THE MEXICAN EARTHQUAKE OF 19TH SEPTEMBER 1985

A FIELD REPORT BY EEFIT

This report was made possible by generous funding, which EEFIT records with sincere gratitude, from the following:

Ove Arup & Partners, Consulting Engineers, London
The British Government, through the Science and Engineering Research Council
Allott & Lomax, Consulting Engineers, Manchester
Reinsurance Offices Association, London

EEFIT's work would also have been impossible without the help of many individuals and organisations, particularly the following:

David Blagbrough, British Council, Mexico
Ing Abraham Ellstein, Laboratories Tlalli, Mexico
Dr Jorge Prince, UNAM, Mexico
David Oxley, Davy McKee, Lazaro Cardenas.

The kindness and helpfulness shown to the team by many Mexicans during their time of national crisis is also recorded with gratitude.

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Summary of EEFIT's conclusions

a) Near the epicentre, engineered structures, including dams, generally survived the motions well, and damage to weak masonry and adobe buildings was less than might be expected in the near field of a magnitude 8.1 earthquake.

b) The greatest concentration of damage to engineered structures occurred on the Lake Zone of Mexico City, 400km from the epicentre. The area of maximum damage in the Lake Zone was that in which the density of medium to high rise construction was greatest. All medium to high rise construction appeared equally at risk within this area of maximum damage.

c) The bedrock motions at Mexico City, which had attenuated to a harmless amplitude, were amplified to an exceptional degree by the local soil deposits of the Lake Zone and were modified to consist mainly of a frequency of 0.25 to 0.5Hz. Similar amplification may be expected during future events.

d) Preliminary analysis strongly suggests that the soil amplification at Mexico City took place largely due to one dimensional effects in the 40m or so of soft superficial clay. These superficial deposits, though unusual, are not unique, and earthquake prone sites with similar deposits will have to be carefully examined for site effects in future. The contribution of the underlying sands and gravels, or of two dimensional basin effects to the amplification was probably much less important than the one dimensional effects.

e) Even though Mexico City is well known for its foundation problems, foundation failures did not play a significant part in the scale of the damage.

f) The Mexico City Lake Zone motions were exceptionally damaging to medium rise construction both because of their frequency content and their long duration.

g) Lack of ductility characterised the structural failures. Many failed buildings appeared to have been designed for low or intermediate levels of ductility, as defined in the 1977 Mexico City code. The alternative option of designing for a high level of ductility but lower lateral forces, which is provided for by the 1977 code, has apparently seldom been used in Mexico City.

h) The 1977 Mexico City earthquake code has no mandatory provision for specifying high levels of ductility in medium to high rise construction on the Lake Zone. It is recommended that consideration be given to making such a mandatory provision, which however does not appear in the Emergency Regulations introduced soon after the earthquake.

i) Based on preliminary analysis by EEFIT, the increase in the level of force resistance required for highly ductile structures by the Emergency Regulations, compared with the 1977 Mexico City code, may be unnecessarily conservative.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

1.0 INTRODUCTION

1.1 BACKGROUND TO THE EEFIT MISSION

The Mexican earthquake of 19th September 1985 was one of the most destructive this century in terms of damage to modern, medium to high rise construction. It was also unusual in that major damage occurred at an epicentral distance of 400km, due to abnormally large local soil amplification effects. The earthquake therefore held the promise of exceptional richness in its lessons for the earthquake engineering community.

On the day after the earthquake, EEFIT (Earthquake Engineering Field Investigation Team) met to prepare for a field mission to Mexico. EEFIT had been founded in 1981 (EEFIT 1983) as an association of earthquake engineers, architects and scientists, with the aim of reporting back to the UK and international community the lessons to be learnt from damaging earthquakes, based on field investigations. Despite the fact that many other Mexican and international teams were studying the event, its enormous significance was felt to make a UK presence essential, although it was recognised that resources would allow only a selective survey. Other international reports on the event are listed with the references cited at the end of this report.

Accordingly, the first EEFIT team arrived in Mexico City on Saturday 28th September, 9 days after the main shock. The primary objective was to learn as much as possible from the earthquake, particularly in its relevance to civil, structural and geotechnical engineers, and the findings form the main body of this report. A secondary objective was to offer any assistance that the team could to the Mexican authorities, and this aspect is also briefly summarised in the report.

1.2 MAIN EEFIT MISSION

The main EEFIT team consisted of Edmund Booth (structural engineer) and Jack Pappin (geotechnical engineer) both of Ove Arup and Partners, Consulting Engineers, London, R. Scott Steedman (geotechnical engineer) of Cambridge University Engineering Department and John Mills (structural engineer) of Allott and Lomax, Consulting Engineers, Manchester. Steedman's expenses were met by the Science and Engineering Research Council and the others by their respective employers. The team spent 13 days in Mexico City. A further day was spent outside the city, when the team split to visit Lazaro Cardenas on the coast near the epicentre, and Ciudad Guzman, an inland town west of the capital which was reported to be unusually badly damaged.

The field methods used by the team in its reconnaissance objective were very simple. As many of the interesting damage sites as possible were inspected and photographed externally and (where possible) internally, and as much relevant information as possible was gleaned from local parties. Some theoretical desk studies were performed by the team on its return to the UK, and some samples of Mexico City clay were tested at Nottingham University.

Before departure, EEFIT contacted investigators of the earthquake from both Mexico and the United States, while in Mexico, liaison with the local authorities was established through the good offices of the British Council and Embassy, and contact was made with engineers from private and public practice and from the national university, UNAM.

1.3 NOTTINGHAM UNIVERSITY SURVEY

Martin Degg, of Nottingham University Geography Department, arrived in Mexico City on 7 November 1985, after the departure of the main EEFIT team, and spent 6 weeks carrying out a systematic damage survey of buildings in Mexico City, based on external visual examination. The survey was carried out for the Reinsurance Offices Association, of London, and thanks are due to them for allowing the findings of the survey to be included as Section 3.2. of this report.

2.0 SEISMOLOGY OF THE EARTHQUAKE

2.1 INTRODUCTION

In this report, only a cursory survey of the seismological aspects of the event is presented. Mexico is an area of complex, high earthquake activity and many specialised articles have been, and will continue to be, produced on this topic.

The earthquake which occurred at 0718 local time on 19 September 1985, had a rupture area near the south coast of Mexico as shown on Figure 2.1. In this region, the Cocos Plate is subducted under the North American plate at a rate of about 75mm/year. This tectonic activity plays a major role in the overall geological structure of Mexico. Figure 2.2 shows a typical section through a subduction zone which leads to the characteristic offshore trench, earthquakes occurring on the boundary and volcanic activity some distance from the trench. The central highland areas of Mexico are predominantly volcanic (see Figure 2.1). At present there are about 7 active volcanoes in Mexico.

The subduction zone stretches over 1500km and about 30 large earthquakes (Ms ÷7.0) have occurred over its length this century. The zone is generally seismically active but four significant seismic gaps had been previously identified - see Figure 2.1 (Singh et al, 1981). Before 1985, the Michoacan and Tehuantepec Gaps had not had major earthquakes in the last two centuries, except for a magnitude 7.3 earthquake recorded in the Michoacan gap in 1981 (see Figure 2.1). This indicates either areas that are aseismic or areas with very long recurrence periods. In the Guerrero and Jalisco Gaps, however, large earthquakes have previously been recorded but with a gap since the last one of the order of the usual recurrence period of 30 to 70 years. These two areas are therefore considered to have a high earthquake potential.

Preliminary results suggest that the earthquake of 19 September was a multiple event consisting of two subevents, the second occurring 26 seconds after the first. The ruptured area covered about 70 x 170 km as shown on Figure 2.1 (Rosenblueth and Meli, 1986). The depth of the event was about 18km and its magnitude has been determined as Ms = 8.1.

2.2 MEASURED GROUND MOTIONS

Several strong motion instruments recorded the motions resulting from the event. The Universidad Nacional Autonoma de Mexico (UNAM) were very helpful to visiting engineers and produced strong motion time history records within a week of the event (Mena et al, 1985, Prince et al, 1985). For the purposes of this report however, only a few typical strong ground motion records are discussed.

Near the epicentre, the ground motion had a typical frequency rich pattern with peak accelerations up to about 30%g horizontally and 15%g vertically. Figure 2.3(a) shows the accelerations measured by an instrument at Zacatula a few kilometers inland near the epicentre (see Figure 4.1). Figure 2.3(b) shows the velocity and displacement variations with time of the north south ground motion. The acceleration response spectrum of the north south motion is plotted on Figure 2.4.

At Mexico City, which is about 400km ENE of the epicente, the recorded ground motion varied greatly depending on the underlying soils. At the University (referred to as CU on Figure 3.1) which is founded on volcanic rock the measured ground motion showed maximum accelerations of about 3%g horizontally and 2%g vertically as shown in Figure 2.5. This level of acceleration and the type of ground motion is not exceptional for an event of this magnitude and at this distance from the epicentre. The acceleration response spectrum for the east west motion is plotted on Figure 2.4.

As described in the next section, the central area of Mexico City was previously a lake bed and is underlain by up to 40m of soft to firm Lacustrine clays and in these areas the ground motion was greatly modified. Figure 2.6(a) shows the accelerations measured at the Department of Transport site (referred to as SCT in Figure 3.1). It is seen that while the vertical motion is not significantly different to that measured by the instrument founded on rock, the horizontal motion has a peak acceleration of about 20% g and consists mainly of a single frequency of about 0.5Hz. The direction of greatest motion is S60°E. Figure 2.6(b) shows the velocity and displacements with time of the east west motion at SCT. A notable feature of the motion is the number of large peaks. The instruments also showed that perceptible ground motions continued over three minutes. The acceleration response spectrum of the east west motion is plotted on Figure 2.4. The response spectra clearly illustrate the difference between the lake bed and surrounding rock motion and the period content of the lake bed motion which has a very distinct peak at 0.5Hz.

Descriptions of the event by residents also confirmed the cyclic nature of the motion in the central area of the city. The motion was described as beginning with two faint rolls and then suddenly becoming a dramatic circular motion, whereas observers on the rock referred to sudden erratic motion.

A witness at Lazaro Cardenas described the earthquake as occurring completely without warning of any kind. The event commenced with a rumbling noise and violent side to side motion. After a little time the shaking felt as if it was coming to an end but then stronger shaking started again. During the event it was difficult to stand and movements could be sensed horizontally and vertically, but mainly the former. The event had a long duration, seeming to last 5 minutes or so, after which time movements gradually reduced.

No strong motion records are available for the Ciudad Guzman area, but a witness described the event as beginning with a gentle movement and becoming more erratic and stronger with time. The motion is said to have lasted about one minute.

2.3 AFTERSHOCKS

The earthquake was followed by a small number of aftershocks. The only major altershock, with Ms = 7.5, came 36 hours after the main event. Its rupture area was east of the main event as shown on Figure 2.1. It is reported to have caused little additional damage.

3.0 EFFECTS OF THE EARTHQUAKE IN MEXICO CITY

Mexico City has a long history, being the capital of the Aztec empire until the area was occupied by the Spanish in 1521. At that time the city was built on many islands surrounded by Lake Texcoco and centred on the site of the present Cathedral. The Spanish demolished most of the Aztec structures and rebuilt the city to their own design. The population gradually expanded from its sixteen century total of around 30,000 and the area of the city was increased by draining the lake and filling. In 1900 the population of Mexico City was 500,000. This increased to 5 million by 1960 and then increased dramatically to about 18 million today. It now has a modern city centre with numerous medium to high rise buildings.

The city has experienced many earthquakes with accounts dating from at least the fifteenth century. In the twentieth century there have been major shocks in 1911, 1932, and 1957 and many other minor shocks. These shocks generally originated from the subduction zone under the south west of Mexico. The 1957 event was the first to cause major damage when about 7 modern buildings up to 10 storeys high collapsed. A few engineered structures were also damaged in minor earthquakes in 1979 and 1981. The event in September 1985 caused a large amount of damage in the central city area primarily to modern structures between 6 and 20 storeys high. Over 200 buildings collapsed, thousands were damaged and an estimated 5 to 20 thousand people were killed, mainly by being crushed or entombed in collapsed structures.

3.1 SOIL CONDITIONS

3.1.1 Geology

Mexico City is at an elevation of about 2250m above sea level. The City is in a closed valley surrounded on all sides by mountains of up to 5000m high which form a ring around the City with a diameter of about 40km. A plan and sections through the Mexico City basin are shown on Figure 3.1. Marsal and Mazari (1957) describe the geological history as follows.

All the existing mountains are volcanic and most were formed in the Middle and Upper Tertiary and Pliocene. There followed a long period of erosion during the Upper Pliocene during which alluvial fans formed within and to the west and east of the existing basin. The alluvial material consists of sharp andesitic fragments of sand and silt. This material is referred to locally as the Tarango deposits and deposition continue to the Lower Pleistocene. There followed a period of glaciation during which some of the alluvial material was removed.

Volcanic activity started again in the Pleistocene. Eventually the Mexico City area became a closed basin, a lake formed, and a thick layer of volcanic ash was deposited. This material decomposed to a lacustrine clay which is referred to as the Tacubaya clays and is composed of montmorillinite and illite. At the edge of the basin the material deposited was sandy and is shown as Tacubaya Alluvial material on Figure 3.1. Overlying the Tacubaya are materials referred to as the Becerra, strata of alluvium and volcanic dust, the Totoltsingo, composed mainly of calcareous silts, and fill, containing many remains of previous human habitation dating back over 3,000 years. Figure 3.2 shows a typical section through the upper soil layers in central Mexico City. Volcanic activity is still present in the area with the two most recent eruptions being 2400 years ago and in 1920.

3.1.2 Regional Variations

It is established practice that the area within and around Mexico City has been subdivided into three zones, namely:-

The Hill Zone (Zona de Las Lomas)
The Transition Zone (Zona de Transicion)
The Lake Zone (Zona del Lago)

These are shown on Figure 3.1 and are discussed below.

The Hill Zone includes the hilly areas around Mexico City. It is made up of either volcanic materials (eg the Ciudad Universidad site) or areas where the Tarango outcrop. They consist of volcanic rocks or dense sands and silts.

The Transition Zone is the area between the Hill Zone and the Lake Zone and includes areas which previously formed the shores of the lake. A variable thickness of the Tacubaya clay is found on this zone.

The Lake Zone is the area previously occupied by the lake. Towards the centre of the Lake Zone is Texcoco Lake which is all that remains of the original lake area. The reduction in size of the lake is basically due to the influence of man with various areas being infilled and drained. The detailed stratigraphy of the Lake Zone is described in the next section.

3.1.3 Stratigraphy of the Lake Zone

As will be discussed later most of the damage to structures during the earthquake was limited to the Lake Zone. Marsal and Mazari (1957) reported the findings of an extensive borehole survey carried out in and around the city. They showed that the ground consisted of the following:-

Stratum		Thickness (m)
FILL	clays and sands with archaeological remains	0 to 10
Totoltsingo & Becerra	clayey silty SANDS	about 4
Tacubaya	extremely compressible soft to firm lacustrine CLAY interbedded with thin sand lenses.	20 to 30

Tarango	medium dense clayey SAND;	3
	less compressible CLAY;	4 to 14
	medium dense becoming dense GRAVELS and SANDS with layers of silt or sandy clay	

A log of a typical borehole in the central part of the city is shown on Figure 3.2. It can be seen that the Tacubaya clay material starts at about 6m below ground level and is about 25m thick at this location. The thickness of this layer at various other locations in the City is presented on Figure 3.1 where it can be seen that the layer becomes thicker to the east. The properties of the Tacubaya and Tarango are given in the following sections.

3.1.4 Soil Properties

a) Tacubaya Clays

These lacustrine clays are derived from volcanic ash and consist of montmorillinite and illite. They are described as being extremely compressible and as can be seen on Figure 3.2 have high natural moisture contents of between 200 and 400%. Marsal (1975) has shown that most mechanical properties of the clay can be related to the moisture contents and Figure 3.3 shows the summary from 11745 tests of various soil properties related to moisture content.

Figure 3.3a shows the interval of moisture contents (w_i) and the number of samples tested at each value. Initial void ratio varies linearly with w_i (Fig 3.3b) and the specific gravity tends to decrease slightly with w_i (Fig 3.3c). The Atterberg limits reveal a close relationship to w_i with the moisture content being marginally less than the liquid limit (Fig 3.4d). Unconfined compressive strengths from undisturbed and remoulded samples show the sensitivity to be about 8 (Fig 3.3, g and h). Angles of friction have been derived from consolidated undrained triaxial tests (see Fig 3.3i). A typical void ratio (e) versus pressure (p) curve from a one dimensional compression test together with the corresponding coefficient of compressibility a_v is shown in Figure 3.4j. This shows equivalent compressibility (m_v) values of up to 6m_/MN at the preconsolidation pressure which is generally equal to the in situ vertical effective stress. The compressibility index (C_c) versus w_i is shown on Figure 3.3k. The coefficient of consolidation C_v varies from about 3 to 7m_/year as shown on Figure 3.3m.

In situ standard penetration test results from the Tacubaya Clays are shown on Figure 3.2. As can be seen the uncorrected SPT blowcounts (blows/300mm) are generally 0 to 1. It is believed this is due to the test remoulding the clay and therefore reducing its strength to about 10kN/m^2 (Fig 3.3h). In situ vane tests show that the unconfined compression strength tests underestimate the undrained shear strength c_u (Marsal and Mazari, 1957). The vane tests give values of about 35 to 50 kN/m_ between 5 and 15m below ground level increasing to around 100kN/m_ at a depth of 20m. The EEFIT team brought back four undisturbed samples of Mexico City clay and quick undrained triaxial tests showed similar undrained stress strengths to those recorded by vane tests (see Table 3.1).

b) Tarango Sands and Clays

The clays near the top of the Tarango formation are similar to the Tacubaya Clays but with moisture contents reduced to between 100 and 200%. The values given on Figure 3.3k shows that this material will be significantly less compressible than the overlying strata.

The principal data available on the sands are SPT results, typical values of which are shown on Figure 3.2. The layer at 31m is often referred to as the "first hard stratum" and the layer at 41m the "second hard stratum". The SPT results show both layers can be considered as medium dense.

3.1.5 Dynamic properties of the soil

a) Tacubaya Clays

Martinez et al (1974) reported on a series of in situ shear wave and compression wave velocity tests in the Tacubaya Clays. They recorded compression wave velocities of about 900m/sec throughout and shear wave velocities of 37m/sec to a depth of 18m and 50m/sec below that depth. For an average bulk unit weight of the soil of 12.1kN/m^3, the small strain dynamic shear modulus value is 1740kN/m^2 for the upper 18m and 3290kN/m^2 for the deeper material. Poisson's ratio was measured to be just below 0.5 which is expected for a saturated clay deposit. Martinez et al (1974) also carried out a series of resonant column tests on the clay and measured lower shear modulus values which led them to conclude that sample disturbance was significant, especially for deeper samples.

As mentioned above, the EEFIT team brought four samples back to the UK. By kind permission of Professor Brown, they were tested in a repeated load triaxial test apparatus at the University of Nottingham. The samples were cut down to 75mm diameter and the vertical strain was measured directly on the samples using small LVDTs (linearly variable differential transformers) connected to small brass studs pressed into the sides of the specimens (see Plate 3.1). Each was subjected to a constant confining pressure approximately equal to the in situ horizontal total stress (see Table 3.1). A sinusoidal vertical stress with a frequency of 0.5Hz was then applied to the sample using a servo-controlled loading system (see Plate 3.2). The mean vertical stress was kept equal to the cell pressure and the amplitude was steadily increased until the strain capacity of the apparatus (±5% shear strain) was reached. The sample was then subjected to a conventional quick undrained triaxial compression test and the undrained shear strength c_u determined (see Table 3.1).

Results of shear modulus versus shear strain amplitude are given on Figure 3.4. As can be seen the shear modulus values are very similar to that measured by Martinez et al (1974). To establish whether the clay is rate dependent the cyclic load frequency was varied between 0.2 and 1Hz on one sample and no discernible difference in stiffness was observed. In addition on one sample the mean vertical stress was increased so that the vertical stress was always greater than the cell pressure and again the shear modulus was not noticeably affected.

To compare the shear modulus results with those of the other clays they have been plotted as a ratio of c_u together with typical ratios for other clays in Figure 3.5. The Mexico Clay is unusual in that its stiffness to strength ratio at low strain is much lower than for other clays. A similar exercise has been undertaken with the material damping which is represented as a damping ratio plotted against shear strain amplitude on Figure 3.6. The damping ratio was calculated assuming that the rules developed by Masing (Pyke, 1979) apply when constructing hysteresis loops. Again the Mexico Clay is showing markedly different results to those of other clays with very low damping being measured until large strains are reached.

The Mexico Clay therefore is different to most clays in that it has a lower stiffness and lower material damping than other clays with similar undrained shear strengths.

b) Tarango Deposits

No direct measurements of the Tarango deposit dynamic properties have been found. It is likely however, that the sands will comply with the standard relationships published by Seed and Idriss (1970) which give the damping and the ratio of shear modulus to small strain shear modulus for a range of shear strain amplitudes. To estimate the small strain shear modulus G_o, the relationship G_o = 5N MN/m^2, where N is the SPT blowcount/300mm, would probably give reasonable values based on measurements at other sites.

The clays are likely to behave in a similar manner to the overlying Tacubaya Clays providing a suitable increase in the c_u value is used. Figures 3.5 and 3.6 can be used to estimate the behaviour.

3.1.6 History of settlement in Mexico City

In this century water has been pumped from below Mexico City to augment both domestic and industrial water supplies. The pumping has caused an increase in effective stress in the compressible clays which consequently have settled. Figure 3.7 shows the build up of settlement with time at the Cathedral and Alameda Square. Figure 3.8 shows contours of settlement between 1891 and 1970. Figure 3.9 shows an overall plan of Mexico City and the Cathedral is at Site 20.

Casings sunk to various depths in borings show over which depths the settlements are occurring. A set adjacent to Alameda Square (Site 14, Figure 3.9) shows that in the period 1951 to 1959 the ground surface settled 1.2m, 0.92 m of which occurred between 0 and 50 m below ground level and 0.28m occurred between 80 and 160m below ground level. At the Juarez housing estate (Site 83, Figure 3.9) between the same period the ground surface settled 1.3m and at 40m below ground level only 0.05m settlement was observed (see Figure 3.24). Therefore, as expected, the upper compressible clay layers are largely responsible for the settlement.

3.2 DISTRIBUTION OF DAMAGE WITHIN THE CITY

3.2.1 Introduction

The EEFIT team were only in the City for 12 days and there was insufficient time for a systematic study of the distribution of damage to be made. The team did, however, record the locations of any building which they had seen had substantially collapsed. The site location numbers shown on Figure 3.9 give a fair indication to the distribution of the failures. Meli et al (1985) have produced a more systematic map and this is shown on Figure 3.10.

It is noticeable that the region of damage is generally in the Lake Zone with very few failures out of that area. The distribution of damage within the Lake Zone is clearly not uniform but the impression formed by the EEFIT team was that the damage occurred where there were tall buildings rather than some areas being unaffected and other areas badly affected.

This impression was confirmed by Martin Degg, a research student at the Geography Department of the University of Nottingham, who spent seven weeks in Mexico City working on behalf of the Reinsurance Offices Association (ROA). He carried out a systematic study of a large area of the City outlined in Figure 3.11 and his observations are summarised in the following sections.

3.2.2 Objectives of the Nottingham University survey

The main objectives of the damage survey were:

1) To define the nature and distribution of damage within Mexico City by category of building.

2) To determine as precisely as possible those factors responsible for controlling the distribution of damage experienced in the City.

3) To examine the vulnerability of the different types of construction found in the City to the type of ground shaking experienced during the earthquake.

A total of 6 weeks was spent on a detailed damage survey, in addition to meeting representatives of various Mexico-based insurance/reinsurance companies and members of several Institutes at UNAM. The following sections give a brief account of the results of the damage survey.

3.2.3 General Description of Damage Distribution

From articles and newspaper reports published shortly after the earthquake, it was possible to outline the approximate area of greatest intensity of damage. This was restricted to the western part of the Lake Zone, with damage in the adjacent Lake Zone areas to the north, south and east being very sporadic. It is interesting to note that this area of the lake bed was also the part worst affected during the 1957 Mexican earthquake. Figure 3.12 compares the area of the major damage experienced in 1985 with that which was affected in 1957. The close correspondence between the two areas would seem to suggest a greater vulnerability of the structures in this part of the city to the type of ground shaking experienced in the 1957 and 1985 earthquakes.

Figure 3.11 shows that most of the serious earthquake damage occurred within a radius of 2 to 4 kilometres from Alameda Square. The residential zone of Tlatelolco, the National Medical Centre and the General Hospital are also indicated.

3.2.4 Field Procedure

The area in which a detailed survey of the distribution of earthquake damage was carried out is outlined in Figure 3.11. It can be seen that this encompasses a large part of the zone of heaviest damage. The limits of the investigation area were deliberately extended off the Lake Zone in a westerly direction to include parts of the Transition and Hill Zones. The purpose of this was to establish how strong a controlling influence subsoil conditions exerted on the distribution of damage, and to allow comparisons to be drawn between the relative performances of buildings in the different Zones.

Within the area of investigation a census of damaged buildings was made. All buildings showing evidence of damage when viewed from the street were marked upon maps and a note was taken of their type and height of construction. It was unfortunately not possible to inspect the buildings from the interior, and so it is inevitable that a number of damaged buildings have not been detected in the survey. It is however reasonable to assume that any errors arising as a result of this limitation will apply uniformly across the area of survey, and that the overall pattern of damage distribution will not have been affected by any such omission.

Each building was assigned to a damage category according to the severity of observed damage. Five categories were used:

1) Total Collapse
2) Partial Collapse
3) Heavy Damage
4) Moderate Damage
5) Light Damage

The first two categories are self-explanatory. Buildings that had been demolished by the time of the survey were deemed to be in category 1. Buildings in the third category remained standing after the earthquake but showed significant structural damage. Following the emergency introduction of a stricter building code in October 1985, it now seems likely that many of the structures showing this class of damage will have to be demolished simply because of the great expense involved in trying to repair them to conform to the new standards. Buildings in the fourth category showed light external structural damage, whereas those of the fifth category showed only minor damage that was largely superficial. Based on the observations of the initial EEFIT mission, many of these buildings probably suffered more severe internal damage.

It is most important to note that all evaluations were performed by the same investigator, thereby ensuring uniformity in the application of the five categories.

3.2.5 Damage Distribution in Detail

Within the area of investigation, the damaged buildings are so numerous that it has unfortunately not been possible to produce a legible small scale map showing all of the five damage classes. It has therefore been necessary to plot each damage class upon a separate map.

Figure 3.13 shows the location of those buildings with light damage, whilst at the other end of the scale, Figure 3.14 shows partially or totally collapsed buildings. These three classes have been chosen to illustrate the nature of the damage distribution as they represent extremes in the overall pattern.

The light damage class is the largest in the five, and is the one that is most evenly and widely distributed across the area of investigation. The relationship between the spatial distribution of lightly damaged buildings and the soil zones referred to in Section 3.1.2 in the area is highlighted in Figure 3.13. It is apparent that only a small number of buildings with light damage occur in the Hill and Transition Zones. Of the 575 buildings in the study area showing this class of damage, only 11 in fact occur outside the Lake Zone.

Buildings with categories of damage other than light are restricted exclusively to the Lake Zone. With each successive damage class, the area distribution of affected buildings is reduced slightly until the situation shown on Figure 3.14 is arrived at in the case of buildings that experienced failure.

3.2.6 Damage Statistics

The statistics relating to all the damaged buildings in the study area are summarised in Table 3.2. Figures pertaining to collapsed buildings have been omitted as it was frequently not possible to determine the former elevation of a totally collapsed structure.

The table shows that three construction types predominate in the area of study:

1) Concrete frame buildings, usually with brick infill panels or with glass curtain walling.
2) Buildings with brick load-bearing walls.
3) Buildings with predominantly stone masonry construction.

Few buildings belonging to categories 2 and 3 exceed 5 storeys in height. Very few buildings of steel frame construction were identified though it is not always possible to identify these positively from an external inspection only.

As regards the vulnerability of particular construction types to damage, it would obviously be unwise to draw too many conclusions from the figures given in Table 3.2. This is because they relate only to damaged buildings, and do not give any indication of numbers not affected by the earthquake. Perhaps the most useful statistics that can be derived from the results are those shown in Table 3.3. For each of the dominant construction types affected by the earthquake, this table lists the percentage of buildings of a particular height showing a particular class of damage. It is evident that as far as several of the damage classes are concerned, there are quite significant differences between buildings of an identical construction type but different heights. This is highlighted by the curve in Figure 3.15 which shows that of those 12 to 14 storey concrete frame buildings damaged in the earthquake, 44% experienced the heavy class of damage. In contrast, only 7% of the affected buildings less than 3 storeys in height suffered such damage. Generally, it appears that concrete frame buildings in the height range of 9 to 17 storeys were more likely to experience the heavy category of damage than buildings of lesser or greater height.

3.2.7 Building Vulnerability to Shaking in the Area of Major Damage

In order to examine in detail the relative performances of different types of construction during the earthquake, it would be necessary to compare the damage figures obtained from the survey with an inventory of all the buildings that occur in the area of study. Unfortunately such an inventory could not be obtained and so as a substitute, a system of sampling by use of transects was devised.

a) Procedure

The location of the transects is shown in Figure 3.16. A total of five transects were taken across the Lake Zone, the area of major damage, these being positioned to provide as complete a sample coverage as possible. In addition, one transect was taken across the Transition Zone for comparative purposes.

Along each of the transects, a survey of the external characteristics of every building was made in the following information recorded:

1) Construction type - usually one of the four types referred to in Section 3.2.6.
2) Height of construction.
3) Whether damaged or not - no attempt was made to distinguish between damage classes.

It was hoped that the transects would enable meaningful overall percentage damage statistics to be determined for different types and heights of construction.

b) Factors influencing building vulnerability in the Lake Zone.

The results of the five transects taken in the Lake Zone have been combined and are shown in Table 3.4. For each construction type the percentage damage statistics for specified intervals of building height have been calculated, and these are illustrated graphically in Figure 3.17.

c) Construction Type

Figure 3.17 clearly illustrates that in buildings of similar heights, there are quite significant differences in damage percentages experienced by different construction types. In particular, for buildings between 1 and 5 storey in height concrete frame structures seem to have been most vulnerable to the ground shaking, with damages being successively reduced in buildings of brick load-bearing wall and stone masonry structure.

d) Height of Construction

In addition to the effects of construction type, Figure 3.17 highlights the marked effect of building height in controlling the vulnerability of buildings to damage during the earthquake. The damage curve for concrete frame structures shows that buildings in the height range of 6 to 20 storeys were particularly badly affected by the shaking, with highest incidence of damage occurring in those between 9 and 11 storeys.

e) Transect across the Transition Zone

The results of the transect taken across the Transition Zone are shown in Table 3.5. It can be seen that except for stone buildings, all the types and heights of construction observed along the Lake Zone transects also occur in this zone. This is very important, because it removes the possibility that the marked contrast in amount of damage experienced in the two zones is due to a difference in the types of construction found within them. These remarks apply to the superstructures; foundation design is known to be different in the two zones because of the different soil conditions.

3.2.8 <u>Building Height and the Area of Major Damage</u>

In the previous section, the greater vulnerability of medium to high rise structures to the types of ground motion experienced on the lake bed during the earthquake was demonstrated. It is now worth considering the extent to which the presence or absence of these buildings can be used to explain the overall distribution of major damage experienced in the Lake Zone.

Figure 3.18 shows three ranges of building height in and around the area worst affected by the earthquake. For each, the percentage of buildings over 6 storeys in height has been calculated, so as to provide an indication of the relative vulnerabilities of the different ranges to the type of ground shaking experience in the earthquake. It can be seen that the area of major damage is largely confined to Ranges I and II where the percentage of tall buildings is greatest, and that it is very much restricted in Ranges III where 98.5% of the buildings are less than 6 storeys in height. Unfortunately no data are available for the remainder of the Lake Zone, though it is known that beyond the margins of Range III buildings are predominantly low-rise.

It would therefore seem that within the Lake Zone, the area of maximum damage is confined to that part of the city where the density of medium to high rise construction is greatest.

3.3 EARTHQUAKE EFFECTS ON FOUNDATIONS

3.3.1 Review of Foundation Types

Because of the difficult soil conditions and the range of building types built in Mexico City, various types of foundation are used. The types and their performance under static loading are as follows.

a) Pad Footings

Pad footings are used for low rise buildings. Standard practice is to use allowable bearing pressures of $50kN/m^2$ where buildings have previously existed and $30kN/m^2$ where virgin loading is occurring (Marsal and Mazari, 1957). This practice has been adopted as the compressibility of the upper clays have been found to reduce significantly with preloading.

Many large old colonial buildings were built on pads and it is obvious from visual inspection that many of these buildings have settled dramatically relative to the ground surface. The Guadalupe Cathedral is an example of this and is shown on Plate 3.3.

b) Rafts

To limit differential settlement many larger buildings are founded on rafts which usually include deep beams for added rigidity. Depending on the size of the building various degrees of compensation are employed. Compensation involves forming one or more basement levels so that the mass of soil excavated partially or completely balances the mass of the building being built.

There are many construction difficulties associated with forming compensated raft foundations. With the soft clay material surrounding the excavation, slope stability problems frequently arise. Associated heave of the base of the excavation is unavoidable and is typically about 1m for an 11m excavation. Special measures must be used to protect existing structures adjacent to the excavations.

Settlement records of structures on rafts show that for a typical 10 storey structure at least 1m settlement relative to the ground surface is expected if compensation is not used. Up to 1m is expected with partial compensation and some small emergence often occurs with full compensation. Emergence is the term used to describe the condition where the ground surface settles more than the structure.

c) Piles

Piles are used for many types of buildings. The conventional type of pile in use in Mexico City is driven or jacked timber or, more recently, precast concrete piles. In the central area of the City it is common practice to jack or drive piles to the "first hard stratum" at about 30m below ground level. For larger buildings (greater than 25 storeys) it is common to drive the piles to the "second hard stratum".

Not surprisingly, piled buildings can suffer badly from emergence. The Independence Monument (Site 54, Figure 3.9) was constructed in 1919 and is founded on 4652 timber piles driven to 23m below ground level. Since construction the monument has emerged by about 2m as shown on Plate 3.4. To minimise the emergence several measures have been attempted as follows:-

i) Friction piles only: The piles are stopped above the first hard stratum to attempt to achieve a balance between excessive settlement and emergence.

ii) Piles failing in end bearing: piles with reduced area at their toes (see Figure 3.2) are driven to the "first hard stratum". As the ground surface settles the extra load on the piles leads to failure of the sand at their toe causing them to penetrate the sand.

iii) Control piles: (see Figure 3.19) A beam is inserted over the pile with an adjustment facility so the building can be raised or lowered relative to the pile head.

iv) Interlaced piles: (see Figure 3.20) Friction piles are interlaced with fixed piles that bear on the hard stratum. The fixed piles are cut down so that only the friction piles support the structure.

With all types of piles it is standard practice to make the lowest slab a rigid grillage or solid raft system to ensure that relative settlement across the building is minimised. It should also be noted, that where piles are used in conjunction with basements the heave associated with the excavation stage can be reduced by up to 5 times if the piles are driven prior to the excavation.

d) Other Foundation Types

i) In medium rise structures a raft is often used as the principal mode of support combined with a few piles that act to resist excessive settlement.

ii) The Latin American Tower is a major 43 storey structure adjacent to Alameda Square (Site 22, Figure 3.9). This structure is founded on piles to the "second hard stratum" as the prime means of support combined with a grillage raft about 13m below ground level. In addition, to control and allow for differential settlement of the structure relative to the ground surface, the following measures were added.

   - The pore pressure under the raft is maintained at $100kN/m^2$ to reduce the effective stress in the soil. This is achieved by having a gravel layer under the slab connected to a supply tank just below street level (Zeevaert, 1983).

   - The ground floor slab can be raised or lowered independently of the main structure to match the level of the pavement. As yet this has not been necessary.

3.3.2 Foundation behaviour during the earthquake

For many buildings there was no evidence of foundation failure. Where it was observed it was generally to such a limited extent that no damage was induced to the structure. The types of failure and associated effects were as follows:-

a) Emergence

Many piled buildings exhibited signs of emergence occurring during the earthquake. This was generally less than 100mm (see Plate 3.5) and probably would have occurred eventually without the earthquake.

b) Settlement

Several buildings settled excessively often inducing a tilt to the structure. This generally occurred to older buildings and it is likely they were founded on raft foundations but this can not be stated with any confidence. Plates 3.6 and 3.7 show examples where this effect has occurred. It is likely that the rocking motion induced by the earthquake caused partial bearing failure of the foundation soil. In no instance was it clear that soil had heaved up adjacent to the buildings but accurate surveying before and after the earthquake would be required to resolve whether this had occurred or not. Two buildings which suffered from this were inspected in detail and these are described in the next section.

c) Failure

There was only one instance of foundation failure observed that was caused by the onset of a mechanism within the soil. This consisted of a bearing capacity failure of a building (Site 78, Figure 3.9) which was founded on a raft at about 3m below ground level. Plates 3.8 and 3.9 show views of the building after the failure. It is not known precisely how many storeys the building had prior to the earthquake, but it was probably ten. It can be seen from the plates that the whole building rotated during the earthquake. Whether the upper storeys collapsed because of the foundation failure or had collapsed prior to the foundation failure is not known.

A few piles were revealed by the collapse (see Plate 3.9) but it is not considered they had a significant role in the collapse. The piles were probably added as a settlement control measure only and the main form of support to the structure was the raft on the underside of the basement.

3.3.3 Case Studies

The EEFIT team had the opportunity to study two piled buildings that had suffered from excessive settlement. These will be discussed in turn.

a) The Insituto de Seguridad y Servicios Sociales de Los Trabajadores de Estado Building (Social Security Building)

This building was sixteen storeys high with a two level basement and was in the central area of the City (Site 15,

Figure 3.9). The borehole illustrated on Figure 3.2 was sunk on the site. The piles used are also shown on the figure and are of the type that were designed to penetrate the "first hard stratum" as the ground surface settled. From outside there was little evidence of damage to the building as shown on Plate 3.10. At the north east corner of the building however the glazing bars were badly buckled and it was clear that relative movement between the building and the pavement had occurred at this point. Inside the basement it was apparent that the floor slab had come up and the building columns and piles had settled (see Plate 3.11). A 3m deep grillage of beams connected the pile caps and therefore no dramatic relative settlement, causing distortion, had occurred. The floor slab had failed however and come free of the grillage and the columns. It was clear that the earthquake loads on the building had induced an overturning force which had failed the piles on one side of the building. Once the piles failed the only other part of the structure that could resist the force was the floor slab. The floor slab however had not been designed to carry loads greater than the conventional imposed live loading. Therefore the floor slab failed and the building rotated. The remedial measures being discussed at the time comprised adding further piles to the settled area of the building. Eventually the settlement of the ground would cause the other side to settle and the building would become level. At that time it was proposed to disconnect the extra piles and the building would continue to settle with the ground as originally intended. The ground slab should also be upgraded.

b) Office Building in Iztacalco

This building was an eleven storey building with a single level basement and was about 6km south east of the city centre (Site 87, Figure 3.9). At this location the Tacubaya Clays are about 40m thick. The building was founded on interlaced piles (see Figure 3.20) which were 500mm diameter precast concrete driven piles. The friction piles were 36m deep and the fixed piles started at 10m below ground level. The building also had a grillage of beams joining the pile caps. In addition a raft slab was cast at the underside of the grillage leaving a void between the raft and the underside of the basement slab. The voids should have been dry but many were suspected to be filled with water. During the earthquake the west side of the building settled by about 1m (see Plate 3.12). In the aftershock 36 hours after the main quake the other side of the building settled by about 0.4m causing the building to become more level. A witness to the aftershock reported that a large amount of air came out from under the basement when the building settled suddenly. There was no sign of any heave to the ground around the building and within the building the basement slab was undamaged. The slab under the grillage could not be inspected however and it is possible that the slab had failed.

Immediately adjacent to the eleven storey building was a four storey building which was an adjoining car park to the taller building. This building was founded on the same type of piles but as it was shorter only the friction piles were used.

Again a grillage of beams was used to join the pile caps but in this instance a raft slab was not used and the basement slab was in contact with the soil. This four storey building did not settle during the earthquakes leading to problems where the two buildings met (see Plates 3.13 and 3.14). There was no major structural damage to either of the buildings.

3.4 EARTHQUAKE EFFECTS ON BUILDING SUPERSTRUCTURES

3.4.1 Building Types in the Lake Zone

The majority of medium to high rise construction is in reinforced concrete, and mostly consists of braced or unbraced beam and column frames. In many cases, the external frames (whether braced or unbraced) have rigid infills of bricks or blockwork. Shear wall construction as the principal seismic resisting element is not a typical form of construction.

Steel frame buildings are found, though they are not common. A number of older steel buildings have riveted connections.

Masonry construction, often of poor quality, is common in low rise older construction and there are also many masonry monumental buildings, such as churches and theatres.

No timber construction was observed.

3.4.2 Failures in Reinforced Concrete Moment Frame Buildings

A number of different modes of failure were observed in r.c. moment frame buildings as follows:

(i) "Top down" collapse of upper storeys, observed in both older (Plate 3.15) and more modern (Plate 3.16) buildings. Typically up to 6 of the top storeys collapsed in buildings up to 12 storeys in height.

An example of a top down collapse was experienced in the telephone exchange building, which is described in more detail in Section 3.4.10(d).

(ii) Collapse of intermediate floors. In some cases, this may have been associated with buffeting damage from adjacent buildings (Plate 3.17), but was also observed in isolated buildings (Plate 3.18). It may be postulated that top down collapse was initiated by this form of intermediate failure, which then spread to upper storeys, but there is no direct evidence for this.

(iii) Several cases of collapse of a ground floor were observed (Plate 3.19); at least some of these were associated with "soft ground storeys".

(iv) Total collapse (Plate 3.20).

(v) Buffeting damage between adjacent structures. Local damage at points of contact was quite widely observed, but many structures of dissimilar height survived close proximity (Plate 3.21) and collapse due to buffetting, although observed, did not seem to be widespread.

A number of other observers (e.g. Rosenblueth & Meli, 1986) report punching shear failure of columns through flat slabs as occurring in half a dozen buildings, but these were not inspected by EEFIT.

3.4.3 Failures in Reinforced Concrete Braced Frame Buildings

Reinforced concrete moment frames with concrete X-bracing in the short direction is a commonly observed form of construction in Mexico City. Usually, the external panels are rigidly in-filled with unreinforced brickwork, which were observed being used as permanent soffit formwork for the concrete bracing in some buildings under construction.

Although distress in such buildings was widely observed, the only collapse recorded by the EEFIT team was in the 15 storey Nuevo Leon building in the Tlatelolco housing estate in the north east of the city (Plate 3.22). This building is described in more detail in Section 3.4.10(a).

It is interesting that both this building and the K-braced steel tower at Pino Suarez (Plate 3.33) failed near the base and fell <u>over</u> - i.e. underwent significant rotation, whereas most other collapsed buildings, with moment frames, fell <u>down</u> - i.e. flattened without significant rotation.

An interesting non-failure was observed in a 7 storey X-braced car park with an open ground storey (Plate 3.23). This would have been expected to be highly vulnerable, being situated in an area of heavy damage, with a poor structural form, and in a height range which generally suffered badly. No damage of any sort could be found from an external inspection, which included close examination of the ground floor columns. No explanation is offered, though it may serve as a warning against reading too much into isolated examples of non-failure.

3.4.4 Failure in Reinforced Concrete Shear Wall Buildings

Reinforced concrete shear walls as the principal means of resistance to seismic loads were only found in one building described in more detail in Section 3.4.10(e). This is understood to have been originally of moment frame construction, which, after earthquake damage in 1981, was strengthened by shear walls. It appeared from an external examination to have survived the 1985 earthquake well, though the warning at the end of section 3.4.3 against reading too much into isolated non-failures should be remembered.

Internal examinations were made of three moment frame buildings (Sites 15, 86, 93 on Figure 3.9) which had concrete shear wall cores which clearly did not form the main element of seismic resistance, and one moment frame building (Site 58) which had an isolated shear wall along part of one end. In all four buildings, the damage to the r.c. moment frame structure appeared slight to negligible, but there were major horizontal and vertical cracks in the shear walls in all four cases (Plate 3.24) but no collapse. The lack of local collapse may have been partly due to support from the non-failing moment frame, but the apparent stability after failure of these shear walls is a noteworthy contrast to the behaviour of many moment frame structures.

3.4.5 Failures in Steel Frame Buildings

Although not nearly as common as concrete structures, steel frame structures of varying ages were observed.

Total collapse in an older 3 storey building with steel joists and brick bearing walls was observed (Plate 3.25). The failure appeared a classic example of lack of horizontal tie between joists and wall, which had separated, causing collapse.

Three collapses in medium rise (6-8 storey) older steel frame buildings were observed, and it is understood that there were other examples. Plate 3.26 shows the debris from a 7 storey building, part of the Televisa complex, which had riveted battened column members, and bolted connections. Failure appeared to have occurred in connections.

A 21 storey building, dating from the late 1970's, collapsed after structural failure at about 4th floor level. The structure was of welded steel, and had a K-braced central core. It is described in more detail in Section 3.4.10(b). No other collapses in recent steel frame buildings were positively identified by the EEFIT team.

Two very high steel frame buildings are sited in the Lake Zone. One, the 43 storey Latin American tower, was completed in the early 1950's. It consists of a riveted steel moment frame, with composite concrete floors. Damage in both the 1985 and previous earthquakes was trivial. The other very tall building, the PeMex tower of around 50 storeys, is also understood to be undamaged.

3.4.6 Non Failure in Engineered Building Structures

Although the majority of engineered building structures between 9 and 20 storeys on the lake bed zone suffered some degree of earthquake damage (Table 3.4), the incidence of serious structural damage - and, in particular, collapse - was smaller (Table 3.3). It is clear that a substantial minority of structures of this type performed satisfactorily. Plate 3.27 shows a view of the central area of the city, taken from the Latin American tower, which indicates that many high rise buildings survived.

3.4.7 Low Rise Brick and Blockwork Housing

A high proportion of the housing stock on the lake zone is in this form of construction, with heights of 1 to 3 storeys. Often, the quality of construction, and standard of maintenance is low. Adobe (mud brick) as well as fired brick is used.

Many examples of undamaged buildings of this sort were observed by the EEFIT team, even where of low quality and in areas of heavy damage to high rise buildings (Plate 3.28). In particular, two areas with large concentrations of adobe buildings were visited by the EEFIT team, and in neither was the damage significant. Isolated examples of damage were observed, but the general impression that low rise buildings fared well is confirmed by the low damage ratios in such buildings (Table 3.4).

One area is reported where extensive damage, including total collapse, occurred in adobe buildings - see Figure 3.10. This area was not visited by the EEFIT team, but it is understood that the buildings were of low quality and in particular that the bottom courses of brick had deteriorated due to dampness. The concentration of damage into a limited area needs explanation; it might be that for historical reasons, the quality of the housing stock was particularly low, or there might be some reason for an amplification of the motions in that area. The EEFIT team was not able to resolve this.

3.4.8 Monumental Buildings

There are many massive buildings on the Lake Zone, including churches dating back to the colonial era. In the older buildings, there are cases of very large settlements in these buildings, which have taken place over many years.

All the major monuments survived the earthquake without damage (e.g. Plate 3.29), and it was surprising that relatively slender appendages (turrets, belfreys and the like) also appeared quite unaffected. Solidly built masonry office buildings dating from the 19th and early 20th century also appeared to have suffered little or no damage.

3.4.9 Other Factors Influencing Earthquake Resistance

a) Quality of Building Construction

In general, the standards of construction in both failed and non-failed buildings appeared at least adequate, and though some examples of poor construction were observed, it seemed unlikely that this will prove a major factor in the scale of the damage.

b) Detailing of Reinforcement in r.c. Members

No steel detailing drawings of failed building were available to the EEFIT team. However, external inspection alone can often give some indication of the steel detailing in failed r.c. members. The impression gained by the EEFIT team was that in many cases, the quantity of confining steel in failed columns was less than that required by the Mexican code for high levels of ductility. These code requirements are similar to those of the United States code ACI 318-83, Appendix A. Although it seems likely that a more general adoption of high levels of confinement steel would have reduced damage levels, this factor should not be overemphasised, for the following reasons.

1) The quantity of confining steel will have only a minor impact on the places at which plastic hinges form in moment frames. In particular, high levels of confinement will not force the hinges to form (favourably) in beams rather than (unfavourably) in columns.

2) Even with high levels of confinement, concrete columns display only limited ductility.

c) Damage from Previous Earthquakes

Damaging earthquakes were experienced in Mexico City in 1957 (when there were a number of collapses of engineered structures) 1979 (where there was widespread damage and 1 collapse) and 1981 (no collapses, but significant damage). Buildings experiencing these events may have been weakened, and there are reports that standards of post earthquake repair were not always very high. The Nuevo Leon (Section 3.4.10(a)), Pino Suarez (Section 3.4.10(b)) and Benito Juarez (Section 3.4.10(c)) buildings are all reported to have been damaged during 1979, and the subsequent collapses may have been associated with this damage.

It is understood that the Mexican national university UNAM is assembling detailed data on this aspect, on which the EEFIT does not have sufficient information to comment. Previous damage is often found to correlate with earthquake induced collapse (eg. EEFIT Chile report, 1986) and this earthquake is likely to prove no exception. However, it should be remembered, that significant collapses occurred in post 1981 buildings which could not have been affected by previous earthquakes.

The influence of previous earthquakes on performance serves as a reminder that seismic damage is cumulative, and that earthquakes of long duration, such as the 1985 event, are particularly damaging.

d) Damage Due to Previous Settlement

Previous long term differential settlements can weaken structures, and have been associated with earthquake damage (eg. EEFIT Liege report, 1984, Donovan, 1985). Large settlements are endemic in Mexico City, but the impression of the EEFIT team was that these were rarely associated with significant structural damage since the structures are typically much stiffer than the foundations, and hence the former settle as a rigid body without experiencing major distress. The buildings that the EEFIT team inspected which had settled significantly in the earthquake rarely had significant damage to their main seismic resisting structural elements. Therefore, it seems unlikely that previous settlements had a major influence on failures in the earthquake. In particular, the settlement problems reported at the Tlatelolco housing estate (Section 3.4.10(a)) were probably not a major factor in the collapses experienced there.

e) Gross Overloading of Floors

Some reports (e.g. Rosenblueth and Meli, 1986) suggest that failures in some structures may have been aggravated by gross overloading, for example by the presence of industrial equipment (eg. for clothing manufacture) for which the structures were not designed. The EEFIT team gathered no first hand information on this. Although it may well have been a factor in some collapses, it is unlikely to be connected with failures in buildings so recently completed they were not occupied at the time of the earthquake (eg. Banco de Mexico and Social Security buildings).

As described in Section 5.2, the emergency regulations introduced after the earthquake have doubled design live loadings in office buildings.

f) Plan and Elevational Eccentricities

Rosenblueth and Meli (1986) report that 42% of the buildings that suffered collapse or severe damage were on corner sites. Since buildings on corner sites might be expected to have more open facades fronting onto the two streets than the corresponding facades at the rear, their centre of stiffness is likely to have been eccentric from their centre of mass and so significant torsional loading would have resulted. Rosenblueth and Meli attribute the poor performance of such buildings to these effects. These eccentricities are more likely to arise from secondary or non-structural elements (glazing, cladding etc.) than from eccentricity in the primary structure. An additional contribution to the vulnerability of corner sites may have been the non-symmetric buffeting that they received from adjacent buildings.

Chandler (1986) provides a more general analysis of torsional response in the Mexico earthquake, and concludes it is likely to have been a significant contributor to damage.

3.4.10 Case Studies

A number of buildings where the EEFIT team was able to gain access, or could inspect in a more detailed way than in general are discussed below.

(a) Nuevo Leon Building, Tlatelolco Estate (Site 4, Figure 3.9)

The Tlatelolco housing estate covers an area of some 150 hectares, and consists of about 100 reinforced concrete apartment blocks. Most of these take the form of high rise linear structures, typically 15 storeys high. The estate dates from 1961 and was part of a government slum clearance scheme. Structural consultants of international repute were associated with the scheme.

Settlement problems were reported in many blocks, and underpinning was carried out on some, including, it is understood, the Nuevo Leon building. Some buildings, including the Nuevo Leon, were damaged during the 1979 earthquake; details of the repairs are not known.

Some 24 of the Tlatelolco blocks are reported to have been significantly damaged by the 1985 event. The only collapse occurred in the Nuevo Leon building described in more detail below. One other block (Plate 3.30) of apparently very similar structural form was inspected, which had no apparent superficial damage. The most obvious external damage in other blocks was to unreinforced external cladding - see Plate 3.31.

A plan of the Nuevo Leon building is sketched in Figure 3.21. It consisted of three structurally separated blocks of dimension approximately 15m x 50m on plan, and 15 storeys (approx 50m) high. The 3 rows of columns at each end of each block were X-braced with reinforced concrete members in the short direction of the building. The structure between these braced end blocks was unbraced. There was a central access core, but photographs of the collapsed structure taken before arrival of the EEFIT team suggest this contributed little to stiffness or strength. From the same source, considerable sway in the end braced section in the longitudinal direction suggests there was no bracing in this direction.

Plate 3.22 shows that deep fascia beams were present at each floor level along the entire front and back faces of the blocks, supported by relatively slender columns. An access corridor at every third floor, which was present on the front facade in the central part of the block, meant the central columns at this section were unstiffened. These columns were considerably distressed in the earthquake (see below), and it is interesting to note that strengthening had apparently been carried out on the corresponding columns in the block structurally similar to Nuevo Leon which did not collapse (Plate 3.30).

The minor axis of the building was oriented at S80°E, which is 20° east of the maximum direction of motion recorded at the SCT building (Section 2.2).

When the EEFIT team arrived, considerable demolition had been carried out on the central and north blocks, both of which collapsed. The end braced frames at the south end of the central block could be seen to have failed at ground floor level and the lower sections had rotated as a rigid body about the long axis of the block (Plate 3.22). There was no evidence of ground movement, and the firm conclusion was this braced tower had suffered a structural failure near ground level. Pictures taken soon after the collapse show that the middle of the central block between its braced end frames pancaked, while its northern braced frame had rotated southwards. The north block appeared to have suffered a more general rotation about its long axis.

There was considerable distress in the central columns at access floor level in the block still standing (Plate 3.32). The distress was probably due to excessive compression due to bending about the long axis. The adjacent distress seen in adjacent fascia beams (Plate 3.32) was probably in a non-structural upstand section, and so not particularly significant. All three blocks of Nuevo Leon, including the south block, have now been razed.

The conclusions (in most cases tentative) about the Nuevo Leon building are as follows:

(i) The collapse was certainly a result of superstructure not foundation failure.

(ii) The failure in the central block was probably initiated by a bending failure of the south braced tower about the building's long axis. Subsequent collapse (including the short axis bending of the north braced tower) was likely to have been triggered by this initial collapse.

(iii) The distress observed in the access corridor columns of the standing block was probably mainly due to bending about the block's long axis. Bending in this direction (it can be assumed) was designed to be resisted by floor diaphragm spanning between the braced end towers. Therefore the column distress is likely to be an indicator that these end towers were close to trouble, and was unlikely to have been the trigger for collapse.

(iv) No obvious signs of settlement could be seen in the standing block, although an adjacent block had tilted appreciably. Although differential settlement could have weakened the braced end towers, observations elsewhere in the city (Section 3.4.9(d)) suggest this is unlikely to have been significant.

(v) The ductility of concrete braced frames under earthquake loading will need careful examination, in the light of the proportioning and detailing used in Nuevo Leon. The restriction by the draft SEAOC requirements (SEAOC, 1985) on the use of concrete braced frames to buildings of 50m in height or less in areas of low seismicity, and their prohibition in areas of high seismicity seems sensible pending further review.

(vi) It is interesting to speculate whether the length and slenderness of the blocks played a part in their failure. Based on a shear modulus of $3500kN/m^2$, a density of $2000kg/m^3$, and a period of 2 seconds, the wave length of vertically propagating shear waves through the clay is around 80m. Horizontal wave lengths may have been of the same order (Martinez et al, 1974). The braced end towers at each end of the failed blocks were separated by approximately half this dimension, raising the possibility that their motions were out of phase, and that significant torsional oscillations may have built up. There is no direct evidence for this postulation.

(b) Pino Suarez Buildings (Site 61, Figure 3.9)

These buildings, which housed judiciary offices, dated from the late 1970's, and formed a group of 5, see Figure 3.22. The buildings are understood to have been damaged in 1979; details of repairs are not known.

In 1985, the southernmost 21 storey block suffered a structural failure at about 4th floor level and fell in a southerly direction, towards a vehicular underpass, crushing the adjacent 14 storey block - see Plate 3.33.

A close inspection was not possible, but the structural form of the failed 21 storey block appeared similar to that of the blocks still standing. These were formed of welded steel plate box columns, and Warren truss floor beams. A central core was braced with welded steel plate box members in a Chevron, or horizontal K pattern.

The three buildings still standing to the north of the collapsed building all showed considerable distress. In both 21 storey blocks, the outer columns at about second floor level appeared to have buckled locally in the box plates. Significant rotation could be seen in the beams in the unbraced bay west of the centre core (to the left, in Plate 3.33) in the block adjacent to the failed block, indicating significant sway.

The damage to the external cladding seen in Plate 3.33 was not due to the earthquake, but was caused by the initial stages of demolition. All remaining blocks have now been razed.

Some observations are as follows:

(i) The failure represents a rare, possibly unique, example, of failure of a modern welded steel frame building in an earthquake.

(ii) The failure was undoubtedly structural rather than geotechnical in origin. Overstressing in the columns near ground level and the capacity of the seam welds in the columns will be among the points to investigate.

(iii) The proximity to the underpass is unlikely to have played a significant part in the failure. Its influence is understood to have been investigated during original design, and found to be small, and the degree of damage to all the buildings tends to support this.

(iv) The ductility of Chevron braced structures will need to be examined. It is instructive that buildings with this as its primary form of earthquake resistance are limited to about 12 storeys or less in areas of high seismicity by the draft SEAOC proposals (1985). The corresponding limit for X braced steel structures in the same document is 15 storeys. These proposals will need examination in the light of the Pino Suarez failure.

(c) Benito Juarez Housing Estate (Sites 82-84, Figure 3.9).

A plan view of the housing estate, which dates from the early 1950's, is shown in Figure 3.23.

Building Type 1 (Plate 3.34) consisted of two structurally separate blocks, each 12 storeys high and about 50m long. The construction appeared to be reinforced concrete moment frame with infill panels. The NE block appeared to have suffered a complete failure at about 1st floor level. The SW block showed some signs of structural distress externally, but generally appeared to have survived quite well. There were no signs of excessive foundation movement.

There were four buildings of type 2, all built across a vehicular underpass. These buildings were seven storeys high. The northern block collapsed entirely, and had been largely removed by the time of EEFIT's visit; Plate 3.35 shows an earlier view. The other three blocks appeared to be only lightly damaged. Once again, the failure was clearly structural, and there were no excess foundation movements.

The 3 buildings of type 3 were of 12 storeys. There was evidence of quite serious structural damage in all of them. Half of the central block had collapsed totally around the central access tower (Plate 3.36). The failures were also structural in origin.

Some observations are as follows:

(i) As with the Pino Suarez and Nuevo Leon collapses, the failures were structural, and did not appear correlated either with proximity to the underpass, or to building orientation.

(ii) Figure 3.24 compares the general settlement in the area (Traces 3 & 4) with that in the northern building of type 3 (Trace 5). Traces 1 & 2 are from tubes to depths of 80m and 40m respectively, and all positions are located on Figure 3.23. It will be seen that the northern type 3 building was settling at about 50% more than the general rate between 1951 and 1958. About 3 months worth of settlement appear to have occurred in the 1957 earthquake. It would be interesting to know if this rate of settlement affected the type 2 buildings, straddling the underpass. It seems likely that the settlements did not play a major part in the collapses.

(iii) EEFIT received second hand reports that the type 3 buildings were damaged in previous earthquakes, and may not have been adequately repaired.

(d) Telephone Exchange Building (Site 27, Figure 3.9)

This 1950's building formed one of 3 on a restricted site. It had six storeys above ground, and one basement. The EEFIT team gained entrance through the good offices of the Royal Engineers 32 Field Squadron, who were engaged in shoring and demolition work. This work has been described by Webb (1986). The other two buildings on the site were not seriously affected.

The building's form was a reinforced concrete moment frame, founded on well maintained control piles (Figure 3.19) which allowed jacking after construction to compensate for settlement. The building appeared well constructed, with good quality concrete and reasonable quantities of reinforcement. The beam sizes remained constant throughout the building, whereas the column sizes reduced from 750mm square at basement level to 350mm square at roof level. The columns contained 40mm high yield bars. The floors were heavily loaded with telephone switchgear.

The top 4 storeys suffered a total collapse. The second floor (the highest surviving) showed some local distress, but this mainly was associated with impact from collapse of the upper floors. The remaining floors had very few signs of distress, and there were none in the basement. The telephone equipment in these floors remained operational.

Three aspects can be mentioned:

(i) The basic soundness of the building is attested by the non collapse of the lower floors, the difficulty found by the Royal Engineers in demolition and the general high standard of construction and maintenance.

(ii) There was no suggestion that the building was not designed for the very high imposed load from telephone switchgear, but this would need confirmation.

(iii) Welded sleeves were used as column bar couplers, and these appear to have performed well.

(e) Direccion General de Administracion de Personal (site 77, Figure 3.9)

This 12 storey building was originally of moment frame construction, and had been damaged by the 1981 earthquake. External concrete shear walls had subsequently been added to strengthen the building; they appeared to be on new foundations, and their layout, as far as could be determined, is shown in Figure 3.25.

Plate 3.37 shows the south and west facades of the building after the 1985 earthquake. About 20% of the glazing on the south face had broken (though other faces seemed much less damaged) and the shear wall on the south face had settled about 200mm. There was spalling in a corner column on the NE face at 4th floor level but this could have been due to the 1981 earthquake.

An internal examination was not made, but the strengthening appears to have been generally successful. The good workmanship and imaginative architectural detailing of the added shear walls are also noteworthy.

(f) Banco de Mexico (Site 7, Figure 3.9)

This 7 storey reinforced concrete structure was structurally complete at the time of the earthquake, and infill exterior panels had been started, but internal finishes and fitting appeared not to have started. The structure consisted of a heavy external frame of beams and columns. The slabs were of waffle construction, except at roof level, where they were flat slabs supported by orthogonal beams.

Plate 3.38 shows that in the east (nearest the camera) and north facades of the building, all the columns had sheared at 2nd and 3rd floor levels. The columns at roof level on the south facade also sheared similarly. There was considerable distress in the east facade columns in the top floor, and also in the east facade beams, particularly at 5th and 6th floor level.

Plate 3.39 shows the SE corner column at 6th floor level. Together with other details, it suggests that confining steel may have been inadequate.

(g) Office Building (Site 93, figure 3.9)

This 13 storey building had been recently completed and was unoccupied at the time of the earthquake. Its structure comprised r.c. columns supporting r.c. waffle slabs with central r.c. core. The building was divided into 3 structurally separated parts, on a single basement supported by end bearing piles 40m deep and about 400mm in diameter.

From the outside, there appeared to be very little damage, although there was some evidence of buffeting damage. Inside, there was extensive damage to columns and beams at the junction between the central core and the rest of the moment frame structure. A number of poor construction details were apparent, including inadequate cover, bunching of column bars at column corners, and column links spacing which varied from adequate to very sparce.

3.4.11 Damage to Non-structural Elements

a) External Infill Panels

Rigidly infilled blockwork panels are commonly seen in moment and braced frame r.c. buildings. Often, the panels are strengthened by secondary r.c. beams and columns (plate 3.40). Blockwork was observed being used as permanent shuttering for r.c. X bracing in some buildings under construction.

Some degree of damage to rigid infill panels was practically universal in 5 to 15 storey buildings. Damage was due both to inplane stresses and probably also out of plane stresses.

b) Glass Curtain Walling

This fared much better than the blockwork, and in general the only serious breakage appears to have occurred in buildings with major structural damage.

c) Internal Elements

The EEFIT team could gather much less information on this since it made internal inspections of relatively few structurally undamaged buildings. The 20 storey Sheraton hotel (Site 49, Figure 3.9) apparently had no structural damage, but suffered extensive internal cracking of plaster, damage to false ceilings, etc.

d) External Tanks

All four roof mounted r.c. water tanks fell from an eight storey block which suffered partial collapse (Plate 3.41). This included the tanks on the surviving parts of the building. The tanks were supported on 4, evidently inadequate, stub columns, which effectively formed a 'soft storey'.

Another dramatic roof tank failure (Plate 3.42) could be seen at the Centro Medico hospital (Site 85, Figure 3.9).

Very similar tank failures occurred in the 1985 Chile earthquake (EEFIT, 1986).

Surprising non failures could also be observed, for example, in a precariously balanced, lightweight tank (Plate 3.43).

3.5 EARTHQUAKE EFFECTS ON OTHER FACILITIES

3.5.1 Lifelines

a) Telephones

At least two telephone exchange buildings were badly damaged by the earthquake, which meant capacity was seriously affected. Despite this, calls within Mexico City were already possible when the EEFIT team arrived, 9 days after the event, and international calls were possible a week later. Telex communications were not affected.

A significant aspect was that all international calls were routed through the basement of a building whose top 4 floors collapsed (Section 3.4.10(d)). Fortunately, the relevant cabling survived, but the lack of redundancy in the design was clearly a poor design feature.

b) Water and Sewage Pipelines

Numerous breaks occurred in buried water supply pipelines, and between 15% to 25% of the population of Greater Mexico City was affected to some degree by inadequate (or in some cases absent) supply. Two major pipelines supplying the east of the city failed, and this area was badly affected. Distribution of clean water supply by tanker had been instituted by the authorities and was much in evidence during the EEFIT team's visit.

Sewage lines were reported less badly damaged, though since they were without full supply, they were not fully tested.

c) Roads and Bridges

There were no bridge failures in Mexico City, and though there were instances of road pavement damage, none appeared to have a major effect on traffic. The road system was most seriously affected by streets being closed because of unsafe buildings.

d) Airports

The international airport was operational a few hours after the earthquake, and other airports were similarly unaffected.

e) Metro

The metro system was also operational after a 24 hours closure for checking.

f) Electricity

A meeting with Ing. Juan Eibenschultz, a sub-director of the Comision Federal de Electricidad, (CFE) confirmed that whilst there were disruptions to electricity supplies, the power generation and transmission system did not suffer significant damage. Disruption to supply was mainly due to damage to distribution lines caused by collapsed buildings, and the loss of power was localised. Many relays were activated by the strong ground motion causing shutdown of generators and some insulators on transformers failed; no transmission structures failed. The figures given by Brune et al (1985) reproduced in Table 3.6 are considered to be a good indication of the limited effect the earthquake had on the power system in Mexico City. It was reported by CFE staff that the power generation system was normal within 24 hours.

g) Oil and Gas Pipelines

Rupture of a propane pipeline is reported to have caused a major explosion at the St Regis hotel. No other significant failures were reported. However, there is no general system of piped domestic gas supply in Mexico City.

3.5.2 Industrial Facilities

The industrial facilities in Mexico City are mostly situated to the north of the City, away from the Lake Zone. Industrial facilities may generally be classified as engineered structures and many major facilities to the north of the City are founded on rock. Ground motions here would probably have been similar to those recorded at the University, i.e. a frequency rich motion peaking around 1 to 2Hz and peak spectral accelerations of less than 0.12g.

Important facilities, such as power stations will generally have been designed, at least by equivalent static methods, for seismic base shear coefficients of between 0.05 and 0.10. The ductility demands on such structures outside the Lake Zone would therefore have been well within their capacity.

A detailed inspection was made of the Valle de Mexico power station situated approximately 15 miles north of the City. The main structures are founded on rock and were designed in the late 1960's for a base shear coefficient of 0.08. The major structures, switchyards and tanks were not damaged. The station was reported to have operated throughout the earthquake and tripped as demand fell suddenly when distribution became disrupted.

In the Lake Zone of the City, industrial facilities of any significant height appeared uncommon. Although in the time available a detailed survey was not possible, it is unlikely that a significant number of facilities would have fundamental frequencies below 1.0 Hz. From Figure 2.4 it can be seen that most facilities would therefore have been subjected to essentially rigid body accelerations of between 0.20g and 0.30g. This is a three or four fold increase on the accelerations suffered to the north but nevertheless, for engineered structures, a relatively modest level of excitation.

Plate 3.44 shows an electricity sub-station in the Lake Zone, which was about 150m from the Banco de Mexico Building described in Section 3.4.10 (f). This sub-station appeared to have suffered no damage due to the earthquake.

Damage was observed to a structure on top of the silos shown in Plate 3.45, again in the Lake Zone, but compared to the scale of damage observed in building structures this was minor.

The performance of the industrial facilities serves as a further important reminder that the effects of the earthquake were critically a function of the dynamic characteristics of individual structures. Industrial facilities were not seriously tested by this event, since they were not in resonance with the earthquake motions.

3.6 FIRE

Around 200 fires are reported by Dames & Moore (1985) to have broken out in the 24 hours after the earthquake, and there was a major propane gas explosion at the St. Regis hotel. Fire blackened buildings were visible in the city. However, there was no major conflagration, as in San Francisco, 1906 or Tokyo, 1922. This was probably due to the absence of wooden buildings. Also, domestic gas is supplied by tanker to roof mounted tanks, so extensive explosions from broken buried gas pipelines could not occur.

The fires, therefore, played no part in the structural damage, but they probably killed trapped people who might otherwise have been saved. The fire station headquarters building was badly damaged by the earthquake and the consequent delay in organising fire services may have had a serious effect in terms of human life.

3.7 EARTHQUAKE EFFECTS ON SOIL STRUCTURES

The only soil structures examined or discovered in Mexico City were various retaining walls to road underpasses. Very little damage was experienced by these structures. The only effect found was some slight movement of the top of an occasional retaining wall panel. The team did not see any major excavation in progress and therefore cannot comment on the performance of these.

A minor point that must be raised was the extensive damage to footpaths and especially kerb stones. In all the areas where tall buildings were present the kerb stones had been disturbed. In many suburban areas where there were only 1 to 2 storey structures again the kerb stones were consistently broken and displaced. The cost of replacing all these will be significant.

4.0 EFFECTS OF THE EARTHQUAKE OUTSIDE MEXICO CITY

4.1 INTRODUCTION

The damage in the epicentral region, particularly to non-engineered structures, was less than would be expected for a shallow reverse thrust earthquake of magnitude around 8. For example the Chile earthquake of March 1985 (Ms = 7.8) (EEFIT, 1986) caused extensive damage to adobe housing over a very wide area, in a way not experienced in Mexico.

Nevertheless, the epicentral region held many points of interest as follows:

a) An extensive array of modern digital accelerograms captured the event. Near field recordings for this type of earthquake are few in number.

b) Recently completed industrial facilites, designed to modern aseismic standards, were sited in the epicentral area.

c) There were also two sizeable dams within 100km of the epicentre.

The time and resources available to EEFIT were such that an extensive survey of the epicentral area was not possible. Two members of the team spent a day visiting the site of the industrial facilities of Lazaro Cardenas which was within 30km of the epicentre - see Figure 2.1. Two other members spent a day at Ciudad Guzman, situated about 500km west of Mexico City and 180km north west of the epicentre. It is understood to be the only inland town outside the capital with a significant degree of damage.

4.2 LAZARO CARDENAS

Lazaro Cardenas is a town with a population of about 150,000, situated on the Pacific coast of Mexico 270km north west of Acapulco (see Figure 2.1). The town lies on the delta of the River Balsas and has been the centre of an area of major industrial investment in recent years.

Close to the town lies the SICARTSA steel plant (see Figure 4.1). A steel mill originally built by a German contractor is operational and extensions are presently under construction by Japanese and British contractors. The island of Cayacal in the Balsas river is also being developed as an industrial area, with new berthing facilities, silos, process plant, and infrastructure. Not far from the island, a new fertilizer plant has been built, and coincidentally was to have been officially opened on the day of the earthquake.

At La Villita, the Comision Federal de Electricidad operate the Jose Maria Morelos hydro-electric scheme which supplies electricity to the industries in the area. There is a further flood control dam in the area operated by the Mexican Department of Hydraulic Resources.

In the town of Lazaro Cardenas there are a few medium height reinforced concrete frame apartments and hotels, but the majority of the domestic construction is of two to three storey insitu reinforced concrete frames with fired brick infill.

Figure 4.1 also shows the location of the Zacatula strong motion station, records of which are shown in Figure 2.3.

4.2.1 Building Structures

(a) General

EEFIT visited the town on 8th October 1985, 19 days after the event and spent 10 hours in the area. Normal business and social life had resumed.

There were reports of only a few casualties due to the earthquake and superficially the town appeared to have suffered little damage. However, as described in more detail below, the damage which occurred, although not as dramatic or life threatening as that in Mexico City, will have significant impact on the industrial economy of the area.

(b) Adobe Housing

Time precluded a full survey but there appeared to be little damage to such housing.

(c) Brick Housing

Isolated collapses did occur, as with the ground floor collapse shown in Plate 4.1 and there were some indications of relatively poor construction standards in this class of building (Plate 4.2). However, generally the damage was light and whilst some damage occurred to brick infill (Plate 4.3), a large number of fired brick 2 to 3 storey buildings with r.c. frames appeared undamaged.

(d) Engineered Buildings

There were few structures of this type within and close to Lazaro Cardenas. No structures of this type had collapsed but there were reports that a medium height building was badly damaged.

Modern multi-storey hotels at the holiday resort of Ixtapa, about 70km south east of Lazaro Cardenas were reported by others (Brune et al, 1985) to be severely damaged. A brief external inspection of these hotels indicated only superficial damage to glazing and infill (Plates 4.4 and 4.5). No collapses of this class of structure occurred.

4.2.2 Industrial Facilities and Dams

(a) Jose Maria Morelos Hydro-electric Dam

This dam is situated on the river Balsas about 20km north of Lazaro Cardenas (Figure 4.1). The dam is shown in Plate 4.6 and is an earth rockfill dam with a central impervious core and concrete cutoff, 60m high and with a crest length of over 400 metres. It is built on alluvial foundations. As shown in Figure 4.1 it is close to the Zacatula strong motion instrument.

The dam is reported to have suffered settlements at the crest of up to 300mm. Longitudinal cracks were visible at the crest, up to 300mm wide and 1500mm long, and small pre-cast concrete parapets had toppled (Plate 4.7). None of the concrete structures associated with the scheme were damaged (Plate 4.8) and the mechanical plant and switchgear were also undamaged.

This facility is operated by the Comision Federal de Electricidad who advised that it is intended to back analyse the performance of the dam using the Zacatula record.

(b) Flood Control Dam

A second dam in the area was damaged but time did not permit a visit by the EEFIT team. Reports indicated that the dam, which is operated by the Department of Hydraulic Resources suffered a rotational slip failure on the upstream face, which did not however cause any operational failure.

(c) SICARTSA Steel Mills

The location of the SICARTSA Steel Mills is shown in Figure 4.1.

Major extensions to the original German built SICARTSA facility are currently being constructed by Davy McKee of Sheffield and a Japanese contractor. It is understood that the design of all the facilities is to the SICARTSA Company Standards, which include significant seismic loadings and permit dynamic evaluation of seismic effects.

The Davy McKee contract is built on the delta of the River Balsas and ground conditions vary considerably. In the south there are sands, gravels and clays and foundations are piled. In the north of the site peat layers occur. Groundwater level is about one metre below ground level. Dewatering was being employed in the construction and this was interrupted by the earthquake.

The construction includes over 50,000m^3 of diaphragm walls which at the time of the earthquake were not capped (Plate 4.9). The walls had been inspected by Davy McKee site staff and their alignment checked. No damage occurred and there was only minor leakage at a small number of wall joints.

The steel rolling mill structure was complete and cranes installed prior to the earthquake, and cladding was in progress (Plate 4.10). Inspection by Davy McKee staff had revealed some minor damage to sheeting at movement joints, and a number of fasteners on the sheeting had popped. No structural damage or misalignment was observed.

The rolling mill was designed by equivalent static methods for base shear coefficients of 0.4 horizontally, 0.4 vertically, and a combination of 70% of both. Movement joints allowed for 100mm of seismic movement and special seismic design features included the following:

(a) piles were necked to reduce pilecap fixity

(b) the OHT crane had restraint devices to prevent the crane being dislodged in an earthquake

(c) the longitudinal braced bays were provided with additional portal frames in the lowest lift to increase ductility (see Plate 4.10).

The only significant damage on this site was to the partly completed single storey canteen shown in Plate 4.11. This reinforced concrete structure has a massive roof construction and at the time of the earthquake the brick infill at one end had been taken up to the roof, whereas elsewhere it stopped short. This appears to have produced considerable eccentricity of stiffness with the result that the roof rotated and failed the external columns. A similar, although not identical, structure nearby was not damaged by the earthquake.

Adjacent to the Davy McKee contract was another mill being constructed by Japanese contractors. This structure (Plate 4.12) was essentially complete, with cranes installed, but the base plates of the columns were not grouted and this resulted in local damage to the base plates, holding down bolts and foundation (Plate 4.13).

The steel chimney of the original facility was reported to be damaged but did not collapse.

(d) Cayacal Island Development

Cayacal Island is being developed as a major industrial area, although presently only a few facilities are completed.

Damage was observed to the silos shown in Plate 4.14 where the clear storey of the control structure on top of the silos had collapsed. The berthing facilities associated with this silo are shown in Plate 4.15 where damage to the pile heads was observed. The conveyor structure on the berth can also be seen to have collapsed.

Minor settlements to dock areas, and damage to the crane bogey was also observed, but by far the most significant damage was that to the bridge shown in Plate 4.16. This is one of a pair of bridges, both of which suffered severe damage to the columns just below the cross heads and settlement and rotation of the bridge abutments. The bridges appeared to be near collapse and crossing was controlled by the army to one vehicle at a time, carrying only the driver. As these bridges are the only access to the island, the industrial facilties there are essentially isolated until the bridges are replaced.

(e) Lifelines

No information on the performançe of lifelines was obtained.

4.3 CIUDAD GUZMAN

Ciudad Guzman is an isolated town with a population of around 60,000. It lies about 180km north west of the epicentre - see Figure 2.1. There is some light industry but nothing of major significance. Most of the housing stock is of single storey adobe construction, generally in poor condition. Recent housing is generally of fired brick in a light concrete frame. A town plan is shown in Figure 4.2.

26 people were killed by the earthquake in Ciudad Guzman, and 700-800 were injured. Some 2,000 families were made homeless,

As described in more detail below, most of the damage was to poor quality housing, and masonry churches were another notable casualty. However, buildings of any reasonable strength, including municipal offices, were generally unaffected.

EEFIT visited the town on 8th October, 19 days after the event, and spent 5 hours in the town. Normal business commerce had resumed, and life seemed fairly normal for most people. There were no tented encampments or shanty towns; temporary shelter was provided by relatives, or in local Catholic schools and government buildings.

4.3.1 Soil Conditions and Geology

Ciudad Guzman is in a similar geological setting to that of Mexico City in that there is a raised valley floor surrounded by volcanic mountains (see Plate 4.17). The major difference between Guzman and Mexico City is that Guzman is not situated on the valley floor but is on the western side slopes of the mountain range (see Plate 4.18). The soil is therefore sandy alluvial material that is being eroded by the water coming down from the mountain above the town. The town does extend on to the flat valley floor but there was no noticeable difference to the level of damage to the structures between this and the other part of the town. Damage distribution appeared correlated only to building type - i.e. weak adobe buildings suffered, other stronger buildings did not.

A feature that should be noted is that one of the active volcanoes in Mexico, Mt Colman, is situated only about 30km south west of the City and dominates the topography of the area.

4.3.2 Damage to Building Structures in Ciudad Guzman

a) Adobe Housing

Damage to single and two storey adobe housing was very widespread throughout Guzman, and it is understood that about half this housing stock was rendered uninhabitable. The damage appeared to be just as frequent in the sloping areas to the east, as it was in the lake shore areas to the west.

A common feature was collapse of the roof and front wall (Plate 4.19). The typical roof construction (Plate 4.20) was tiles supported by wooden poles, which were often seen as being rotten and attacked by insects (Plate 4.21).

b) Brick Housing

Buildings of reasonable strength survived well, even where next to collapsed adobe buildings (Plate 4.22). A large number of fired brick 2 storey buildings in r.c. frames were observed, which seemed undamaged (Plate 4.23). The floors consisted of shallow brick arches spanning between steel joists.

c) Engineered Buildings

A 3 storey r.c. building suffered a partial collapse (Plate 4.24). However, it appeared that only the upper two storeys were r.c., having been added to an older masonry structure. A corner column of the latter appeared to have failed bringing down the r.c. structure it supported.

A two storey school (J. Clemente Orosco) appeared to consist of a light r.c. frame with rigid brick infill panels. The structure appeared in no danger of collapse, but the infill panels in the long direction of the building (east-west) had all either totally fallen out during the earthquake or else had been so seriously damaged they had been completely removed (Plate 4.25). Evidence of reinforcing bars and restraining beams which connected the failed panels to the structure could be clearly seen.

Neither example could properly be described as serious structural damage to engineered buildings, and none of the other engineered structures in town were damaged, though there were few of them. These engineered structures included single storey steel industrial sheds and a steel guyed radio mast.

d) Churches

All five churches in Guzman were damaged to some extent. The most seriously affected ancient building was the main cathedral (Plate 4.26) which was closed to the public, and had suffered heavy loss of masonry, though no general collapse.

The church of Cristo Re, on the western slopes of the town, collapsed entirely, killing some 20 people. The structure had been totally razed by the time of the EEFIT visit. A pile of steel beams and reinforcing steel to one side of the site suggested that it may have had reinforced elements, but it is noteworthy that the reinforced concrete wall against which the debris was stacked was unscathed.

It was reported that the older churches had been damaged and repaired in many previous earthquakes.

4.3.3 Damage to Lifelines in Ciudad Guzman

Some breaks in water pipelines in the lake shore zone of the town were observed. According to a local teacher, however, water supplies were maintained, and there were no problems with sewage.

Electricity was cut immediately after the earthquake, but restored one day later. Telephones were unaffected, and our informant successfully telephoned Los Angeles, California, on 22nd September. Roads were unaffected except by demolition work on buildings, or by pipebreaks.

4.3.4 Damage between Guadalajara and Ciudad Guzman

The EEFIT team drove the 120km journey from Guadalajara to Guzman without seeing any signs of earthquake damage, although only one major town was passed.

According to a local teacher, two villages in the vicinity of Guzman - San Sebastian and San Andres - were damaged to some extent; but the EEFIT team did not visit them. From the same source, Chulhuapan, a village in the hills above Guzman, was unaffected.

5.0 STATUTORY REQUIREMENTS FOR ASEISMIC DESIGN IN MEXICO CITY

5.1 THE 1977 MEXICO CITY CODE

The loading provisions of the 1977 Mexico City code follow the standard pattern of many international codes. Thus, an equivalent static force analysis is permitted, whereby the seismic base shear depends on building use, structural ductility, structural period and soil conditions. The loading provisions are at least as up to date as the American code, UBC (1982), and in some respects, more so.

Three zones in the city are specified, corresponding to the Hill, Transition and Lake Zones described in Section 3.1.2.

In the Lake Zone, predominant forcing periods of up to 3.3 seconds are allowed for. The ultimate base shear coefficient for a 1Hz frequency building with ductile moment frames in the Lake Zone, is 4%, compared with 6.7% required by UBC (1982) for soft soil sites in Zone 4 (the most seismic areas of the USA, as defined by UBC). The equivalent figures for a 0.5Hz building are 4% (Mexico) and 4.7% (UBC). Rosenblueth (1979) states that 15%g peak ground acceleration was envisaged by the Mexico City code writers as having a 100 year return period in the Lake Zone, compared with 22%g recorded in September 1985, and 25%g envisaged for a 100 year return in Zone 4 of the USA.

Five levels of ductility are allowed by the code. The highest level, corresponding to a displacement ductility of 6, is only permitted for fully ductile moment resisting frames in steel or reinforced concrete, for which the requirements are based on, and similar to, current American requirements for fully ductile moment frames eg. ACI 318-83 Appendix A for reinforced concrete, and UBC (1982) section 2722 for steel.

For a ductility of 6, there are also requirements controlling the regularity of the structure which limit the divergence between required strength and capacity at any level.

The next level, corresponding to a displacement ductility of 4, is permitted for 'intermediate' moment resisting frames, which broadly conform to, or are slightly more stringent than the requirements of ACI 318-83 Appendix A9. The same level is also permitted in dual systems, consisting of shear or braced structures acting in combination with moment frames having at least 25% of the total lateral strength capacity. For both cases, there are similar, but less stringent, requirements for regularity as for the previous ductility level.

A displacement ductility of 2 is permitted for reinforced concrete, steel, wood or reinforced blockwork without special aseismic detailing. Unreinforced blockwork is assigned a ductility of 1.5, and otherwise unspecified systems a ductility of 1.

Unlike current Californian requirements for areas of high seismicity, there are no restrictions on the height of structures with limited ductility in the Lake Zone.

5.2 THE EMERGENCY REGULATIONS OF 18TH OCTOBER 1985

One month after the earthquake, emergency regulations were introduced, which modified certain aspects of the 1977 code. The regulations were directed both at new construction, and repair and retrofit of old construction. The most important changes introduced include the following:

- increases in basic seismic coefficients (lateral load factors) by 70%, for the Lake Zone with correspondingly smaller increases in the other zones

- the detailing requirements for the class of structures with highest ductility have been somewhat strengthened, and the associated ductility factor Q has been reduced from 6 to 4. This class now includes ductile moment frame structures in steel, concrete or timber, and also dual structures, where shear walls or braced fromes contribute up to 50% of the seismic resistance, in combination with moment frames. Previously, only pure steel or concrete moment frame structures were permitted in this class

- the detailing requirements for structures with intermediate ductility have also been slightly strengthened, and the ductility factor Q has been reduced from 4 to 3

- the requirements for structures in ordinary reinforced concrete or masonry, with low ductility, are unchanged, including the Q factor.

The overall effect of the increase in basic seismic coefficient and changes in ductility factor Q is that for medium to high rise buildings in the Lake Zone, with high and intermediate ductility, lateral force requirements have increased by a factor of (1.7 x 6/4) or 2.55, whereas for ordinary structures of low ductility, the factor of increase is 1.7.

Other important changes include

- doubling of design live loads in buildings destined for use as offices

- more detailed specification for infill walls designed to resist lateral forces, with provision for checking associated reaction forces in frame members.

- owners of buildings damaged in the earthquake are required to report the matter to the authorities. They are subsequently required to submit documentation to show the building has been strengthened to meet the new requirements, or they must demolish the damaged building.

These and other new provisions are currently being refined with a view to incorporation in a new edition of the full construction code due out in October 1986.

The emergency regulations are discussed in Section 7.3.4.

6.0 DISCUSSION OF THE GEOTECHNICAL ASPECTS OF THE EARTHQUAKE

There are two main areas where geotechnical aspects affected the response of structures to the earthquake. These are the modification of the lake bed soils to the free field motion and the soil structure interaction affecting the natural frequency of the structures.

6.1 FREE FIELD MOTION

The lake bed soils clearly had a major influence on the earthquake motion. The one measured ground motion on the lake bed which is discussed in this report (Section 2) shows that the soil filtered the motion to consist mainly of a 0.5Hz frequency and also amplified the motion by about a factor of 6. There has been much discussion about this effect with suggestions that basin effects (the shape of the surface of the underlying hard strata affecting the free field motion), surface waves and other factors may have contributed significantly to the observed behaviour. As the upper soft clay (Tacubaya) layers forming the Lake Zone are so thin (about 30m) compared with the width of the lake (about 15km) and as the edge of the Lake Zone is not distinct it is difficult to envisage basin effects being significant. Strong evidence in favour of the surface motion being mainly the result of vertically propagating waves is that only the horizontal and not the vertical motion was significantly modified by the soil. The horizontal motion at the surface resulting from vertically propagating shear waves would be noticeably affected as the low soil shear modulus would affect shear waves. However the vertical motion resulting from vertically propagating compression waves would not be affected as the saturated soils would appear to be very stiff to compression waves.

As a preliminary investigation into the effect of the lake bed soils on the motion EEFIT carried out a conventional one dimensional analysis using the computer programme SHAKE which models vertically propagating shear waves. The vertical profile modelled was that for the plaza de Republica, as shown on Figure 3.2 and the secant shear modulus and damping values were derived as discussed in Section 3.1.5. As no data were available for the soils deeper than 50m, two shear modulus values were used, namely $500MN/m^2$ and $5000MN/m^2$. A hand digitised record of a horizontal acceleration time history measured on the rock outcrop at the University (CU) was used for the input motion. It was assumed that this CU record represented the free field motion of a fictional outcrop of the underlying material. The aim was to compare the predicted free surface motions with those actually recorded at the Department of Transport (SCT) site on the lake zone, discussed in Section 2.2. The locations of Plaza de Republica, SCT and CU sites are shown on Figure 3.1.

The results of the analysis are shown on Figure 6.1. The response spectra published by UNAM (Prince et al, 1985) are shown for both the CU and the SCT sites. The response spectrum of the hand digitised record at the CU site is also shown and it can be seen that although there are many discrepancies, due to the digitisation process, the agreement with the UNAM spectum is adequate. The response spectra from the two predicted acceleration time histories at the surface are also shown on the figure. The only change to the input between the two predictions is the stiffness of the underlying material.

Clearly there are major discrepancies between the observed motion at SCT and that predicted by the one dimensional analysis. The principal difference is that the predictions are generally only about 60% of that measured. However, the overall shape of the prediction agrees well with that observed. It is likely that with better data and a full parametric study the differences between the predicted and measured response would be reduced.

6.2 SOIL STRUCTURE INTERACTION

The soft soils at Mexico will lower significantly the vertical, horizontal and rotational stiffness of the building foundations. The effect of this reduced stiffness on the fundamental frequency of a typical building has been studied. The foundation stiffness was allowed for by using the conventional stiffness equations for a rigid base on an elastic half space (Lambe & Whitman, 1969) using a soil shear modulus of 3,500kN/m^2. Calculations show that the fundamental frequency of a typical twelve storey building with one basement founded on a raft reduces from 1Hz assuming a fixed base to 0.4Hz using foundation stiffnesses appropriate to Mexico City. An equivalent damping ratio of about 7% was predicted for the soil structure system.

7.0 DISCUSSION OF STRUCTURAL ASPECTS OF THE EARTHQUAKE

7.1 INTRODUCTION

A definitive study of building damage would need to establish at least the following for a representative sample of buildings in the affected zones:-

a) Original design specification, including applicable code and degree of compliance, if possible.

b) Structural repair of building before the earthquake, including any previous earthquake damage or overload.

c) Detailed internal as well as external inspection.

d) Identification of non-damaged structures, in order to quantify failure rates in each category of structure.

Information on original design and structural repair was generally not available to the EEFIT Team, and although they were able to make a number of internal inspections, in the majority of cases, only an external inspection was possible.

Another factor making interpretation difficult was the variability in damage between similar adjacent buildings. A particular example was on the Benito Juarez housing estate where damage in identical, adjacent blocks varied between total collapse and apparently only superficial damage (Section 3.4.10(c)). Similar examples of variability are observed in many earthquakes (see for example, EEFIT Chile report, 1986).

Lack of comprehensive information means that definitive conclusions will have to await the painstaking investigations, back analyses and research that will be done in the months to come. Some observations based on the data available to the EEFIT team are given below.

7.2 DAMAGING CHARACTERISTICS OF THE STRONG MOTION RECORDS FROM THE MEXICO CITY LAKE ZONE

The strong periodicity of the lake zone motion led, as expected, to the damage being most severe in medium to high rise buildings. Based on empirical rules, resonance with the 0.25 - 0.5Hz earthquake frequency would normally be expected in buildings at least 20 storeys high on a fixed base. However, the peak damage was observed in buildings around 12 storeys (see Figure 3.17). Simple calculations reported in 6.2 suggest that the large foundation flexibility due to the soft clays of the lake bed is sufficient to lower the frequency of a typical 12 storey building from 1Hz to around 0.4Hz and so the observed damage is broadly consistent with the recorded motion. Moreover, the widely observed failure of brittle infill panels will have tended to reduce structural frequencies still further.

Moreover, the low frequency of the motion, in comparison with many earthquakes, meant that the elastic fundamental frequency of most buildings less than 10 storeys in height was more than that of the earthquake. A reduced frequency upon yielding of a building under 10 storeys would therefore lead to an increase in load, as the response spectrum (Figure 2.4) demonstrates. For as long as they could remain elastic, many buildings may therefore have survived, whereas once they started to yield, they shook themselves into more and more trouble. The stiffening of structural frames with rigid, brittle infill panels may therefore have been especially dangerous, as discussed further in Section 7.3.3 below.

The peak recorded acceleration of 0.22g was not especially great - accelerations as high as 1.0g are quite commonly recorded in the epicentral regions of major earthquakes. However, it is well known that peak acceleration is not a very good indicator of the damaging power of earthquakes (eg. Campbell, 1985). Table 7.1 compares various indicators of damaging power of the Mexico lake bed trace with El Centro 1940, a Californian earthquake often taken in design as a severe proving event.

It will be seen that on all indicators, apart from peak ground acceleration, the Mexico trace was substantially more severe than El Centro. The number of cycles is important, because the ductility of many structures degrades with cycling - a "high amplitude, low cycle fatigue" type of effect. The Housner spectral intensity (Housner, 1952) is related to the peak velocity (a measure of kinetic energy) of single degree of freedom oscillators, summed over frequency. The Arias intensity is described by Campbell (1974) as "the sum of the total energy per unit weight stored in a population of undamped linear oscillators, uniformly distributed in frequency, at the moment the earthquake record ends". The energy flux is related to the energy input per unit area (Sarma, 1971).

Although comparisons between earthquakes of such different frequency content are difficult, these data confirm that the lake bed motions were exceptionally severe.

7.3 STRUCTURAL COLLAPSE MODES

7.3.1 Column and beam sway mechanisms

The collapse of concrete moment frame structures was characterised by failure in columns rather than beams. "Column sway" mechanisms (see Figure 7.1) are well established as being less ductile than "beam sway" mechanisms because of the reduced ductility of columns by virtue of their compressive loads, the concentration of ductility demand into fewer members, the exacerbation of "P-delta" effects and the more widespread consequences of a column collapse compared with a beam collapse. Such considerations are well known, both in Mexico (eg. Bazan & Meli, 1985) and elsewhere.

Improving column ductility by increasing confining steel is only possible to a limited extent and in many codes, there is a requirement to check at each column/beam junction that the columns are stronger than the beams. The requirements for the highest ductility level in the Mexico City (1977) code include this, on a similar basis to the requirements of the US code, ACI 318-83 Appendix A. However, confidence in achieving a high level of ductility is crucial to safety in places subject to very violent seismic motion. It may be that calculation of isolated beam/column joints is insufficient to establish this confidence, and establishing the overall collapse mode directly is essential in medium to high rise buildings in areas of high seismicity. The associated techniques of "capacity demand" calculations have been developed, especially in New Zealand, and reference can, for example, be made to the New Zealand concrete code, NZS 3101 : 1982 : Parts 1 & 2.

Given the significant failures in braced, as well as unbraced, structures, direct consideration of the collapse modes in these type of structures may also be indicated, to make sure that the assumed overall level of ductility can be withstood by the members which are expected to yield.

7.3.2 "Top down" collapses

A remarkable feature of the earthquake damage was the large number of failures in upper or intermediate storeys of moment frame buildings, where the lower storeys survived intact (Section 3.4.2(i) & (ii)). Such failures were common in isolated buildings, where buffeting from adjacent structures was not possible, and also in buildings where there was no obvious discontinuity at the lowest level of collapse. Similar types of collapse were evident in Mexico City after the 1957 event (Zeevaert, 1983).

The influence of higher modes on response might be considered as a cause for these failures. Many codes (eg. UBC, Clause 2312(e)) include a concentrated top force to allow for the effects of higher mode response on tall buildings. It is understood that pre 1977 Mexican codes did not include such a provision, and so a possible mechanism for the top down failures suggests itself. However, on reflection, it seems unlikely that higher modes would be important in what was essentially a monochromatic, long period excitation. A response spectrum analysis, using the SCT spectrum (figure 2.4), on a typical 10 storey building has confirmed this. The shear distribution predicted by a rigid translational mode (equivalent to neglecting the top concentrated force) corresponds closely to that predicted by the full modal analysis - see Figure 7.2.

Other explanations for top down collapses must therefore be sought, and two such are presented below. It should be emphasised they are speculative in nature, and would need further work to establish definitively.

The first factor contributing to collapses at intermediate levels may have been the ratio of column to beam strength. For the purposes of gravity design, columns have to increase their strength, and hence possibly also their size, towards the bottom of a building. Beams, by contrast, are influenced only by loading on one floor, and can generally remain constant throughout. The ratio of column to beam strength therefore tends to increase toward the bottom of a building. It is possible that collapse occurred at a level where the seismic shears and moments had built up to a level sufficient to cause failure, but at which the ratio of beam to column strength forced failure in the latter.

Top down collapse, on this explanation, followed by a subsequent progressive collapse of the floors above.

No direct evidence exists for this explanation, but the Telephone exchange building (Section 3.4.10(d)), which suffered a top down collapse, had constant section beams, whereas the columns reduced in section with height from 750mm square to 350mm square.

Another factor which may have contributed to intermediate level failures is connected with the stiffening effect of rigid infill panels. Brittle failure of such panels at a certain level would tend to cause a soft storey at that level, with the attendant risk of forming a column sway mechanism. In-plane shearing forces tend to decrease with height in a building whose shear stiffness remained constant with height, but out of plane forces, related to accelerations, increase with height. Failure of rigid infill panels due to out of plane forces may therefore have helped to trigger collapses at intermediate levels. No evidence was obtained to support this theory, which remains speculative.

7.3.3 Failure of Rigid Infill Panels

Some degree of failure in rigid infill panels appeared practically universal in medium rise reinforced concrete frame buildings. The significance of such failures goes beyond the danger from falling masonry, and the cost of repair. The rigid but brittle nature of the infill causes a substantial, and unpredictable alteration to structural behaviour. Failure of the panels at a given level could cause an effective 'soft storey' to form at that level, with the attendant risks of an enormous increase in ductility demand in the columns at the same level, leading to subsequent collapse.

It is reasonably certain that 'softening' of buildings (i.e. lengthening of their period) following brittle failure of their infill panels caused many buildings to shake themselves into resonance with the fundamental period of the lake bed motions. This type of 'knife edge' effect could be one reason for the observed variability in damage.

These dangers are well recognised in Mexico and elsewhere. The problem is that blockwork infill is a simple technology, which provides good acoustic and thermal insulation properties. However, detailing to provide separation of panels from the structural frame leads to complications in ensuring out of plane stability for the panels. Lightweight flexible cladding, the more usual solution in California, may not always be technically appropriate in Mexico. An appropriate solution seems an important task.

The emergency provisions introduced after the earthquake make the requirements for infill panels more stringent.

7.3.4 Adequacy of Force Levels in the Mexico City Code

In order to compare the force requirements of the 1977 Mexican code with the actual demands of the 1985 earthquake, a single degree of freedom system with an elasto-plastic spring was analysed for the S60°E motion recorded at SCT in the Mexico City Lake Zone. Since the time history used in the analysis was based on a hand digitisation of the acceleration trace, the results are necessarily preliminary, but the elastic response spectra obtained from the time history corresponded well with the spectra published by UNAM.

Figure 7.3 compares the responses obtained for a displacement ductility of 1 and 4 with the corresponding 1977 Mexico City code requirement for ultimate design base shear, for use with an equivalent static analysis. It will be seen that at 0.5Hz, the following results are obtained.

Actual force/code force = 2.6, for ductility = 1
" " " " = 2.0, for ductility = 4

It also can be seen that, at resonance, a ductility level of 4 reduces the elastic response by a factor of over 5, rather than the factor of 4 assumed in the code. This is due to the 'detuning' effect of the onset of plasticity, for such a monochromatic signal.

The ratios quoted above apply to ultimate design forces. However, design to the code forces implicitly contains a number of safety factors, which apply to systems with both high and low levels of ductility as follows.

a) Code capacity is based on characteristic (i.e. lower bound) static strength to which a capacity reduction factor is applied. The actual flexural strength of r.c. beams at 0.5Hz is greater by perhaps 20% because

   i) The "capacity reduction factor" provides a safety factor of at least 10%
   ii) The average steel strength is about 10% greater than the guaranteed minimum

b) The single degree of freedom results do not apply directly to a multi-storey building, which typically would have an effective mass of between 80% and 90% of its total mass. This reflects the reduced participation of the mass at lower levels.

Additional safety factors apply to systems achieving high ductility, which do not apply to brittle systems, as follows.

a) A structure designed for a nominal ductility of 4 or 6 should be able to withstand one or two cycles to higher ductility, whereas low ductility systems are much less likely to be tolerant of such excursions.

b) An overall displacement ductility of 4 implies substantial local excursions into yield. A significant degree of strain hardening can therefore be anticipated, which could cause the strength of flexural members to increase by around 30%, although this may be partly offset by a loss of strength due to concrete degradation at high strain.

As a check, the lateral load requirements of UBC(1982) for ductile shear wall systems (for which a ductility of about 4 is probably appropriate) were compared with the response calculated by Veletsos and Newmark (1960) for El Centro 1940, for a ductility of 4 - see Figure 7.3. The maximum ratio of response to ultimate code force is 1.8, very similar to that found for a ductility level of 4 in Mexico.

As described in Section 5.2, emergency regulations were introduced a month after the earthquake. For medium to high rise buildings on the lake bed zone, these have increased the required seismic design loads required for cases of high or intermediate ductility by 155%. However, for cases of low ductility, the increase is only 70%. From the previous discussion, the relative increases appear to apply in the wrong order.

The general impression gained by the EEFIT team was that the failed r.c buildings had been designed to low or intermediate levels of ductility, rather than high levels of ductility. This is an option permitted by the 1977 Mexico City code, which entails less stringent detailing at the expense of higher lateral force resistance, compared with the high ductility option. It was understood from local engineers that the high ductility option has seldom been adopted in Mexico City.

Some important conclusions emerge:

1) For buildings designed and detailed with a high level of ductility to the 1977 Mexico City Code, the 1985 lake bed motions would not have made ductility demands substantially out of line with that provided for by the code.

2) For low levels of ductility, the code force levels were significantly low, for structures near resonance with the lake bed motions.

3) As is well known, systems capable of achieving high levels of ductility contain reserves of safety greater than a simple reduction of elastic response by the ductility factor would indicate. The absence of a prohibition on structures with low ductility for medium to high rise buildings in the Lake Zone seems a significant omission from the Mexico City code (1977), which should be rectified, and has not been addressed in the emergency regulations of October 1985.

4) The emergency regulations have increased force levels, but the increase is greatest for systems with the highest ductility, whereas preliminary analysis has indicated that systems with low ductility should have the greatest increase in force.

8.0 LIAISON OF EEFIT TEAM WITH MEXICAN AUTHORITIES

Through the good offices of the Scientific Officer of the British Council in Mexico, David Blagbrough, the EEFIT team held discussions on possible UK/Mexican collaboration in post earthquake engineering operations and research. The discussions were held with representatives of the following organisations.

UNAM	:	Faculty of Engineering (Dr Romo)
CONACyT	:	Consejo Nacional de Ciencia y Tecnologia (Dr Resendiz)
DDF	:	Departamento de Distrito Federal (Ing. Alejandro Rivas)
SEDUE	:	Secretaria de Desarrollo Urbano y Ecologia (Lic Beatriz de la Vega)

The minutes of the final meeting with SEDUE, which represented the culmination of these discussions, are reproduced in the Appendix.

9.0 CONCLUSIONS

9.1 DAMAGE DISTRIBUTION

Near the epicentre, engineered structures, including dams, generally survived the motion satisfactorily. Damage to weak masonry and adobe buildings was less than might be expected from an earthquake with a magnitude of 8.1.

Away from the epicentre, the earthquake generally caused little damage. However there was some damage at Ciudad Guzman, some 180km north west of the epicentre and major damage was caused to medium to high rise buildings in the central part of Mexico City some 400km north east of the epicentre.

9.2 SITE RESPONSE

The damage at Mexico City was caused by site response effects as follows.

a) The Lake Zone lacustrine clay material underlying the affected areas of Mexico City caused a major amplification of the ground motion leading to a 6 fold amplification of the surface acceleration compared with bedrock motions recorded nearby. In addition, the surface motion was filtered to consist mainly of 0.25 to 0.5Hz motions.

b) All significant damage was restricted to an area of the Lake Zone. The distribution of damage was correlated mainly to the building height - i.e. medium and high rise buildings sensitive to low frequency motion appear to have been affected irrespective of their position within the area of the Lake Zone.

c) The Lake Zone clay material has a very high moisture content (about 250%) and is highly elastic in that it has a low stiffness modulus compared with its strength, and exhibits little internal hysteretic damping.

d) The major amplification observed in the Lake Zone seems to have been mainly caused by vertically propagating shear waves. The nature of the observed motion and studies with a simplified one dimensional model of the Mexico City Lake Zone soils, strongly support this conclusion. Similar amplification may be expected during future events.

9.3 FOUNDATION BEHAVIOUR

a) Only one foundation failure is known to have led to collapse of a structure.

b) A few buildings suffered excessive settlement during the earthquake leading to tilting and problems at entrances and service connections.

c) Many of the foundation systems were designed to enable the buildings to settle at the same rate as the surrounding consolidating ground. This is frequently achieved by using piles to a stronger underlying stratum which are designed to progressively fail as the surrounding ground consolidates. During an earthquake there may therefore be little spare capacity to resist the dynamic forces and local failure of some of the foundations can occur. It is somewhat surprising that more foundation systems did not fail during the earthquake.

9.4 CAUSES OF STRUCTURAL COLLAPSE

The major causes of collapse in engineered structures in the Lake Zone of Mexico City were the following:-

a) Resonance of medium to high rise buildings with the predominant frequency of the earthquake at Mexico City, namely 0.25 to 0.5Hz. Low rise buildings were generally unscathed, even where they were of low strength.

b) The particularly onerous nature of the motions, particularly with respect to the large number of damaging cycles.

c) Insufficient structural ductility.

   The causes of lack of structural ductility included the following:

   i) Inadequate detailing of connections in reinforced concrete buildings.

   ii) Inappropriate structural form, especially "weak column, strong beam" structures in reinforced concrete moment frames. Buildings with plan and elevation irregularities also appear to have been particularly badly affected.

d) Brittle failure of block infill panels in unbraced frames, which probably led to a lengthening of structural period, and hence for medium rise buildings, an increasing level of force. Local failure of panels at one level may have also led to the damaging formation of soft storeys.

   Other factors contributed to the scale of the damage, including poor construction in some (but by no means all) buildings, buffeting between adjacent buildings, excessive live loads on floors and damage from previous earthquakes. However, these are likely to prove less important than the causes listed above. Damage due to previous foundation settlements does not appear to have played a significant part.

9.5 CODE PROVISIONS

The EEFIT team was not able to establish in detail how closely the buildings examined conformed to the 1977 Mexico City code, although many of the failed concrete buildings appeared to have less stringent reinforcement detailing than those required for high levels of ductility in the Mexican code. The latter are broadly equivalent to the provisions of the United States Code ACI 318-83. The following observation on code provisions in general, and the

Mexico code in particular are therefore necessarily somewhat tentative.

a) Low levels of ductility are not appropriate for medium to high rise buildings in areas of high seismicity , such as the Lake Zone in Mexico City. However, structures with low ductility were permitted in that zone by the 1977 Mexico City code, albeit with high requirements for lateral force resistance, and this appears to have been a serious omission in that code. It does not appear to have been corrected in the Emergency Regulations of October 1985 (due to be revised in October 1986).

b) The ultimate design forces specified in the Lake Zone by the 1977 Mexico City code were substantially below the peak forces that would have been experienced by single degree of freedom systems with appropriate levels of ductility and a structural period of around two seconds, had they been subjected to the lake bed motions recorded at the SCT building. The Emergency Regulations have effectively increased these ultimate design forces by at least 70%.

   Preliminary analysis by EEFIT suggests that the exceedence by actual forces in the earthquake over the 1977 code design forces was much less serious for highly ductile structures than for those with limited ductility. In this respect, the increase in design forces specified by the Emergency Regulations may be unnecessarily conservative for highly ductile structures.

c) Where it is intended to achieve a high level of ductility, consideration should be given to including code requirements for direct analysis of the collapse mode of a building structure, in order to ensure the following:-

   1) That the collapse mechanism is a ductile one (e.g. plastic hinges form in beams, not in columns for moment frame structures).

   2) That provisions for ductile detailing are adequate in the identified regions of plastic deformation.

d) No evidence was collected to suggest that current internationally accepted requirements for detailing highly ductile reinforced concrete structures are inadequate, though it would be prudent to check that the details can withstand the large number of damaging cycles experienced in the 1985 Mexican earthquake.

e) The dangers of failure in rigidly connected unreinforced block infill panels were recognised by the 1977 Mexico City code, and further emphasised by the Emergency Regulations. However, development of practical details to conform to the code requirements appears to be a serious need.

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GLOSSARY OF TERMS AND ABBREVIATIONS

a_v	coefficient of compressibility = -de/dp
C_c	compression index = $-de/d(\log_{10}p)$
c_u	undrained shear strength
C_v	Terzaghi's coefficient of consolidation
CFE	Commision Federal de Electricidad
CONACyt	Consejo Nacional de Cienca y Tecnologia
CU	Ciudad Universitaria
e	voids ratio
DDF	Departamento de Distrito Federal
EEFIT	Earthquake Engineering Field Investigation Team
g	acceleration due to gravity, 9.81 m/s^2
G_o	shear modulus at low strain
LVDT	linearly variable differential transformer
Ms	surface wave magnitude
m_v	coefficient of volume change = $a_v/(1+e)$
N	SPT blowcount per 300mm penetration
OHT	overhead travelling
p	effective overburden pressure
Q	a factor in the 1977 Mexico City code to allow for the favourable effects of structural ductility
SCT	Department of Transport
SEAOC	Structural Engineers Association of California
SECED	Society for Earthquakes and Civil Engineering Dynamics
SEDUE	Secretaria de Desarrollo Urbano y Ecologia
SERC	Science and Engineering Research Council
SPT	standard penetration test
UBC	Uniform Building Code
UNAM	Universidad Nacional Autonoma de Mexico
w_i	water content

TABLE 3.1 **PROPERTIES OF MEXICO CITY CLAY**

Depth (m)	Moisture Content (%)	Undrained Shear Strength (kN/m^2)	Confining Pressure (kN/m^2)	Shear Strain Amplitude (%)	Secant Shear Modulus (kN/m^2)
7.6 to 8.6	150	50	120	0.14 to 4.33	1830 to 530
9.8 to 10.8	286	77.5	150	0.10 to 2.05	2570 to 1830
12.0 to 13.0	187	(80)*	190	0.05 to 1.52	4270 to 2470
16.4 to 17.4	86	85	250	0.04 to 4.81	5810 to 1000

* Sample failed at a dynamic shear strain amplitude of 1.52(%) with liquefaction (remoulding) occurring at bottom of sample. Assumed static undrained shear strength shown in brackets.

TABLE 3.2 <u>NUMBERS OF DAMAGED BUILDING IN EACH DAMAGE CLASS, BY TYPE AND HEIGHT OF CONSTRUCTION</u>

Construction Type	Damage Class	Number of Storeys <3	3-5	6-8	9-11	12-14	15-17	18-20	>20
Concrete Frame	Severe	7	15	29	16	5	5	-	-
	Heavy	3	35	50	50	31	10	2	-
	Moderate	13	42	77	41	24	14	5	2
	Light	20	76	84	36	11	8	5	7
	Total No	43	168	240	143	71	37	12	9
Brick Load-bearing Wall	Severe	46	9	-					
	Heavy	62	21	1					
	Moderate	129	40	2					
	Light	219	80	2					
	Total No	456	150	5					
Stone Masonry	Severe	1	-						
	Heavy	4	-						
	Moderate	3	1						
	Light	16	5						
	Total No	24	6						
Steel Frame	Severe	3	-	-	-	-			
	Heavy	2	-	-	-	3			
	Moderate	-	1	-	1	-			
	Light	1	5	-	1	-			
	Total No	6	6	0	2	3			

TABLE 3.3 **PERCENTAGE OF DAMAGED BUILDINGS IN EACH DAMAGE CLASS, BY TYPE AND HEIGHT OF CONSTRUCTION**

Construction Type	Damage Class	Number of Storeys <3	3-5	6-8	9-11	12-14	15-17	18-20	>20
Concrete Frame	Severe	16	9	12	11	7	14	-	-
	Heavy	7	21	21	35	44	27	17	-
	Moderate	30	25	32	29	34	38	42	22
	Light	47	45	35	25	15	22	42	78
	Total	100	100	100	100	100	100	100	100
Brick Load-bearing Wall	Severe	10	6	-					
	Heavy	14	14	20					
	Moderate	28	27	40					
	Light	48	53	40					
	Total	100	100	100					
Stone Masonry	Severe	4	-						
	Heavy	17	-						
	Moderate	12	17						
	Light	67	83						
	Total	100	100						
Steel Frame	Severe	50	-	-	-	-			
	Heavy	33	-	-	-	100			
	Moderate	-	17	-	50	-			
	Light	17	83	-	50	-			
	Total	100	100		100	100			

TABLE 3.4 DAMAGE STATISTICS OBTAINED FROM 5 TRANSECTS ACROSS THE AREA OF MAJOR DAMAGE

Construction Type		Number of Storeys							
		<3	3-5	6-8	9-11	12-14	15-17	18-20	>20
Concrete Frame	No. Observed	75	143	93	36	26	18	8	3
	No. Damaged	10	25	37	28	20	13	5	0
	% Damaged	13	17.5	40	78	77	72	62.5	0
Brick Load Bearing Wall	No. Observed	214	108						
	No. Damaged	18	9						
	% Damaged	8	8						
Stone Masonry	No. Observed	21	28	1					
	No. Damaged	1	2	0					
	% Damaged	5	7	0					

TABLE 3.5 DAMAGE STATISTICS OBTAINED FROM A TRANSECT ACROSS THE TRANSITION ZONE

Construction Type		Number of Storeys							
		<3	3-5	6-8	9-11	12-14	15-17	18-20	>20
Concrete Frame	No. Observed	14	19	19	3	6	4	3	0
	No. Damaged	0	0	0	1	1	0	0	0
	% Damaged	0	0	0	33	17	0	0	0
Brick Load Bearing Wall	No. Observed	17	11						
	No. Damaged	0	0						
	% Damaged	0	0						
Stone Masonry	No. Observed	-	-						
	No. Damaged	-	-						
	% Damaged	-	-						

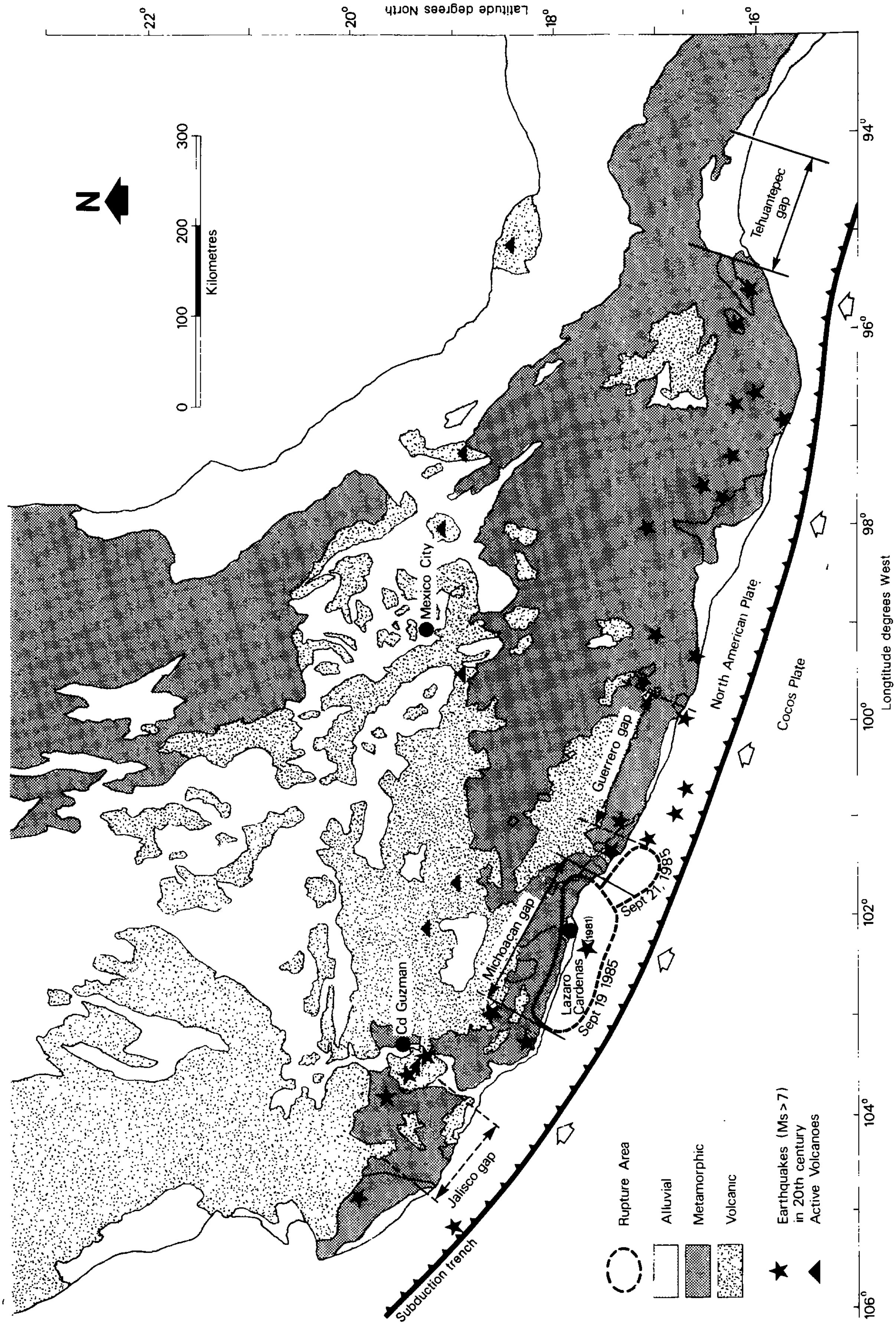
TABLE 3.6 **DAMAGE TO POWER SYSTEMS IN MEXICO CITY**
(after Brune et al, 1985)

Damaged Transformers	113 out of 26,000
Poles affected	50 out of 350,000
Lines damaged	6km out of 10,500km
Secondary distribution	10km out of 15,750km
Consumer connections	15,000 out of 3,200,000

TABLE 7.1 **COMPARISON OF STRONG MOTION DAMAGE INDICATORS**

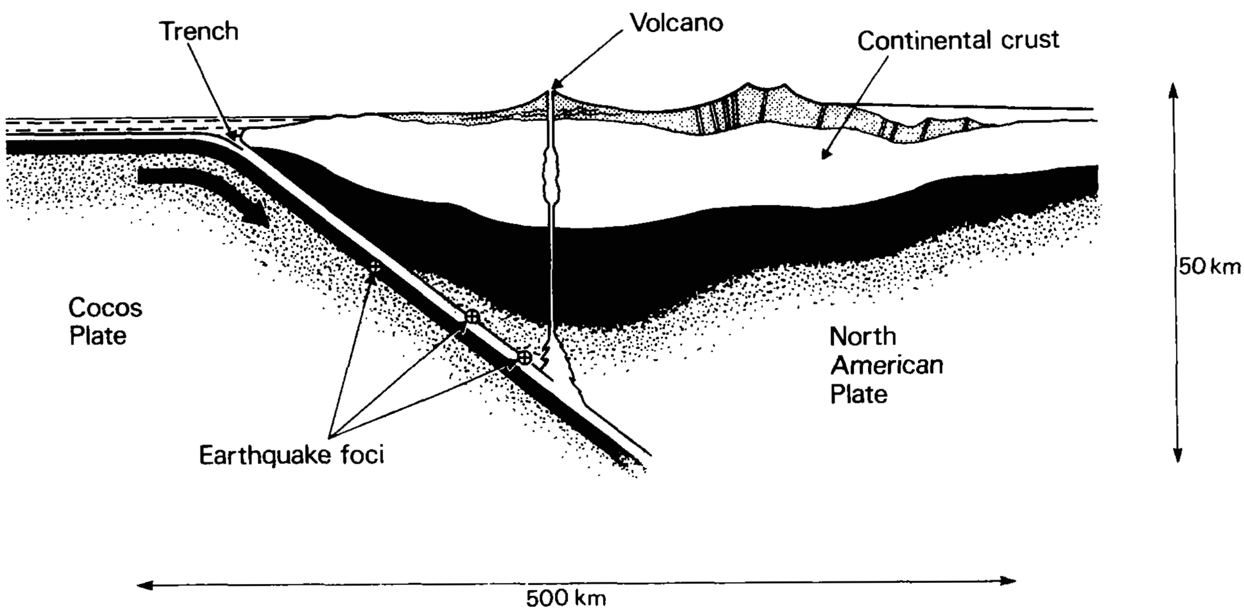
	Mexico City Lake Bed 1985 (SCT, S60°E)	El Centro 1940 (NS)
Peak Ground Acceleration	0.22g	0.33g
Peak Ground Velocity	0.65m/sec	0.38m/sec
No of Acceleration Peaks ÷70% of max	8	4
Housner Spectral Intensity (2% damping)	3.73m	1.76m
Arias Intensity	10.7m/sec	6.6m/sec
Energy flux	1.3m^2/sec	0.4m^2/sec

Note: the Mexico values are preliminary estimations for comparison purposes.



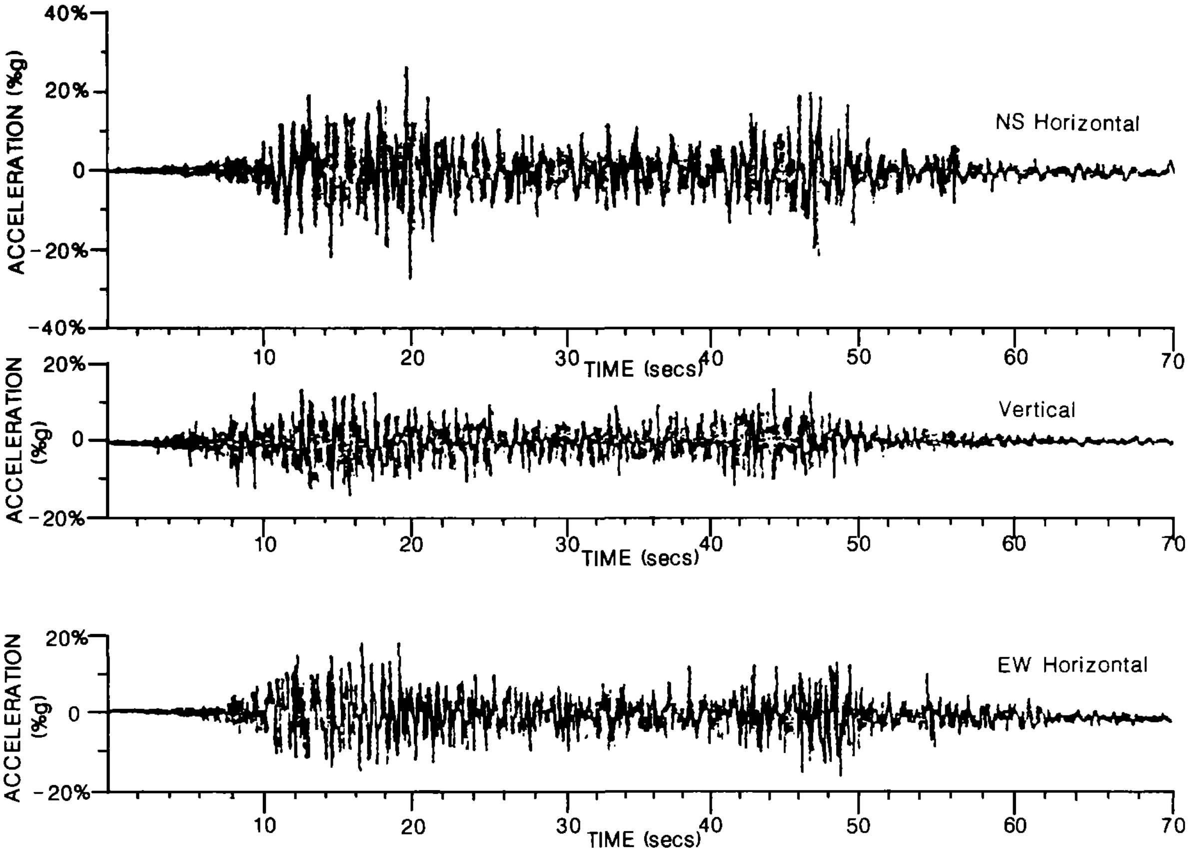
TECTONIC MAP OF MEXICO

FIGURE 2.1

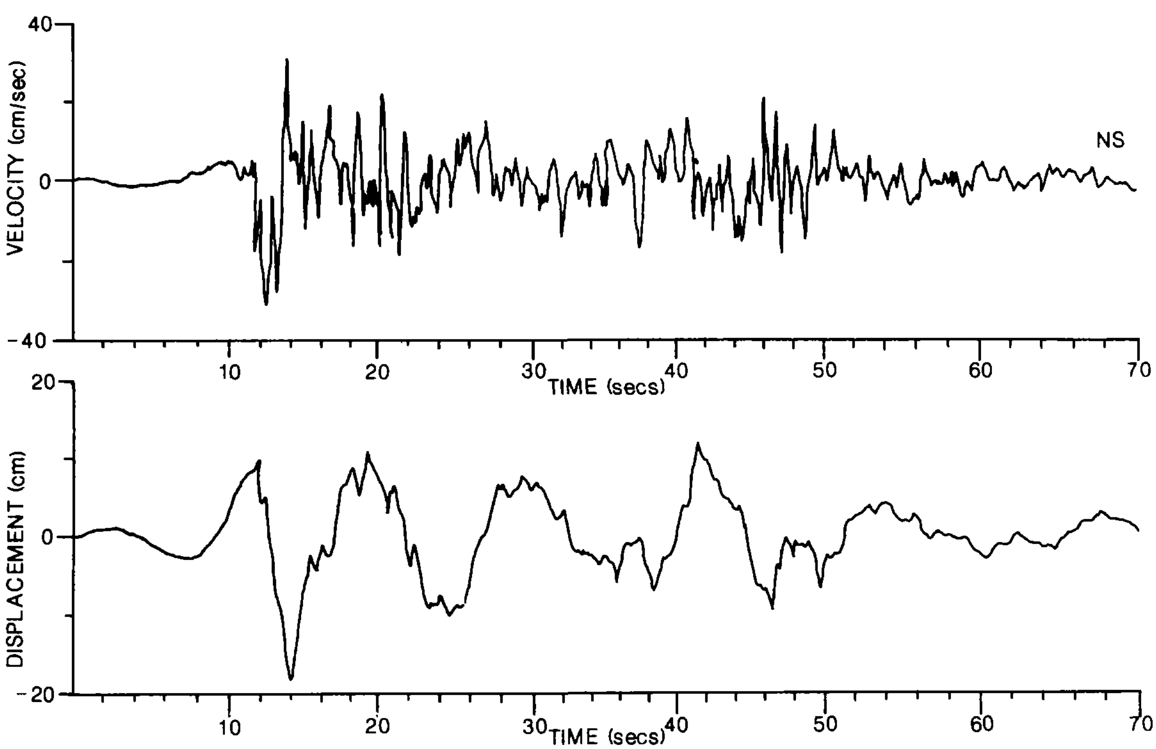


SCHEMATIC SECTION THROUGH
TYPICAL SUBDUCTION ZONE

FIGURE 2.2



(a) Acceleration

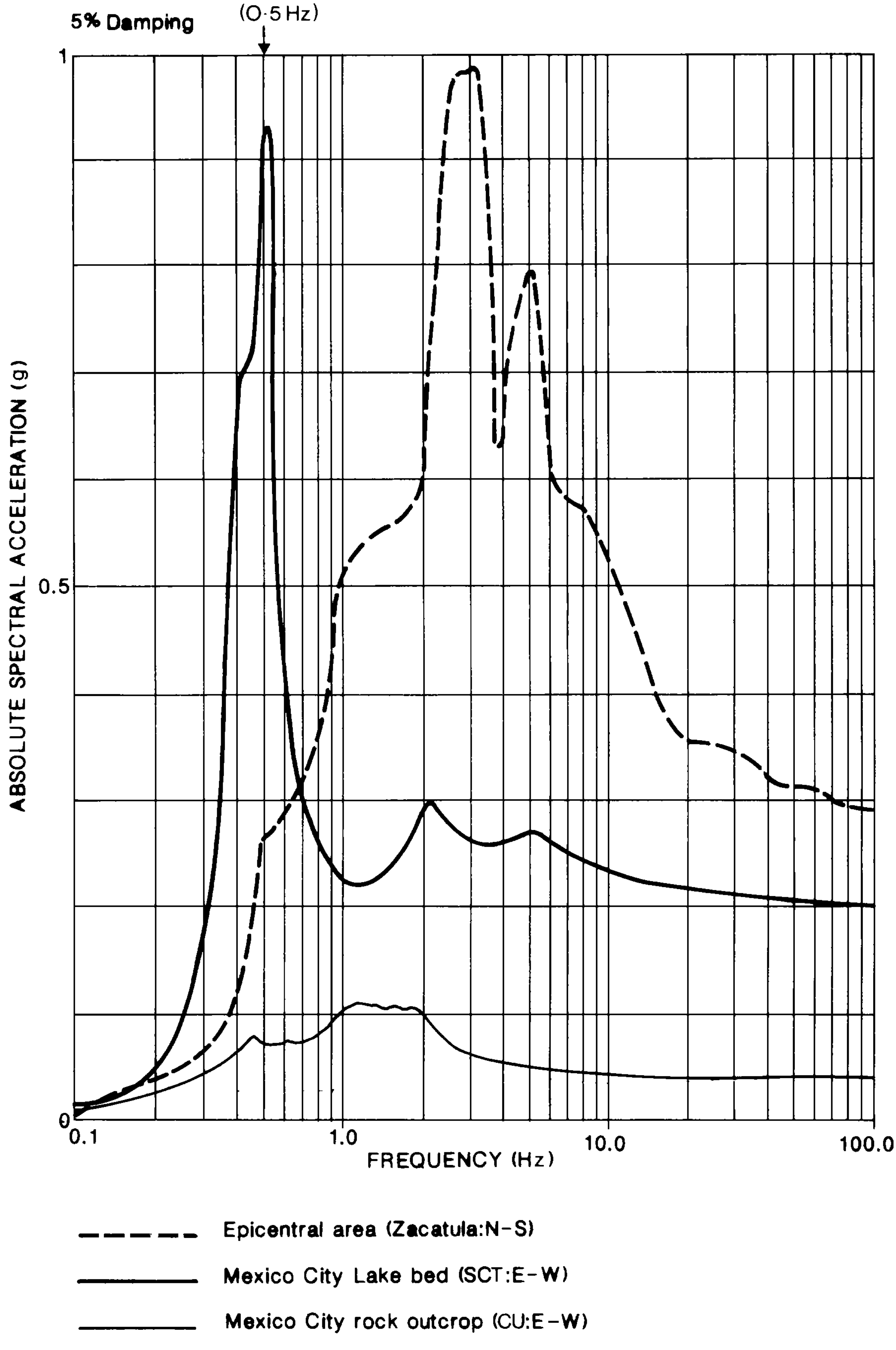


(b) NS Horizontal - Velocity and Displacement

Source - Mena et al (1985) UNAM:IPS-10E

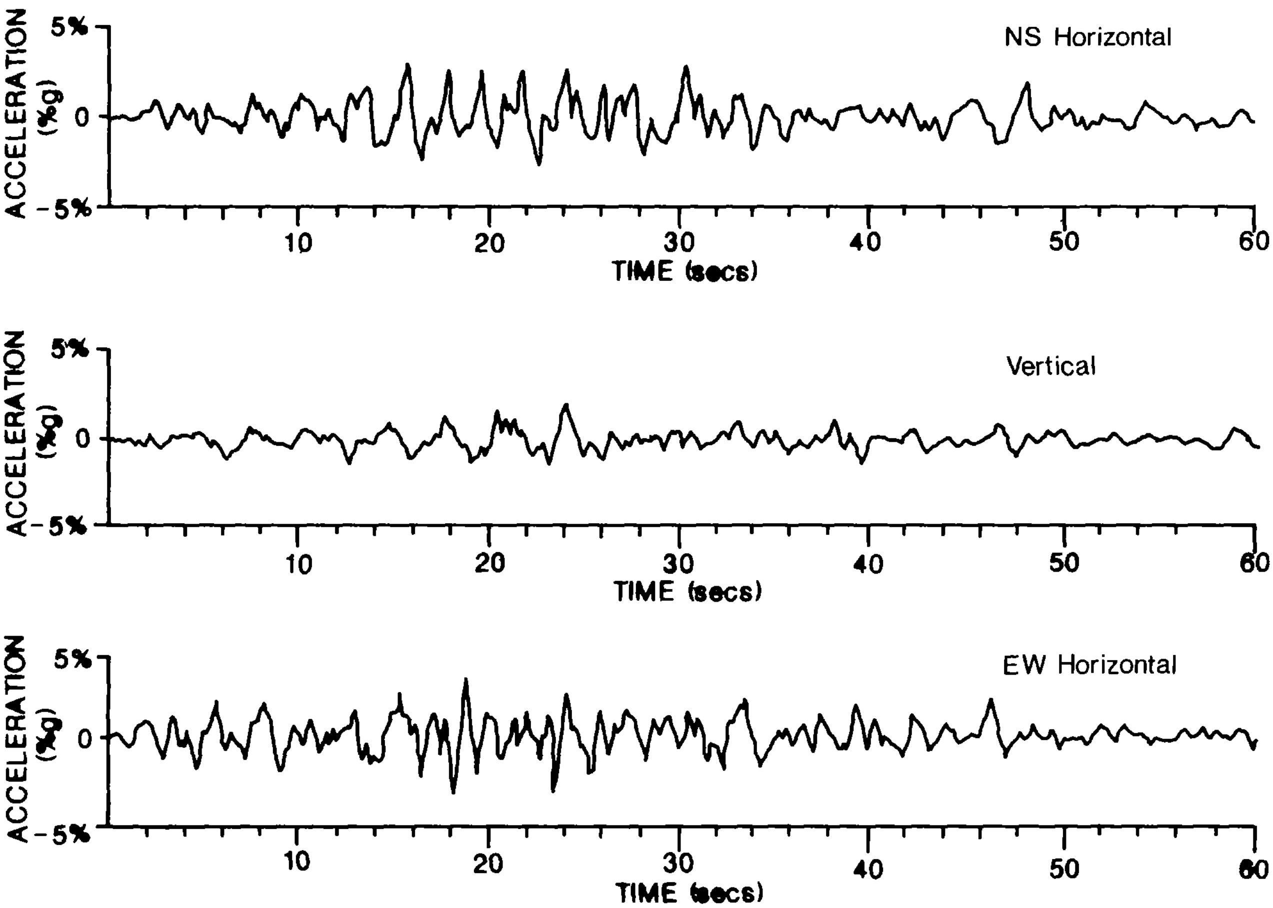
STRONG MOTION RECORDS FROM ZACATULA NEAR LAZARO CARDENAS

FIGURE 2.3



ACCELERATION RESPONSE SPECTRA

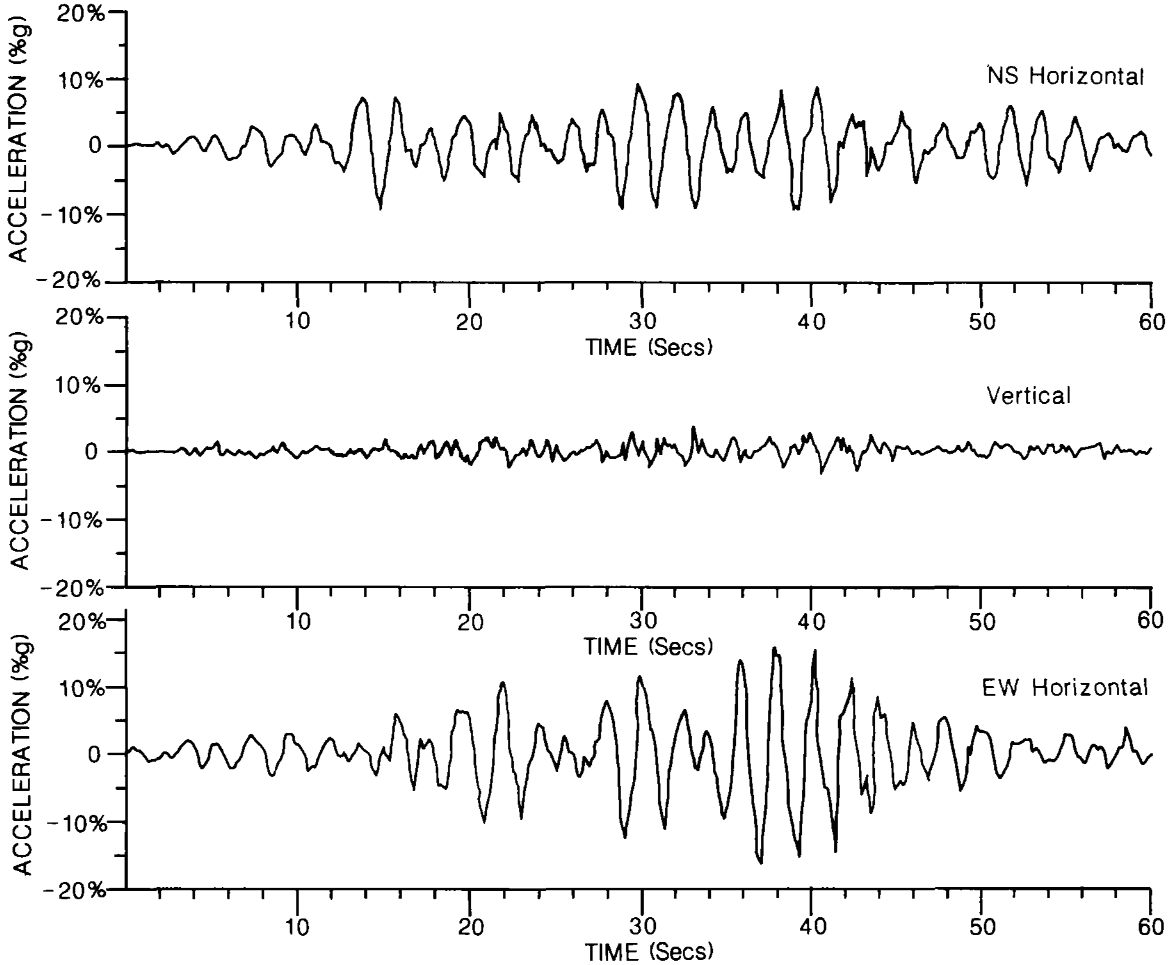
FIGURE 2.4



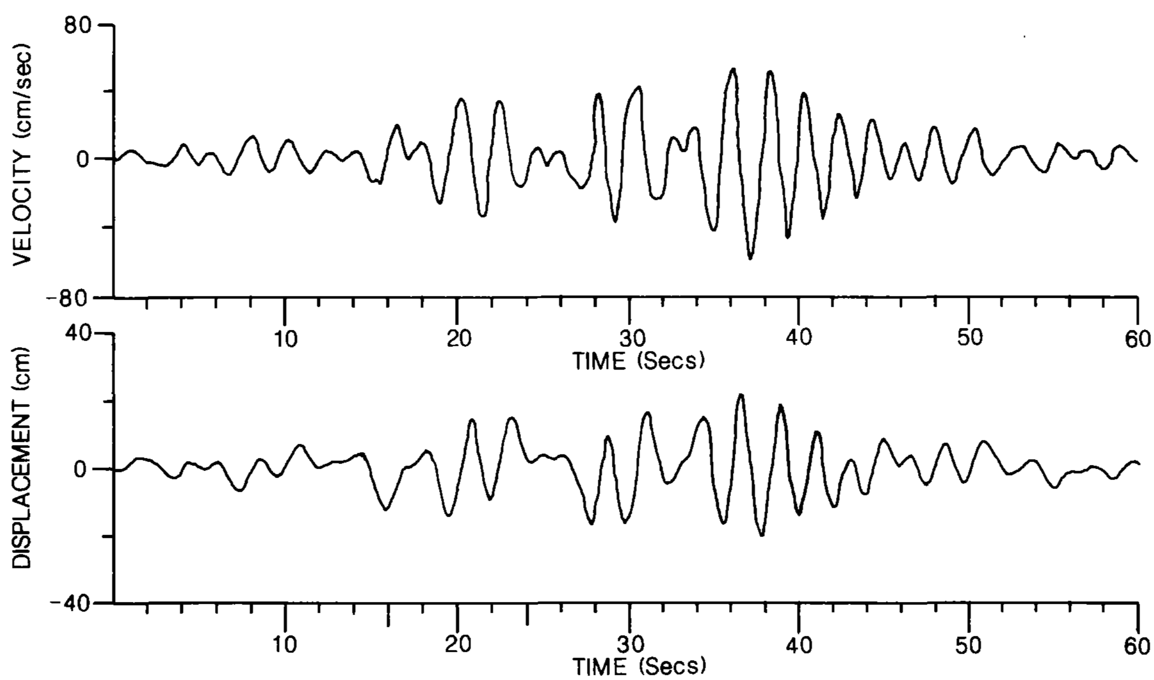
Source - Prince et al (1985) UNAM : IPS-10A

STRONG MOTION RECORDS FROM
ROCK OUTCROP AT CU SITE MEXICO CITY

FIGURE 2.5



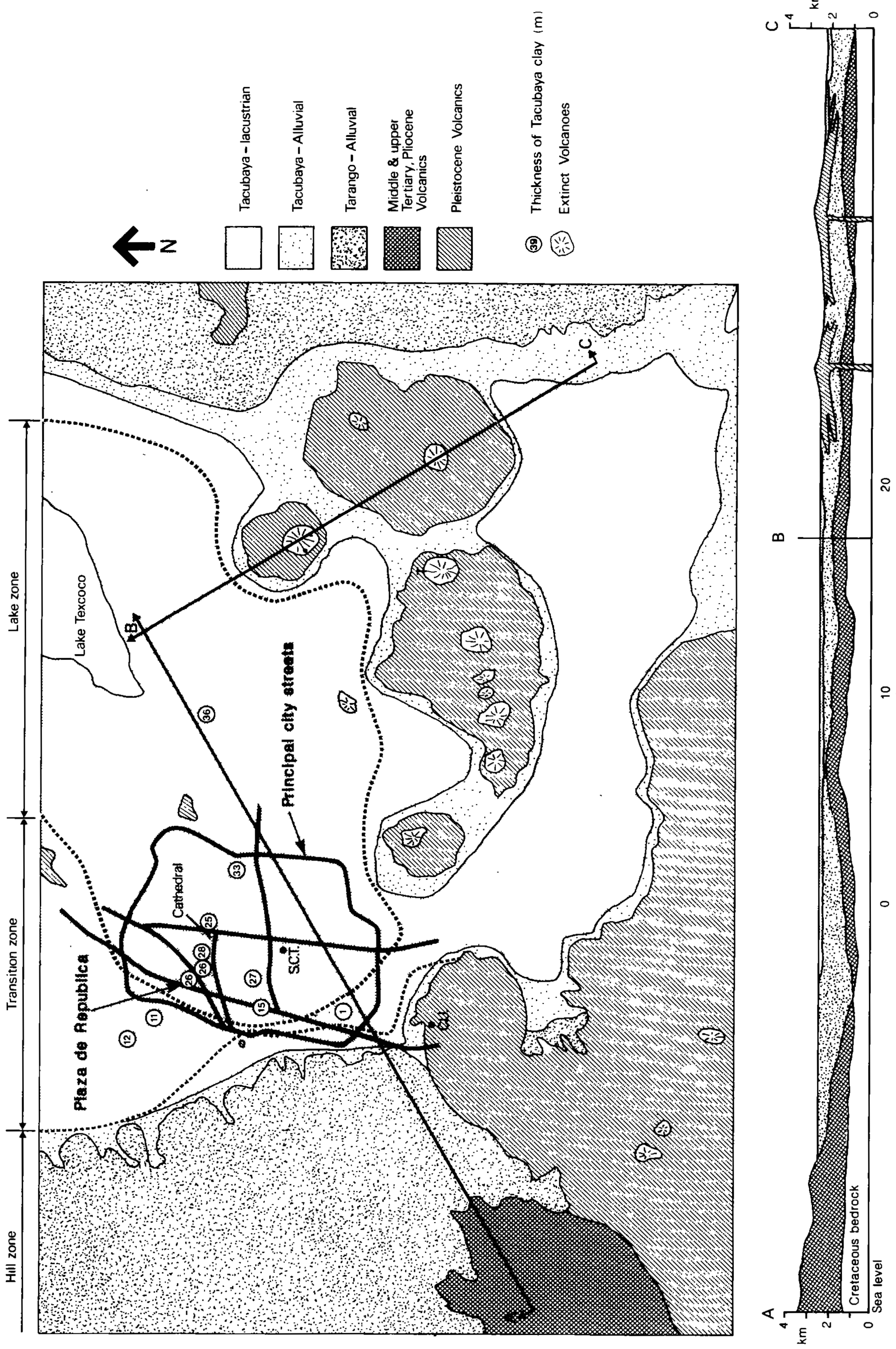
(a) Acceleration



(b) EW Horizontal - Velocity & Displacement
Source - Mena et al (1985) UNAM:IPS-10B

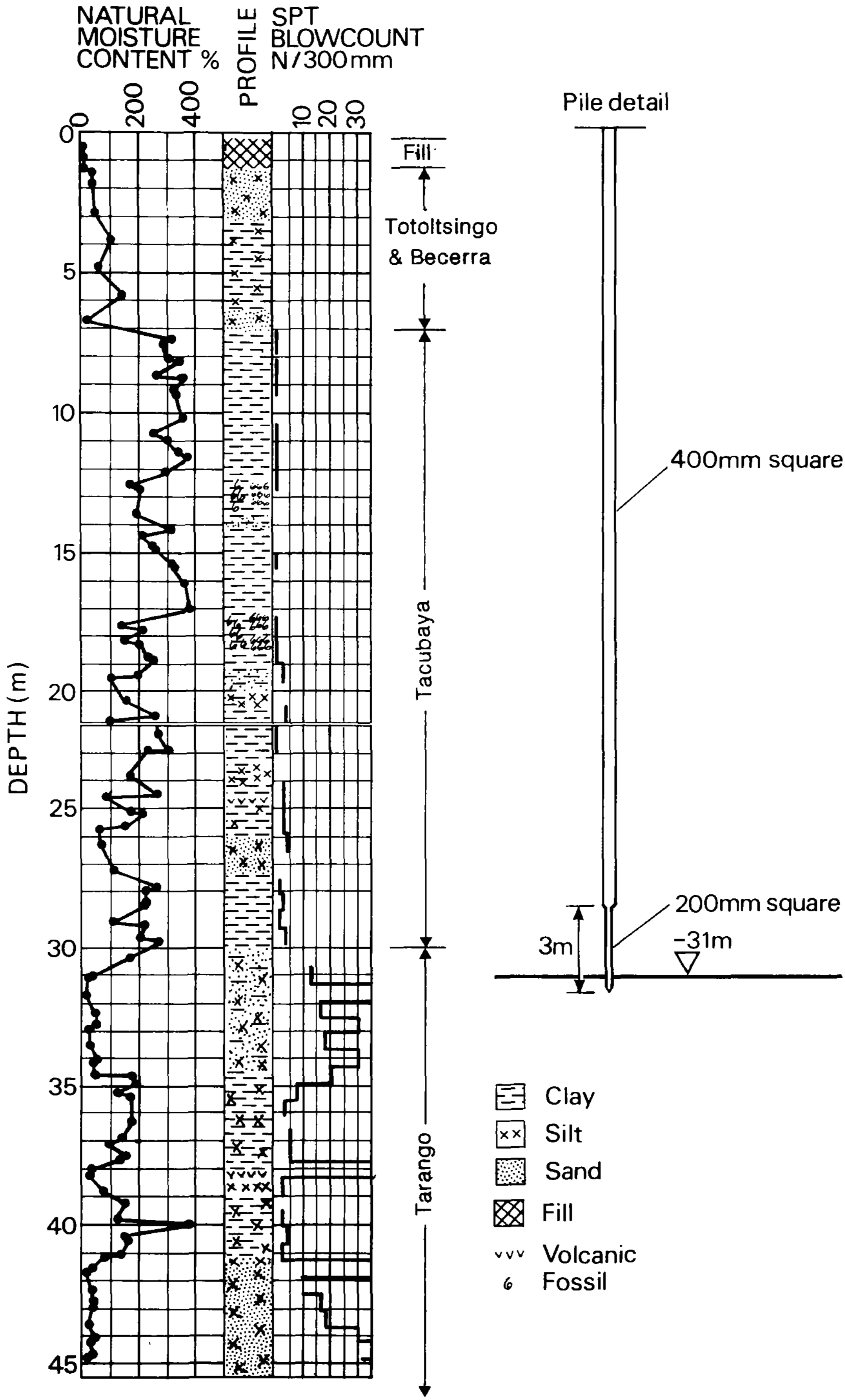
STRONG MOTION RECORDS FROM
LAKE BED AT SCT SITE MEXICO CITY

FIGURE 2.6

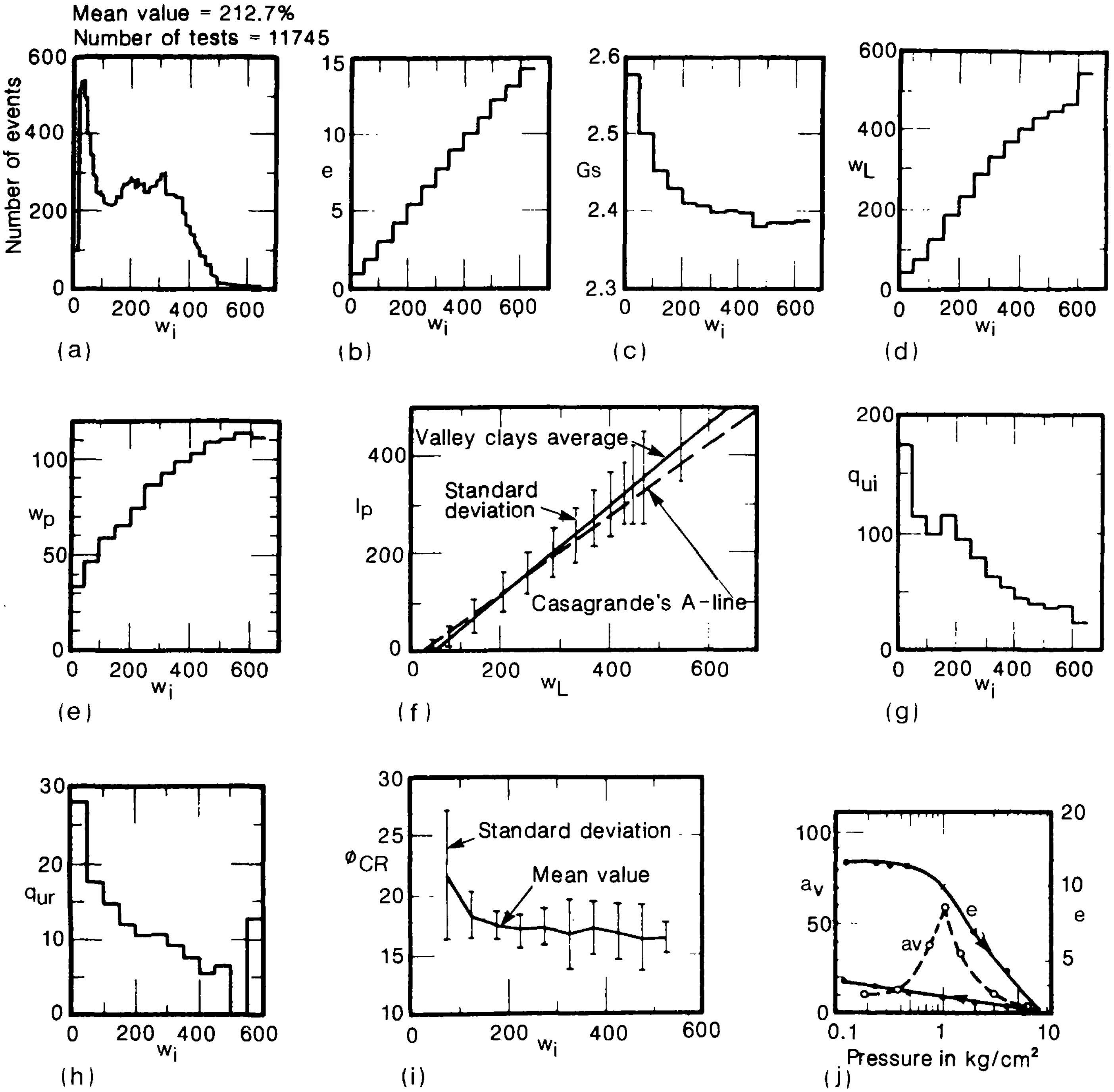


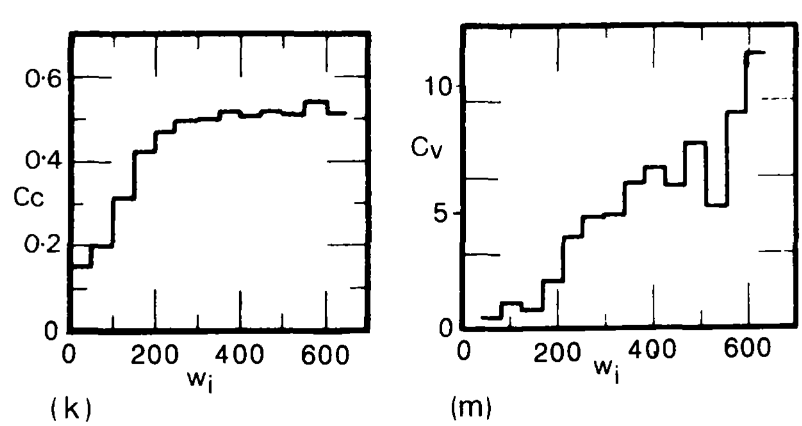
MEXICO CITY GEOLOGY

FIGURE 3.1



GROUND CONDITIONS
AT PLAZA DE REPUBLICA
FIGURE 3.2



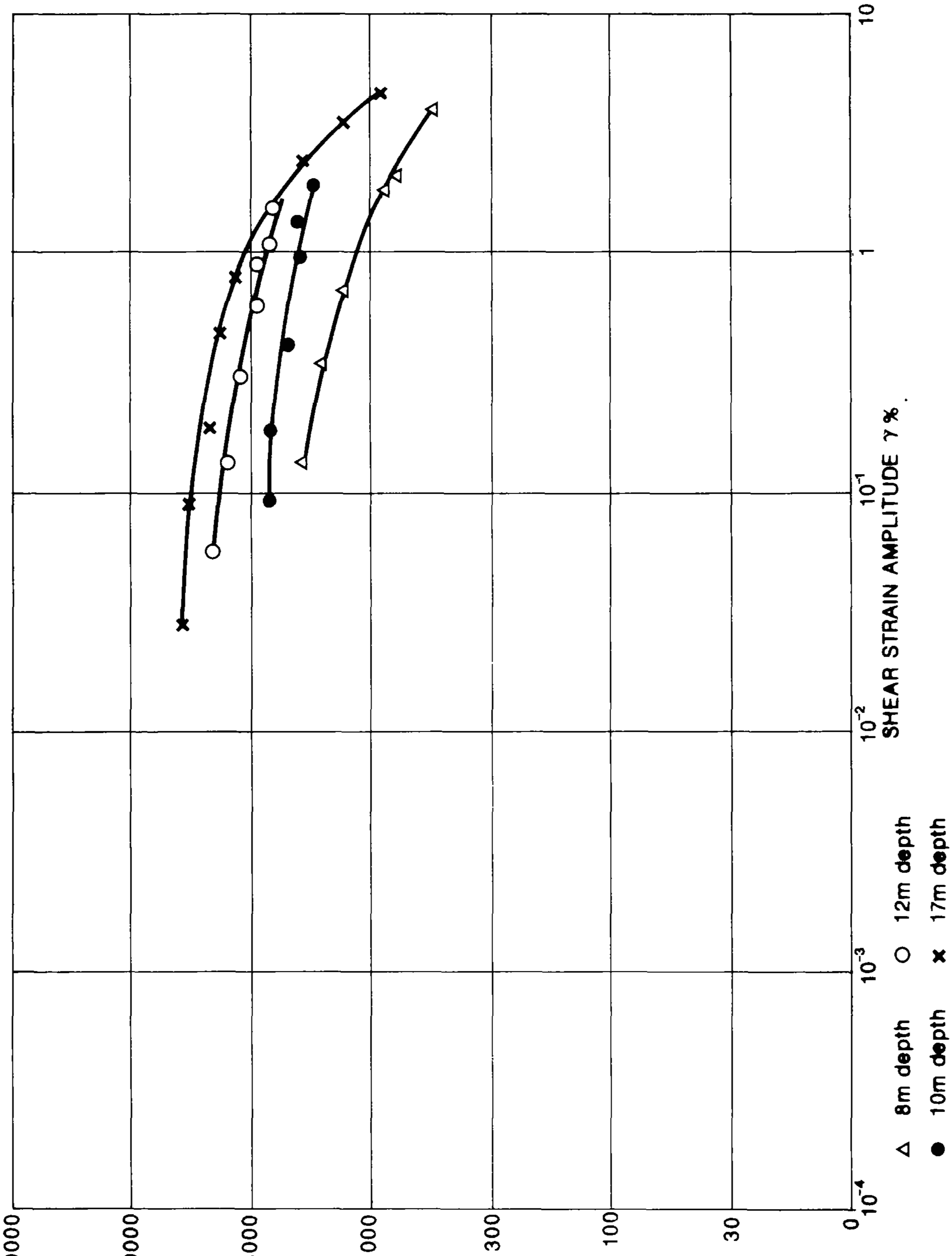


REFERENCES

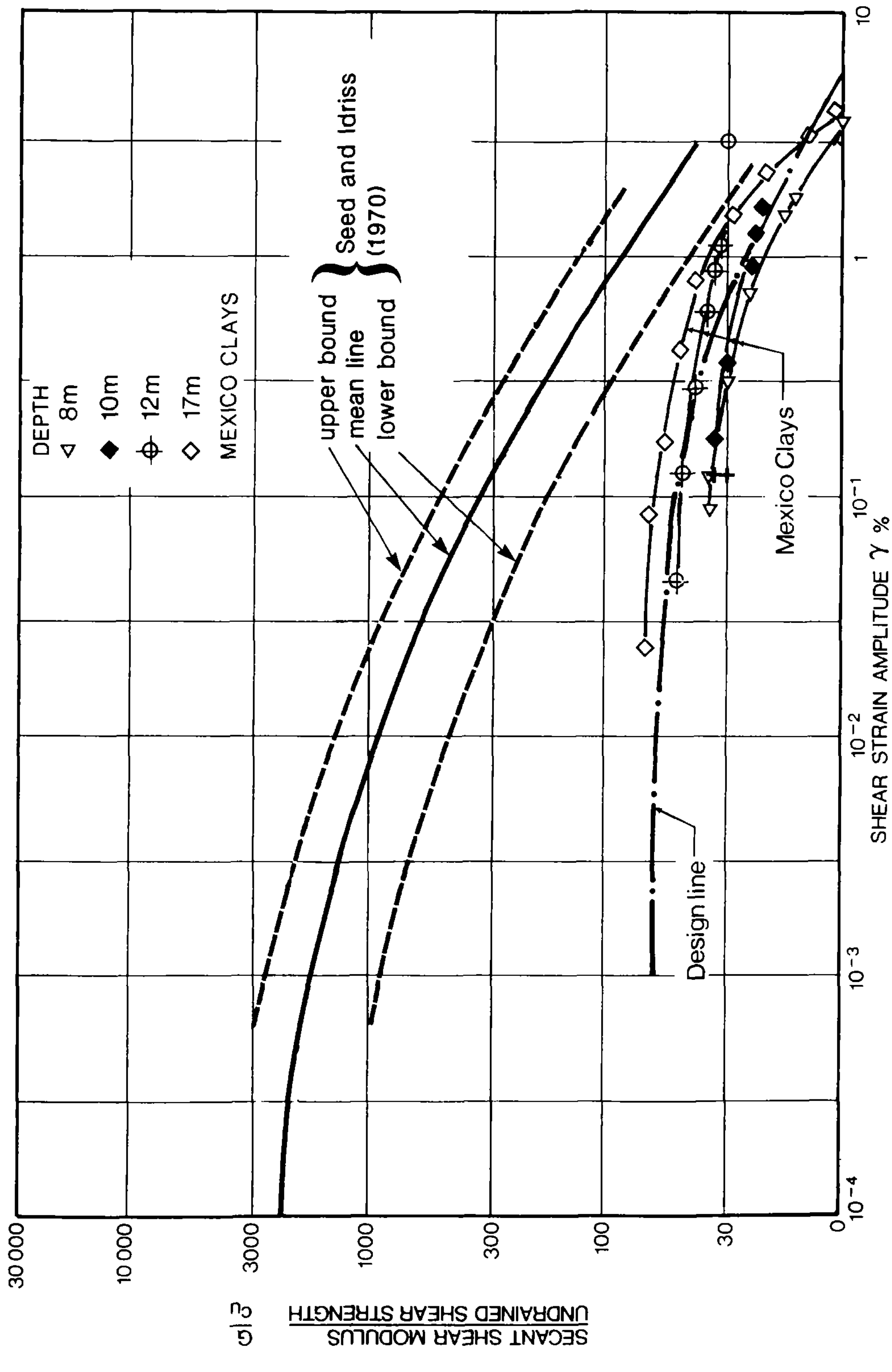
w_i	natural water content, in per cent
e	void ratio
G_s	specific gravity
w_L	liquid limit, in per cent
w_p	plastic limit, in per cent
I_p	plasticity index
q_{ui}	compressive strength (undisturbed) in kN/m^2
q_{ur}	compressive strength (remolded) in kN/m^2
ϕ_{CR}	friction angle, consolidated-undrained tests
a_v	coefficient of compressibility in m^2/MN
C_c	compressibility index
C_v	coefficient of consolidation in m^2/year

VARIATION OF MECHANICAL PROPERTIES WITH NATURAL WATER CONTENT

FIGURE 3.3

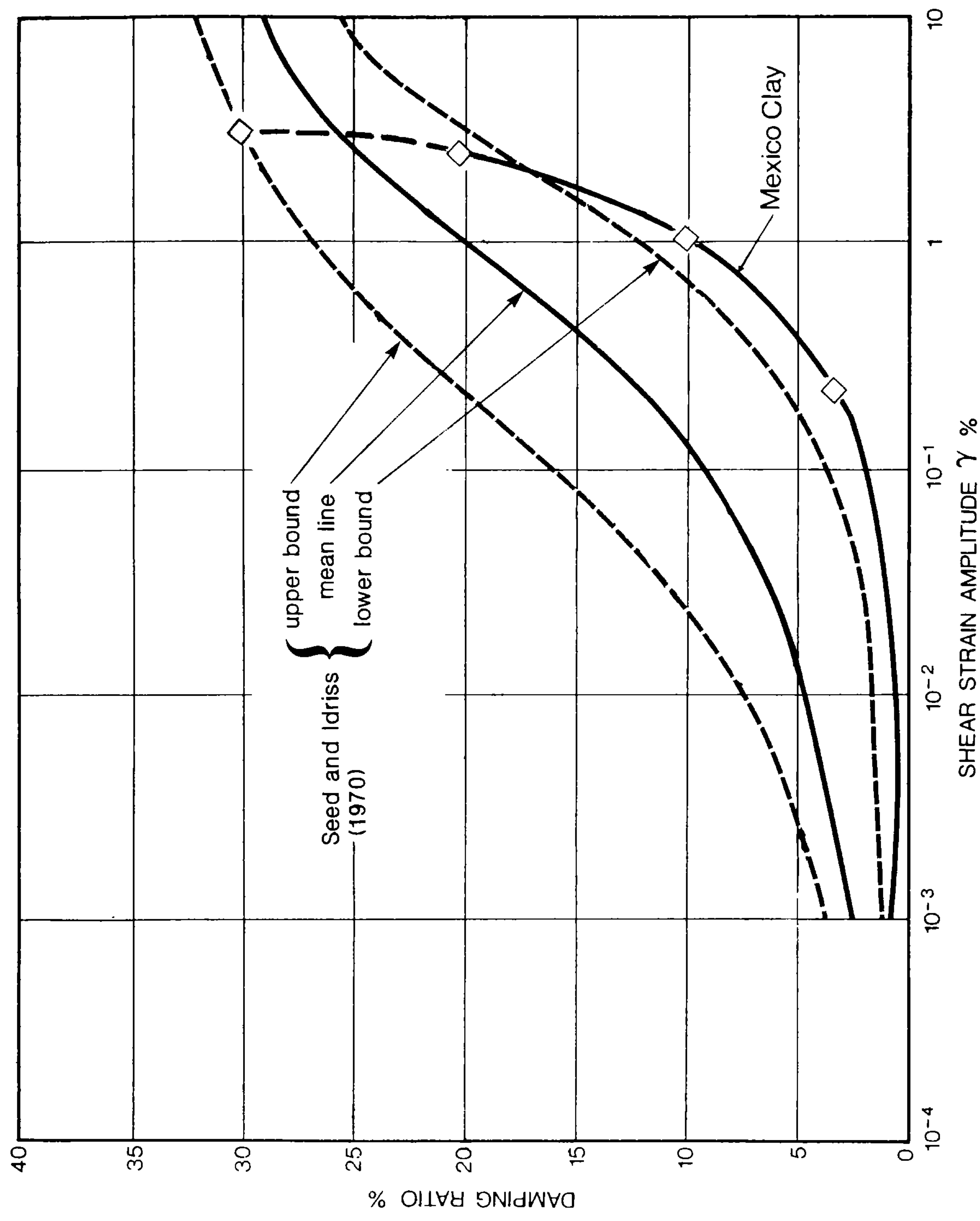


SHEAR MODULUS VALUES
FOR MEXICO CITY CLAYS
FIGURE 3.4



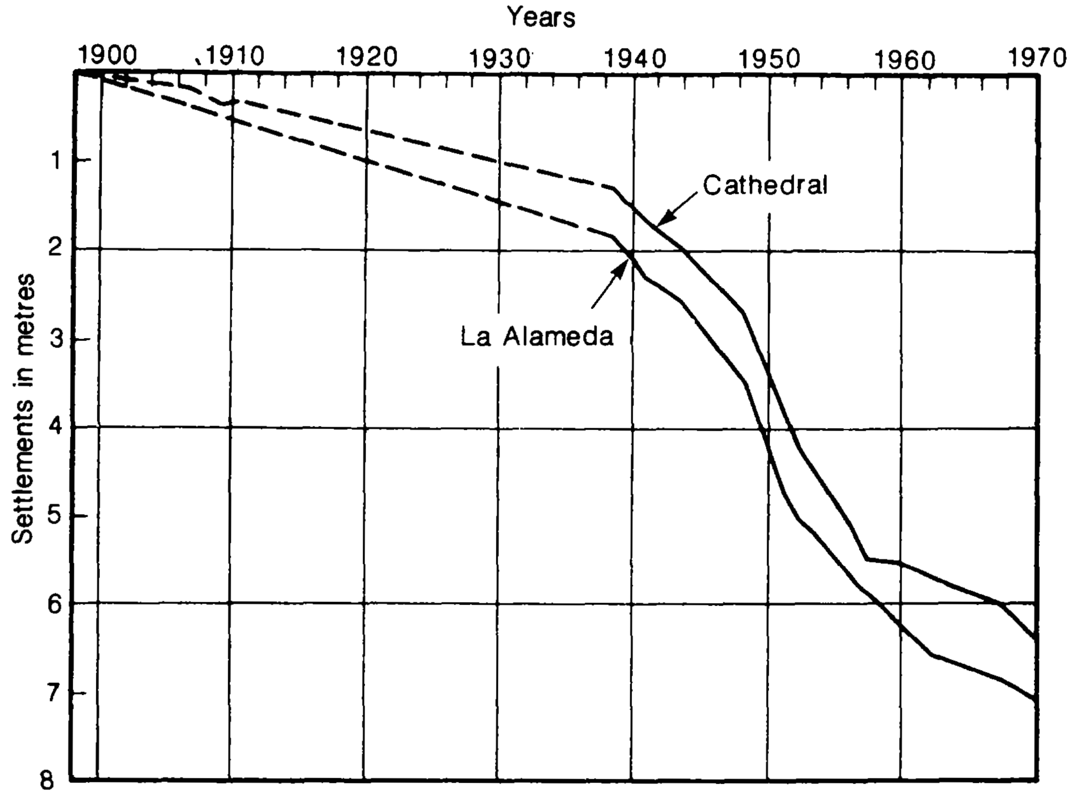
NORMALISED SHEAR MODULUS
VALUES FOR CLAYS

FIGURE 3.5

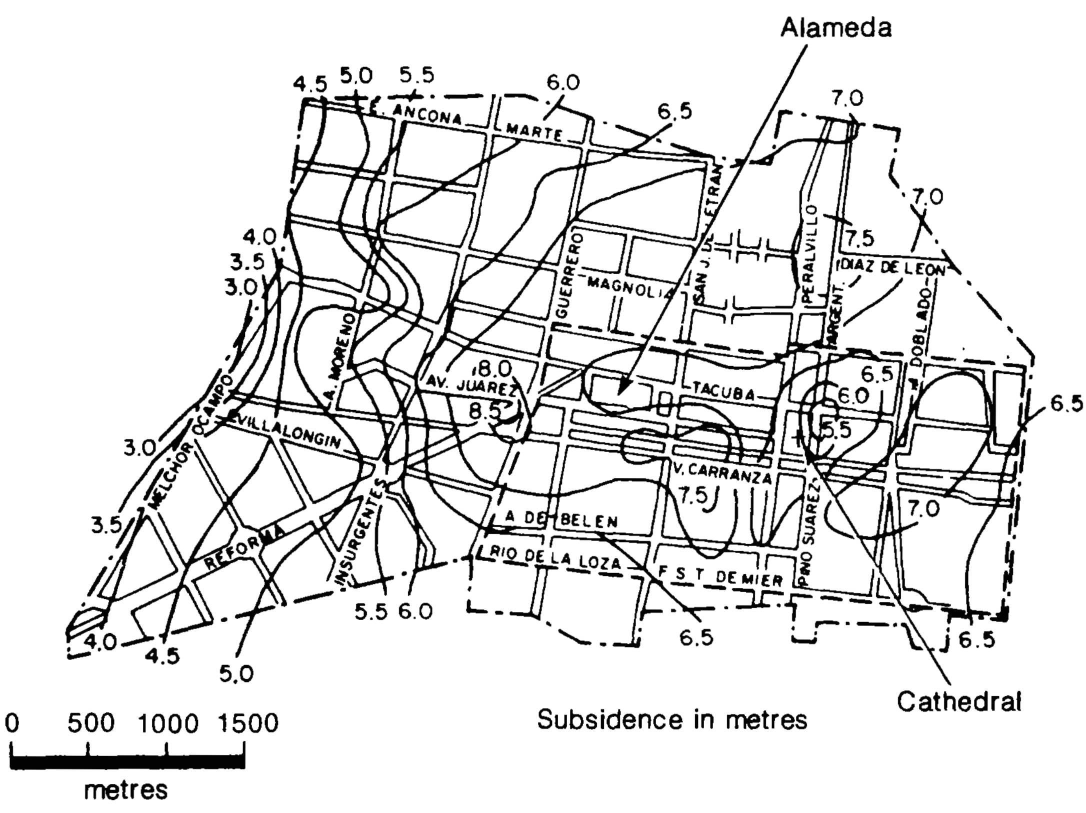


DAMPING RATIOS FOR CLAYS

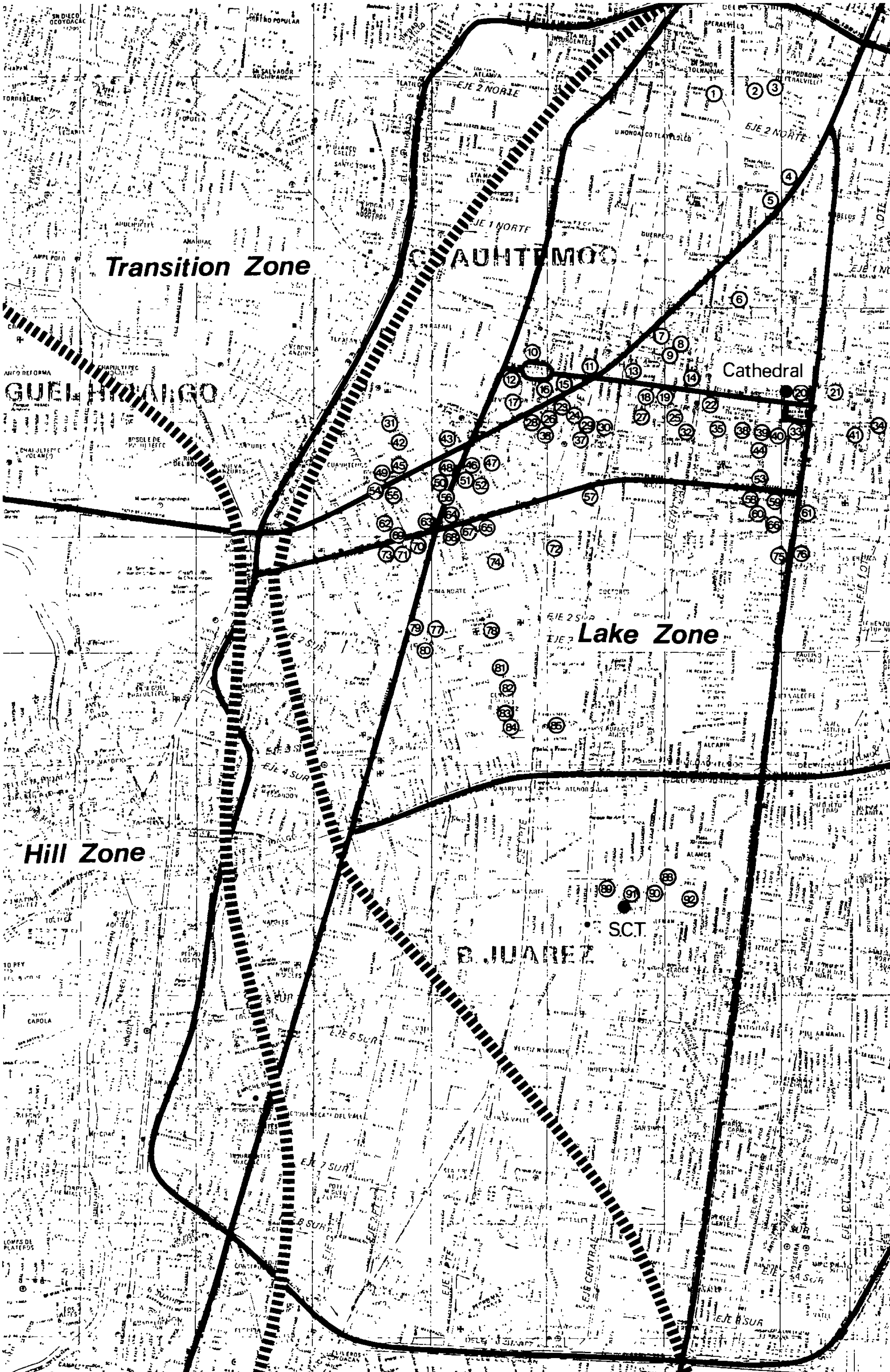
FIGURE **3.6**

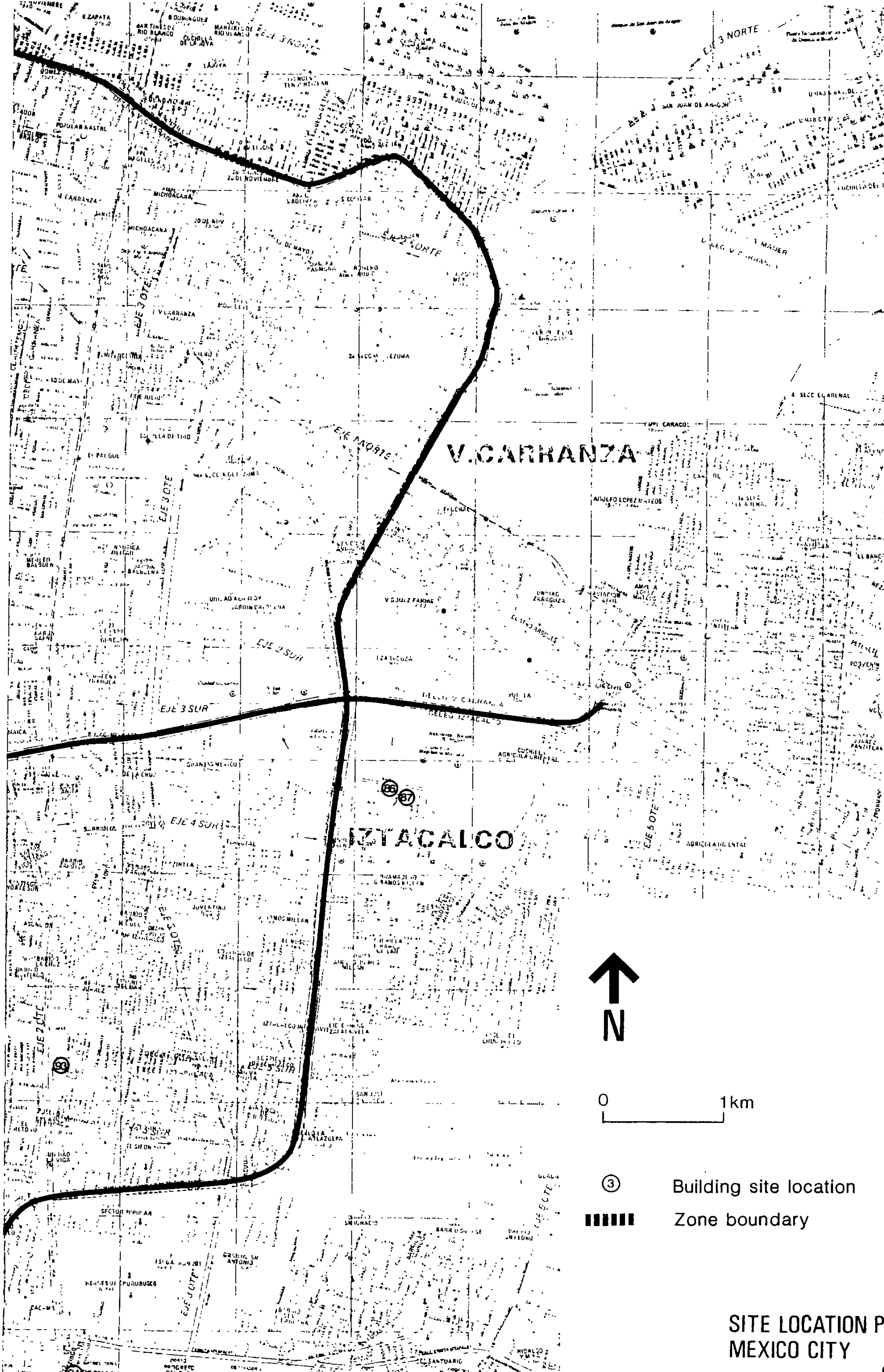


TYPICAL SETTLEMENTS IN MEXICO CITY
FIGURE 3.7

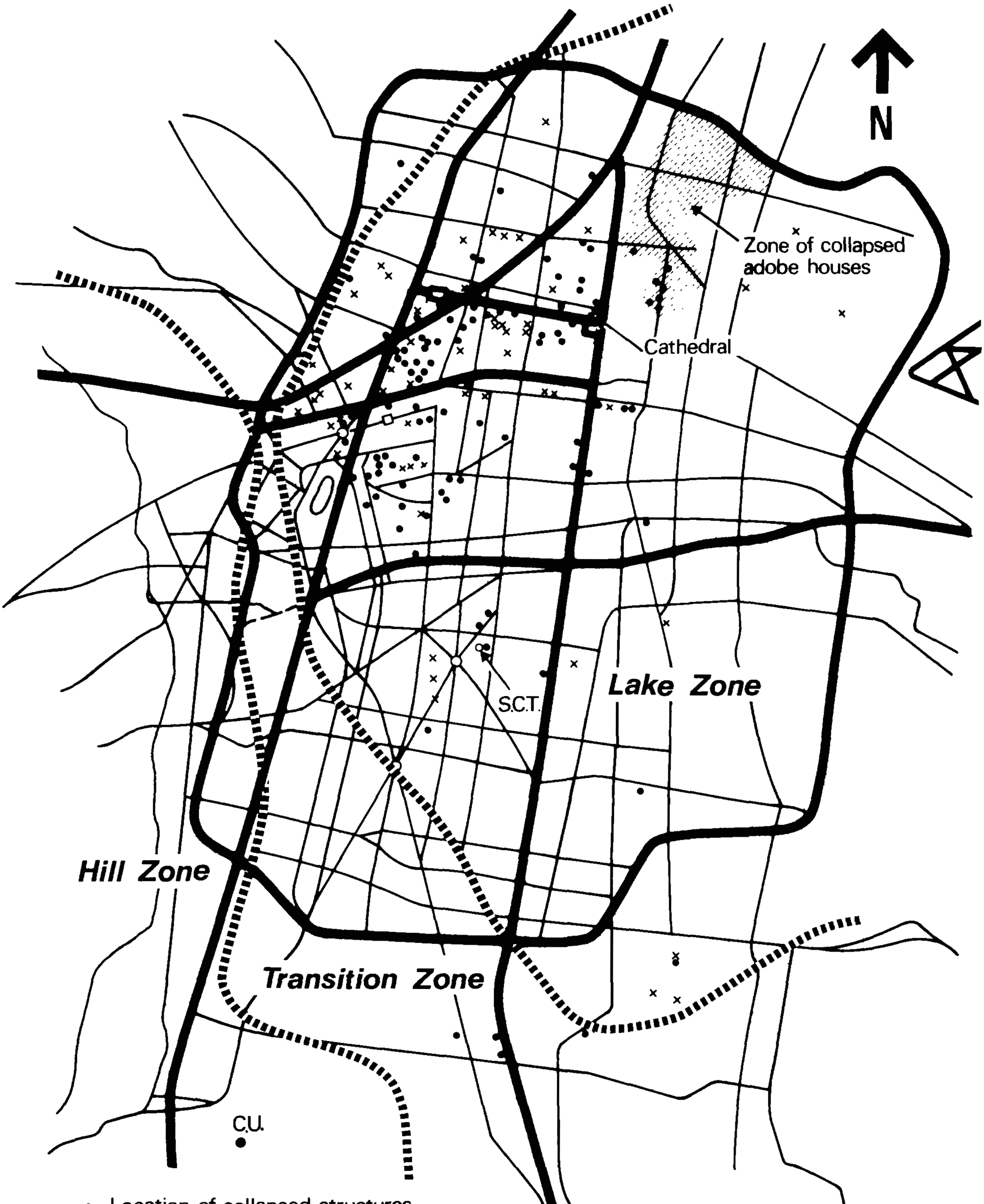


CONTOURS OF EQUAL SUBSIDENCE
1891 — 1970 FOR MEXICO CITY
FIGURE 3.8


Transition Zone
Lake Zone
Hill Zone
Cathedral
S.C.T.
CUAUHTEMOC
B. JUAREZ
EJE 2 NORTE
EJE 1 NORTE
EJE 2 SUR
EJE 4 SUR
EJE 6 SUR
EJE 7 SUR
EJE 8 SUR
EJE CENTRAL



SITE LOCATION PLAN
MEXICO CITY
FIGURE 3.9



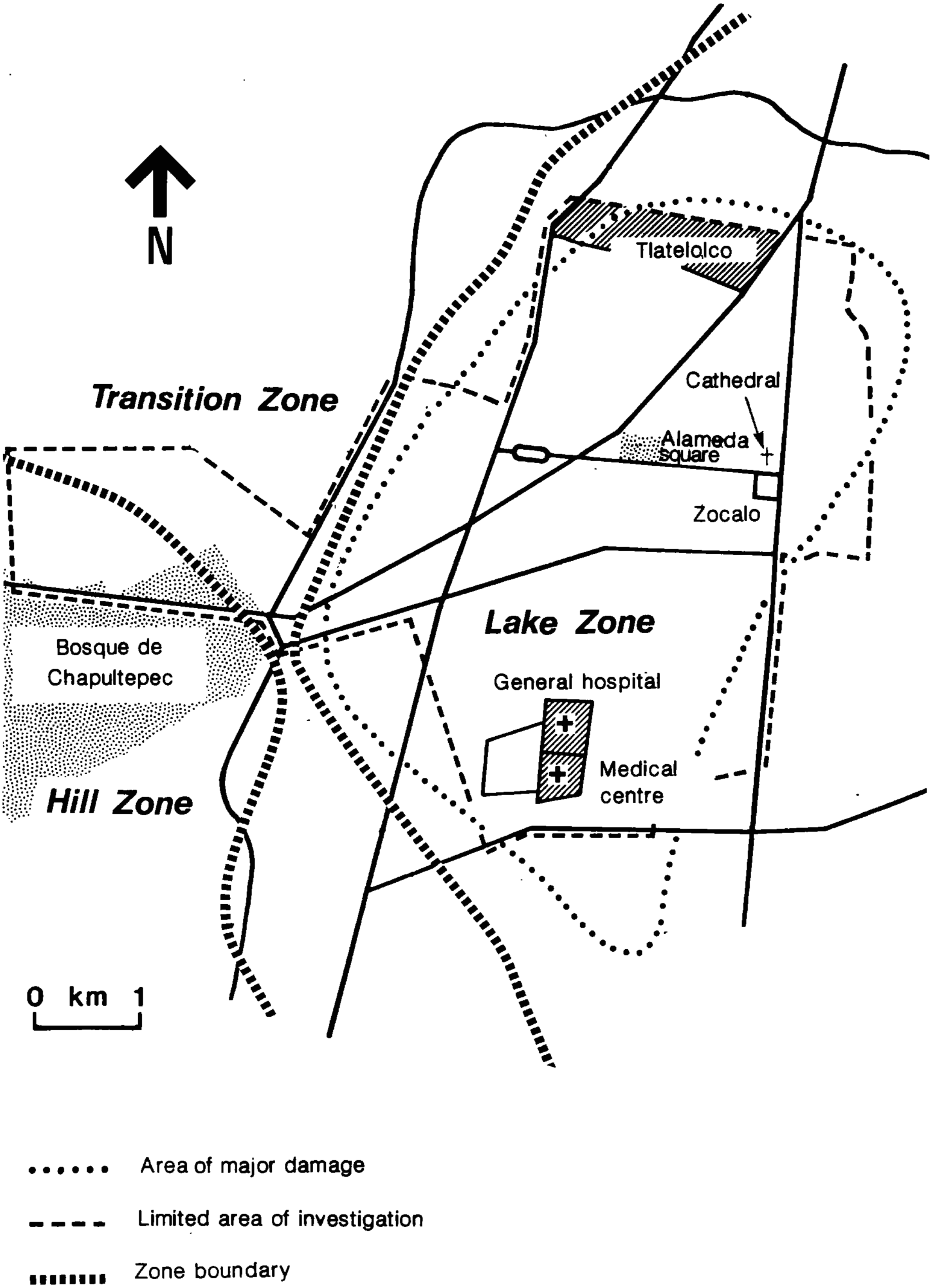
• Location of collapsed structures
× Location of severly damaged structures

0 1 2 3km

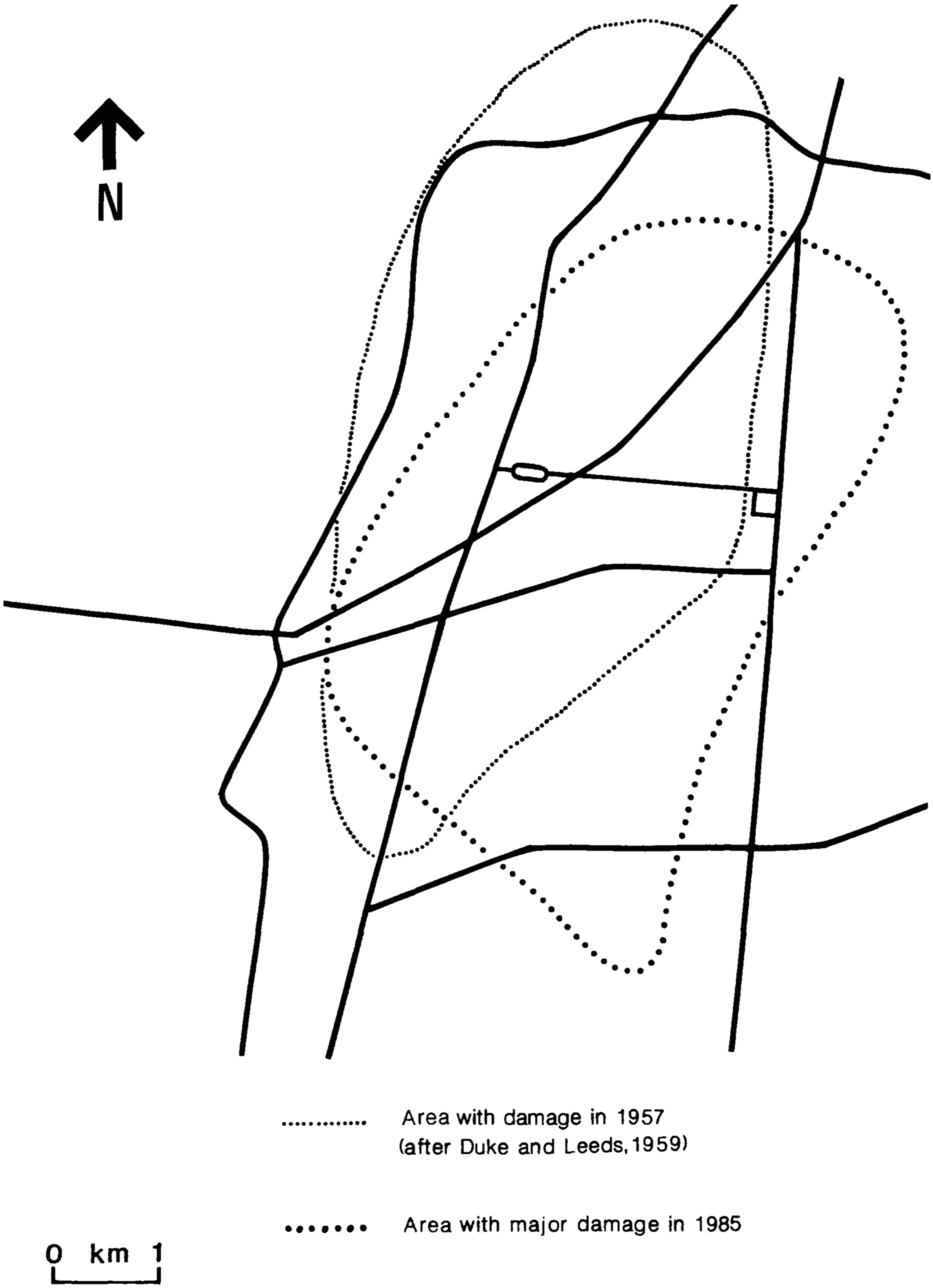
Source : Meli et al 1985

LOCATION OF COLLAPSED
OR SEVERELY DAMAGED BUILDINGS
FROM UNAM SURVEY

FIGURE 3.10



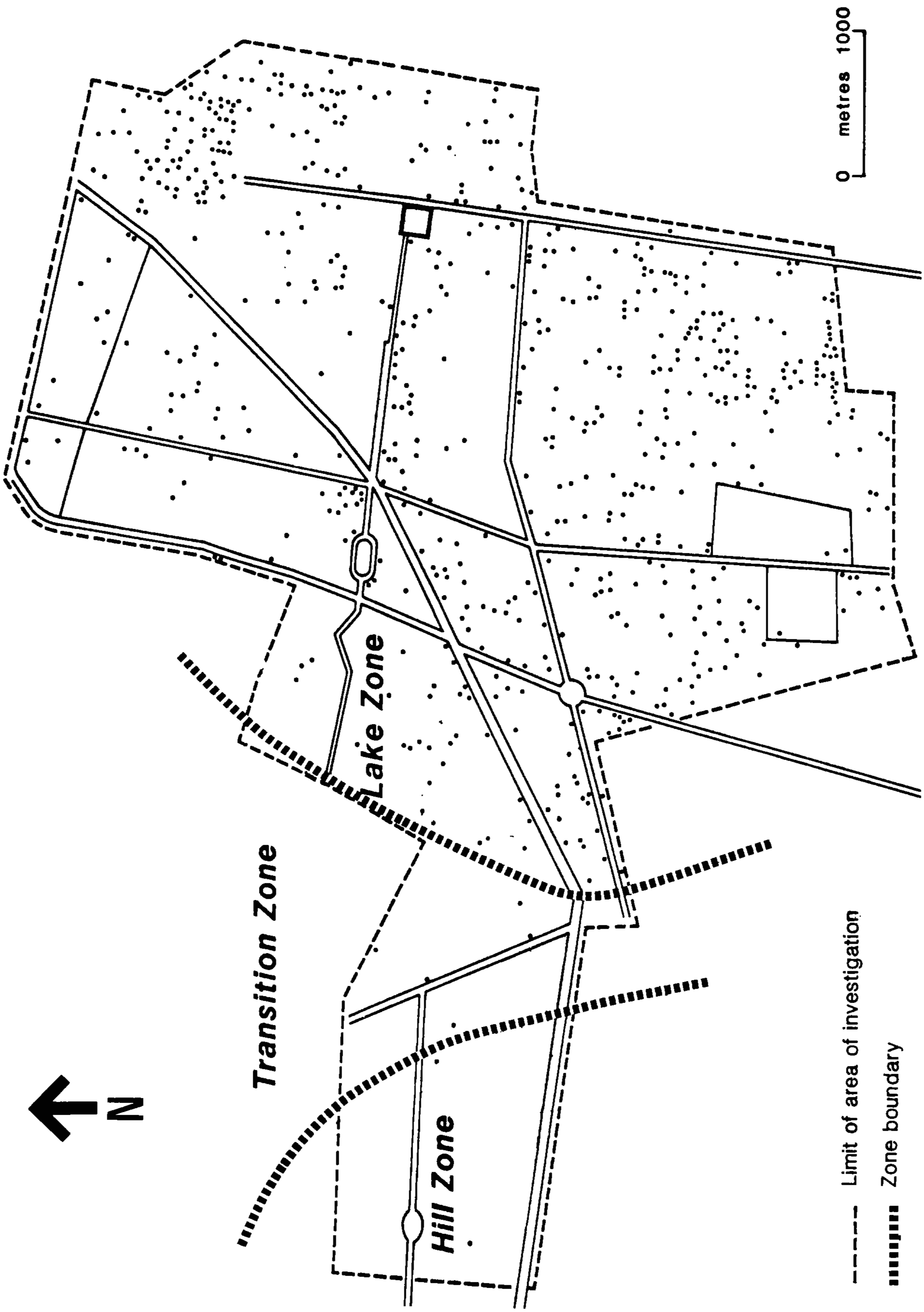
LOCATION OF DAMAGE ZONES
& AREA OF DETAILED SURVEY
FIGURE 3.11



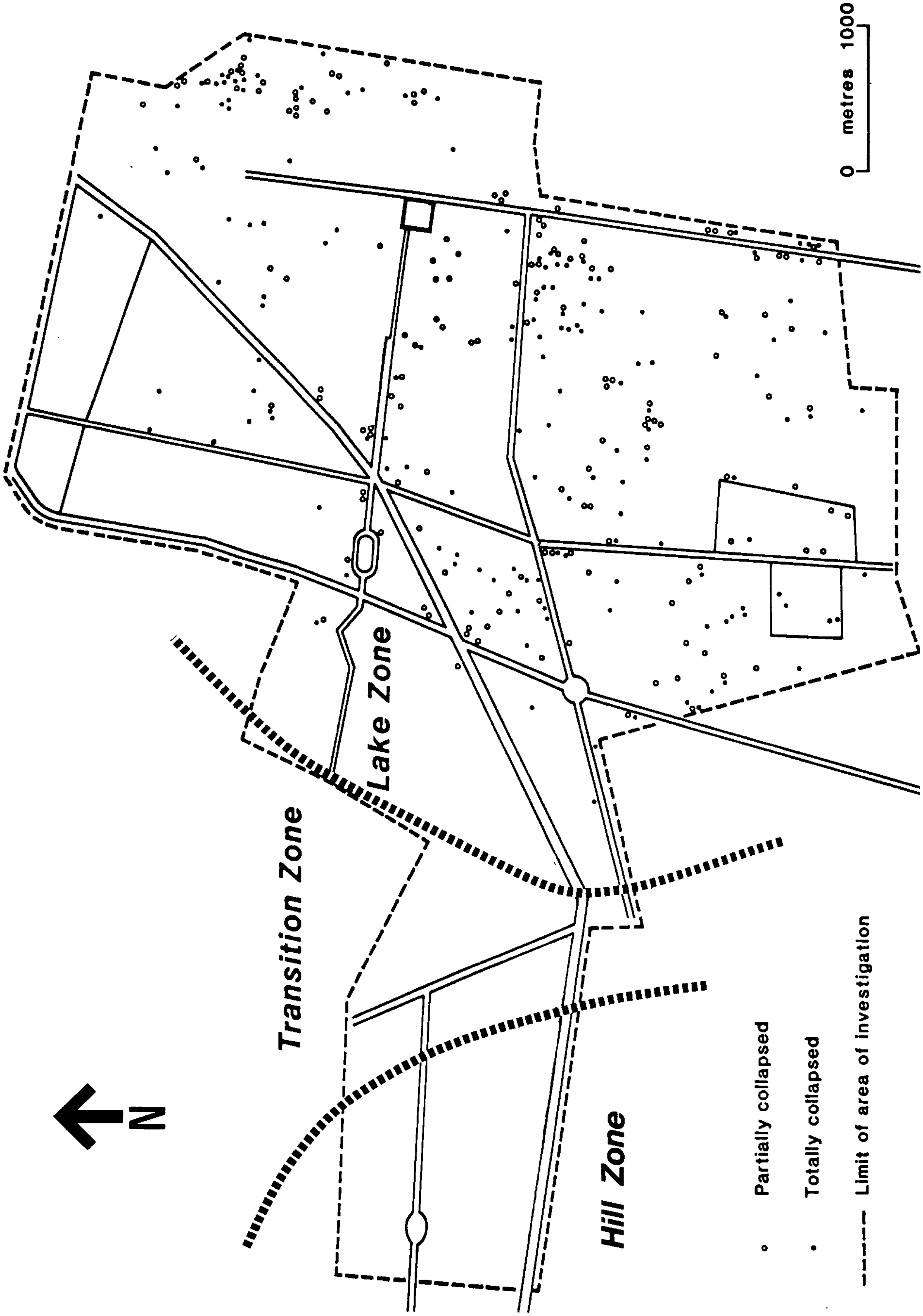
Area with damage in 1957
(after Duke and Leeds, 1959)

Area with major damage in 1985

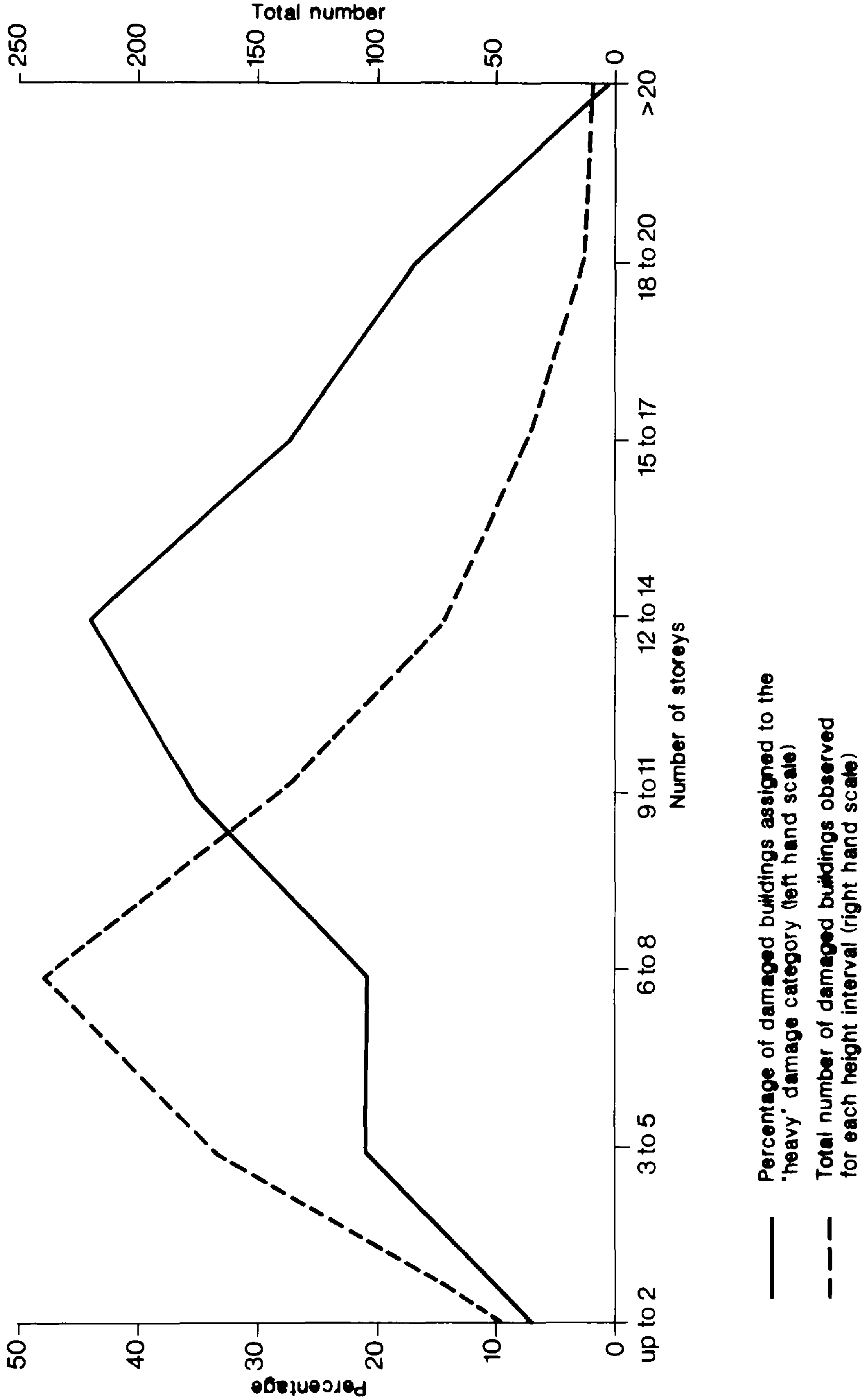
LOCATION OF DAMAGE ZONES
1957 & 1985 EARTHQUAKES
FIGURE 3.12



LOCATION OF LIGHTLY
DAMAGED BUILDINGS
FIGURE 3.13

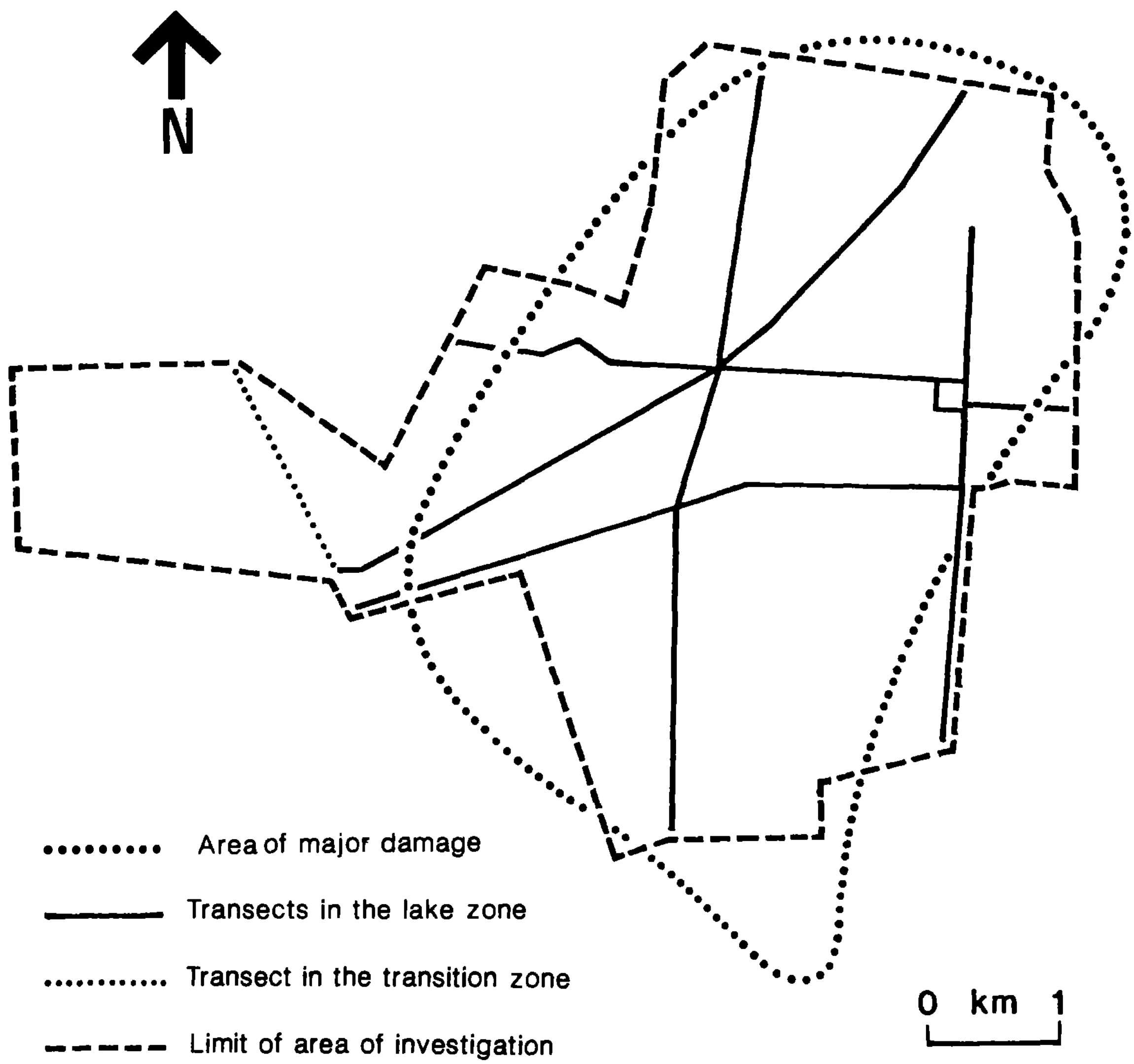


LOCATION OF PARTIALLY OR
TOTALLY COLLAPSED BUILDINGS
FIGURE 3.14

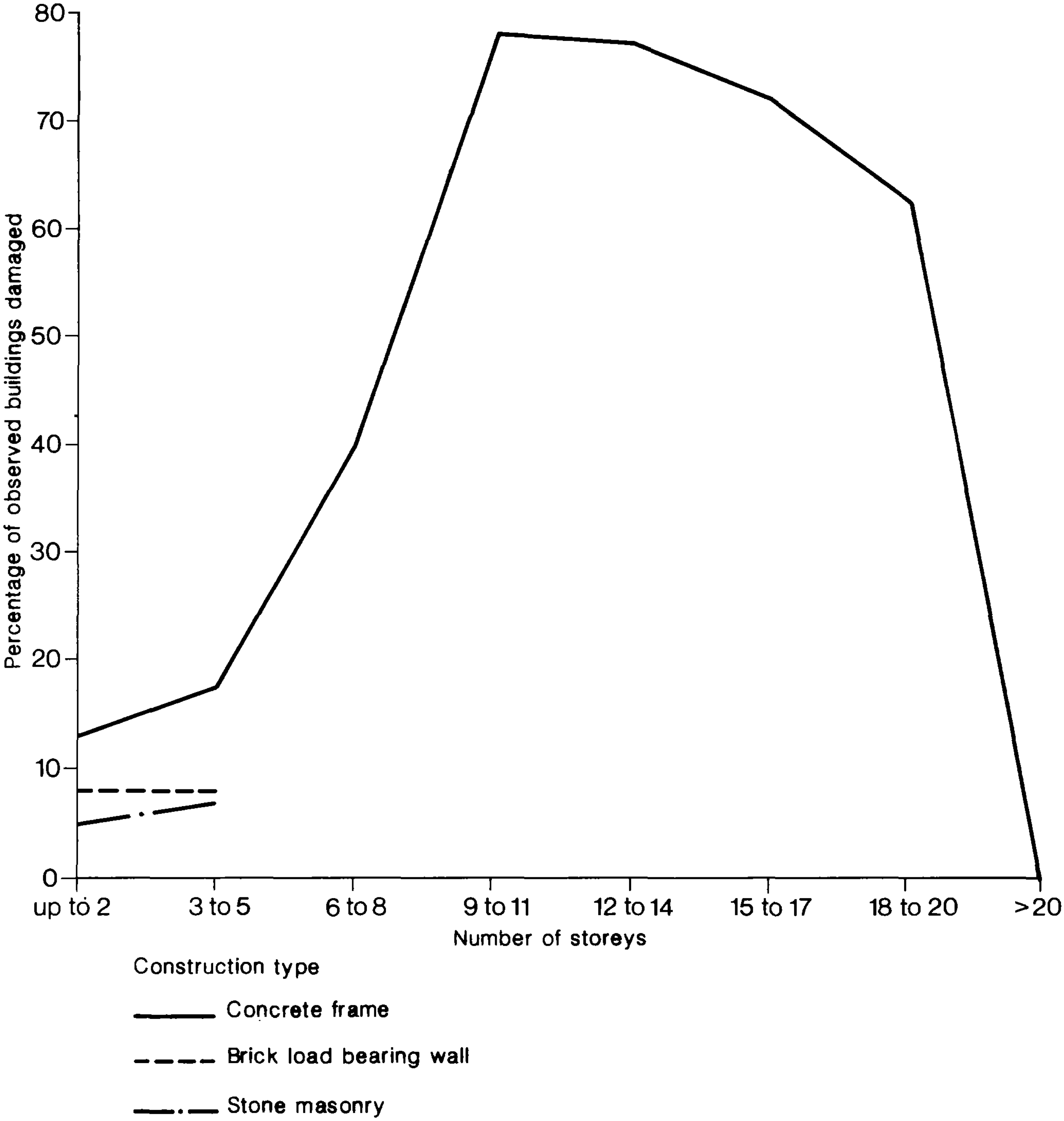


RELATIONSHIP BETWEEN DAMAGE AND NUMBER OF STOREYS

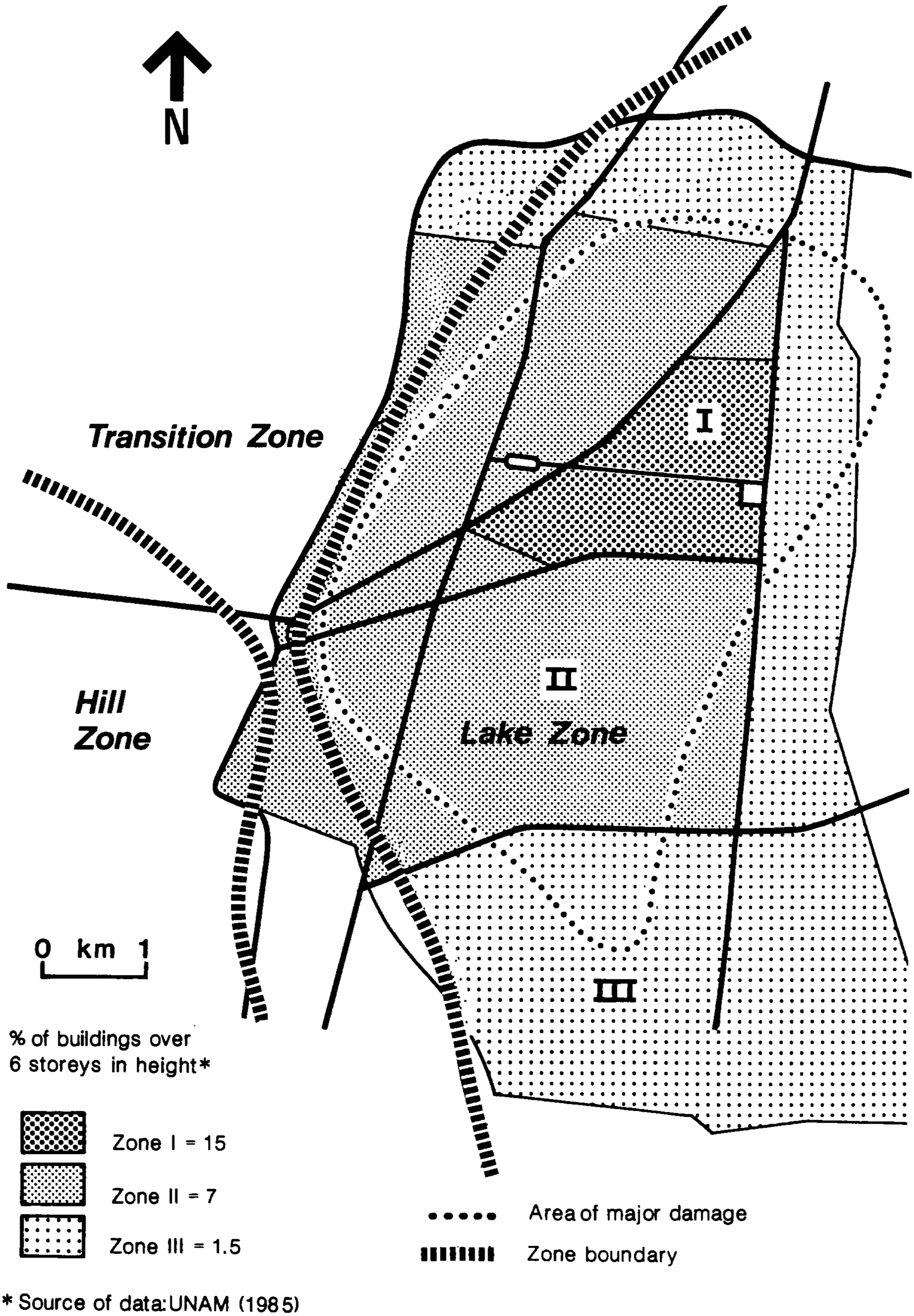
FIGURE 3.15



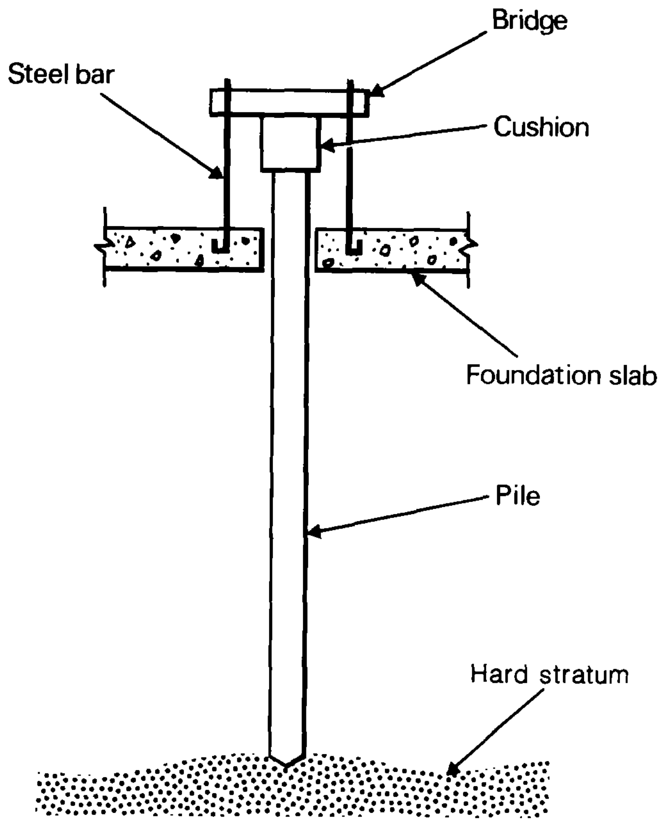
TRANSECT LOCATIONS
FIGURE 3.16



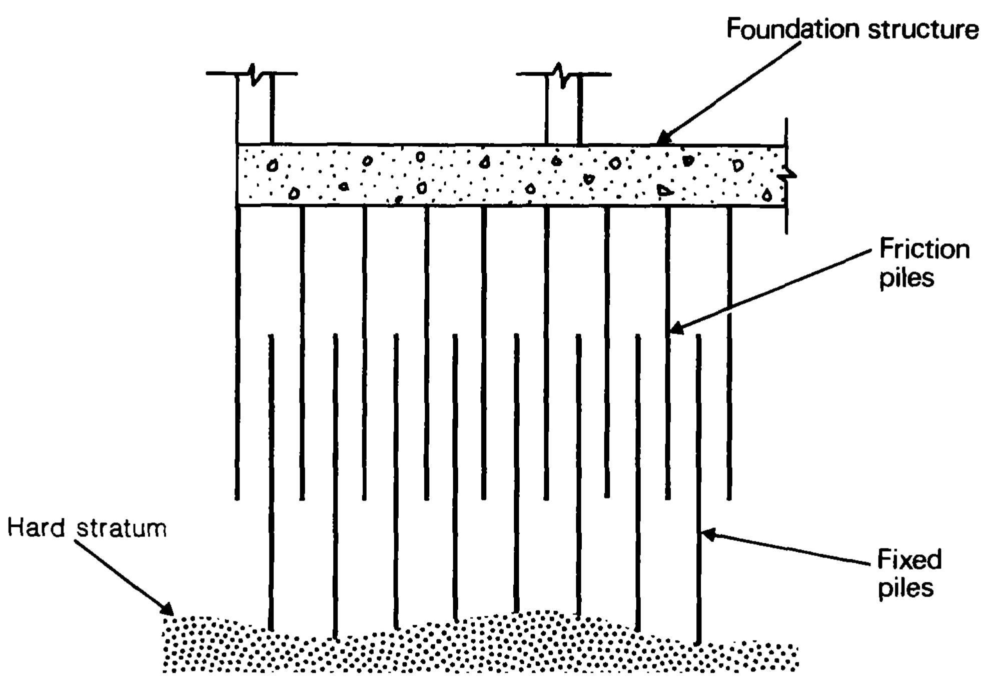
PERCENTAGE DAMAGE
AGAINST NUMBER OF STOREYS
FIGURE 3.17



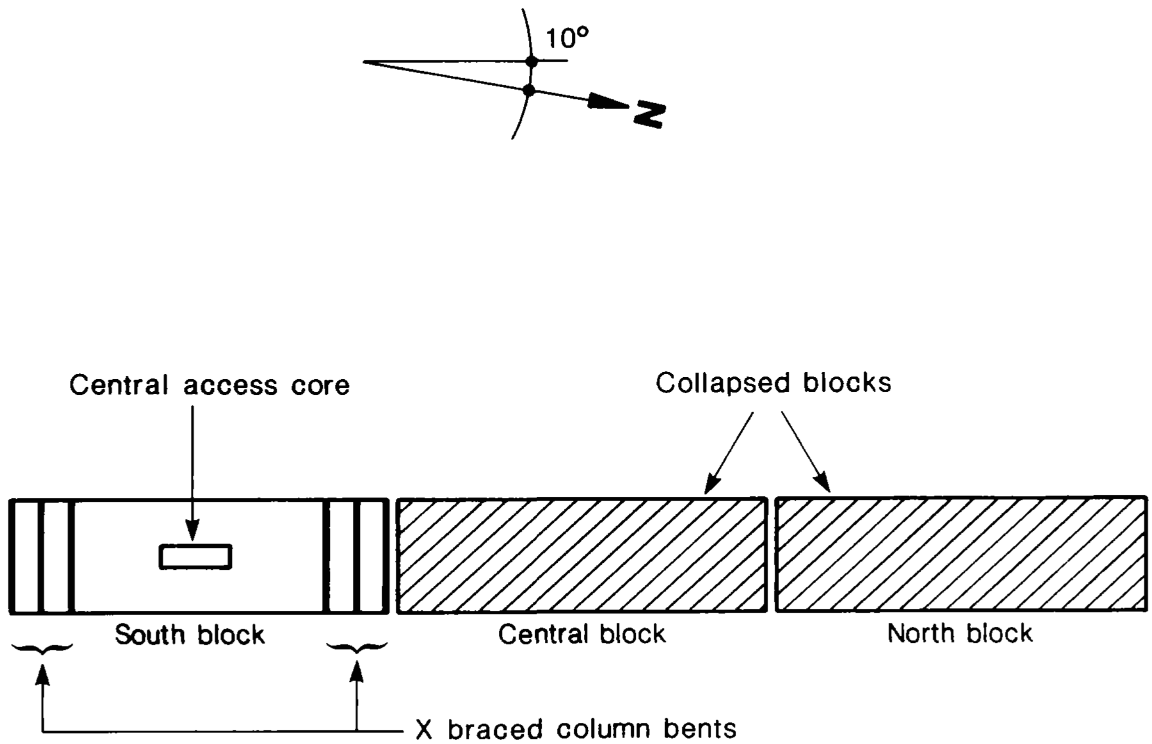
DISTRIBUTION OF
HIGH RISE BUILDINGS
FIGURE 3.18



CONTROL PILES
FIGURE 3.19

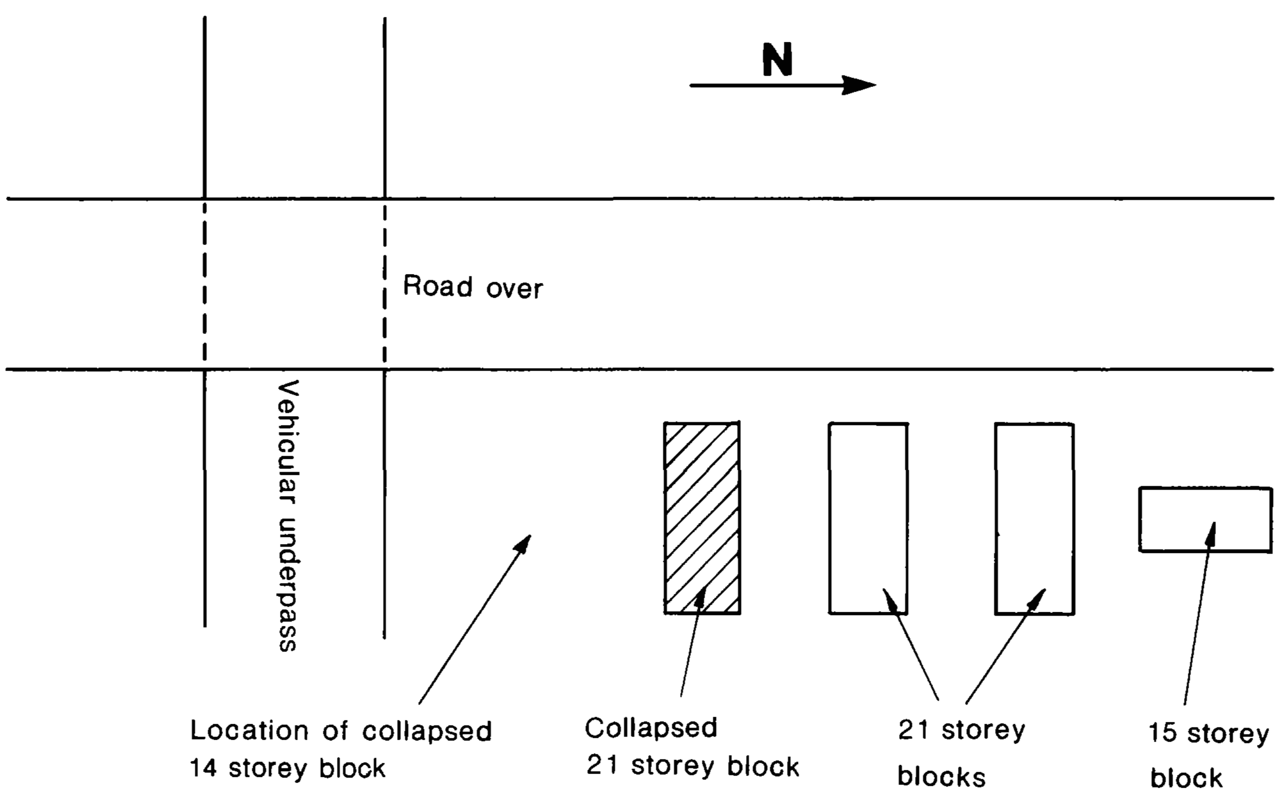


INTERLACED PILES
FIGURE 3.20



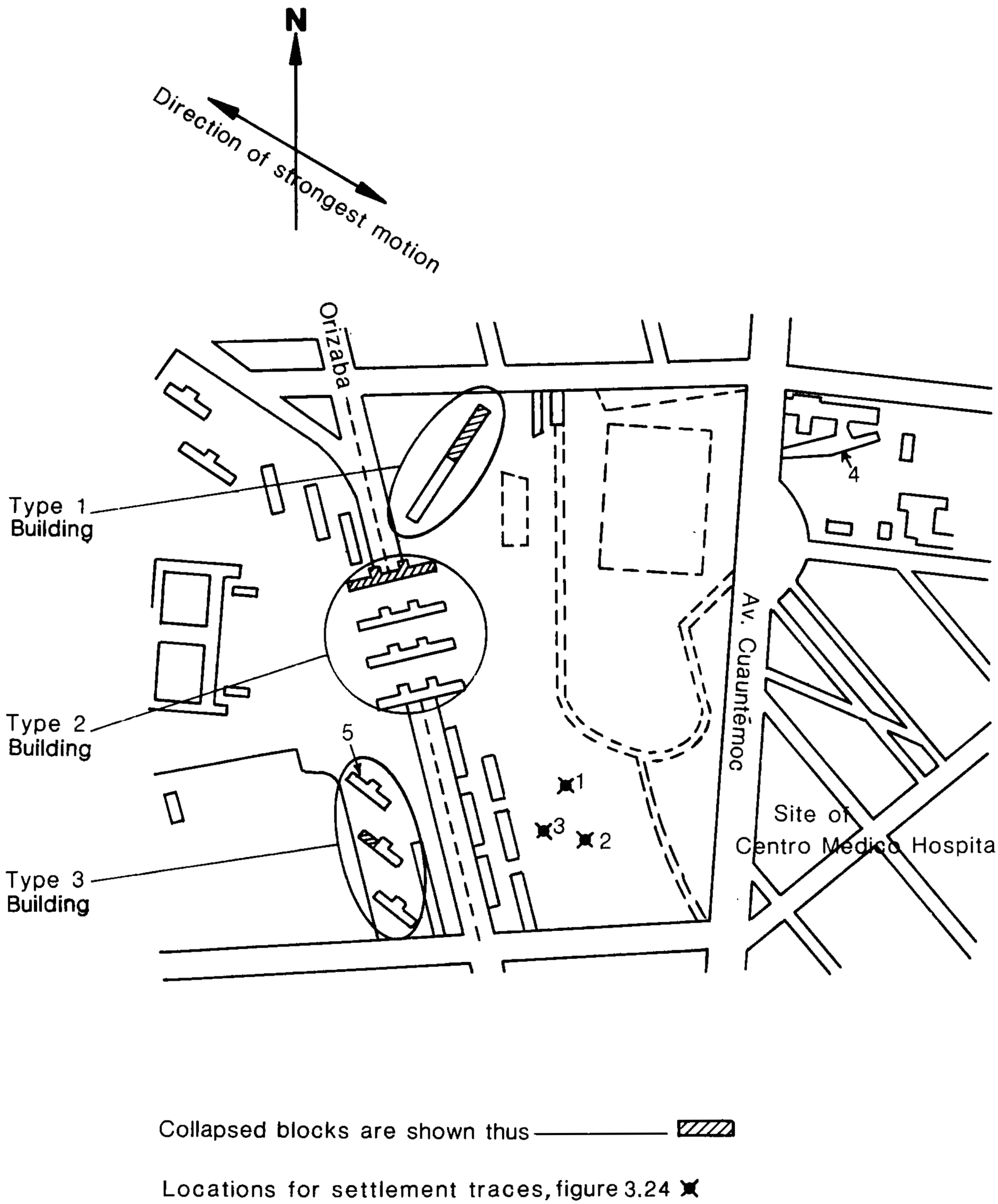
SKETCH PLAN OF NUEVO LEON BUILDING (SITE 4)

FIGURE 3.21

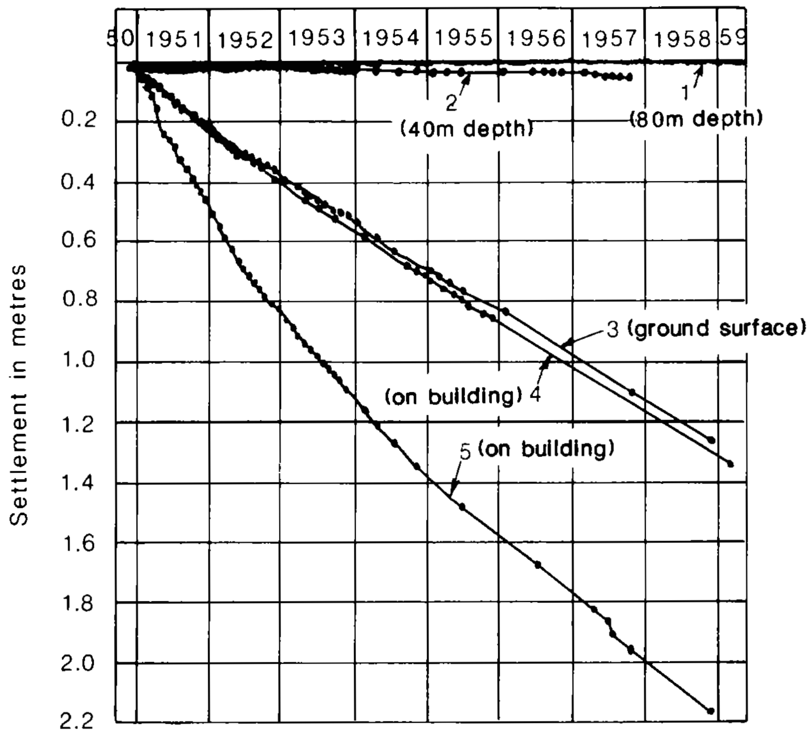


SKETCH PLAN OF PINO SUAREZ BUILDINGS (SITE 61)

FIGURE 3.22

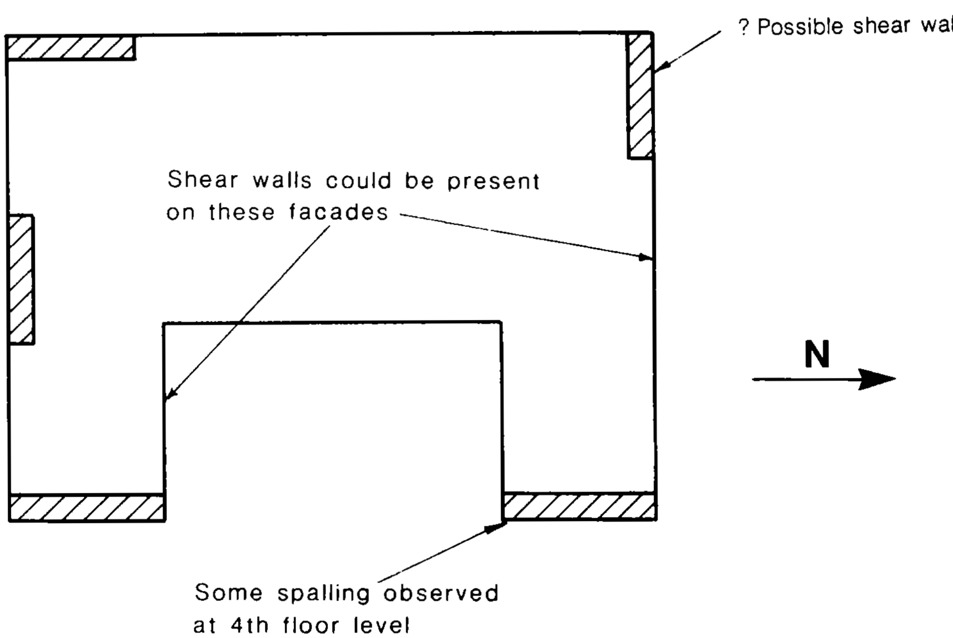


PLAN OF BENITO JUAREZ
HOUSING ESTATE (SITE 82)
FIGURE 3.23

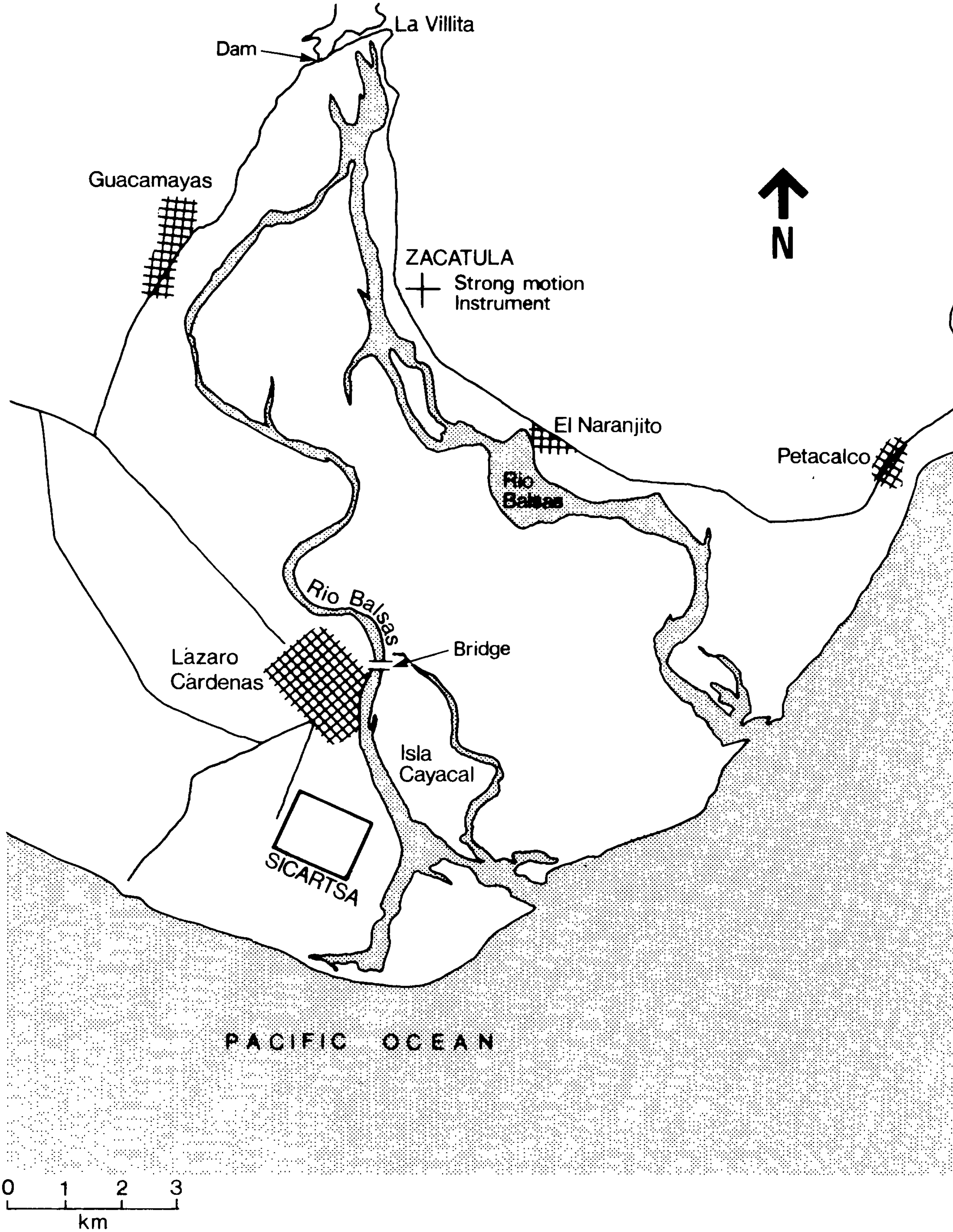


For positions of settlement see figure 3.23

SETTLEMENTS IN BENITO JUAREZ
FIGURE 3.24

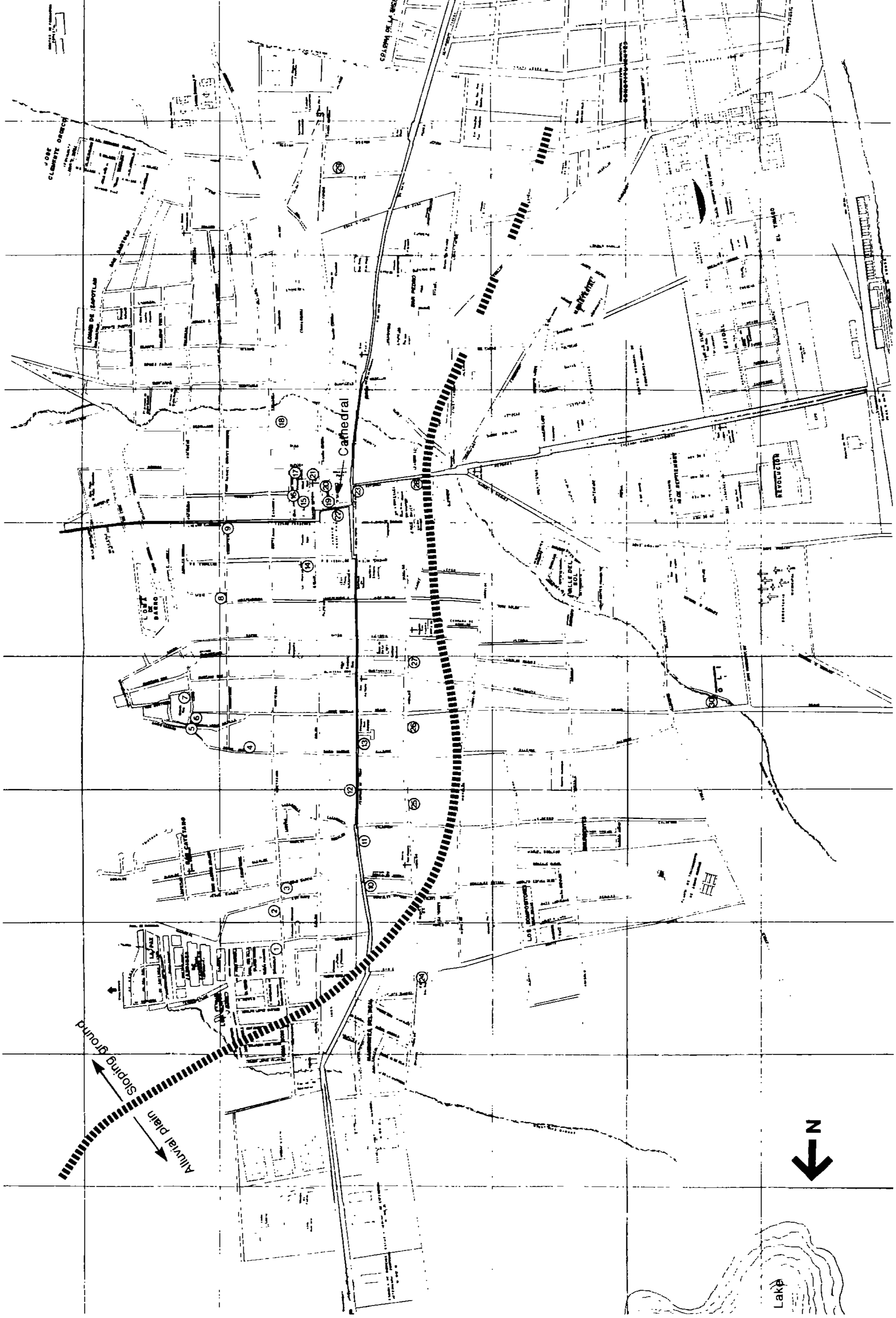


SKETCH PLAN OF SHEAR WALL STRENGTHENED BUILDING (SITE 77)
FIGURE 3.25



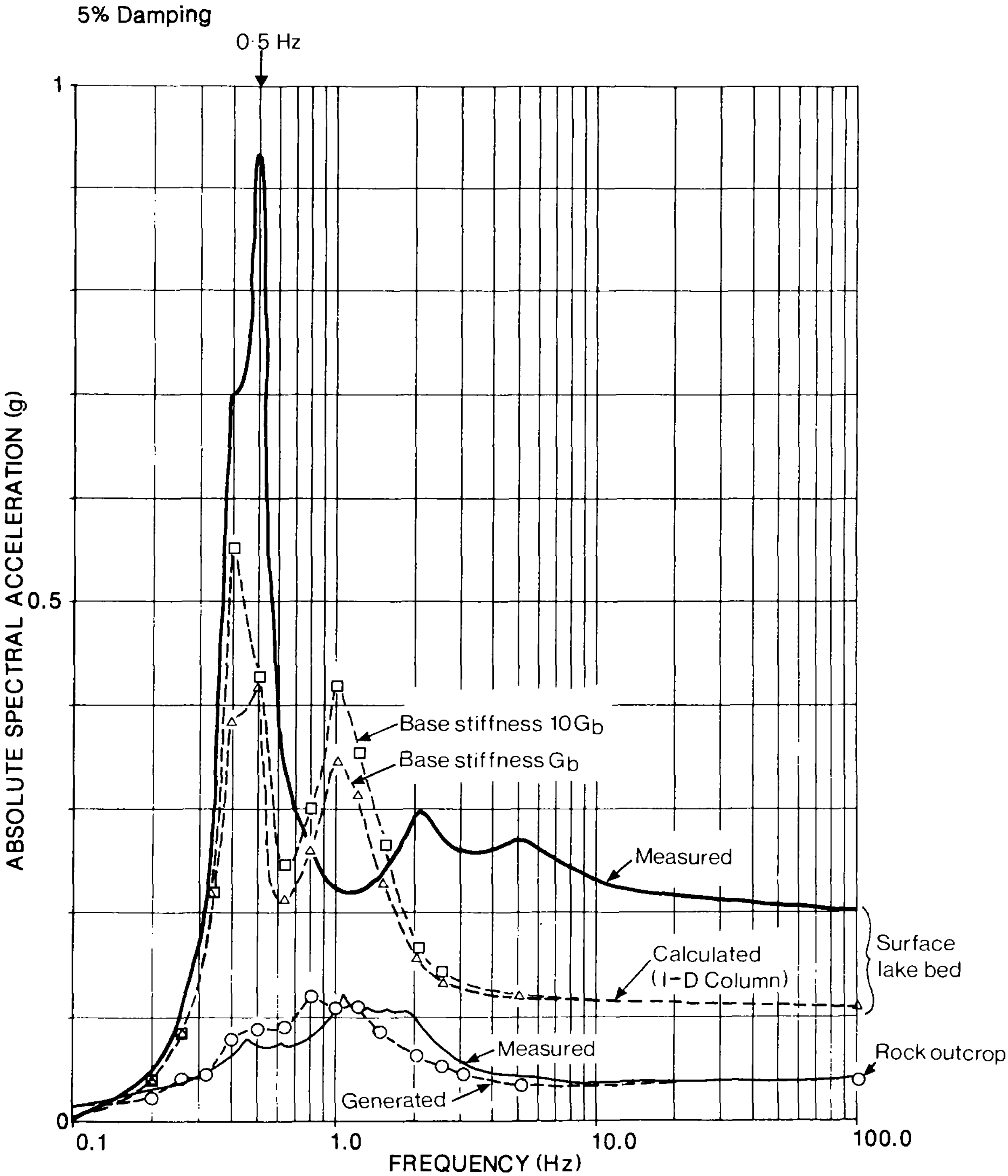
MAP OF LAZARO CARDENAS
AND ITS ENVIRONS

FIGURE 4.1



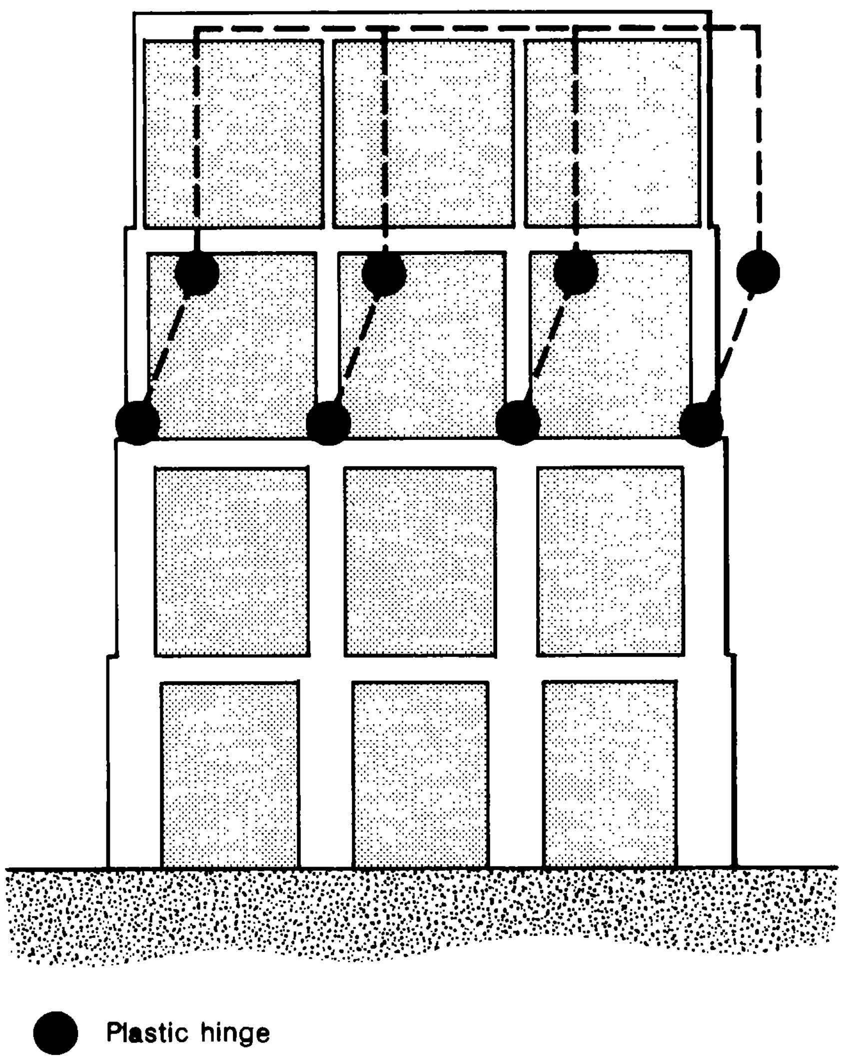
MAP OF CIUDAD GUZMAN

FIGURE 4.2

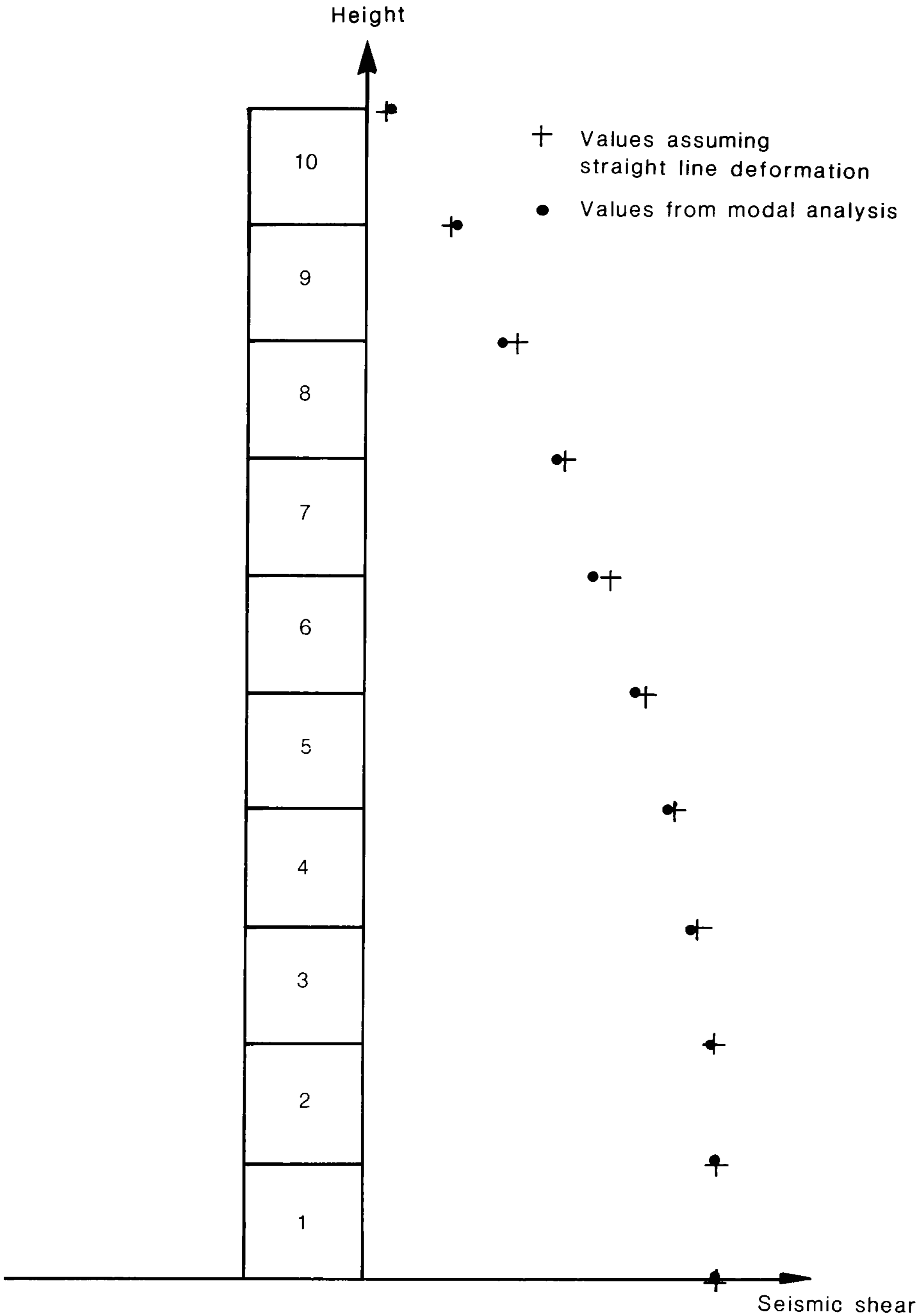


PREDICTED AND MEASURED ACCELERATION RESPONSE SPECTRUM

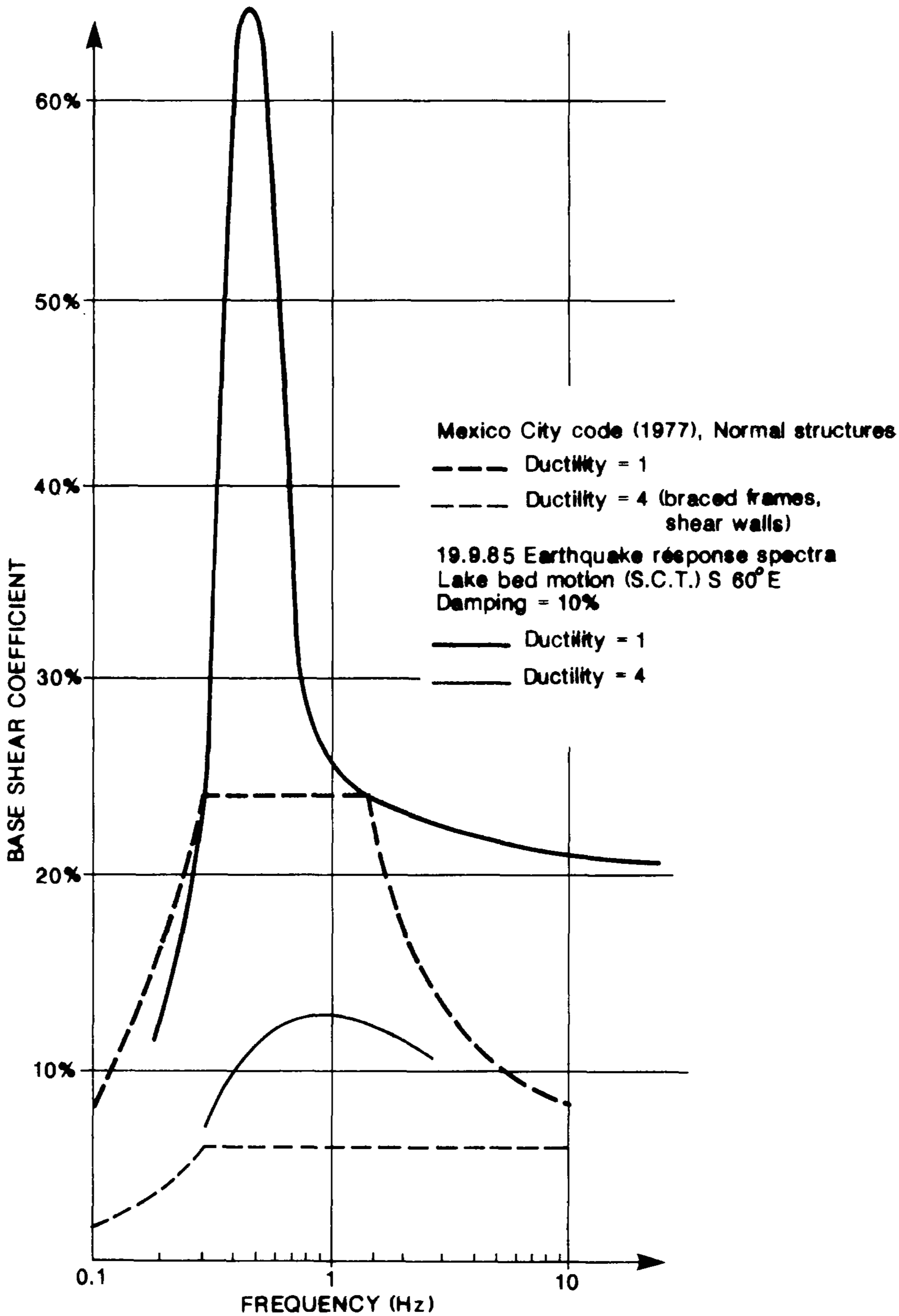
FIGURE 6.1



COLUMN SWAY MECHANISM
FIGURE 7.1

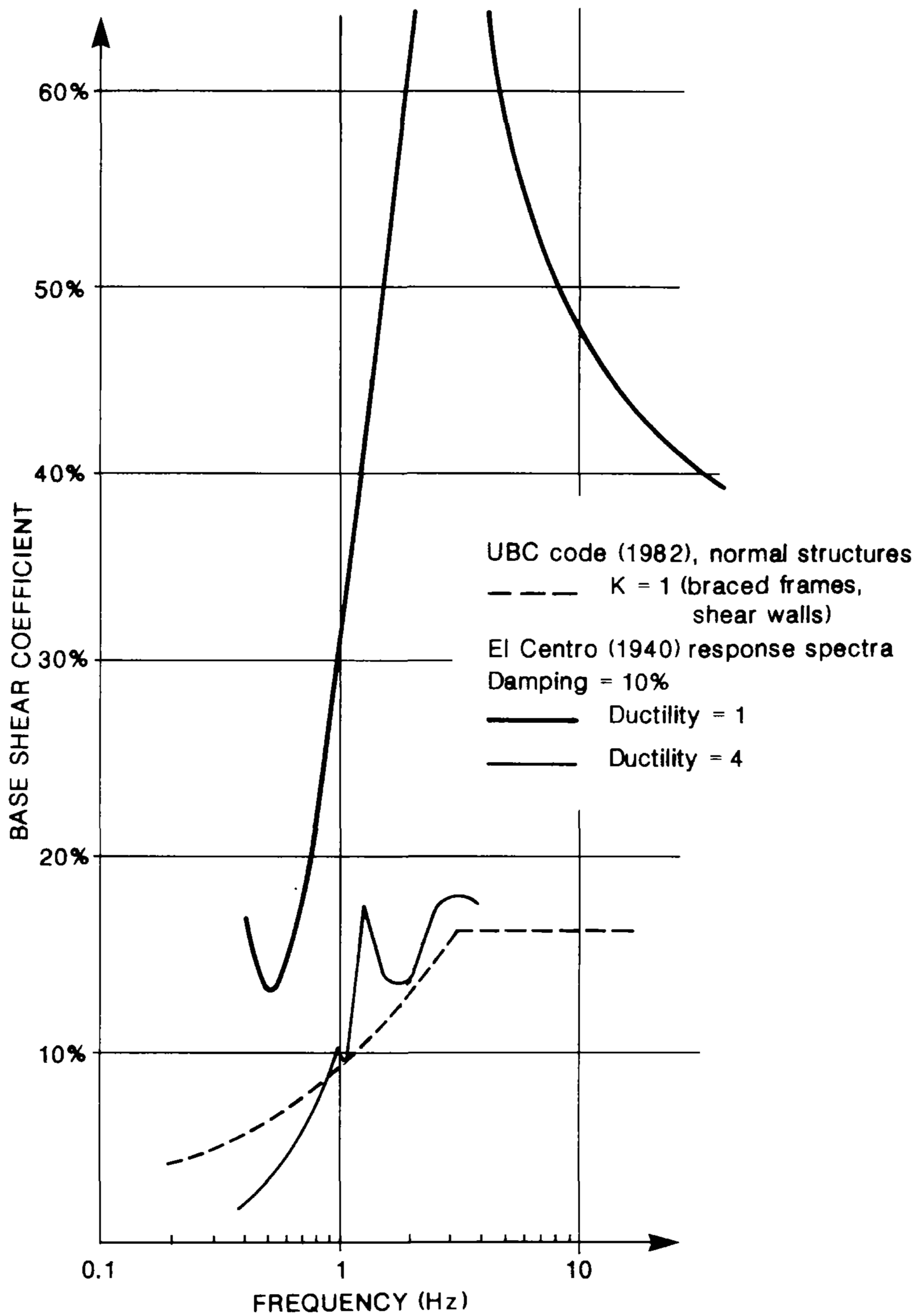


DYNAMIC SHEARS IN
A 10 STOREY BUILDING
FIGURE 7.2



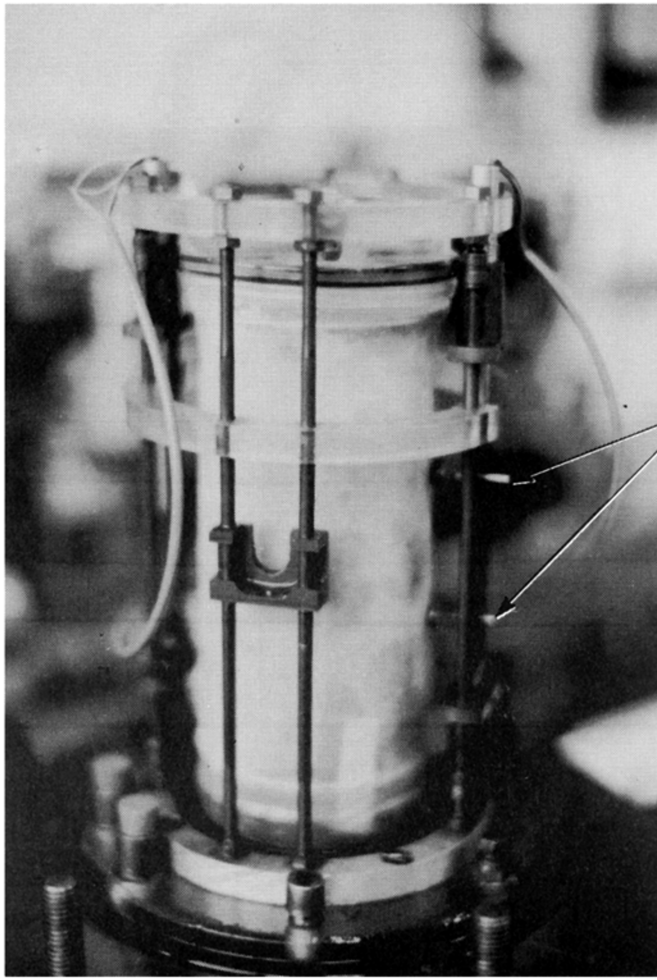
COMPARISON OF CODE FORCES
WITH RESPONSE SPECTRA
FOR LAKE BED MOTIONS

FIGURE 7.3



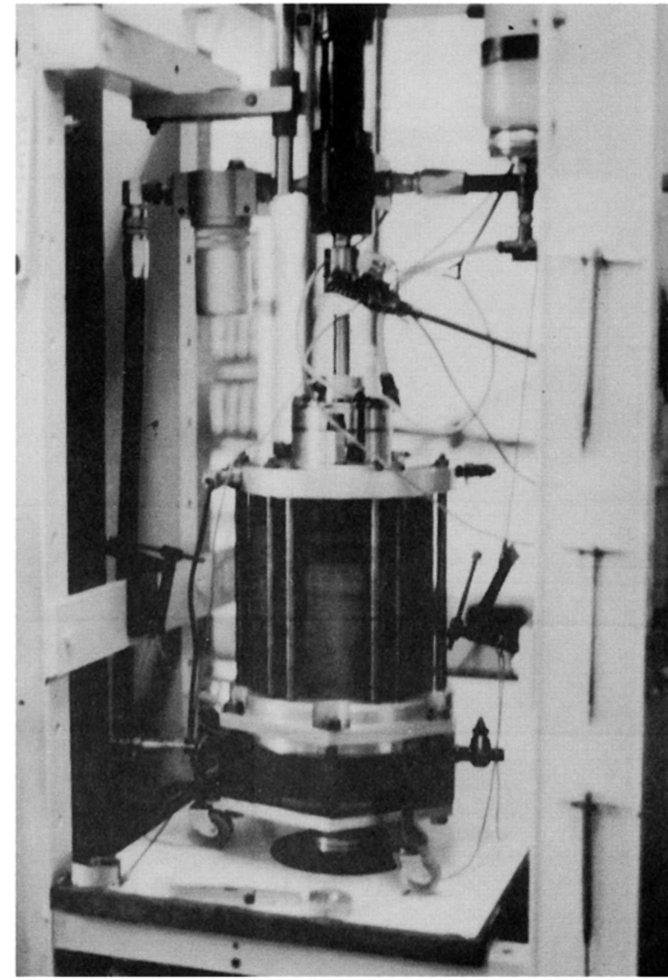
COMPARISON OF UBC CODE FORCES WITH RESPONSE SPECTRA FOR EL CENTRO 1940

FIGURE 7.4



Triaxial clay sample with monitoring stubs

PLATE 3.1



Clay sample in repeated loading triaxial apparatus

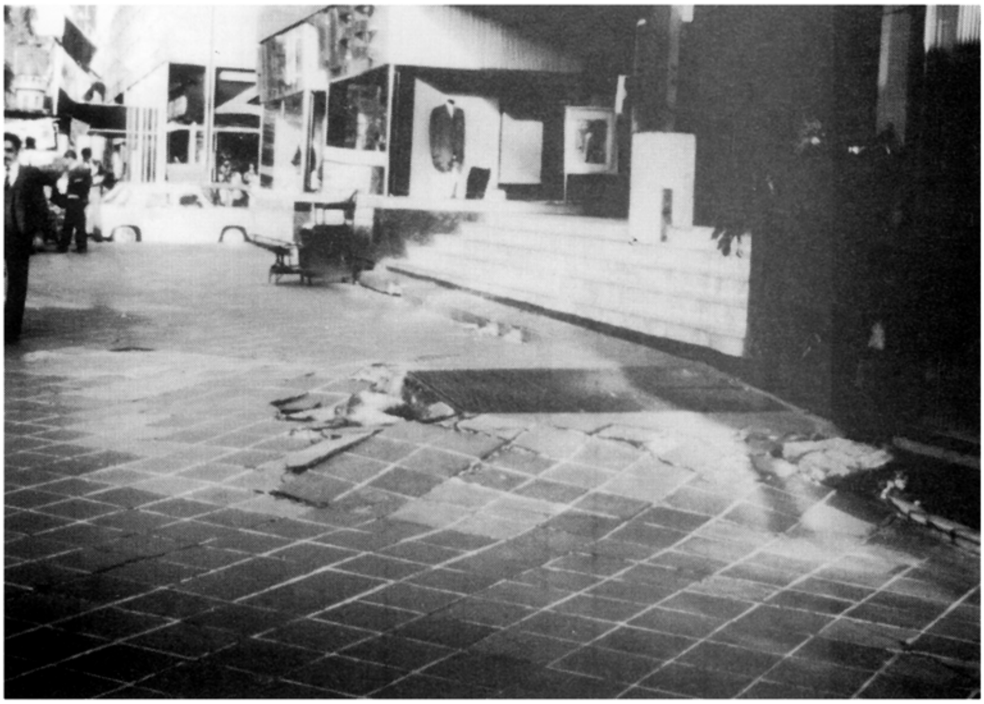
PLATE 3.2

Previous settlement at Guadalupe Cathedral

PLATE 3.3

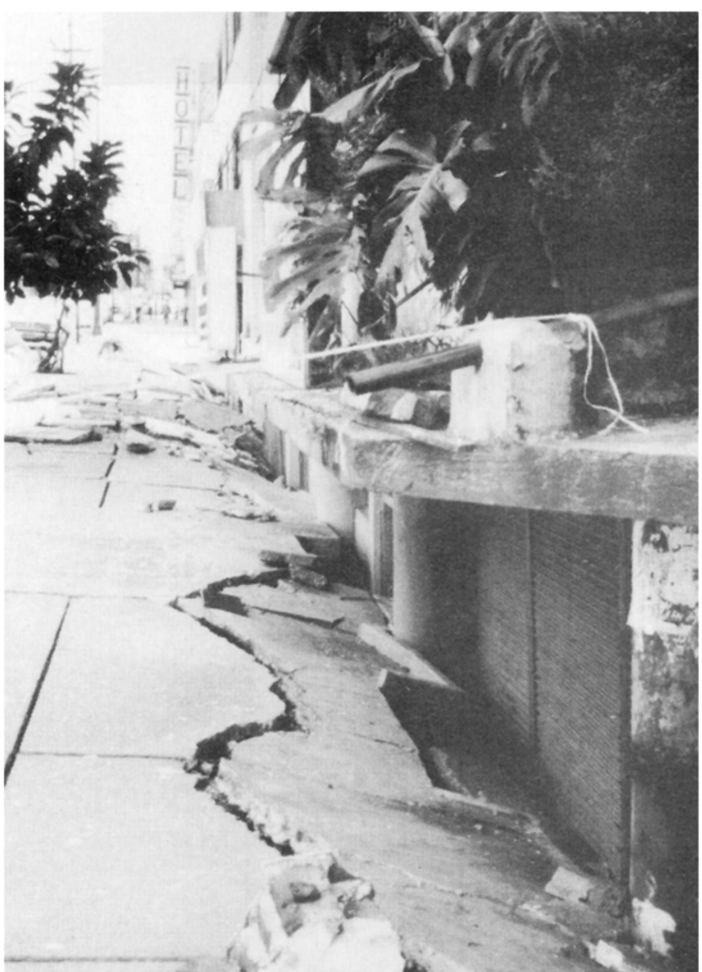
Previous emergence at Independence Monument (Site 54)

PLATE 3.4



Slight Emergence Damage (Site 28)

PLATE 3.5



Settled Building (Site 74)

PLATE 3.6

Sloping Building (Site 29)

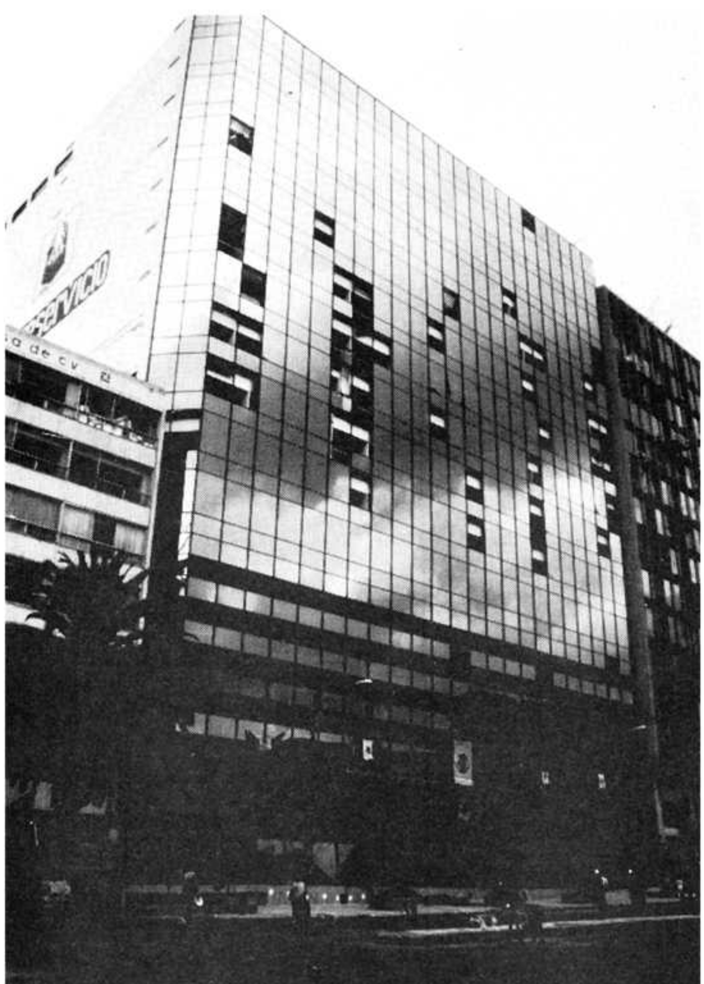
PLATE 3.7

Foundation Failure by Rotation (Site 78)

PLATE 3.8

Foundation Failure by Rotation showing extruded pile (Site 78)

PLATE 3.9



Exterior of Social Security Building (Site 15)

PLATE 3.10

Inside Social Security Building (Site 15)

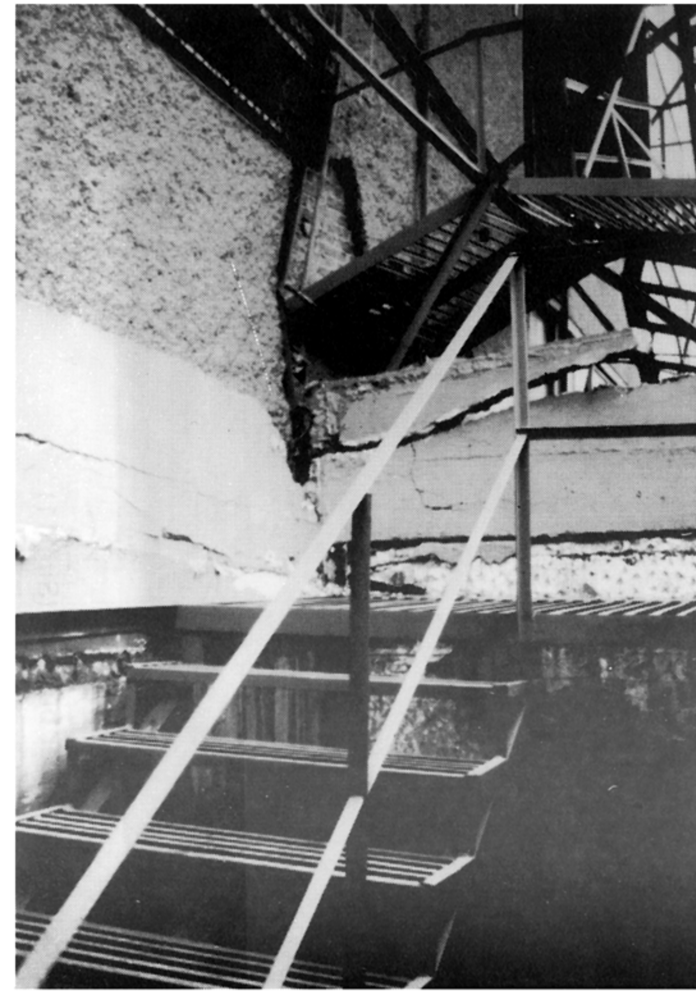
PLATE 3.11

Settled Building at Iztacalco (Site 87)

PLATE 3.12

Ground slab joint at junction of eleven and four storey building

PLATE 3.13



Staircase - Structural damage at joint of eleven and four storey building (Site 87)

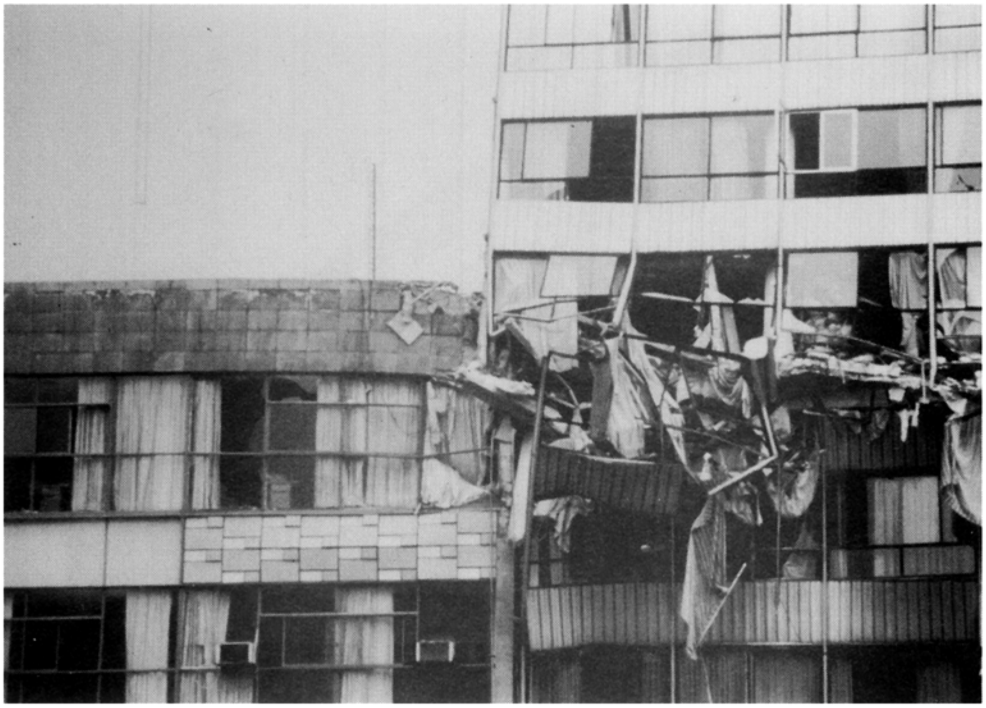
PLATE 3.14

Top down collapse of older building (Site 11)

PLATE 3.15

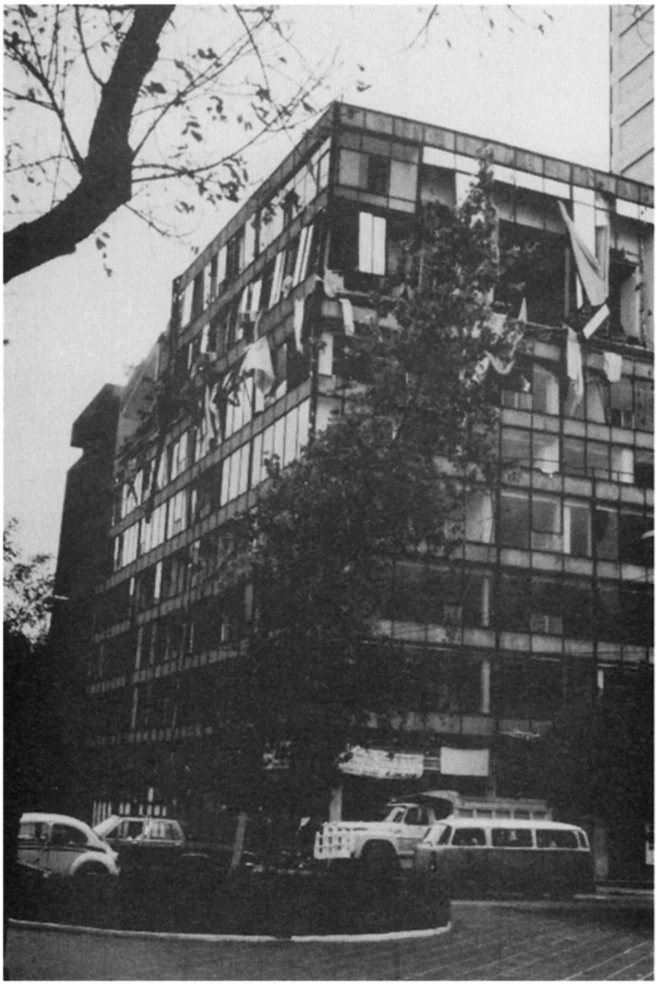
Top down collapse of newer building (Site 60)

PLATE 3.16



Intermediate collapse - buffetting Hotel de Carlo (Site 10)

PLATE 3.17

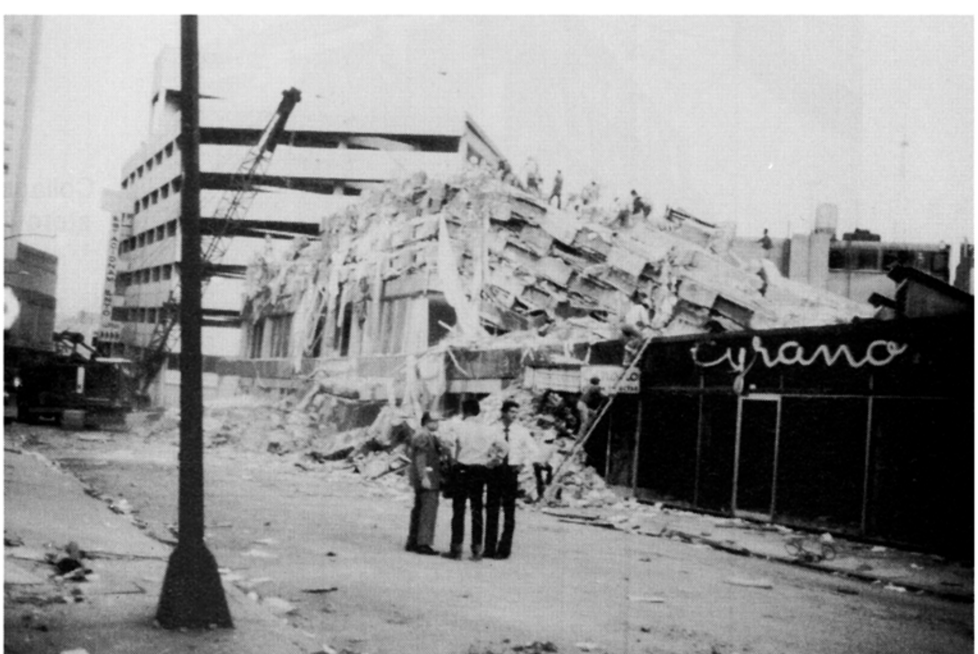


Collapsed storey

Intermediate collapse - isolated building Costa Rica Airlines (Site 43)

PLATE 3.18

Soft ground floor failure, reinforced concrete building (Site 37) PLATE **3.19**



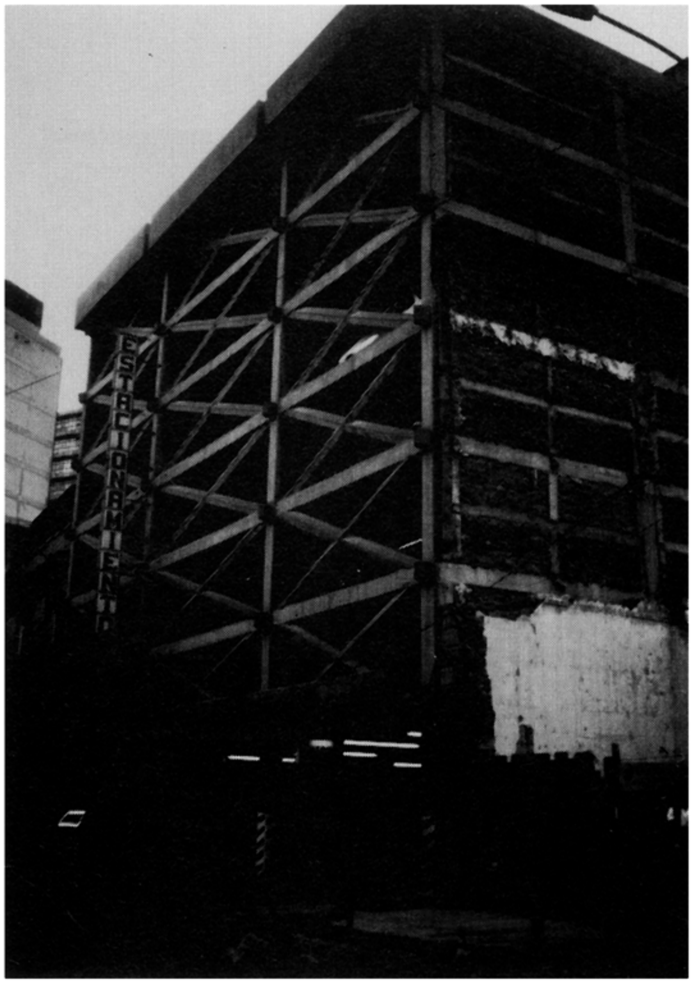
Total pancake collapse (Site 36) PLATE **3.20**

Non-failure in adjacent structures of dissimilar height (Site 24)

PLATE 3.21

Failure of reinforced concrete x-braced
Nuevo Leon Building,
Tlatelolco estate (Site 4)

PLATE 3.22

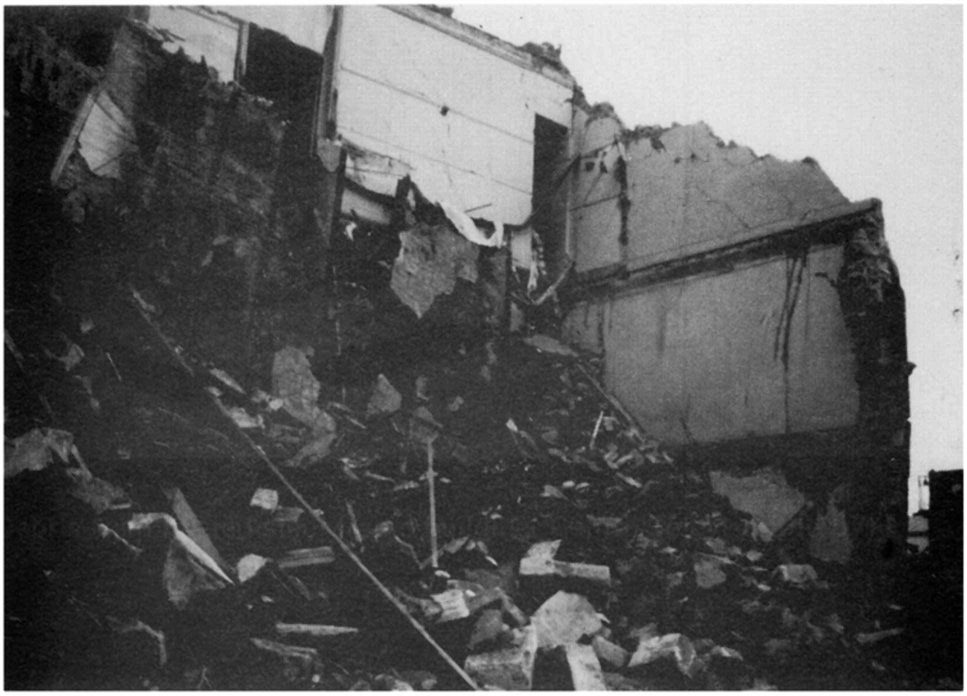


Non-failure of vulnerable soft storey building (Site 23)

PLATE 3.23

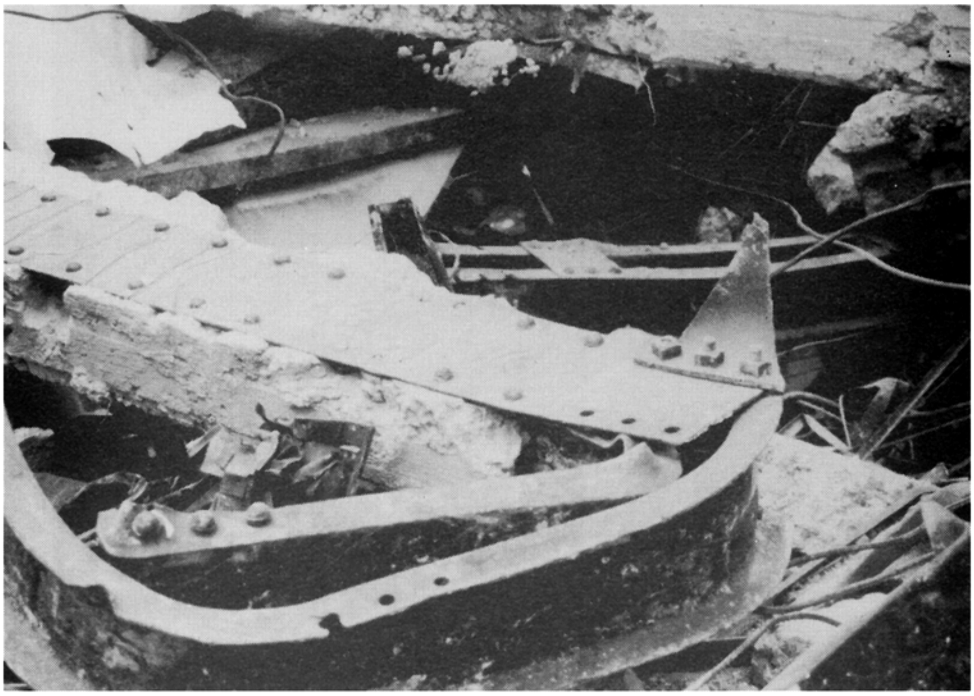
Failure in reinforced concrete shear wall (Site 58)

PLATE 3.24



Failure in 3 storey masonry building with steel joists (Site 30)

PLATE 3.25



Debris from riveted steel frame building, Centro Televiso Site 57)

PLATE 3.26

Downtown Mexico City from the Latin American tower (Site 22)

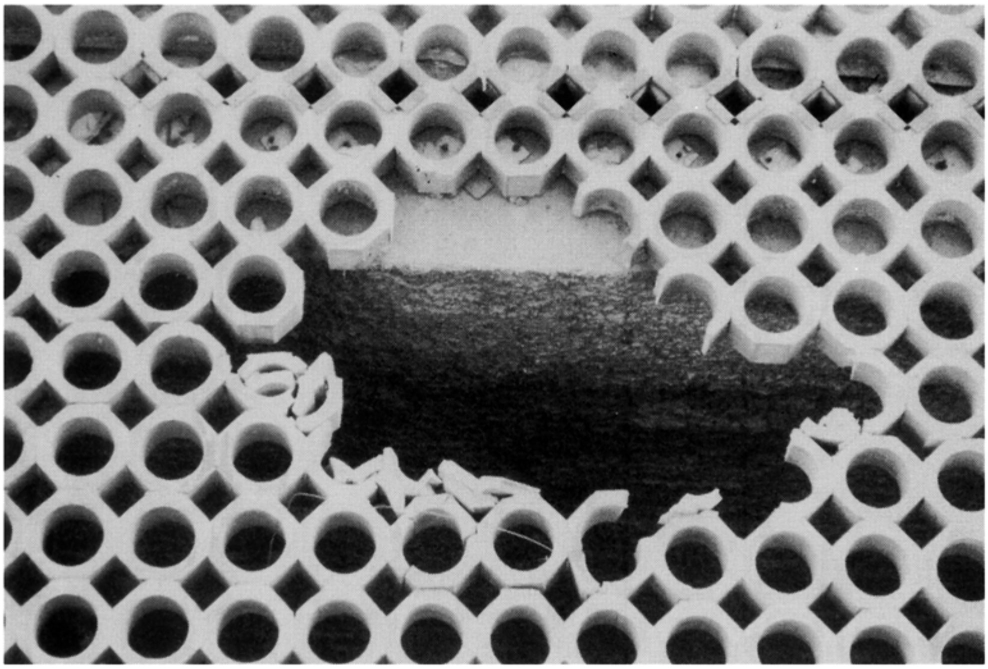
PLATE 3.27

Lightly damaged adobe/brick building (Site 2)

PLATE 3.28

Main Cathedral (Site 20) PLATE 3.29

Building on the Tlatelcolco estate similar to the failed Nuevo Leon PLATE 3.30



Failure of decorative cladding, Tlatelolco estate

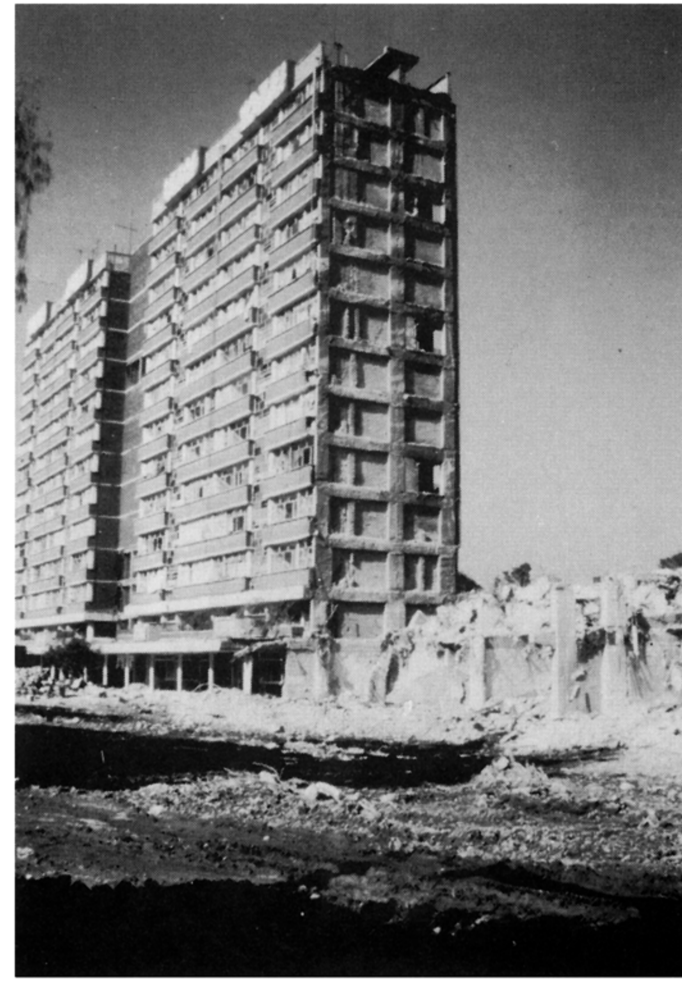
PLATE 3.31

Nuevo Leon - column failure in surviving section

PLATE 3.32

Pino Suarez buildings (Site 61)

PLATE 3.33

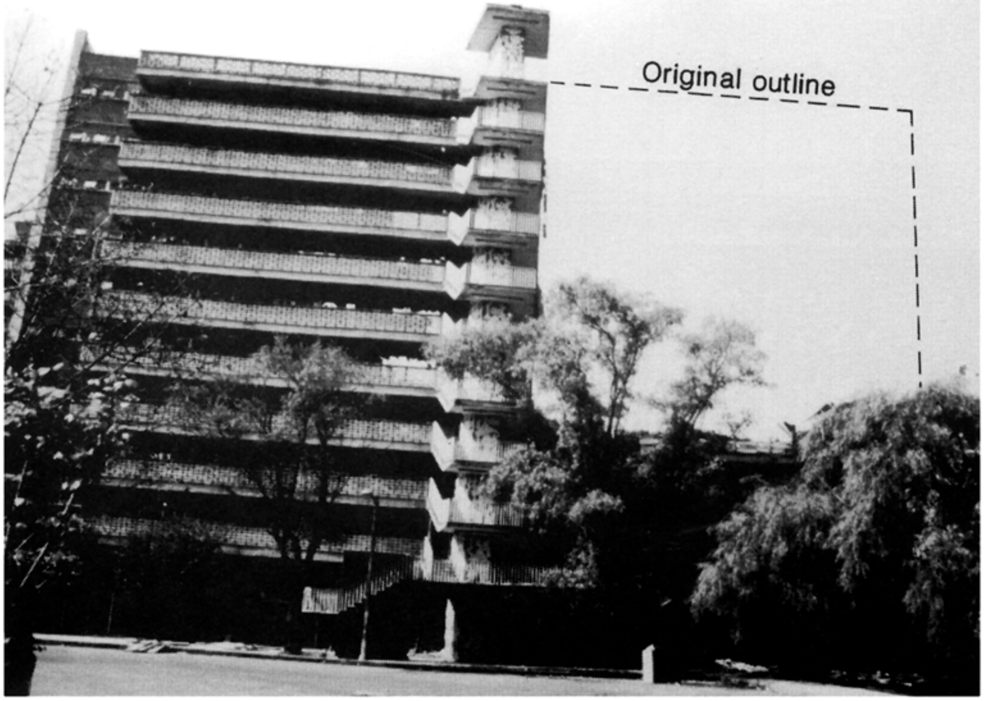


Benito Juarez estate - building type 1 (Site 81)

PLATE 3.34

Benito Juarez estate - building type 2 (Site 82)

PLATE 3.35



Benito Juarez estate - building type 3 (Site 83)

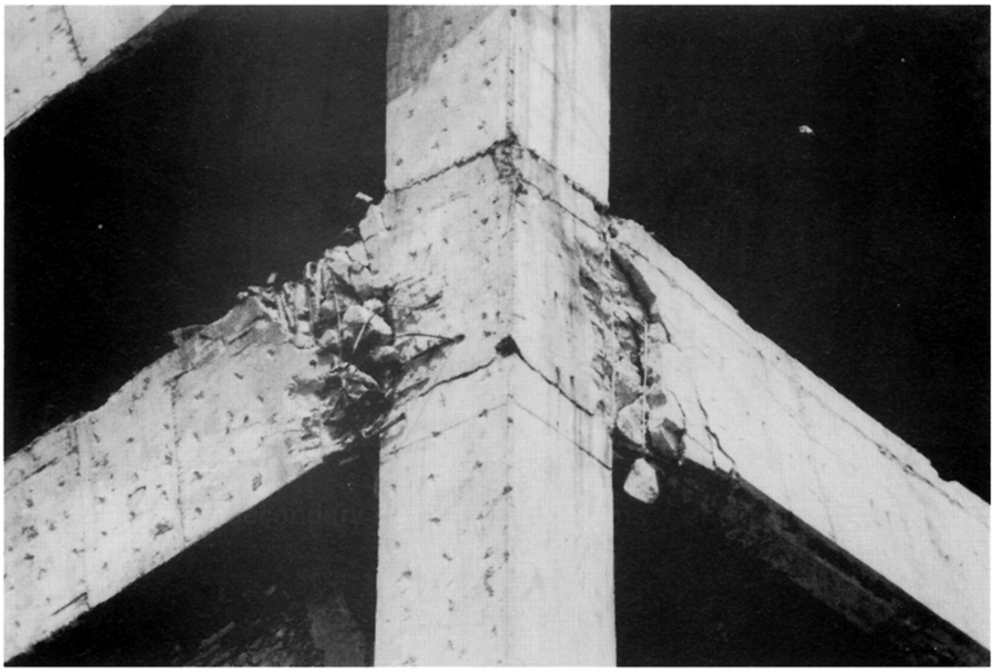
PLATE 3.36

Shear wall strengthened building - south and west facades (Site 77)

PLATE 3.37

Banco de Mexico - east and north facades (Site 7)

PLATE 3.38

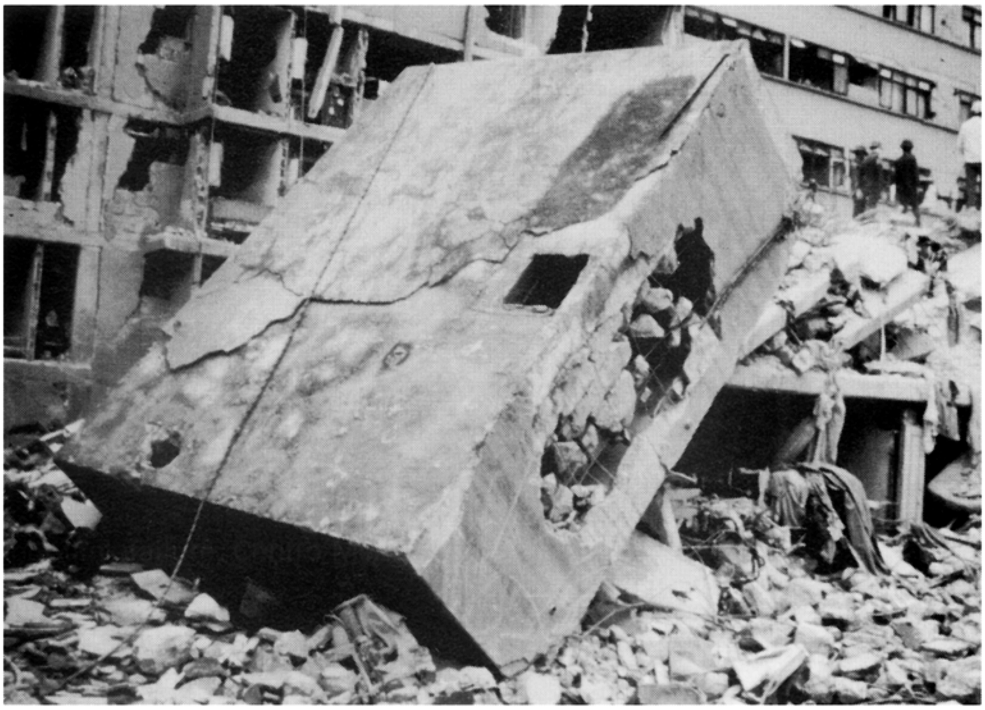


Banco de Mexico - column detail

PLATE 3.39

Failed infill with secondary columns and beams, Centro Medico (Site 85)

PLATE 3.40

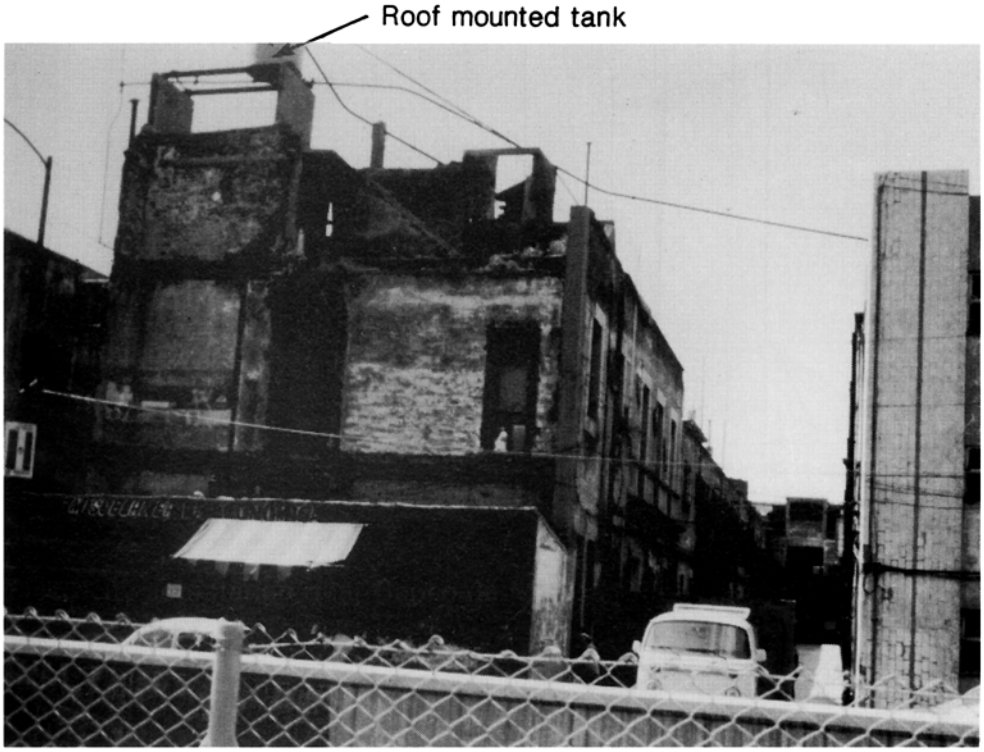


Fallen water tank, Piazza Garibaldi (Site 6)

PLATE 3.41

Roof tank failure, Centro Medico (Site 85)

PLATE 3.42



Non-failure of roof mounted tank (Site 75)

PLATE 3.43

Electricity sub-station near Banco de Mexico (Site 7, Plate 3.38)

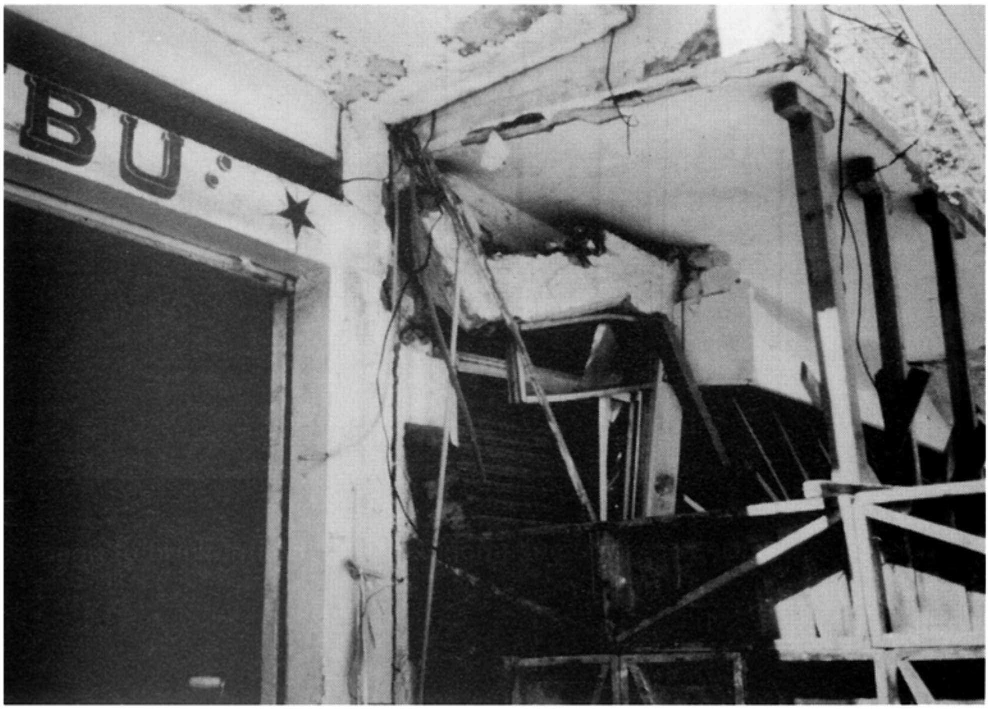
PLATE 3.44

Damage to structure on top of silos (Site 1)

PLATE 3.45

Ground floor collapse of domestic structure in Lazaro Cardenas

PLATE 4.1



Damage to beam indicating standard of construction of domestic structures in Lazaro Cardenas

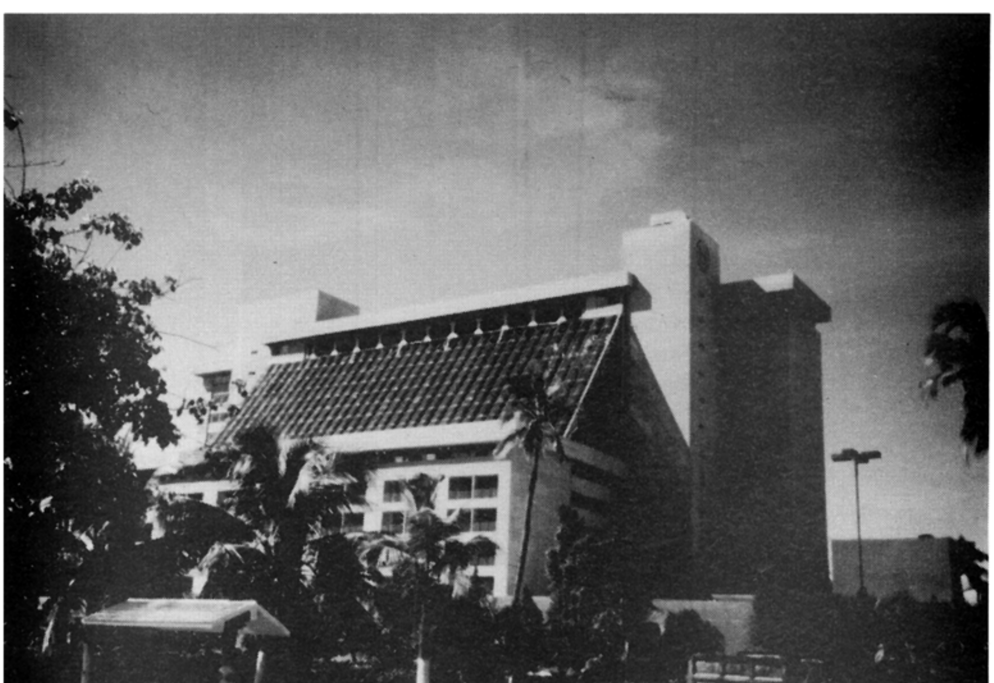
PLATE 4.2

Damage to brick infill in 2 storey domestic structure at Lazaro Cardenas

PLATE 4.3

Modern hotel at Ixtapa

PLATE 4.4



Modern hotel at Ixtapa

PLATE 4.5

Jose Maria Morelos Hydro-electric Dam

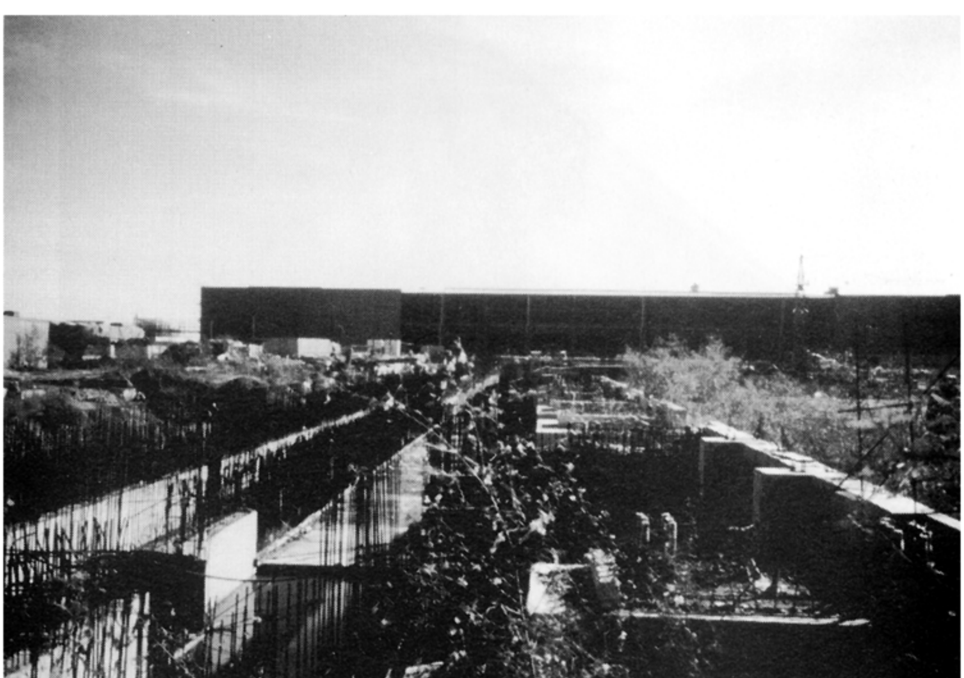
PLATE 4.6

Crest of Jose Maria Morelos Hydro-electric Dam showing longitudinal cracks

PLATE 4.7

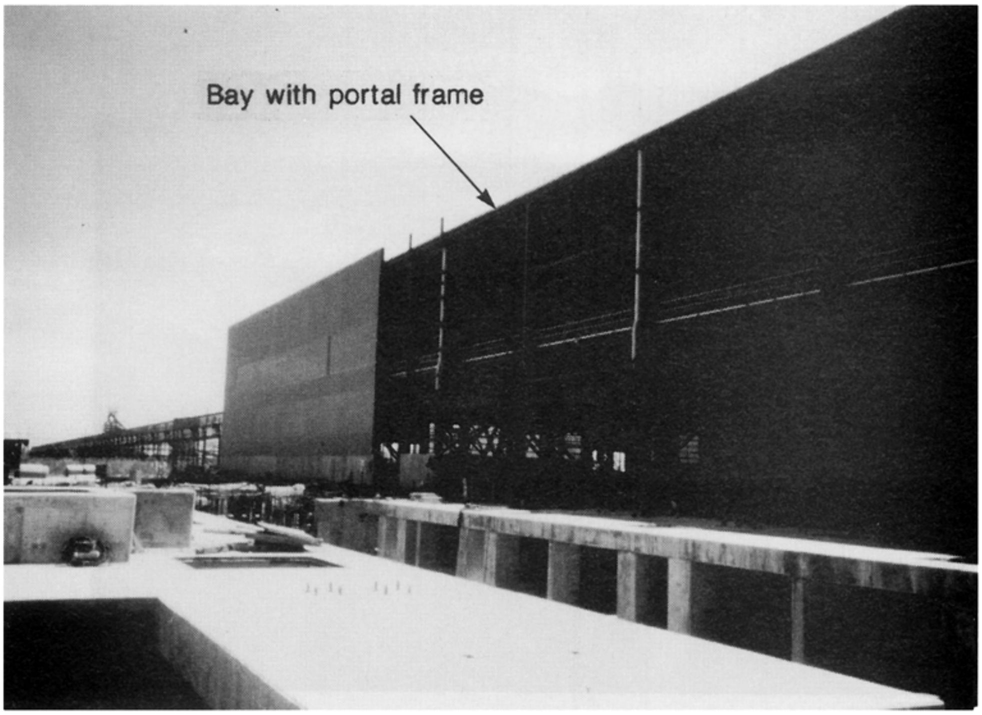
Concrete spillway at Jose Maria Morelos Hydro-electric Dam

PLATE 4.8



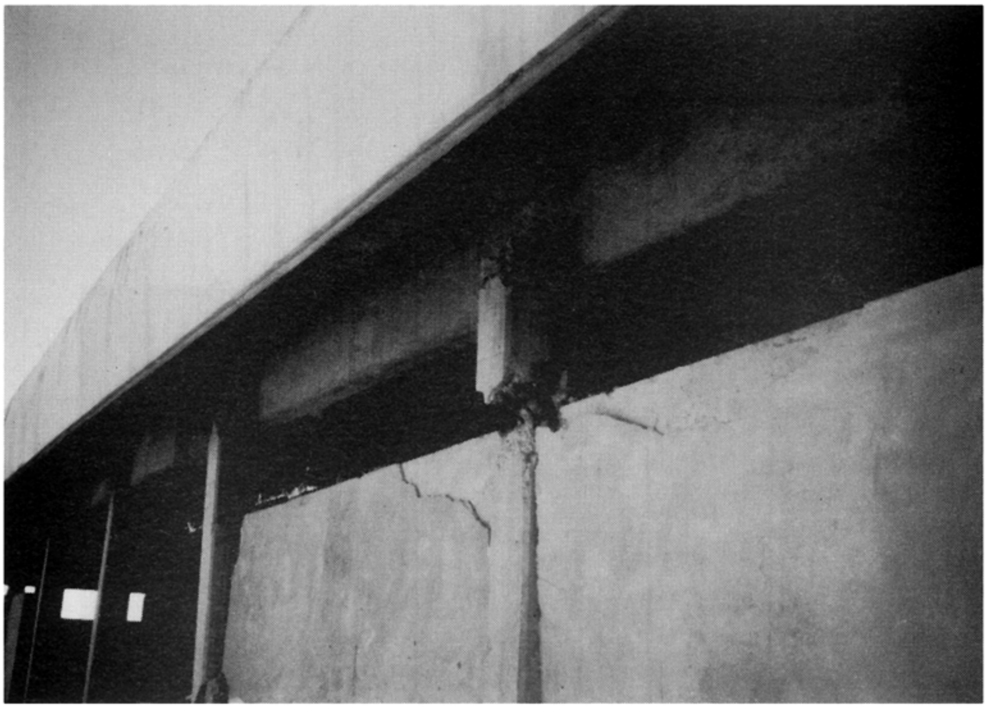
Diaphragm walls at Sicartsa

PLATE 4.9



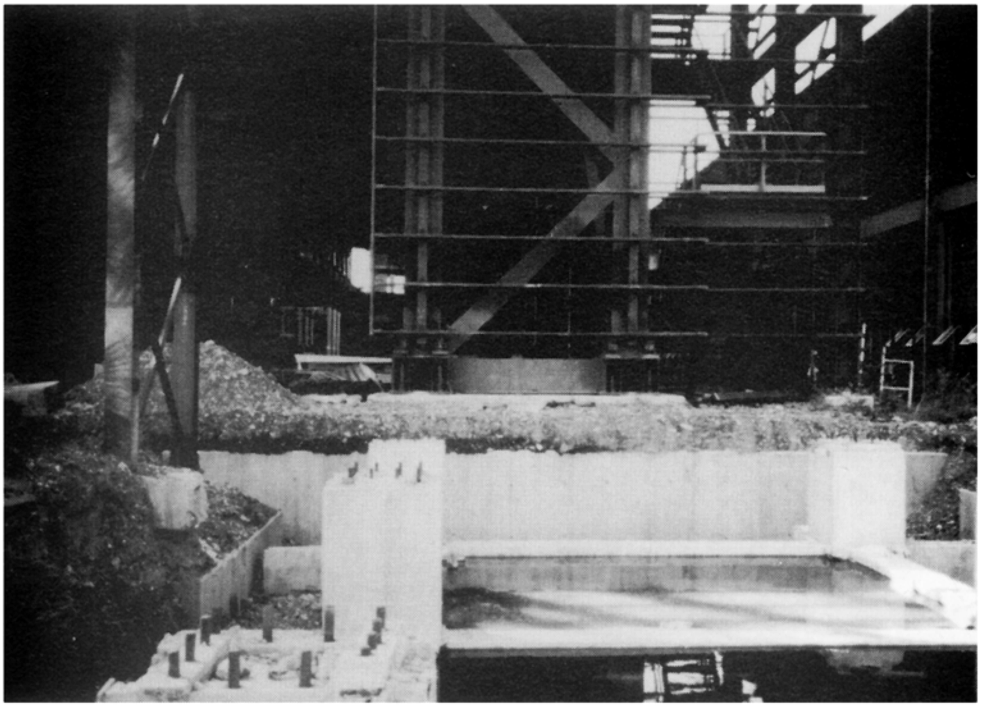
Steel rolling mill built by Davy McKee at Sicartsa

PLATE 4.10



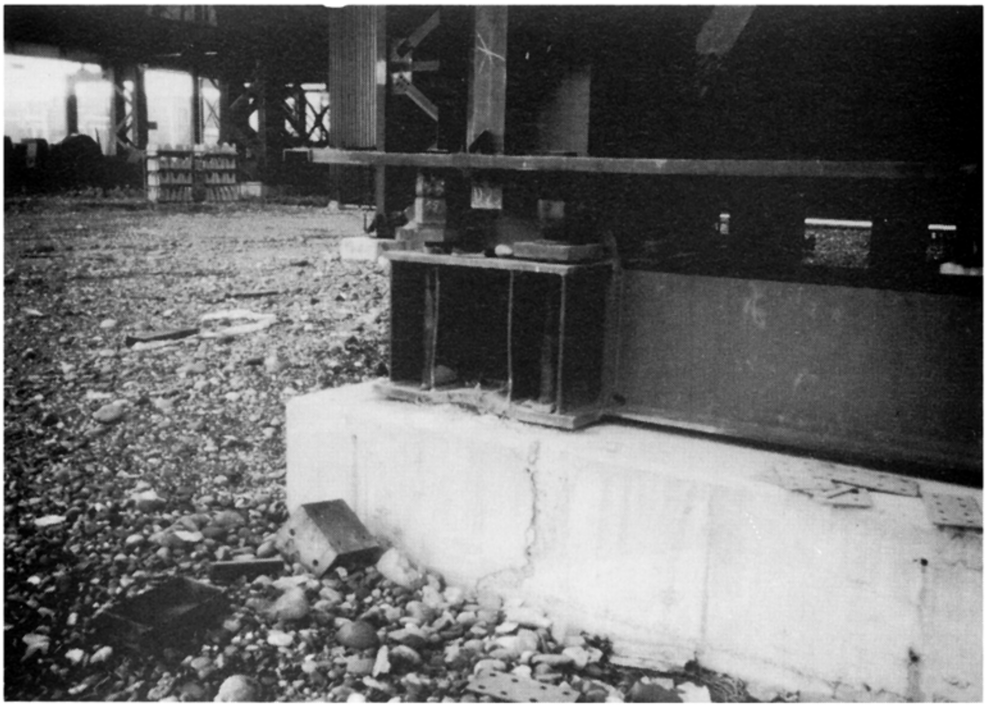
Reinforced concrete canteen at Sicartsa

PLATE 4.11



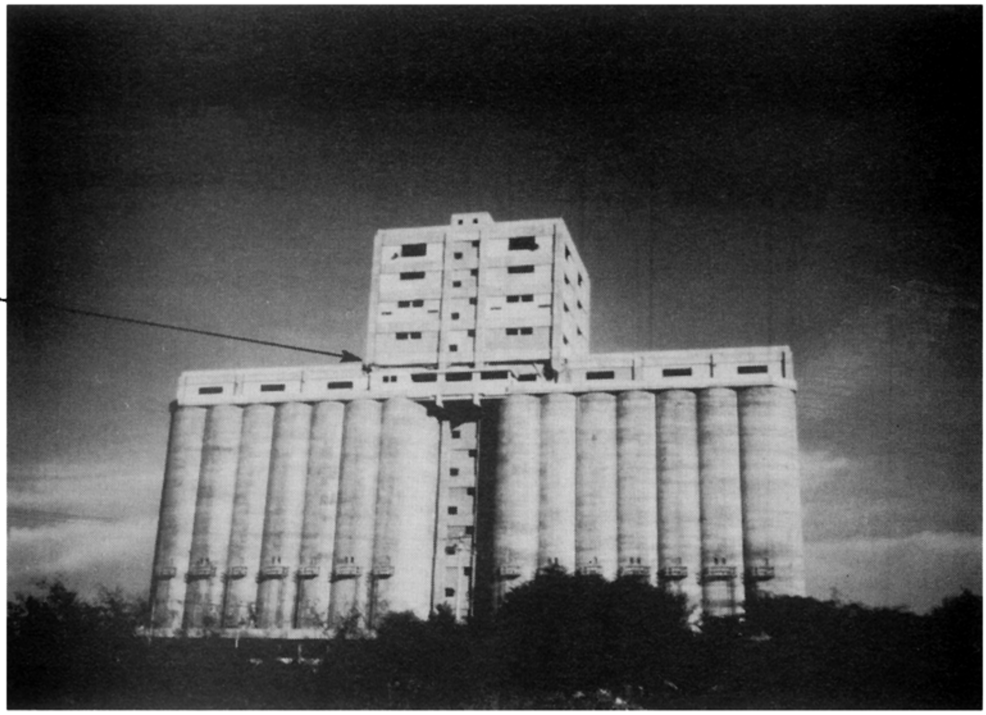
Steel rolling mill built by Japanese at Sicartsa

PLATE 4.12



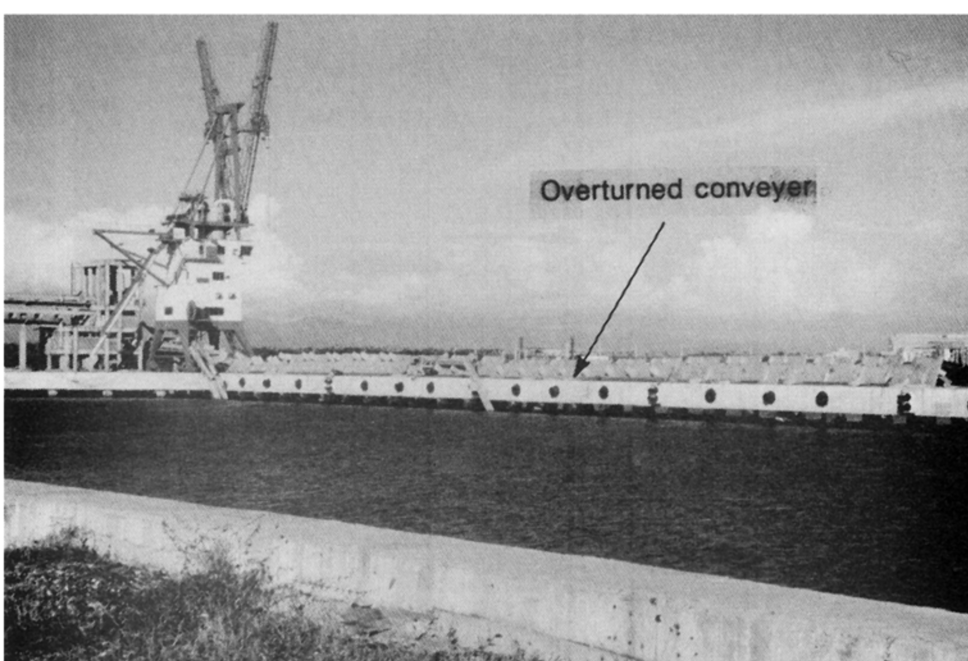
Damage to ungrouted base plates

PLATE 4.13



Damage to silos on Cayacal Island

PLATE 4.14



Berthing facilities on Cayacal Island

PLATE 4.15

Bridge to Cayacal Island

PLATE 4.16

Topography similar to that at Cuidad Guzman

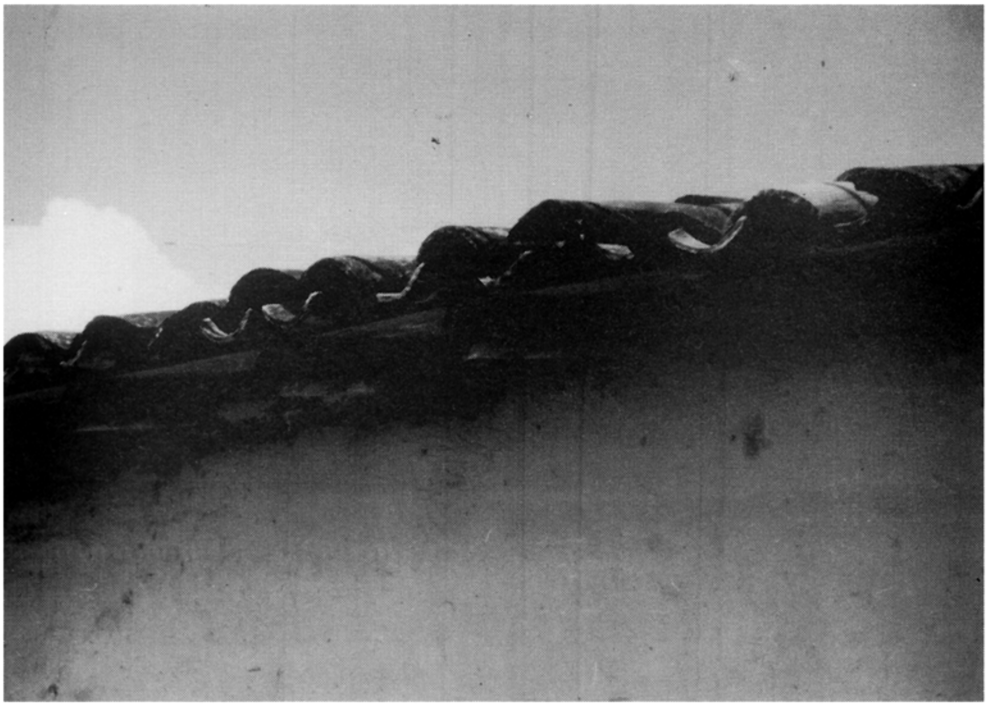
PLATE 4.17

General view of Cuidad Guzman (Site 5)

PLATE 4.18

Adobe front wall collapse (Site 24)

PLATE 4.19



Typical roof construction of adobe building (Site 25)

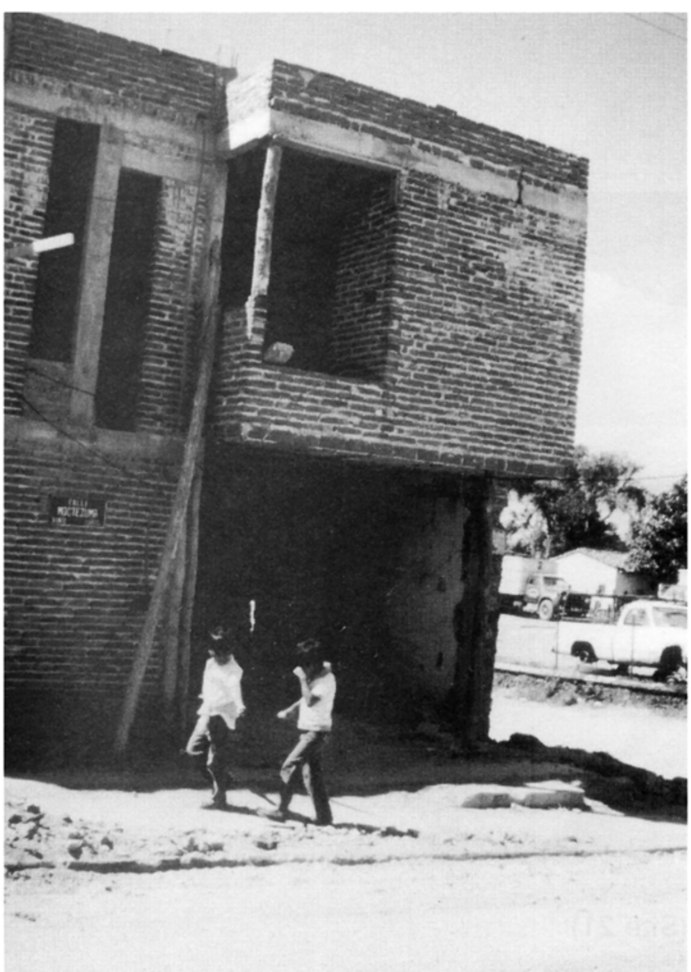
PLATE 4.20

Rotten roof timbers (Site 21)

PLATE 4.21

Comparison of adobe and brick building (Site 8)

PLATE 4.22



Brick building under construction (Site 3)

PLATE 4.23

Failure in 3 storey reinforced concrete building (Site 20) **PLATE 4.24**

J Clemente Orozco School (Site 17) **PLATE 4.25**

Ciudad Guzman Main Cathedral (Site 19)

PLATE 4.26

APPENDIX

MINUTES OF MEETING HELD AT SEDUE'S OFFICES 9 OCTOBER, 1985

PRESENT:

Lic. Gloria Maria Valdes Alcantara (SEDUE)
Arq. Roberto Barnard Amosurritia (SEDUE)

Mr Edmund Booth (EEFIT)
Dr Jack Pappin (EEFIT)

Lic. Beatriz de la Vega (SEDUE)

Purpose of Meeting:

To discuss possible areas of UK/Mexican technical cooperation.

1.0 National Commission for Reconstruction

Lic. Gloria Valdes explained that a National Commission for Reconstruction has been established by the President. The Commission comprises six Committees as follows:

a) Committee for the City of Mexico
b) Committee for Decentralization
c) Committee for Financial Matters
d) Committee for Social Security
e) Committee for Coordination of International Assistance
f) Committee for Civil Security

Lic. Valdes explained that all international assistance would be coordinated by the fifth of the Committees listed above.

2.0 Possible Areas of UK/Mexican Technical Coorperation

The areas of possible UK/Mexican technical cooperation in post-earthquake reconstruction activities were discussed. It was agreed that although a high level of expertise existed within Mexico in most relevant areas, the scale of the problems facing Mexico was such, that outside assistance would be of great value. It was also agreed that there were many possible fields in which Britain could provide assistance, covering many aspects, including engineering, planning, telecommunications and building control regulations. It was therefore important that some specific projects should be defined where the need of Mexico was greatest.

Six possibilities for such projects were identified as follows:

a) Soil mechanics projects, in collaboration with the Instituto de Ingenieria of UNAM.

b) Equipment and personnel for inspection of earthquake damaged buildings.

c) Earthquake resistant design of brick infill panels in buildings.

d) Assistance with drafting building codes for earthquake resistance.

e) Expert advice on town planning/decentralization.

f) Design studies for low cost, low rise dwellings, including prefabricated and self-build-housing.

These possible projects were discussed in greater detail as follows.

3.0 Soil Mechanics Projects

The Mexico City clay and the shape of the Mexico City basin, are both unusual features and the recent earthquake has highlighted the need for detailed studies to be made of their dynamic or seismic behaviour.

Both the projects described below have been discussed with Dr M. Romo of the Instituto de Ingenieria, UNAM, and details of them have been sent to Dr Daniel Resendiz, Secretary General of CONACyT.

3.1 Repeated Load Testing of Mexico City Clay

This project would involve a program of repeated load triaxial tests on samples of the soft clay under Mexico City. The Instituto de Ingenieria, UNAM, do not have facilities for this test at present and consider the results would complement the tests they will be carrying out shortly. The results will be useful for both the study of the seismic properties of the Mexico City Basin and the analysis and design of structures built on this material. It is envisaged that an engineer from Mexico will visit Britain to carry out the tests and that the program of work will take about three months.

3.2 Centrifuge Model Testing

This would be a longer term project involving physical model studies carried out on the centrifuge testing facility at Cambridge University. Studies could include modelling the seismic properties of the Mexico City Basin and also a structure, or a group of structures, with varying foundation types built on the soft Mexico City clay. The results of the tests would be used to help understand the characteristics of the behaviour and to calibrate the mathematical models currently being developed at UNAM. The work would ideally be associated with Mexican students doing post-graduate research at Cambridge.

4.0 Equipment and Personnel for Building Damage Inspection

The UK, with its large stock of buildings over 50 years old, has many years of accumulated experience in the repair and upgrading of old buildings. It also has extensive expertise in the strengthening of ancient momuments (cathedrals, historic buildings, etc). With this background, a possible project would consist of the following:

a) Identify the equipment for use in the inspection of earthquake damaged buildings.

b) Supply this equipment to Mexican authorities.

c) Supply UK personnel to Mexico for say three month periods to demonstrate use of the equipment and train Mexican personnel in its operation.

5.0 Aseismic Detailing of Brick Infill Panels

Cracking and collapse of unreinforced brick infill panels in concrete frame structures was very wide-spread during the Mexican earthquake of 19.10.85. Not only did this result in a danger from falling masonry, but it also may have led to weakening of the structure at the level of the damaged panels, with the risk of a concentration of structural damage at certain discrete levels and subsequent structural collapse. The present code forbids the use of rigidly connected brick infill panels without adequate strength, but in practice, it is difficult to achieve a separation of infill panels from the structural frame. There is therefore a need to develop simple, practical details for achieving a safe separation. It is an area where the UK has expertise and could assist in the following ways:

a) Development of practical details, including working drawings, by means of desk studies and calculations.

b) Testing proposed details on shake table facilities in the UK (e.g. Bristol University, Imperial College) or Mexico.

6.0 Building Codes for Earthquake Resistance

Since the seismicity of the UK is low, there are no British code requirements for earthquake resistance. However, UK industry is actively involved in the design of structures in seismically active areas all over the world, and so has considerable knowledge of international practice for aseismic building codes. The UK could therefore provide impartial advice and assistance to the Mexican authorities in their code drafting. Such advice could probably best be supplied through the British Standard Institute, an internationally regarded body, which is currently providing such advice in the drafting of the European Economic Community (EEC) code for earthquake resistant design, Eurocode 8.

7.0 Expert Advice on Town Planning and Decentralisation

UK experience in town planning and new town development extends back over 50 years. This could be made available to the Mexican authorities by the secondment to Mexico for periods of say 2 months of British Experts to act as expert consultants to the National Reconstruction Commission's committee on decentralisation.

8.0 Design Studies for Low Cost, Low Rise Buildings

The UK has considerable experience of prefabricated housing, including single family units. The Building Research Establishment, a UK government body, may be able to assist by providing both literature and standard designs, and also by seconding personnel to Mexico for short periods. The Martin Centre for Urban and Architectural Studies at Cambridge University, has extensive experience in low cost, low rise housing for seismically active areas, and could assist in similar ways.