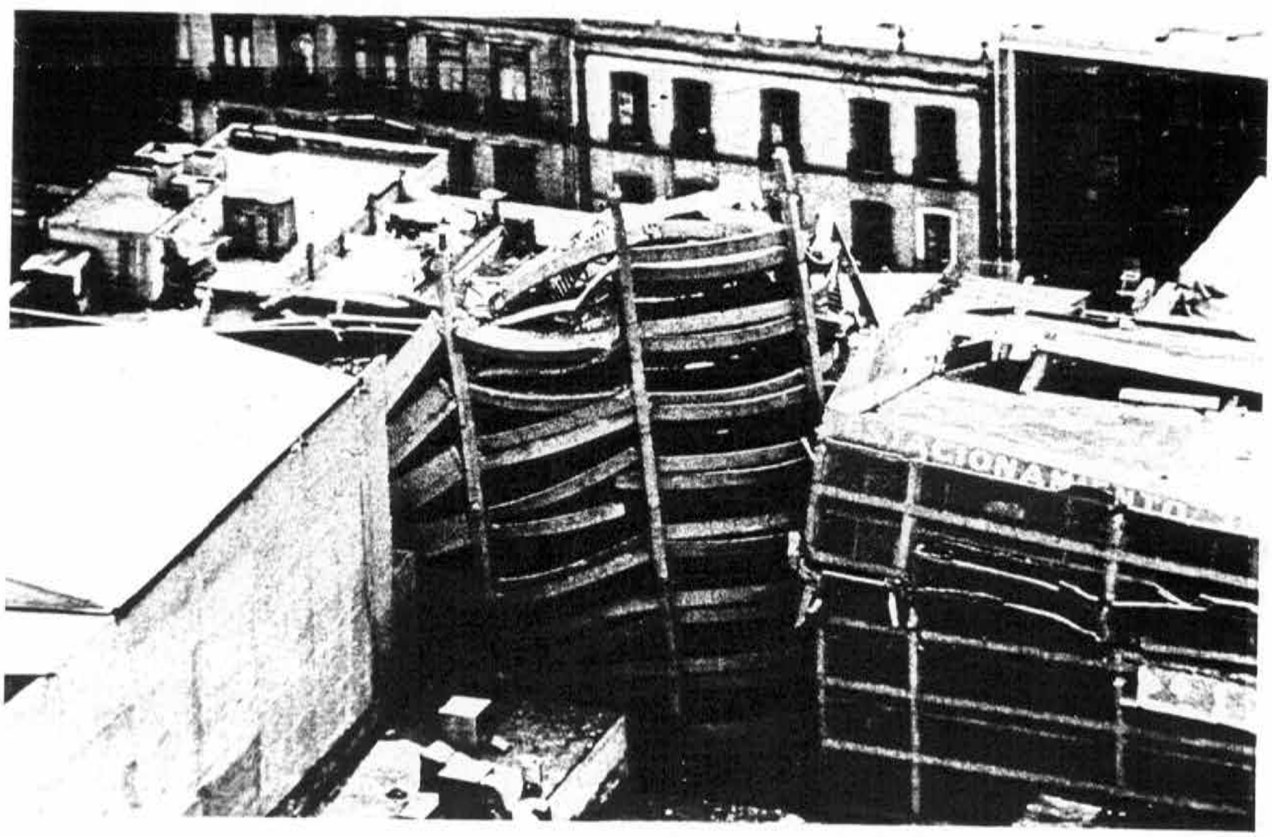


Fig. 6.10b and 6.10c Top Storey Collapse and Collapse of Precast Concrete Parking Structure

Fig. 6.11 Regis Hotel Collapse

The collapse of the Regis Hotel also resulted in ruptured propane gas lines and a fire which hampered rescue operations.

Fig. 6.12 Undamaged Triangular Building

This relatively new 28 storey structure has steel columns and a centrally located reinforced concrete elevator core. This structure is located close to the collapsed Regis Hotel and is in the region of severe damage. Not a single pane of glass in the glass curtain wall was broken.

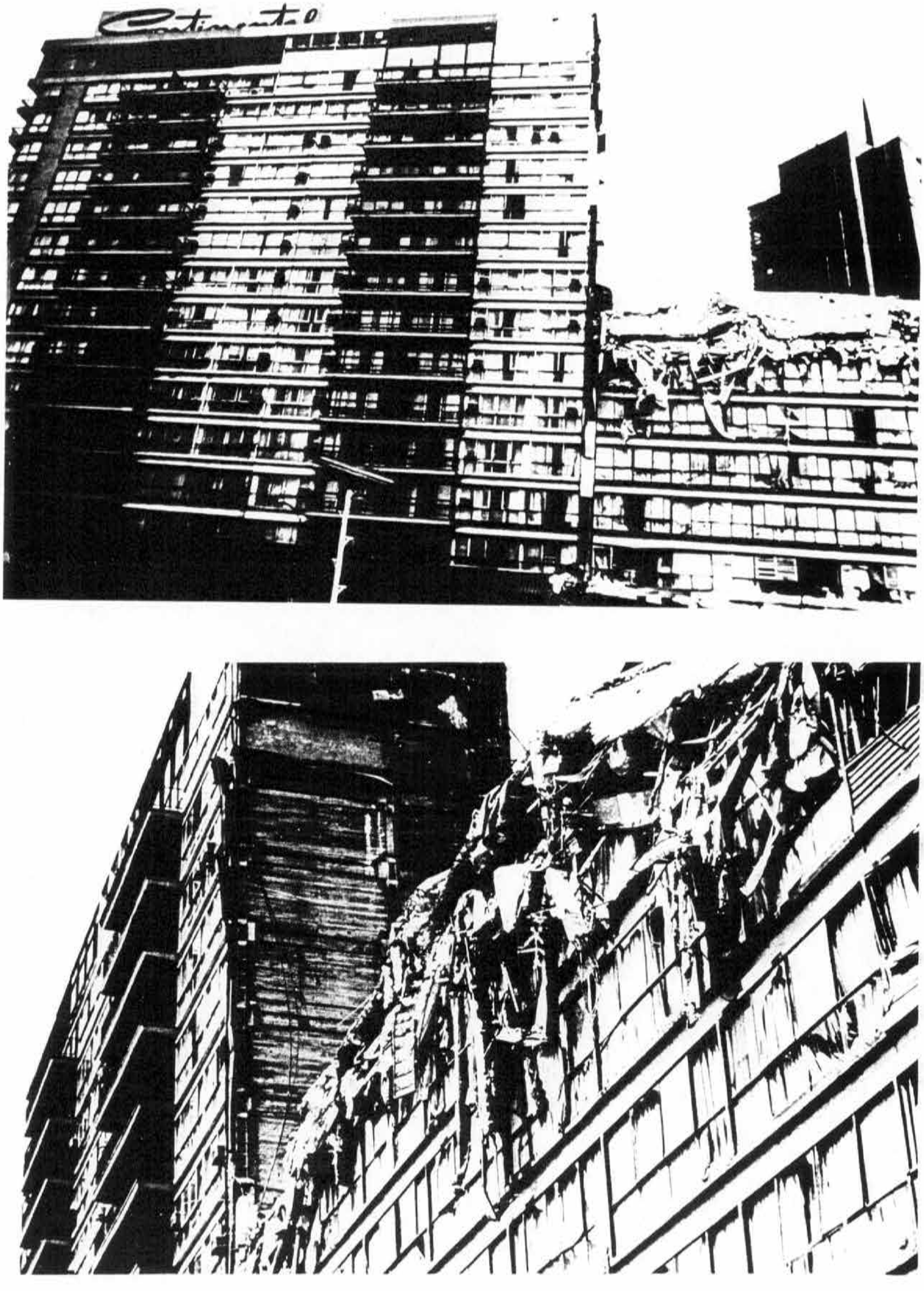


Fig. 6.13a and 6.13b Collapse of Top Six Storeys of Continental Hotel

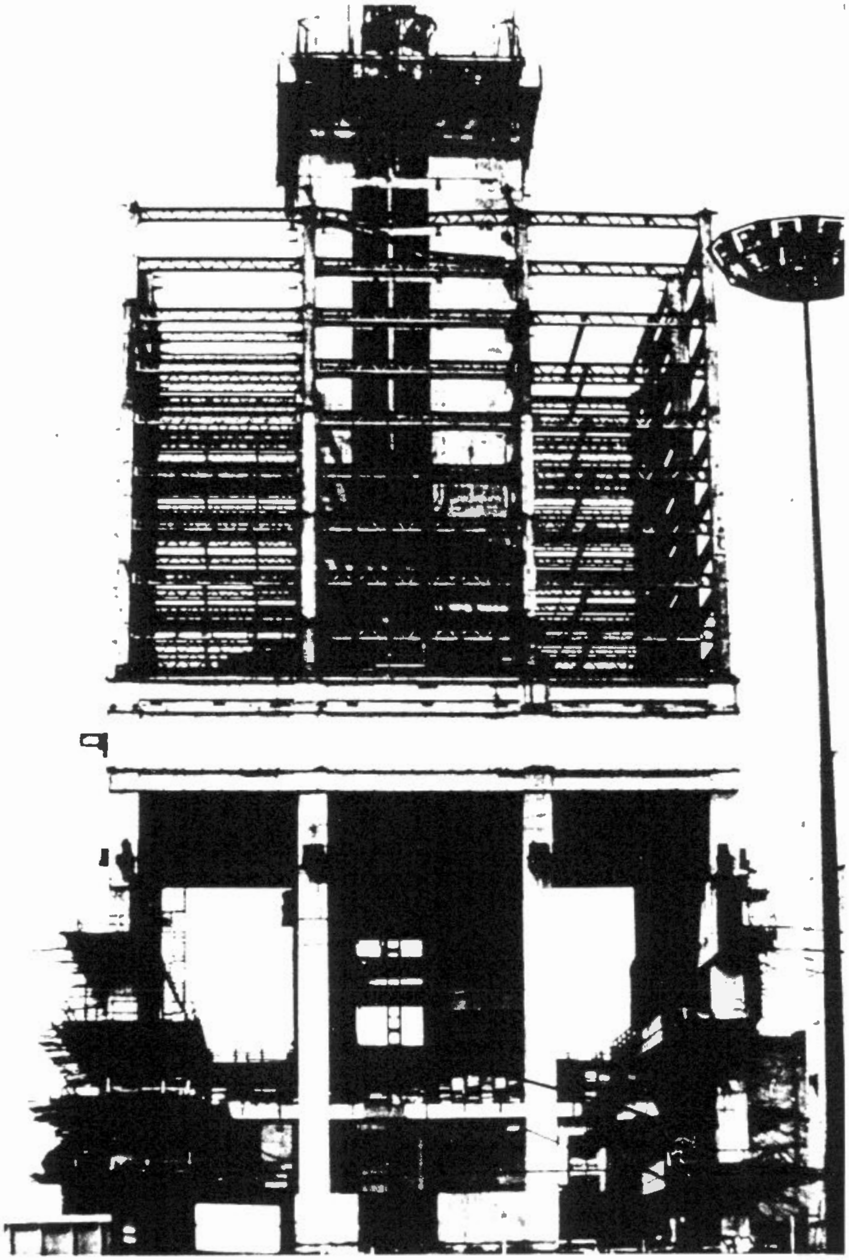


Fig. 6.14a Partially Completed Intercontinental Hotel

This structure was intended to be the largest Holiday Inn in Latin America. Construction was stopped due to financial problems after the peso was devaluated. The construction crane on top of the structure collapsed. Some of the steel truss framing members near the top of the structure were damaged. The shear wall coupling beams in the lower part of the structure were severely damaged in shear as shown in Fig. 6.14b.

Fig. 6.14b Damage to Shear Wall Coupling Beams of Incomplete Intercontinental Hotel

Fig. 6.15a Foundation Failure of Eight Storey Building

This relatively slender eight storey reinforced concrete apartment building completely overturned due to failure of the foundation. The photograph shows only the remains of the structure, the first storey and the 2.5 m deep foundation box. The foundation box has undergone a rotation of about 45 degrees and has resulted in pull-out of the 500 mm diameter concrete piles. Each pile had large tension cracks through the thickness as shown in Fig. 6.15 b and c.

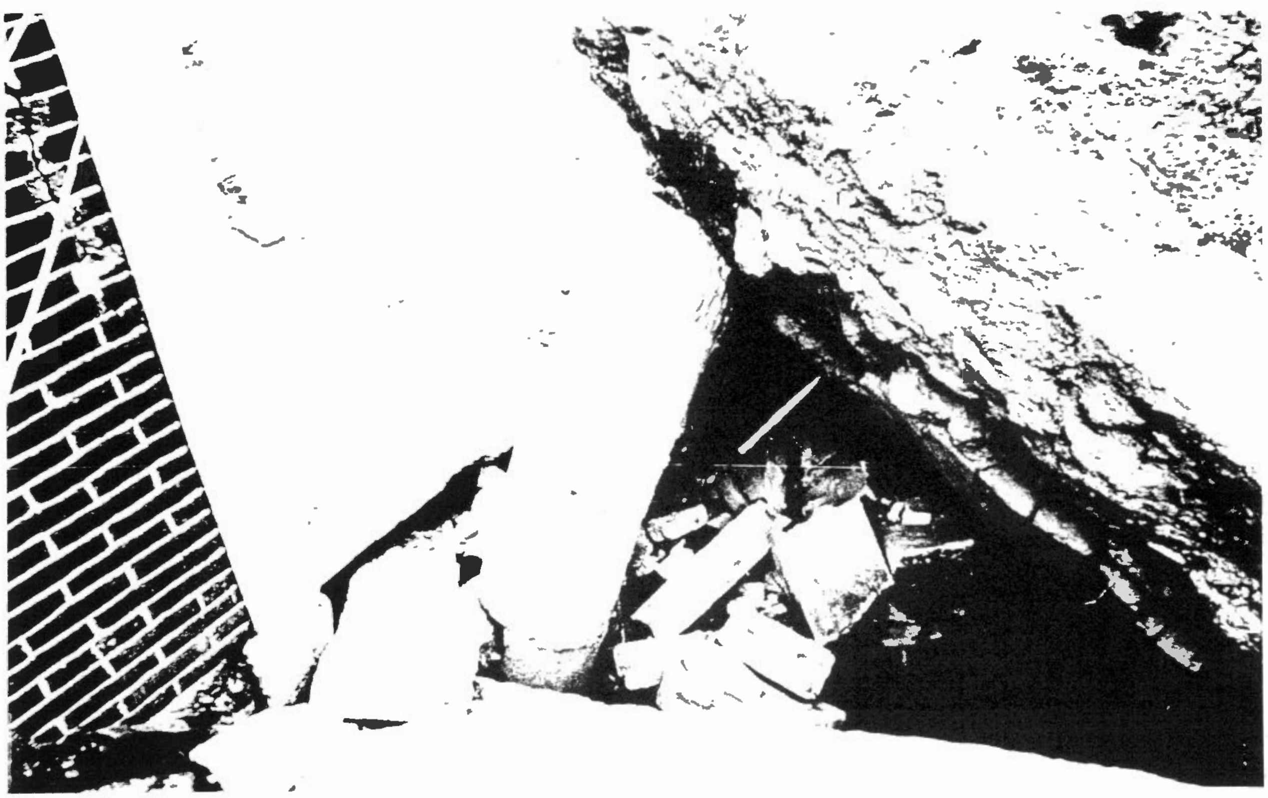


Fig. 6.15b and 6.15c Rotation of Foundation Mat and Pull-out of Piles

Fig. 6.16a Flat Plate School Structure

This flat plate structure was constructed with masonry blocks embedded in the concrete floor. It is believed that punching shear together with moment transfer contributed to the collapse.

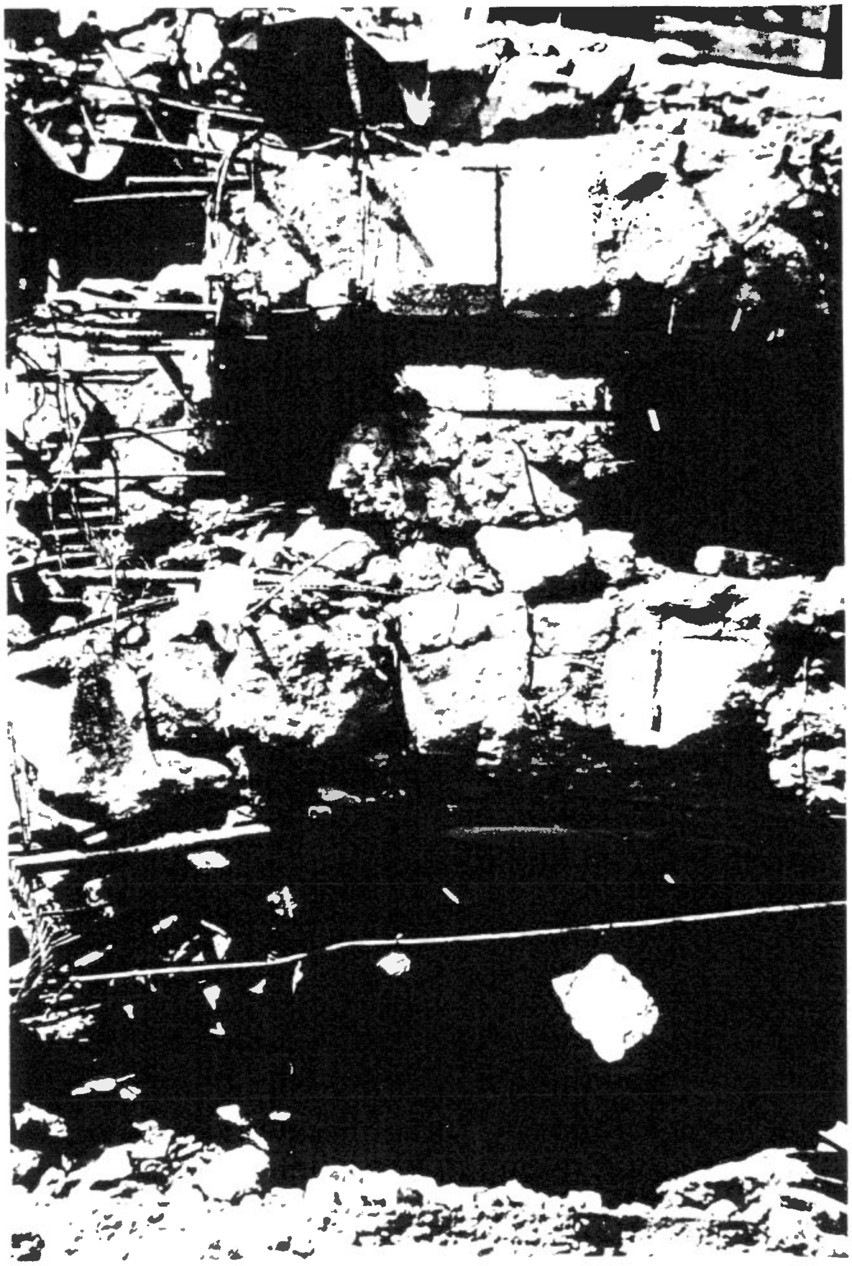


Fig. 6.16b Close-up of Punching Shear Failure in School Structure

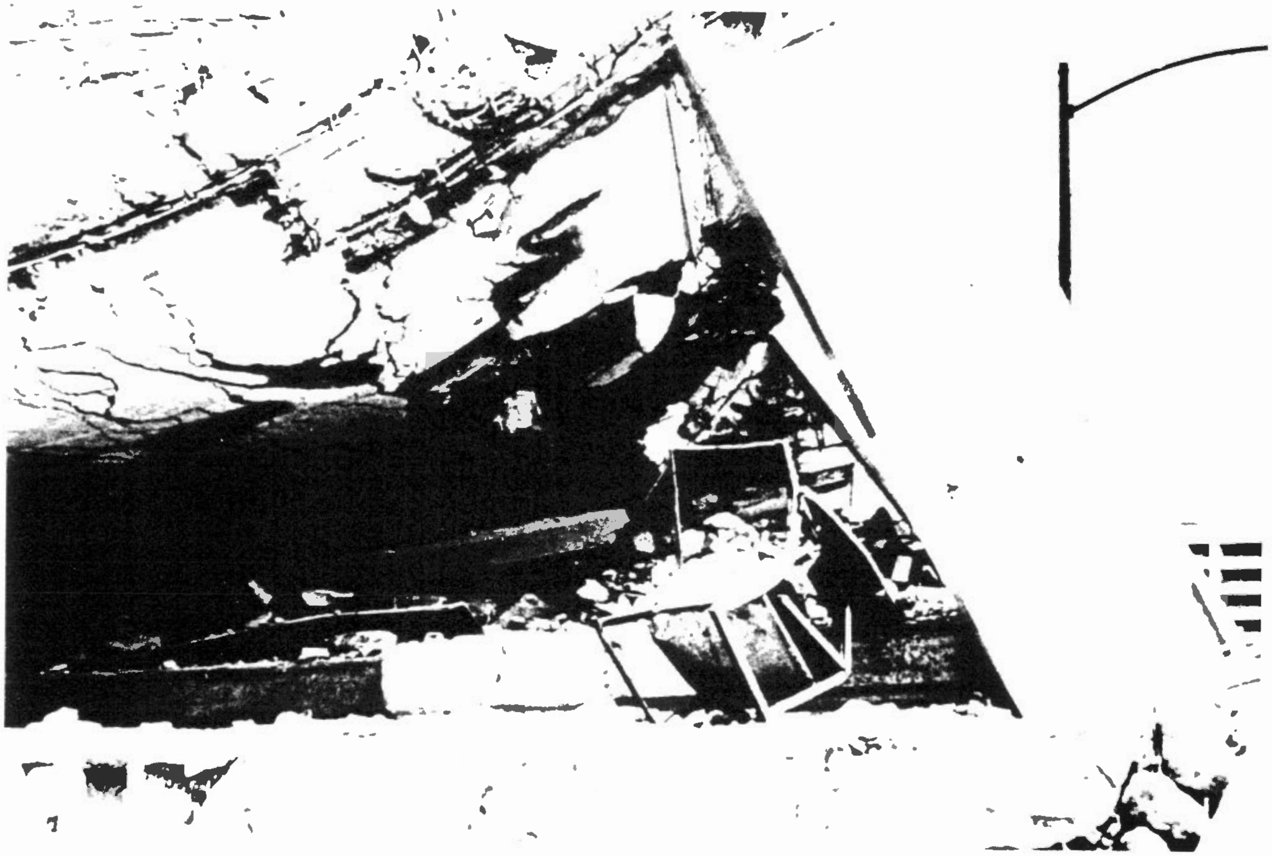
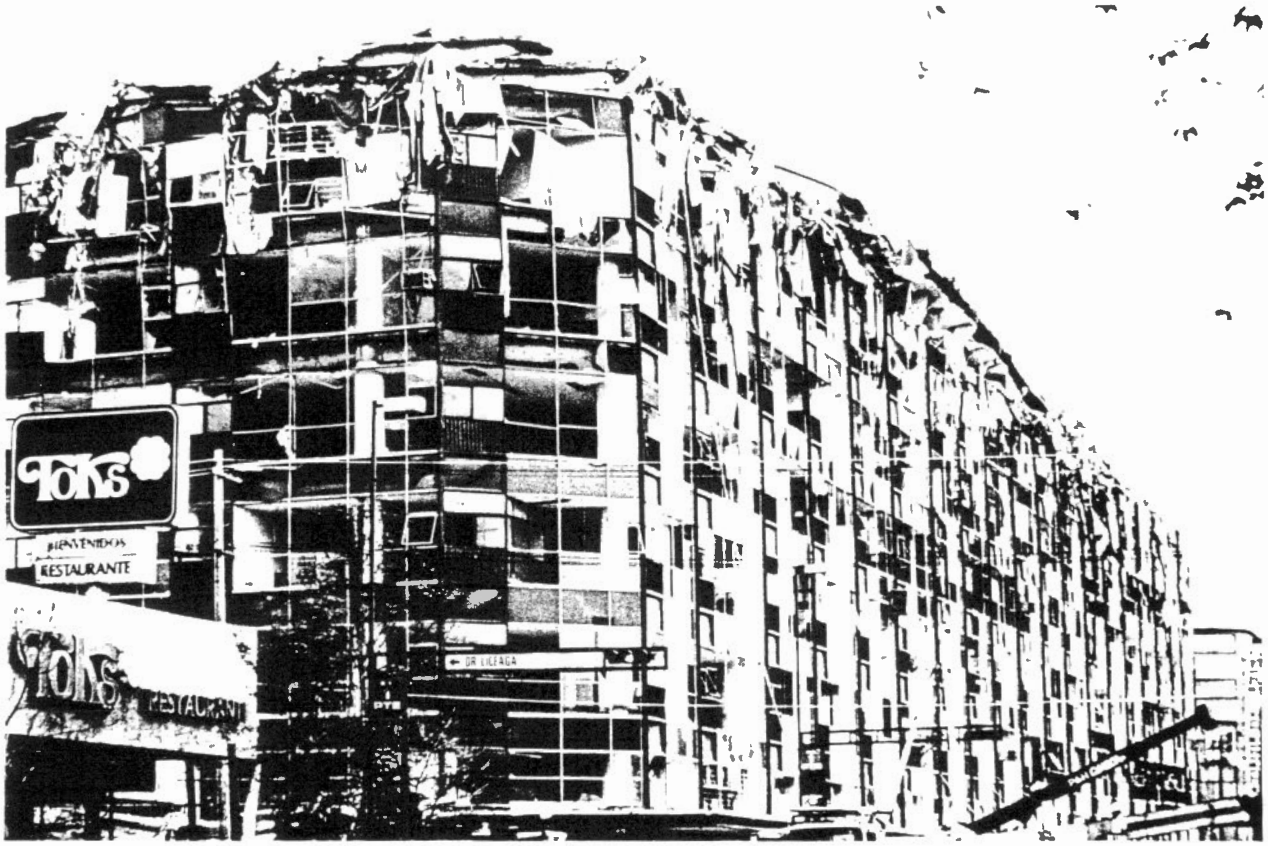


Fig. 6.16c Exterior Column-Slab Connection in School Building

Fig. 6.18 Collapsed Theater Structure

This structure had masonry walls with light steel trusses and a metal deck roof.



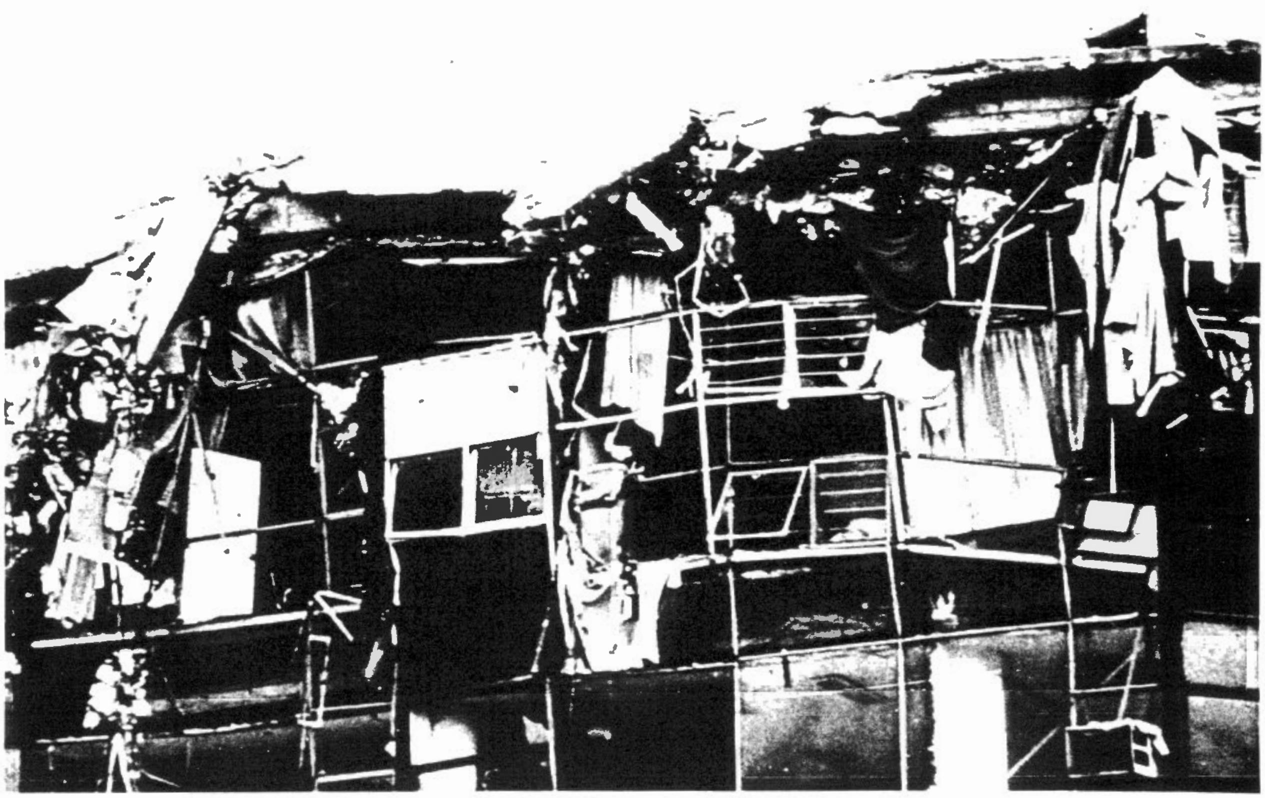
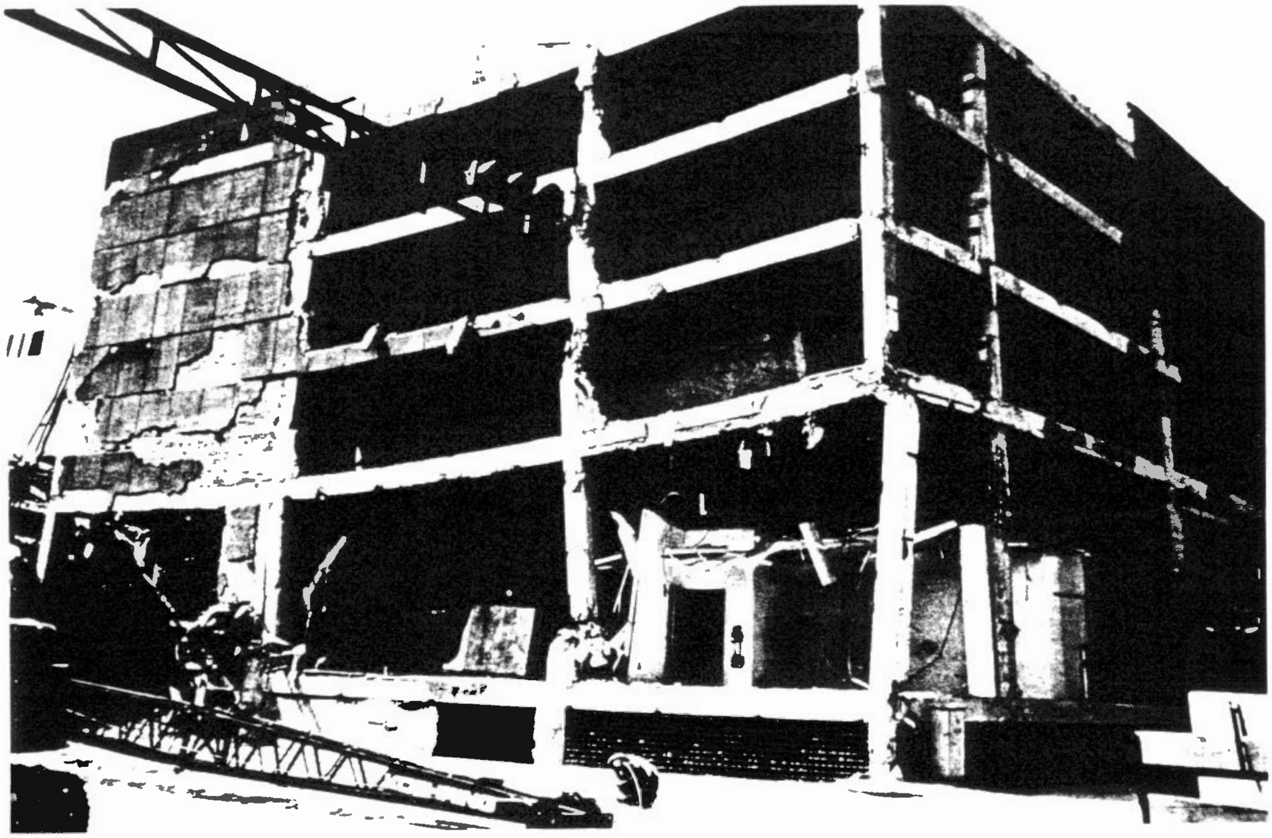


Fig. 6.17a and 6.17b Eight Storey Flat Plate Structure with Collapsed Top Storey



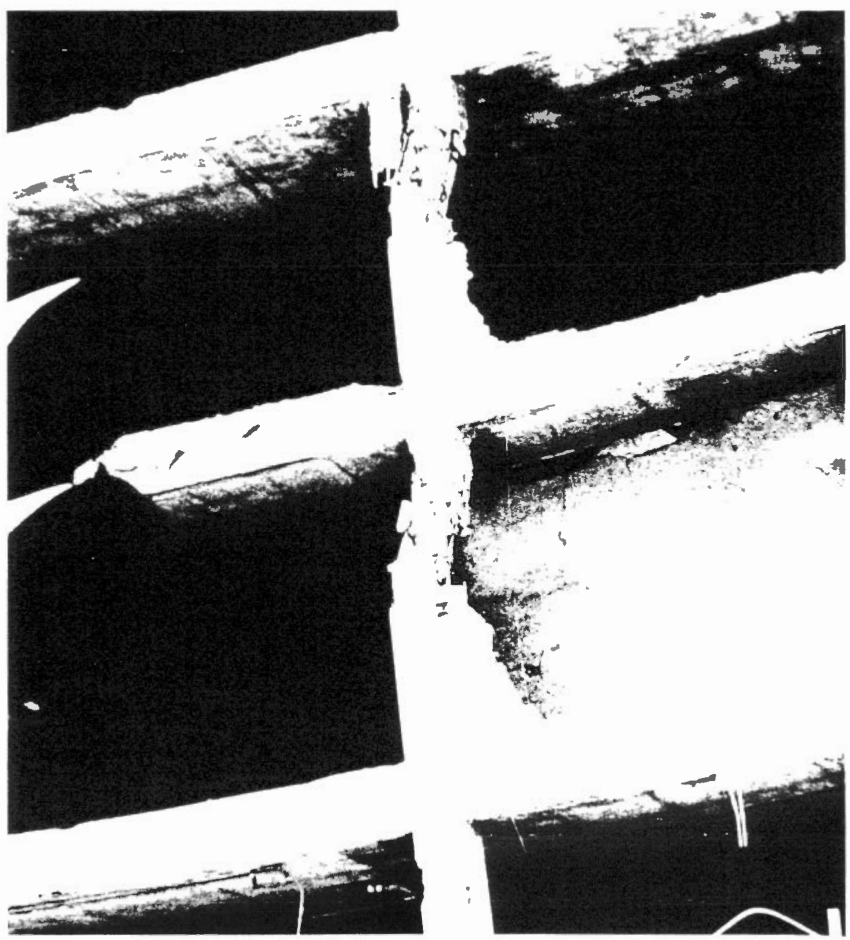


Fig. 6.19a amd 6.19b Four Storey Theatre with Reinforced Concrete Frame and Masonry Infilled Walls

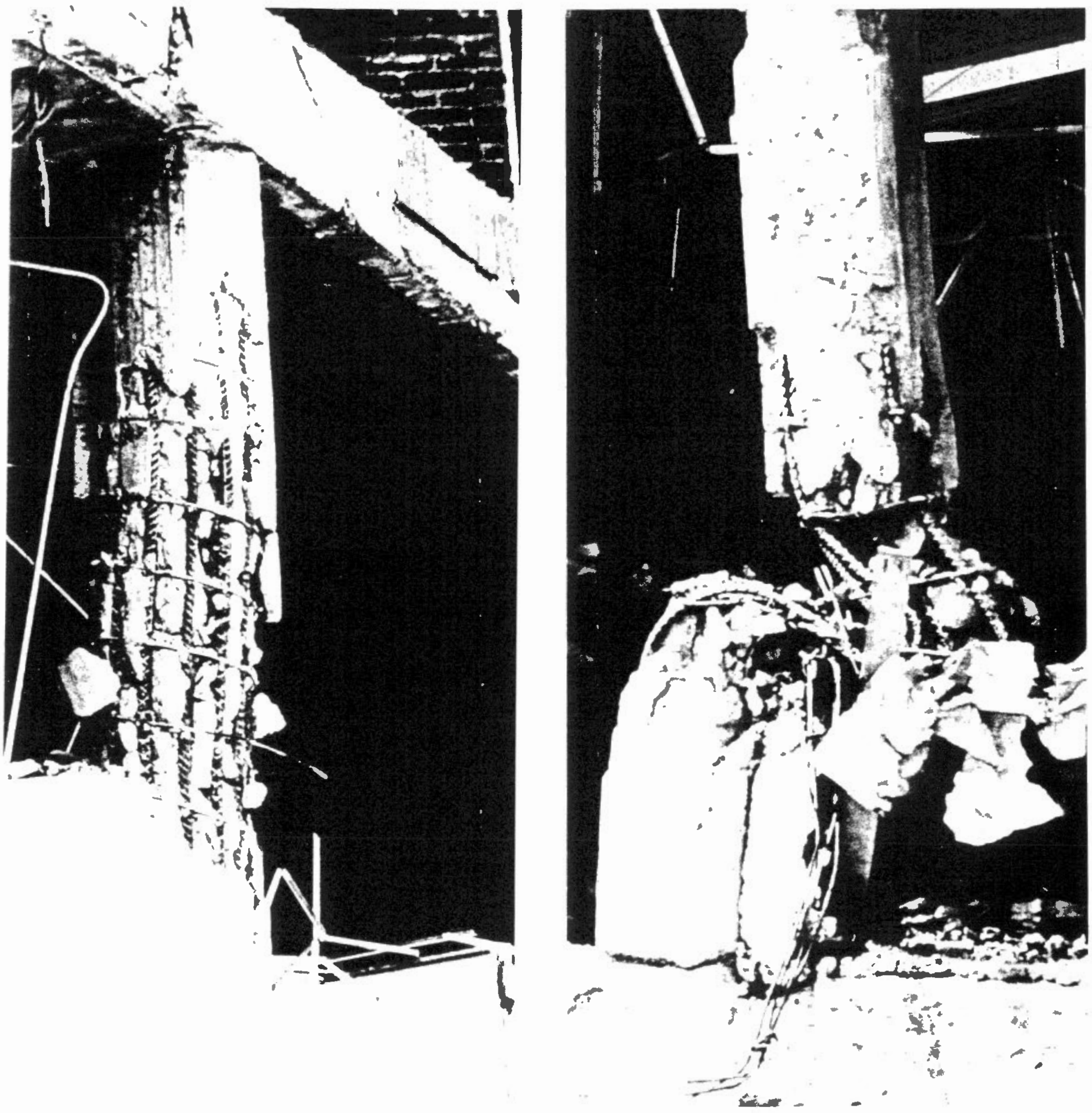


Fig. 6.19c and 6.19d Column Failures in Theatre (Televiteatro)

Fig. 6.20 The Tlatelolco Housing Development

The 23 year old Tlatelolco housing development contains 102 apartment and condominium buildings with about 105,000 inhabitants. This development is in the soft soil region close to the transition zone. According to the National Housing Agency only 59 out of the 102 buildings are habitable.

Shown in Fig. 6.20 is the Chihuahua apartment building which is rectangular in plan and is composed of three parts. In the Nuevo Leon condominium complex, which was identical to the Chihauhua structure, 2 of the three adjacent parts had completely collapsed. Details are given in Fig. 6.21 a, b and c.

Fig. 6.21a Part of the Nuevo Leon Building Still Standing

Two thirds of this very large structure totally collapsed. The 14 storey structure consists of a reinforced concrete frame with reinforced concrete diagonal bracing members with infilled masonry walls. A close-up of one of the many column failures is given in Fig. 6.21b.

Fig. 6.21c shows the foundation for the structure.

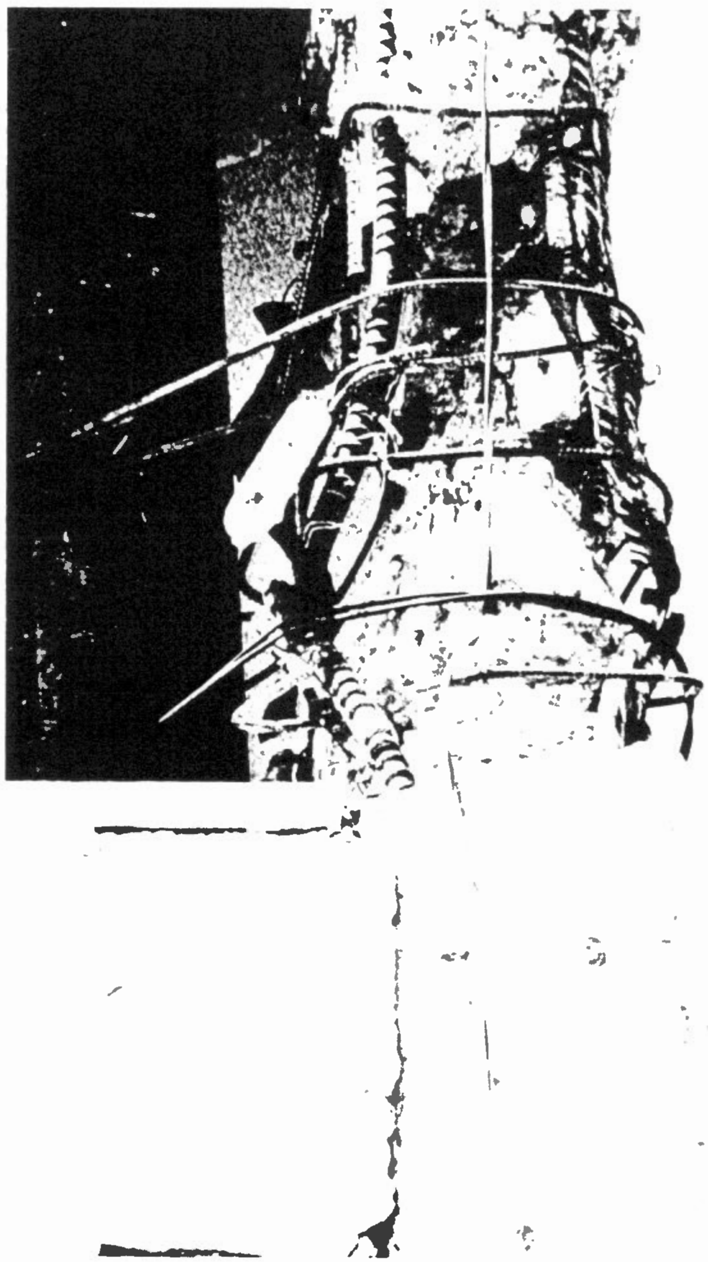


Fig. 6.21b Column Failure in Nuevo Leon Condominium Structure

This column shows a lack of confinement reinforcement and the column longitudinal bars have actually ruptured. The steel pipe column to the left of the failed column is helping to support the severely damaged structure.

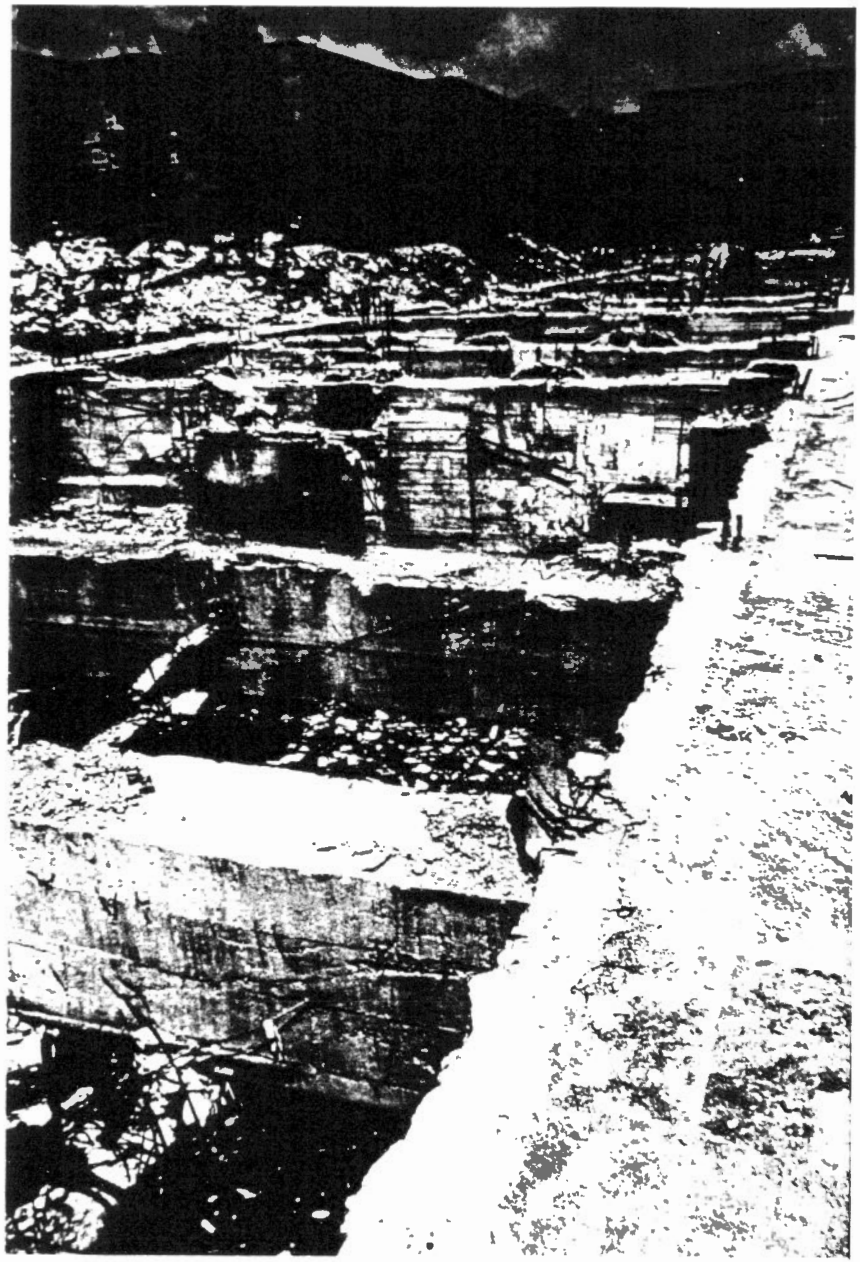


Fig. 6.21c Foundation of Collapsed Nuevo Leon Building

This rigid-box foundation seemed intact in spite of reports that there might have been a foundation failure. There were reports of many problems in the past with the foundations of this building. This figure shows the length of the collapsed structure.

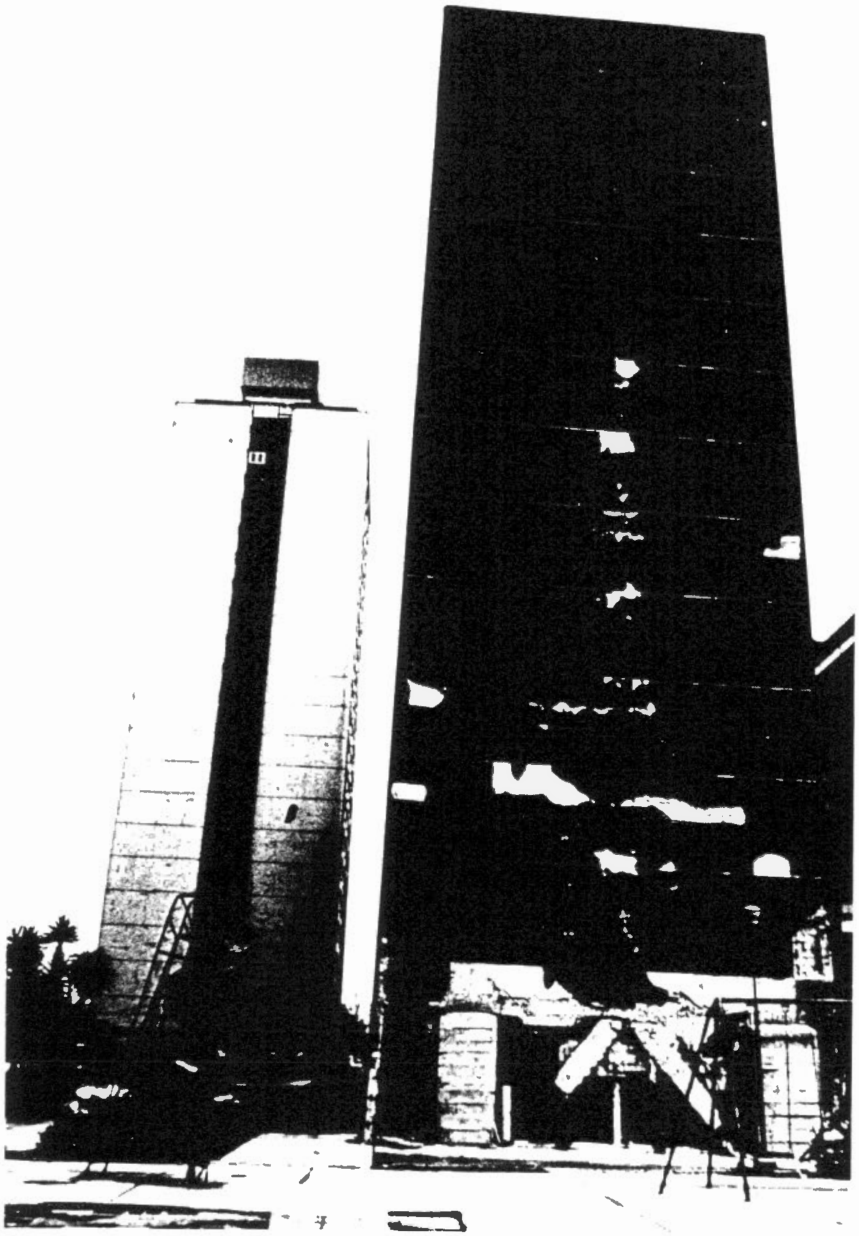


Fig. 6.22a Severe Damage to 13 Storey Apartment Structure

This 13 storey apartment building in the Tlatelolco housing development suffered severe damage in the first storey level. The structure consisted of a reinforced concrete frame with reinforced concrete bracing members and infilled masonry walls. The structure was supported by timber shoring and then large concrete pillars were cast at the extremities of the wall. The elevator core was located towards the other end of this rectangular shaped building and therefore torsional eccentricties may have contributed to the damage. Close-up views are given in Fig. 6.22 b and c.

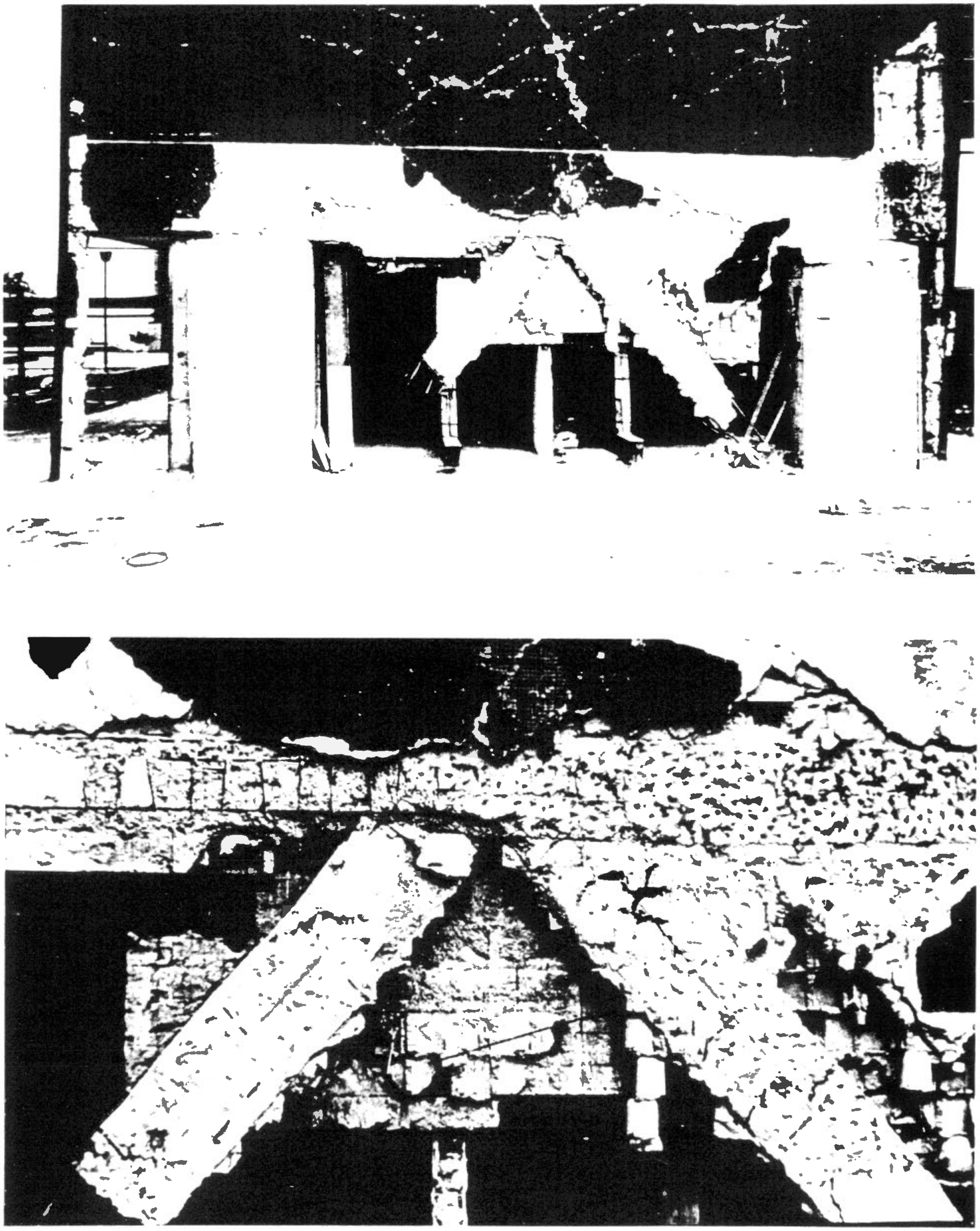


Fig. 6.22b and 6.22c Close-up of Failed Wall and Bracing Members

Fig. 6.23a Fourteen Storey Apartment Building with Offset Floors

The offset floors of this 14 storey apartment building in the Tlatelolco housing development creates an interesting architectural feature but results in a poor structural system as shown in Fig. 6.23b.

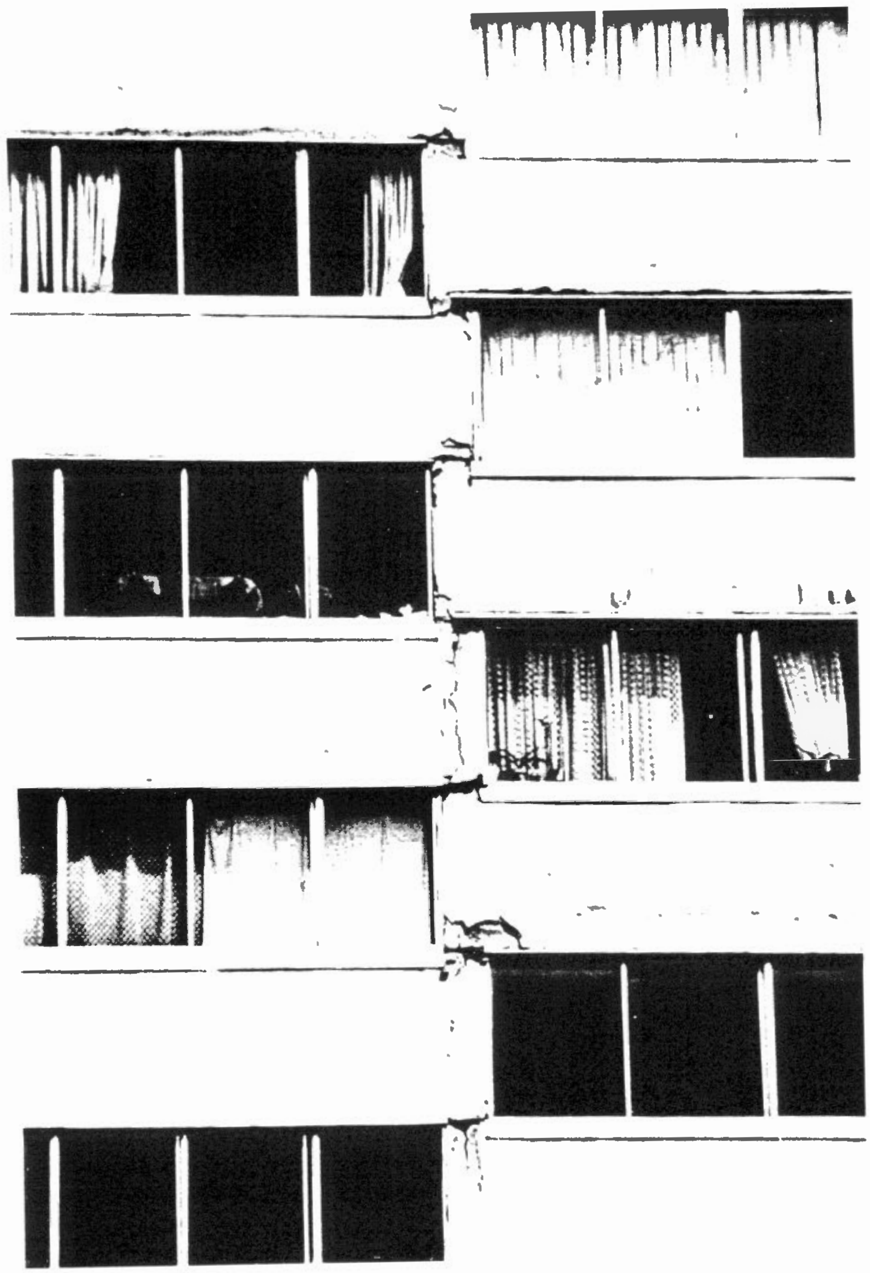


Fig. 6.23b Damage due to Offset Floors

This close-up view of the 14 storey apartment building with the offset floors emphasizes the need to properly connect the spandrel beams. The local damage at the connections is apparent.

6.5 References

6.1 Zeevaert, A. and Cuevas, L., "La Torre Latinoamericana", 1983 Edition, available from the Latin American Tower, 84 pp.

6.2 Zeevaert, L. and Newmark, N.M., "Aseismic Design of Latinoamericana Tower in Mexico City, Proceedings of the World Conference on Earthquake Engineering, Berkeley, California, June 1956, pp. 35-1 to 35-11.

6.3 Zeevaert, L., "Foundation Design and Behaviour of Tower Latinoamericana in Mexico City", Géotechnique - The International Journal of Soil Mechanics, The Institution of Civil Engineers, Vol. VII, No. 3, Sept. 1957, pp. 115-133.

6.4 Zeevaert, L., "Foundation Engineering For Difficult Subsoil Conditions", Van Nostrand Reinhold Company, New York, 1972, 652 pp.

6.5 Rosenblueth, E., "The Earthquake of 28 July 1957 in Mexico City", Proceedings of the Second World Conference on Earthquake Engineering - Vol. 1", Japan 1960, pp. 359-379.

6.6 Institute of Engineering, "El Tremblor del 19 Septiembre de 1985 y sus Efectos en las Construcciones de la Ciudad de Mexico (The Earthquake of 19 September 1985 and its Effects on the Construction in Mexico City)", a preliminary report prepared by the Institute of Engineering at the National Autonomous University of Mexico, Sept. 30, 1985, 30 pp.

CHAPTER 7

EMERGENCY CODE CHANGES

7.1 Details of Code Changes

On the 18 October 1985 a Presidential Decree was published in the "Diario Oficial" (Ref. 7.1) giving emergency code changes for construction in the Federal District. The articles of this decree have been translated and are summarized below. Comments on some of these articles are clearly indicated, being enclosed by square brackets.

ARTICLE 1 All buildings in the Federal District 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 for the Federal District including the Complementary Codes.

ARTICLE 3 The owner or the occupants of buildings with knowledge of structural damage (including damage to walls) must report this to the authorities.

ARTICLE 4 The owners must submit a technical report of the damage to the authorities. If the stability of the structure is not in question then the report must identify means of local repair or strengthening. Any significant repairs must be designed in accordance with existing codes and a proper inspection made by first removing any elements covering the structural components. Authorization must be obtained for any repair or any new construction from the authorities.

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 buildings. All structures currently being constructed classified in Group A must be revised according to the emergency changes.

ARTICLE 6 The resistance factor, F_r, for 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 F_r may be taken as 0.6 in calculating the resistance under flexure and compression.

The resistance factor of 0.35 used for foundation design must also be used for the frictional forces between soil and caissons or piles.

ARTICLE 7 The live loads for office buildings are to be taken as follows:

$$w = 140 \text{ kg/m}^2$$

$$w_a = 180 \text{ kg/m}^2$$

$$w_m = 120 + 420A^{-1/2} \text{ but not less than } 250 \text{ kg/m}^2$$

[where w is the mean live load used in calculating differential settlements, w_a is the live load to be used in conjunction with seismic and wind loadings, w_m is the mean live load to be used in the structural design for gravity loads and for immediate soil settlements and A is the tributary area in square metres.]

ARTICLE 8 The simplified method for determining lateral forces may only be applied for buildings not higher than 8.5 m.

ARTICLE 9 The seismic coefficient c for structures in group B 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). [The values of a_o are increased from 0.045 to 0.054 for Zone II and from 0.06 to 0.10 for Zone III.]

Table 7.1 gives the modified seismic coefficients to be used in the simplified method of analysis for Group B structures.

Zone	Brick Walls Height of Structure, H(m)		Hollow Block Walls Height of Structure, H (m)	
	<4	4 < H < 8.5	<4	4 < H < 8.5
I	0.06	0.08	0.07	0.11
II	0.09	0.10	0.11	0.15
III	0.12	0.15	0.13	0.17

Table 7.1 Modified Lateral Force Coefficients for Simplified Analysis .

The values in Table 7.1 must be multiplied by a factor of 1.5 for structures in Group A.

[The reduced height of structure for which the simplified analysis is applicable together with the increased lateral force coefficients for Zones II and III (comparing Table 5.7 with Table 7.1) is coupled with an increase in the importance factor from 1.3 to 1.5 for Group A (important) structures resulting in a large increase in the design lateral forces.]

ARTICLE 10 The ductility factor Q is to be modified as follows:

(a) Q = 4

In order to use Q = 4 the following requirements must be met:

(i) The resistance in all levels is supplied by unbraced frames of concrete, wood or steel together with braced frames or with concrete walls in which the capacity of the frames without the walls or the bracing 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. The resisting capacity of one storey will be calculated taking into account all of the elements that can contribute to the resistance.

(iii) Columns with tie Reinforcement - 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 700 $d_b/\sqrt{f_y}$ where d_b is the longitudinal bar diameter and f_y is in kg/cm^2. These limits are reduced by one-half in both extremes of the column in a length equal to the larger column dimension and at least 600 mm. [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

$$A_v = 0.4\ p'd_c\ s_h$$

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) Steel Frame Structures - 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.

(b) 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(a) must be satisifed

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

(c) 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 10a and 10b. Also solid brick masonry walls confined by pilasters, bond beams, columns or beams of reinforced concrete or steel may be used.

(d) 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 (a) to (c) above.

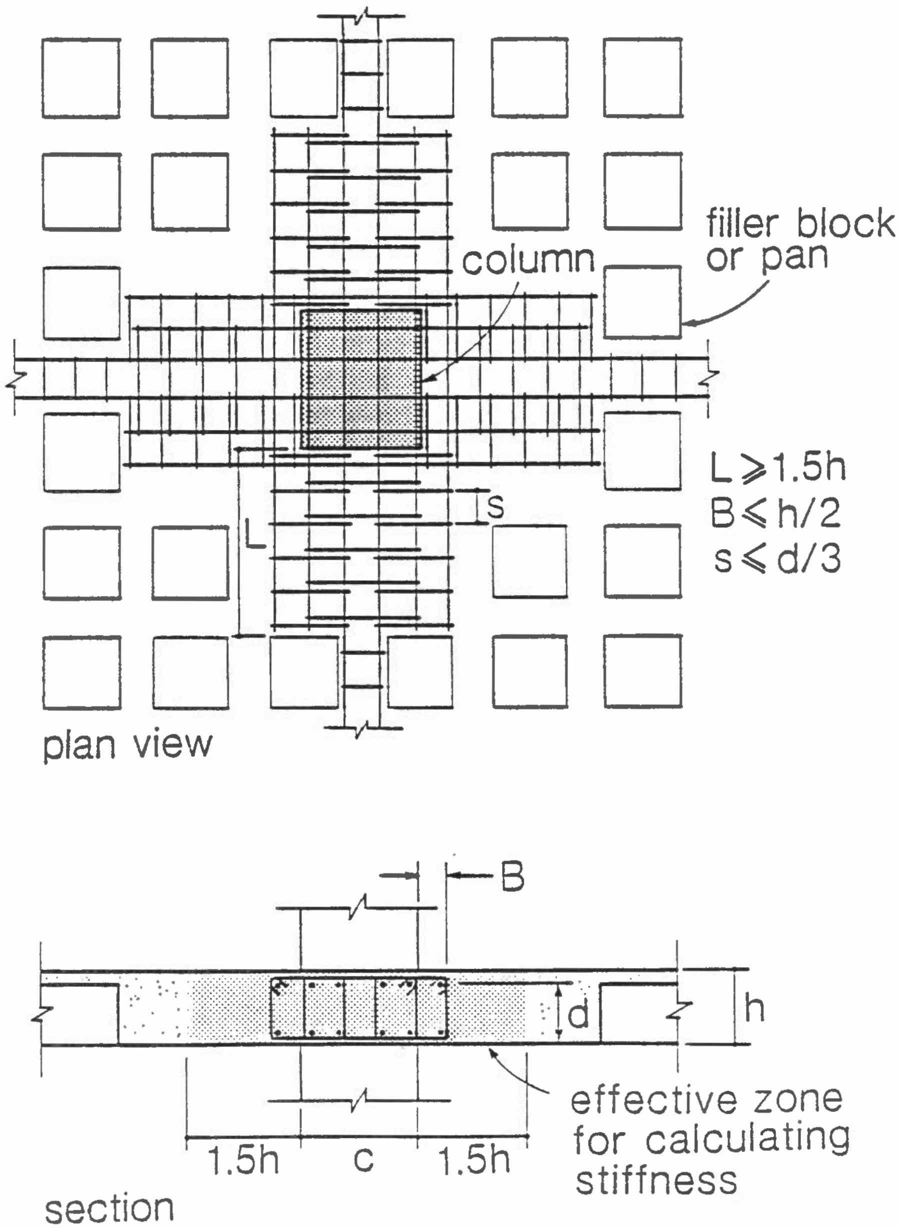


Fig. 7.1 Details of Waffle Slab Required by Emergency Code Changes

(e) 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 analysis with lateral loads an equivalent beam width of slab 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. 7.1].

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 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 structures of Group A 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.

7.2 Period of Transition

The Decree will be in effect on October 19, 1985. All of the current codes will continue to be applied as long as they do not contradict these emergency provisions. The Decree was signed on the 17 October 1985.

7.3 References

7.1 Miguel De La Madrid H., Presidente Constitucional de Los Estados Unidos Mexicanos, "Decreto por el que se establecen las normas de emergencia en materia de construccion para el Distrito Federal" ("Decree establishing the emergency provisions for construction in the Federal District"), Diario Oficial, 18 octubre, 1985, pp. 26-30.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Value of Site Visits

The Canadian National Committee on Earthquake Engineering should continue to send site-visit teams to significant earthquakes which are relevant to the Canadian experience. This will enhance Canadian expertise in the various areas of earthquake engineering and the lessons learned from these visits will provide valuable data in updating future Canadian codes. The reporting of the findings of site-visit teams also serves to educate engineers and the general public concerning earthquake hazards and structural damage.

8.2 Tectonics

The 1985 earthquake occurred in a seismic gap where subduction could have been argued to be aseismic. Several other seismic gaps along the Mexican subduction zone may soon be approaching maturity; with their rupture expected to shake Mexico City in a similar fashion to the 1985 earthquake.

Until recently, some have argued that subduction beneath the Pacific Northwest is also occurring aseismically. This view may now need revision and an earthquake off the coast of British Columbia, of a magnitude similar to the Mexican Earthquake cannot be excluded.

8.3. Epicentral Ground Motions

Damage in the epicentral area was surprisingly moderate. Accelerations recorded on competent rock were about 12 to 17% g which is consistent with the assessed intensities of VII to VIII along much of the coast. Local amplification was probably responsible for the intensities of IX to X observed on the Balsas River delta.

8.4 Effect of Poor Soil Conditions on Strong Ground Motion

The unique soil conditions in the Lake Zone of Mexico City led to resonance and harmonic motion in the highly compressible soil. At the SCT Communications Building the harmonic motion with a 2-second period reached 20% g and lasted for about 60 seconds. This represented amplification of up to 5 times the motion on firm ground and increased the duration of strong ground motion.

In view of the potential for large earthquakes near Vancouver together with the thick recent sediments in the Frazer River Delta, similar ground motion effects should be expected. Similar considerations apply to the St. Lawrence Valley.

8.5 Duration of Ground Motion

Although ground motion amplitudes saturate in the mid M 7 range the strong motion durations will increase proportionally to the rupture length of the fault. Thus a rupture velocity of 3-4 m/sec along 150 km of fault length gives about 1 minute of strong ground motion on hard soil. At Abastos in the lake zone region of Mexico City, the strong ground motion reverberated longer (about 3 minutes duration).

The duration problem is not addressed in the Canadian Code and in view of the potential for large earthquakes in some areas of Canada durations do deserve renewed discussions.

8.6 Value of Subzonation

The 1985 damage in Mexico City was more severe than in previous earthquakes, but was centered in the same area. The characteristics of the ground motion and the resulting structural damage in the soft subsoil regions confirms the need for subzonation that had been established long before the 1985 earthquake.

The near source hard rock accelerations were unexpectedly low for an earthquake of this size, falling well below all relevant ground motion relations such as Hasegawa (1981), Schnabel and Seed (1973) or Joyner and Boore (1981). It is difficult to believe that such low grond motion levels are typical of large subduction earthquakes, and one example to the contrary was the 1985 Chile, M 7.8 earthquake that yielded (although with a large scatter) peak accelerations at less than 100 km distance, that were at least 4 times higher than the Mexican ones.

On the other hand, at disances of 300 to 400 km, on hard sites on Teacalco and Mexico City, the recorded accelerations are considerably higher than predicted by Schnabel and Seed and Joyner and Boore's relations but do fit the Hasegawa relation for M 7.5. This M 7.5 is the specified saturation magnitude for the latter relations, that is, larger magnitudes are not expected to yield higher amplitudes in the short period frequency band that is considered here.

8.7 Digital Strong Motion Recorders

The value of digital strong motion recordings was convincingly demonstrated in Mexico where processed strong motion records were available within days of the earthquake. In Canada modern digital instruments should be installed at a few strategic locations.

8.8 Factors Contributing to Structural Damage

One very important factor leading to severe damage in the lake zone of Mexico City was the so-called double resonance of both the soil and many structures. The almost pure harmonic motion, with a period of about 2 seconds together with the high accelerations and long duration led to unprecedented damage and collapses of structures in the 8 to 20 storey range. Due to the many cycles of loading and cumulative damage the period of these short buildings lengthened and therefore led to increased lateral forces. The emergency code changes resulted in a 35% increase in lateral forces for the transition zone and a 66% increase in the lateral forces for the lake zone.

In many situations damage or collapse occurred at intermediate storey levels or at the top of structures. Factors which contributed to this phenomenon were heavy storage loads at upper levels and the influence of higher modes and foundation rocking which contributed to pounding of adjacent structures not having sufficient separation between them. There were many cases of severe damage caused by effects such as large torsional eccentricities (primarily due to eccentrically located walls), soft storeys, shear failures of short columns and P-Δ effects in flexible frame structures. In some cases a combination of the above factors led to the structural damage.

Although there were very few examples of foundation failures there were some dramatic examples of pull-out of friction piles. Some of the structural damage however, may have been due in part to the response of the foundations on soft soil.

8.9 Performance of Different Structural Systems

The collapse of a 21 storey steel structure is significant because it is the first high-rise steel structure to collapse in an earthquake. The emergency code changes required that beam-to-column connections in ductile steel structures be designed to resist forces associated with plastic hinges in the beams. Both of these facts suggest the need to have some specific Canadian code requirements for achieving high levels of ductility in steel structures.

There were many examples of severe damage of reinforced concrete frame structures with masonry infills. The damage to these structures, some of which employed reinforced concrete diagonal braces together with the masonry infills, had many contributing factors such as large torsional eccentricities, poor confinement details in the columns, brittle steel reinforcement and poor connection details (between the columns, beams and braces). The 1985 Canadian codes require more stringent detailing requirements in ductile and nominally ductile frame members, particularly for the confinement details of the columns.

A common form of construction in Mexico are flat plate structures consisting of either waffle slabs or slabs with hollow masonry filler blocks cast in the slabs. Many of these slabs failed by punching shear and were subjected to large moments transferred through the slab-column connections due to P-Δ effects generated by these flexible structures. The emergency code changes call for a decrease in the ductility assigned to these types of structures, lower stiffnesses for analyses and improved detailing of the slab-column connections (stirrups in the slab and minimum amounts of steel passing through the column). The design and detailing requirements of the emergency changes provide useful guidance for Canadian engineers for improving the ductility of slab-column connections.

The majority of concrete structures that suffered damage or collapse were not designed and detailed to exhibit significant levels of ductility and were different than typical Canadian structures, designed and detailed according to the 1985 codes. The damage does raise concern over older Canadian buildings which would not meet the more stringent design and detailing requirements of our modern codes. More attention should be placed on bringing our older structures, particularly post-disaster structures, up to a minimum standard more consistent with the goals of our present code requirements.

8.10 Post-Disaster Preparedness

The large number of buildings that collapsed in the 1985 earthquake led to desperate rescue attempts in order to save lives. The problem was compounded due to the damage and collapse of several medical facilities and school buildings. Rescue operations were further hampered by blocked roadways and the lack of water supply after the earthquake. The estimated 10,000 deaths led to problems of identification and humane disposal of the bodies. The over 200,000 homeless lived in tent camps until relocation was possible. The loss of many structures housing governmental departments caused a large disruption which led to immediate plans to decentralize the government. The airport and metro suffered no damage and these facilities were operational immediately after the earthquake. Television and local telephone communications were not severely disrupted but long distance communications were disrupted for over three weeks. Safety switches on the electrical power supply facilites such as dams and substations worked very well enabling restoration of electrical service to undamaged areas soon after the earthquake.

In Canadian regions of high seismic hazard steps should be taken to improve post-earthquake preparedness.

8.11 Further Studies Required

The Canadian National Committee on Earthquake Engineering (CANCEE) should participate in ongoing Mexican studies on the implications of the 1985 earthquake. The results of these studies will enable CANCEE to take appropriate actions, if necessary, in updating the National Building Code of Canada.