THE SEPTEMBER 1985 MEXICO EARTHQUAKES: FINAL REPORT OF THE NEW ZEALAND RECONNAISSANCE TEAM

SUMMARY

This report on the September 1985 earthquakes in Mexico describes the earthquakes themselves, their effects and the resulting damage in considerably more detail than did the preliminary report of the reconnaissance team.

TABLE OF CONTENTS

1. INTRODUCTION	8.4 Lateral Loads
2. THE TEAM	8.5 Effects of September 1985 Earthquakes
3. THE RECONNAISSANCE	9. STRUCTURAL DAMAGE
4. SOILS OF MEXICO CITY	9.1 General
5. SEISMOLOGICAL FEATURES	9.2 Damage in Mexico City
6. STRONG MOTION ACCELEROGRAPH RECORDS	9.3 Damage to Buildings in the Epicentral Region Including: Cd Guzman, Chilpancingo, Acapulco, Ixtapa, Lazaro Cardenas and Playa Azul
6.1 The Guerrero Accelerograph Array	9.4 Case Studies 1 - 22
6.2 Strong Motion Records in Mexico City	10. NON STRUCTURAL DAMAGE TO BUILDINGS IN MEXICO CITY INCLUDING BUILDING SERVICES.
7. SEISMIC DESIGN PROVISIONS IN MEXICO	10.1 General
7.1 Code Status and Implementation	10.2 Ceilings
7.2 Comparison Between Design Spectra and Spectra Derived from Motion Recorded During 19 & 20 September 1985 Earthquake	10.3 Partitions
7.3 History of Development of Seismic Design Provision in the Federal District (Mexico City).	10.4 Stairs
7.4 Material Codes	10.5 Windows and Glazing
8. FOUNDATIONS	10.6 Exterior Cladding
8.1 Soil Conditions	10.7 Building Services
8.2 Common Foundation Types	11. DAMAGE TO CIVIL ENGINEERING STRUCTURES
8.3 Settlement	11.1 Mexico City
	11.2 Epicentral Region
	11.3 Case Studies 1 - 6

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12. DAMAGE TO MAIN FRAME COMPUTER INSTALLATIONS, STORAGE RACKS AND MISCELLANEOUS STRUCTURES

12.1 Main-frame Computer Installations

12.2 Storage Racks

12.3 Transmission Towers

12.4 Microwave Towers

13. PERFORMANCE OF LIFELINE SYSTEMS

13.1 Water Supply

13.2 Electricity Supply

13.3 Gas Supply

13.4 Road and Rail Transport

13.5 Telecommunications

13.6 Mexico City Airport

14. THE RESPONSE OF EMERGENCY SERVICES

14.1 Civil Defence and Rescue

14.2 Medical Services

14.3 Radio and Television Response

14.4 Fire Services

15. LESSONS FOR NEW ZEALAND

16. CONCLUSIONS

17. REFERENCES

1. INTRODUCTION

An earthquake of magnitude 8.1 occurred near the mouth of Rio Balsas on the Pacific Coast of Mexico at 7:18 am local time on 19 September, 1985. Its destructive effects were unusually widely distributed and there were many casualties. At Mexico City, 400km from the source, resonant amplification through layers of flexible lacustrine deposits that underlie parts of the urban area produced damaging intensities (MMIX), shaking that affected especially those structures that responded to low frequencies. This former lake bed zone has a history of vigorous response to excitation from distant earthquakes.

Within the earthquake's epicentral region (fig.1) buildings were damaged selectively according to their construction types and vibration characteristics. Microzone effects were also apparent. There was destruction and damage to Lazaro Cardenas (MMVIII-IX), a newly developed industrial town and port; at Ixtapa (MMVII), a luxury resort; at Playa Azul (MMIX); at Zihuatanejo (MMVII) and at Petatlan. Northwest from the epicentral region the towns of Coalcoman, Ciudad Guzman (MMVIII) and Gomez Farias were all severely shaken and extensively damaged. At Chilpancingo, to the east, a church and some state government buildings were damaged. Acapulco, though almost unscathed, suffered a disastrous loss of tourist revenue in the months after the earthquake.

Vigorous aftershocks aggravated the damage. Several buildings that had been critically affected by the main shock collapsed in the largest of its aftershocks, an earthquake of magnitude 7.5, which occurred at 7:37 pm on 20 September. Perceptible shocks continued for two more weeks.

Within a few hours of the event news services were carrying reports that Mexico City had been stricken. The epicentral region was correctly identified and the magnitude estimated at 8. The United States Embassy reported an estimate of the death toll at 20,000.

Apart from its tragic aspect, this Mexican earthquake held exceptional interest for New Zealand engineers and earthquake engineering researchers. Mexico City, the world's most populous metropolis, is growing at a phenomenal rate. In the 15 years preceeding the earthquake the population doubled to 17 million. New construction needed to accommodate new citizens and to provide services for them was done in nominal compliance with modern building codes. Mexican engineers, some of them acknowledged world leaders in earthquake engineering, had contributed to the design procedures used. Thus there was a rare opportunity to observe how buildings, which should have been well designed and constructed, had behaved when they were subjected to intense shaking. Also, the earthquake source was in a subduction zone like the subduction zones near New Zealand's Fiordland and East Cape and so of interest to us. It is a different tectonic regime from the regime of transcurrent faulting, such as in California and central New Zealand, from which records used to establish our design acceleration response spectra have been obtained.

These were compelling reasons for a New Zealand reconnaissance.

FIGURE 1. DAMAGE ZONES OF THE 19 SEPTEMBER 1985 MEXICAN EARTHQUAKE.

DSIR and MWD proposed sending observers. The NZNSEE, who assumed administrative responsibility, added engineers from consulting firms. The formation of the team was announced by the Ministers of Works and of Science, and the Ministry of Foreign Affairs obtained Mexican Government permission for the visit. The New Zealand Embassy in Mexico City supported the team for the duration of its stay in Mexico. The team's official standing and the support received from government and its agencies contributed very significantly to the success of the mission.

In the early aftermath of the disaster all available resources were allocated to essential work - rescuing victims, treating injured, disposing safely and decently of the bodies of the dead, sheltering the homeless and restoring vital services. The New Zealand Embassy in Mexico City advised the team to defer its arrival until the emergency period was over. This was agreed. The team reached Mexico on 5 October, 16 days after the earthquake, timing that was almost ideal. We were early enough to see everything from which we could learn, yet not so early that we embarrassed our Mexican hosts or impeded vital operations. We were more warmly received than earlier teams had been and were able to get better information than was available to them. We did not see buildings which had been reduced to rubble and cleared away during the rescue operations; but there was nothing to be learned from them that was not already known. We did see partially collapsed buildings that had sufficient toughness to resist total reduction, and we had the generous cooperation of our Mexican hosts to make our opportunity to study these as valuable as it could be.

We left Mexico on Saturday, 26 October, having visited all the damage areas mentioned here, except the remote mountain town of Coahuila, in three weeks.

2. THE TEAM

In the original party were three engineers, Messrs George Butcher of Morrison Cooper and Partners (team leader), Garry McKay of MWD and Robert Jury of Beca Carter Hollings and Ferner, and one engineering seismologist, Dr Graeme McVerry of the DSIR. These were joined (for two weeks) by Dr Tony Gillies, a New Zealander at Lakehead University, Ontario, Canada, and (for 10 days) by Dr David Hopkins, engineer, and Mr Warwick Massey, architect, both from KRTA.

Ms Caroline Forsyth, First Secretary of the New Zealand Embassy in Mexico City, accompanied the party, smoothing the way, arranging meetings and contacts and overcoming many of the linguistic problems that would otherwise have beset us. Much of our success is attributable to her.

Except that it lacked a soils and foundation engineer, who could have made a valuable contribution, the team was a well balanced one of cooperative individuals who worked together harmoniously and efficiently.

FIGURE 2. THE ZONES OF MAJOR DAMAGE IN MEXICO CITY, A SMALL PROPORTION OF THE CITY.

FIGURE 3. THE ZONE OF SEVERE DAMAGE IN MEXICO CITY SUPERIMPOSED ON A MAP OF WELLINGTON.

3. THE RECONNAISSANCE

The first day in Mexico City was for orientation, for identifying the localised areas where there was severe damage and for a superficial appraisal.

The urban area is 1050 sq. km. Important damage was confined to the former lake bed zone, area 23 sq. km. (fig 2) and so larger than downtown Wellington (fig 3), in which some 3% of the structures were severely affected according to a UNAM survey [Reference 1]. Among these were collapsed schools and office buildings, fortunately empty of occupants at the time of the earthquake, so that the death toll, variously estimated in the range 8000-16000, was mercifully smaller than it might have been. Casualty buildings included many that were large and newly constructed, government buildings among them. But in most parts of this city of 1.5 million buildings there was little or no obvious damage. Four weeks after the earthquake, preliminary safety inspections had been made on about 3500 buildings, all of which had been damaged more or less critically.

On the second day the team visited the University and established contacts at the Instituto de Ingenieria there, most notably with Drs Esteva and Prince, whose help was to be invaluable.

The rest of the first week was spent in the epicentral area, from Acapulco in the south to Caleta de Campos at the northern limit of the rupture zone. This town was also the most northerly instrument site along the coast of a recently installed array of strong motion accelerographs.

Damage reports from this region that were filed in Mexico City were sketchy and unreliable, so the team could not preplan its work. Damage was studied as it was encountered. There was, surprisingly, no widespread destruction of the kind one would expect to find in the epicentral region of such a very large earthquake. Inferior unreinforced brick and adobe construction survived with nothing worse than trivial damage in village after village, even though accelerographs recorded energetic high - frequency motion to which the buildings might have been expected to respond vigorously. Larger

would expect to find in the epicentral region of such a very large earthquake. Inferior unreinforced brick and adobe construction survived with nothing worse than trivial damage in village after village, even though accelerographs recorded energetic high - frequency motion to which the buildings might have been expected to respond vigorously. Larger structures were often more seriously affected, having responded to the strong long - period components of the earthquake. Among them were modern hotels at Ixtapa and bridges, grain silos, port facilities and ancillary structures at the steel mill at Lazaro Cardenas.

The team visited accelerograph sites in the epicentral region to observe the topography and geology associated with the records that were made.

Three of the team travelled to Ciudad Guzman and its neighbouring village of Gomez Farias, north of the epicentral region, where many one and two storey houses had collapsed. Extensive damage of this type was confined to these localities. It did not occur in the neighbouring villages.

Back in Mexico City, detailed studies were made of building damage, and there were meetings with engineers, architects and representatives of various authorities and professional groups, from all of whom the team obtained important information. These authorities and groups included the Departamento del Distrito Federal (DDF) which administers the city, the Federal Electricity Commission, the regional power distribution authority, Telefonos de Mexico, the airport authority, the Metro (underground rail system), the Mexican Concrete Institute, US Embassy, radio stations and university researchers.

4. SOILS OF MEXICO CITY

The concentration of earthquake damage in the central city area of Mexico City was closely associated with the unusual soil conditions there. The soils of the central city area have long been infamous for their amplification of the motions from large distant earthquakes and for the very large settlements that occur on them. The damage zone in 1985 coincided with but was larger than the damage zone in the 28 July 1957 and 14 March 1979 earthquakes (fig. 4).

The principal reason for this concentration of damage was amplification, by a factor of about five, of the horizontal components of the incoming rock motion through soil-layer resonance in the soft soils of the central city (see section 6.2). To understand this phenomenon it is necessary to appreciate the geological history of Mexico City.

Mexico City lies in the Valley of Mexico, a naturally closed basin on the Mexican Plateau, surrounded by volcanic mountains. During the Pleistocene era, the basin filled with material eroded from the hills, forming deposits of gravels, sand and sandy clays several hundred metres thick. Natural drainage into the basin created a series of

FIGURE 4. DAMAGE ZONES IN MEXICO CITY IN THE 1957, 1979 AND 1985 EARTHQUAKES, OCCURRING REPEATEDLY IN THE SAME PART OF THE CITY (from [1]). ACCELEROGRAPH SITES ARE ALSO SHOWN.

lakes which later merged into a single large lake occupying virtually the whole basin. Soft fine sediments settled out of the lake. These included volcanic ash which subsequently decomposed into clay. As sedimentation continued to fill the basin, the lake dwindled in size. Its remnant has been further reduced by modern drainage. Today most of the area is a lake-formed plain in which are a few small lakes [Reference 2].

Mexico City has three soil zones: the lake-bed zone, the hill zone and the transition zone (fig. 5). Subsidence and earthquake vibration problems are encountered mainly in the lake-bed zone.

The hill zone consists of alluvial deposits with no compressible clay layers. It is bounded by lava flows towards the south. Buildings in the hill zone generally suffer only minor damage from earthquakes.

Records from rock sites in the hill zone provided a measure of the incoming earthquake excitation. This was greatly amplified in parts of the lake-bed zone.

Between the hill zone and the lake zone is the transition zone. The soil composition is variable, and there can be problems with

FIGURE 5. THE THREE SOIL ZONES OF MEXICO CITY WITH THE ZONE OF SEVERE DAMAGE AND THE DIGITAL ACCELEROGRAPH SITES SUPERIMPOSED (LOMAS = HILL, LAGO = LAKE). THE LOCATIONS OF THE SOIL PROFILES A AND B OF FIG. 6 ARE SHOWN (modified from [1]).

bearing capacities and differential settlements, but the zone is not generally susceptible to major earthquake damage from large, distant events. The boundary with the lake-bed zone is marked by the appearance of a regular series of recent lake deposits.

The lake-bed zone is characterised by thick deposits of highly plastic soft clays interspread with layers of firmer silts, sands and gravels. The harder layers in soil profiles have substantially lower natural water contents than the compressible clays. An example of the variation of the soil properties in the different layers is given in Table 1, which shows data from a bore-hole in Alameda Park in the lake-bed zone.

A typical soil profile for the central city area is as follows [References 2,3,4]:.

A surface layer of clayey or silty sands, or, in some places of man-deposited fill, often containing archaeological artifacts, reaches to depths of from 5 to 10 metres. This is underlain by the upper layer of extremely compressible clay, 15 to 32 metre thick, containing thin layers or lenses of sands. The clay layer has very high natural water contents, between 200 and 500 per cent. Beneath it is the first hard capping

layer, approximately 3 metres of sandy clayey stratum with a water content below 100 per cent. Next is a second volcanic clay layer, similar to the upper layer, of high water content but less compressible, between 4 and 14 metres thick. Then there is a second hard layer in which sand predominates. In some areas a third compressible clay layer has been found starting about 60 metres below ground level. Beneath that, starting at a depth of 80 metres, are compact strata of sandy silts and clays with some gravel and sand, extending to a depth of more than 100 metres, and characterised by a much lower water content than the clays have.

The thickness of the different layers varies considerably within the lake bed zone. The overall sediment thickness increases with distance from the transition zone boundary. Some of the variations in strata thickness are attributed to differential consolidation of the clay layers that was induced by drainage and by the imposition of building loads. Soil profiles across north-south and east-west sections in the central city area are shown in fig. 6. The locations of these sections are shown in fig. 5.

The lake-bed zone responds vigorously to low frequency earthquake excitation because the depth of flexible soils is great. Soil layers have characteristic periods of vibration determined by their thickness, density and stiffness. Thick deposits of flexible soils have long natural periods, causing high amplifications of those components of incident earthquake waves that have similar periods. In consequence Mexico City is affected by large magnitude distant earthquakes because these events generate strong long period wave trains which travel long distances with little attenuation. Much of their energy is in the period band that excites resonant response in the lake-bed soil layers.

Fig. 7 shows the distribution of collapsed and badly damaged structures in Mexico City in the 1985 earthquake. An overlay of the severely damaged zone upon the soil zone map (fig. 5) shows that the severe damage was confined to a portion of the lake-bed zone. Comparisons of the position of the severe damage zone and contours of the depths to the first and second hard layers show very definite correlations. Almost all the severe damage was in the area where the depth to the first hard layer is between 22 and 36 metres and the depth to the second hard layer is between 30 and 50 metres (fig.s 8 and 9).

The correlation between depths to the hard layer and the damage zone is a consequence of the natural period of vibration of the soil layers in this area. Both calculations based on the soil layer properties and observations of the spectral peaks of recorded ground motions have shown that the natural periods in this zone are between about 2.0 and 2.6 seconds. Some calculated and recorded periods are listed in Table 2.

The soil properties and natural periods of vibration given in Table 2 are for a site at Alameda park in the centre of the city.

FIGURE 7. SERIOUSLY DAMAGED (X) AND COLLAPSED (•) BUILDINGS IN MEXICO CITY, ENCLOSED BY THE ZONE OF SEVERE DAMAGE. THE DIGITAL ACCELEROGRAPH SITES ARE MARKED (modified from [1]).

FIGURE 8. CONTOURS OF THE DEPTH (IN METRES) TO THE FIRST HARD LAYER, WITH THE SEVERE DAMAGE ZONE SUPERIMPOSED (modified from [1]).

FIGURE 9. CONTOURS OF THE DEPTH (IN METRES) TO THE DEEP DEPOSITS, WITH THE SEVERE DAMAGE ZONE SUPERIMPOSED (modified from [1]).

Curvas de igual profundidad de depósitos profundos

TABLE 2. SOIL PROPERTIES AND NATURAL PERIODS FOR ALAMEDA PARK IN CENTRAL MEXICO CITY (based on Zeevaert [2]).

Classification	Depth (m)	Thickness di (m)	Wn (%)	ρ ton sec ² /m ⁴	μ ton/m ²	$v_s = \sqrt{\frac{\mu}{\rho}}$ m/sec	$\Delta T = \frac{4di}{v_s}$ sec				
Fill	5.55	5.55	55	0.169	1000	76.9	0.289				
Becerra	9.15	3.60	85	0.153	2000	114.4	0.126				
Silty clay I	15.80	6.65	300	0.119	280	48.5	0.549				
Clayey silty sand	16.50	0.70	42	0.180	700	62.3	0.045				
Silty clay II	19.80	3.30	320	0.118	360	55.2	0.239				
Silty clay III	23.65	3.85	200	0.126	470	61.0	0.253	0.719	1.765	2.054	
Silty clay IV	27.20	3.55	273	0.120	470	62.5	0.227				2.561
Silty clay Va	29.10	1.90	248	0.122	720	76.7	0.099				
Silty clay Vb	33.50	4.40	300	0.119	720	77.7	0.227				
Tarango sand I	38.20	4.70	42	0.180	4000	148.9	0.126				
Tarango silty clay Ia	41.55	3.35	174	0.129	980	87.0	0.154				
Volcanic glass	41.95	0.40	30	0.178	2000	105.9	0.015	0.382			
Tarango silty clay Ib	45.25	3.30	174	0.127	1120	93.8	0.141				
Tarango clayey sand Ic	47.70	2.45	168	0.130	2500	138.4	0.071				
Tarango sand II	>47.70										

W_n = water content; ρ = density; μ = shear modulus;

v_s = shear wave velocity; ΔT = contribution of layer to natural period.

PERIODS OF IMPORTANT LAYER COMBINATIONS

Firm base at 47.7m depth to surface : 2.56 seconds
 First hard layer at 33.5m depth to surface : 2.05 seconds
 Hard layer at 33.5m to base of fill at 5.55m : 1.76 seconds
 Softest layers between 16.5 and 27.2m depth : 0.72 seconds
 Lower clay formation between 38.2 and 47.7m depth : 0.38 seconds

DOMINANT PERIODS FROM EARTHQUAKE RECORDS AT ALAMEDA PARK (from undamped velocity spectra)

Peak Number	11 May 1962		19 May 1962	
	N11W	N79E	N11W	N79E
1	2.40	2.35-2.50	2.60	2.40
2	2.65	2.0	2.20	2.00
3	2.00	1.6-1.75	3.50	1.75
4	1.60			
5	0.75			

The 2.1 second spectral peak of the east-west record from the SCT centre agrees very closely with the 2.05 second period calculated for Alameda Park soil between the surface and the top of the first hard capping layer at a depth of 33.5 metres. The depth to the capping layer at the SCT is between 32 and 34 metres according to the contours of fig. 8, so the soil layer response should be very similar to that at Alameda Park. This expectation is confirmed by the agreement between the observed and calculated periods. The subsidiary peak at 2.6 seconds in the SCT spectrum corresponds to the calculated natural period of 2.56 seconds at Alameda Park for the soil layers from the surface to the top of the second hard layer. The dominant peak in previous earthquakes has been at this period, rather than at the shorter 2.1 second period peak of the 1985 record. The agreement between the dominant periods of motion in the 1985 earthquake and the calculated soil layer periods is remarkable in that the calculations were published before the earthquake occurred.

Further into the lake-bed zone, periods become longer, up to 5 seconds in one record from the Sports Palace [5], obtained in a previous earthquake. However, these extremely long periods are greater than the periods of any structures in the vicinity and so were not associated with major structural damage.

Soil column resonance in Mexico City amplified only the horizontal components of rock motion. Vertical components measured on soil were similar to those measured on rock.

The importance of resonance in generating damaging motion is illustrated by the proportions of buildings of various heights which suffered severe damage or collapse. Most damage occurred in structures of between 6 and 15 storeys. The proportion of buildings of less than 5 storeys that collapsed was small, and those low rise buildings that were damaged or that collapsed often did so because they were struck by taller neighbouring buildings. Few buildings of higher than 15 storeys suffered severely.

The following table is an estimate of the percentage of buildings in various height ranges in the most severely damaged zone (inner 23 sq. km. zone of fig.s 2 and 4) which suffered severe damage or collapse.

TABLE 3 : PERCENTAGE OF BUILDINGS OF VARIOUS HEIGHTS SUFFERING SEVERE DAMAGE OR COLLAPSE IN THE MAJOR DAMAGE ZONE [1].

No of Floors	% Damaged
up to 2	2
3 - 5	3
6 - 8	16
9 - 12	23
> 12	22
all heights	3

Calculations of soil layer periods were made by one-dimensional modelling which accounted for thickness, density and rigidity of various soil layers at a particular location without accounting for change of these properties in any horizontal direction. However, it is obvious from the soil depth contours of fig.s 8 and 9 that the depths change with distance from the lake-bed boundary in the damage zone, so it is likely that the properties of the local soil column do not, by themselves, determine response. The contours suggest focussing effects may be involved, and the concentration of the damage on the edge of the lake-bed zone suggests mechanisms similar to shoaling of water waves on a beach, or amplification of elastic waves in the tip of a wedge. The strong directionality of the SCT accelerogram (see section 6.2) obtained in this earthquake also suggests an amplification mechanism more complex than represented by a one-dimensional soil layer modelling. Complex amplification apparently occurred in a non-uniform depth "bowl of jelly" of irregular shape.

The flexibility of the soil which gave rise to resonant amplification of the incident earthquake motion was the primary cause of structural damage in the lake-bed zone, but other properties of the soil there also contributed considerably to the extent of damage. Subsidence of the soil layers, low strength soil and other consequences of the soil flexibility were important.

Buildings in the lake-bed zone of Mexico City often suffer from severe settlement problems, being founded on highly compressible lacustrine deposits from which water has been extracted in recent times. Subsidence in parts of central Mexico City has been more than 8 metres this century [4] and tilting due to differential settlement is often spectacular. The settlement of structures may be controlled by founding them on piles driven to one of the hard layers. However, when this is done for any building, it may appear to rise out of the ground as the soil column shrinks due to dehydration consolidation. Then large friction forces may be imposed on piles by drag from the sinking soil, increasing the end bearing pressure. Water extraction from the lower soil layers is largely responsible for the consolidation, but the near-surface water table is not affected because of the imperviousness of the underlying clay deposits and because surface layers are continuously recharged with water, so excavations for foundations are almost always in saturated material.

Tilting due to differential settlement of a structure generates stresses which reduce the strength available to resist earthquake excitation. Tilting may also reduce the spacing between neighbouring buildings, thus increasing the likelihood of impact during an earthquake. Whenever neighbours are of different heights, the lower building tilted against the taller may stiffen its bottom portion, thus causing damage to the composite immediately above the level of the top of the lower building. There were many examples these kinds of damage, attributable to pre-earthquake

tilting.

Dynamic shear stress induced in the soil during earthquake shaking may combine with the static shear stress to exhaust the ultimate shear strength in the soil. In sensitive (or "quick") clays, intense shaking might alter the structure of the clay reducing its shear strength and so cause immediate structural failure, or it might increase the rate of settlement and so cause problems in the years following the earthquake. Foundation difficulties are more prevalent in Mexico City than they are in most other large cities.

A mechanism which in some cases improves and in others worsens the performance of a structure in an earthquake is soil-structure interaction. Firstly, foundation flexibility increases the natural period of vibration of a soil-structure system. In most seismic regions this would be an advantage because it would reduce the response accelerations; but not in Mexico City, where the response accelerations are more likely to be increased. Secondly, radiation damping effects are often high for stiff buildings on soft soils, thus helping to protect such buildings; but material damping in the Mexico City clays is surprisingly low, reducing this benefit. Tests indicate that the clays probably remained essentially elastic throughout the earthquake, although the strains were large.

No one in the team was equipped to study these questions in depth.

Reconnaissance teams sent to future earthquakes, especially to earthquakes affecting places like Mexico City, where soft soils modify building behaviour significantly, should include at least one specialist geotechnical engineer.

5. SEISMOLOGICAL FEATURES

The earthquakes of September 1985 occurred along the Pacific Coast of Mexico where the Cocos plate is subducting under the North American plate at a convergence rate of about 75 mm/year. The subduction zone (fig. 10) stretches over 1350 km. The recurrence period for rupture by large (surface wave magnitude $M_s > 7.0$) earthquakes at any point is about 30 to 70 years along much of this length [6].

Subduction zone earthquakes occur in New Zealand, in the Fiordland region, where the Australian plate is diving under the Pacific plate, and in a region stretching from the Hikurangi trench, off East Cape, to near Wellington, where the Pacific plate is subducting beneath the Australian plate. The mechanism of earthquakes generated where one plate is diving underneath another is different from the mechanism of transcurrent faulting systems such as New Zealand's Alpine fault or California's San Andreas system, where two plates are sliding past each other. Most strong-motion accelerometer records used in New Zealand seismic-resistant design are from Californian events, so it was important to obtain records and observe the pattern of

FIGURE 10. THE TECTONIC SETTING OF MEXICO. THE SEPTEMBER 1985 EARTHQUAKES ASSUMED ALONG THE SUBDUCTION ZONE OF THE MIDDLE AMERICAN TRENCH BETWEEN THE COCOS AND NORTH AMERICAN PLATES (from [30]).

FIGURE 11. THE AREA OF THE RUPTURE ZONE OF THE MEXICAN EARTHQUAKE SUPERIMPOSED ON NEW ZEALAND SUBDUCTION ZONES. MEXICO CITY WAS 400 KM FROM THE SOURCE ZONE.

FIGURE 12. ESTIMATED RUPTURE ZONES OF SOME MAJOR MEXICAN EARTHQUAKES SINCE 1950 (modified from [10]).

damage and shaking from the subduction zone Mexican earthquake. To illustrate the large distance of 400 km at which damaging motion was caused in Mexico City by the 19 September 1985 earthquake, the area of the rupture zone of the Mexican earthquake is shown superimposed on various parts of the New Zealand subduction zones at distances of 400 km from major New Zealand cities in fig. 11.

There have been 33 earthquakes this century exceeding magnitude 7.0 between Jalisco and Oaxaca, the part of the Cocos plate subduction zone giving rise to earthquakes which can damage Mexico City. New Zealand has experienced only 13 earthquakes this century exceeding magnitude 7.0, so has been less seismically active than the Mexican subduction zone.

The 1985 earthquake has been exceeded in magnitude by only one Mexico earthquake in this century, the 1932 Jalisco earthquake, magnitude 8.2, which did not cause intense shaking in Mexico City. The intensity of shaking which Mexico City experienced in the 1985 earthquake has been matched there only once this century, in 1911. Then there were no buildings with vibration frequencies in the susceptible range in the

city, so relatively little damage was done. Comparison of damage to water mains in the two events led to the inference that intensities matched [1].

The 28 July 1957 earthquake (fig. 12) which caused widespread damage is well known to earthquake engineers. This magnitude 7.5 event was centred between Acapulco and Chilpancingo, about 250 km from Mexico City, compared with 400 km for the magnitude 8.1 September 1985 event. The seismic design codes of Mexico City account for the strength of shaking experienced in the 1957 earthquake.

The epicentral zone has suffered damage in recent years from the magnitude 7.6 Petatlan earthquake of 14 March 1979, which was reported as also damaging 60 buildings in Mexico City, and the 25 October 1981 Playa Azul earthquake (fig. 12).

Preliminary estimates of the location of the epicentre of the 19 September 1985 main shock were not in agreement. One estimate placed the epicenter off the mouth of the Rio Balsas at 17.68 degrees North and 102.47 degrees West. Others located it onshore near Caleta de Campos. The fault plane is located at depth, probably in the

15 to 30km range. The focal depth was certainly less than 33 km. There have been no reports of surface faulting. At the time of the visit seismologists estimated that the earthquake ruptured a very large fault area for a Mexican subduction zone event, possibly up to 200 km long and 80 km wide, with a maximum fault displacement exceeding one metre.

Later estimates [21] give rupture areas for the main shock and principal aftershock of $170 \times 50 \text{ km}^2$ and $66 \times 33 \text{ km}^2$ respectively, with average slips of 220 and 330cm.

An unusual feature of this earthquake was the generation of a tsunami ("tidal wave"), very uncommon for Mexican earthquakes. We received reports of the tsunami from Ixtapa, where an initial withdrawal of

water was followed by a surge which reached the entrance of beach front restaurants and, in a couple of cases, flooded into hotel swimming pools, estimated to be approximately 3 metres above normal high-tide level; at Lazaro Cardenas, where the tsunami washed away a road embankment to the container terminal; and at Playa Azul, where shellfish beds were exposed by the initial withdrawal of water before the arrival of the surge. The loss of three fishing boats without trace at the time of the earthquake is attributed to the tsunami.

The generation of a tsunami suggests a massive seafloor displacement. A possible cause was slumping induced by the vibration of the earthquake, rather than fault displacement on the sea bed. Tsunami

FIGURE 13. SPACE-TIME PLOT OF ESTIMATED RUPTURE ZONES OF MAJOR EARTHQUAKES OF MAGNITUDE 7 OR GREATER ALONG PART OF THE MEXICAN SUBDUCTION ZONE, SHOWING RECOGNISED SEISMIC GAPS (based on [6] and [7]).

generation is usually associated with long-period motion.

The main shock occurred in the Michoacan seismic gap, an area previously suspected to be seismically inactive because no major earthquakes were known to have occurred there between 1800 and 1981, a 180 year period which far exceeds the typical return period of 30 to 70 years for most of the subduction zone. Fig. 13 [6,7] is a space time plot of large earthquakes along the Mexican subduction zone, showing rupture areas and seismic gaps where no large earthquakes have occurred recently. Fig. 12 also shows the position of the Michoacan and Guerrero seismic gaps. A magnitude 7.3 earthquake occurred in the centre of the Michoacan gap in 1981. It was thought likely that earthquakes filling in the remaining portion of the Michoacan gap to the north and the south would be of similar magnitude to this Playa Azul earthquake, since there was no history of larger events, although the possibility of a major earthquake had not been ruled out [8]. Mexican seismologist Dr Singh believes that great earthquakes might have occurred and been unreported, because the population was sparsely distributed and the instrumentation was poor. A crucial factor in determining the recurrence interval in the Michoacan gap is the true location of a magnitude 7.7 earthquake in 1911, located in the gap by Gutenberg and Richter, but relocated outside it by Singh et al [9]. If Gutenberg and Richter's original location within the gap were correct, this would reduce the recurrence interval in this area to a value similar to that in surrounding zones. Evidence used for Singh's relocation of the 1911 event was the intensity pattern, with the major damage occurring in the Ciudad Guzman area. The 1985 earthquake also caused extensive damage in Ciudad Guzman thus partly discrediting Dr Singh's relocation.

In contrast to the Michoacan gap, the neighbouring Jalisco gap to the north-west and the Guerrero gap to the south-east are considered to have a high potential for a large earthquake. Large events have occurred in both in the past, and the interval since the last of these is of the order of the usual recurrence interval. The strong-motion array being installed at the time of the earthquake was primarily intended to record a large earthquake expected to occur in the Guerrero gap sometime within the next 10 years, although some consideration had been given to the possibility of an earthquake in the Michoacan gap.

The main event appeared to be a double shock, not unusual for an earthquake of this magnitude, but observed only once before for a large Mexican subduction event, in the 1932 Jalisco earthquake [7].

An initial energy release near Caleta de Campos, close to the northern end of the rupture zone, was followed about 26 seconds later by a second burst of energy south of La Villita, near La Union. The extreme rupture length, multiple events and wave dispersion (components of various frequencies travel at different speeds)

over the propagation path to Mexico City contributed to the long duration of shaking which was so damaging there. In the soft soil zone of the city, the duration of the incoming excitation would have been further lengthened by multiple reflections as the waves became trapped in the soils of the former lake basin.

The largest aftershock, 36 hours after the main event, was located at the southern end of the rupture zone of the main shock, overlapping the aftershock zone of the 14 March 1979 Petatlan earthquake (fig. 12). At the time of our visit there was concern among some Californian seismologists that some of the later aftershocks that had occurred and were continuing to occur south of this zone had penetrated into the Guerrero gap, and may be either precursors or triggers of the large event still expected in that zone.

Further south along the subduction zone the M 7.8 Oaxaca earthquake of 29 November 1978 had been successfully predicted. There was no such prediction of the September 1985 events, although their epicentral region had been identified as a seismic gap.

6. STRONG MOTION ACCELEROGRAPH RECORDS

The ground motion in the coastal region near the epicentre was recorded on digital accelerographs of a new network, the Guerrero accelerograph array [10], which had 20 of its planned 30 stations installed at the time of the earthquake. There were a few other instruments installed in the epicentral region, independent of the array. The Guerrero records were the first ever obtained within the rupture zone of a magnitude 8 earthquake. The shaking was also recorded in Mexico City at 15 strong motion accelerograph stations on sites with various ground conditions. Virtually all instruments were located at ground sites. There were no sets of interconnected multi-instrument installations to measure the structural response over the height of any building.

Altogether there are about 150 strong-motion accelerographs in Mexico, analogue and digital types, according to the magazine "Construction Today" (November 1985 issue), and most of these triggered in the earthquake.

Computer tapes of the processed records from Mexico City and the Guerrero array are held by the Engineering Seismology section, PEL, DSIR.

6.1 The Guerrero Accelerograph Array

Planning for the Guerrero accelerograph array began as a result of the International Workshop on Strong-Motion Earthquake Instrumentation Arrays held in Honolulu in 1978. [11.] The installation was a joint project of the Institute of Geophysics and Planetary Physics, University of California, San Diego (UCSD) and the Instituto de Ingenieria (IDEI), Universidad Nacional Autonoma de Mexico (UNAM). The array was designed to record strong ground motion on bedrock from large

FIGURE 14. GUERRERO ACCELEROGRAPH ARRAY (from [12]).

subduction zone earthquakes in the Guerrero gap. A secondary purpose was to monitor the Michoacan gap where the September 1985 earthquake occurred. The status of the array at the time of the earthquake and a preliminary summary of the data obtained are presented in a series of joint reports by UCSD and UNAM [12,13].

The locations of the instrument sites are shown in the map of fig. 14. The 20 stations in operation at the time of the earthquake are shown as heavy stars and triangles, denoting the 2 models of digital accelerograph in use, with the 10 further stations scheduled to be installed by the end of 1985 shown as circles. The shape of the array was a double-headed T, with a dense line of instruments along the coast at a typical spacing of 20 km along the Guerrero gap and at a spacing of 50 km further north-west along the Michoacan gap, a second more widely spaced line parallel to the coast about 100 km inland, and a leg running inland from the coast to Mexico City. Most of the coastal instruments had been installed, but none of the proposed inland instruments at El Balcon, La Comunidad and Filo de los Caballos which would have provided valuable information on the propagation of the earthquake towards Mexico City were in place. Only Teacalco supplied this information because the instrument at Tonalapa failed.

The 20 instruments installed at the time of the earthquake consisted of 8 DCA-333 Terra Technology digital accelerographs and 12 Kinemetrics DSA-1 digital accelerographs. All were installed between February and August 1985, just in time to record motions in the epicentral area. Both types of instrument use 3 triaxially-mounted force-balance transducers and record on digital cassette tapes.

The prototype field station outside the I de I building at UNAM is shown in fig. 15. The accelerograph and batteries are contained in separate compartments of a steel box. The steel box is mounted on a large concrete block which is anchored in the underlying rock by steel bars. An adjacent 9 metre tall guyed tower supports solar panels for recharging the batteries and a receiver for satellite-transmitted time signals.

Six of the accelerograph stations (Caleta de Campos, Ixtapa-Zihuatanejo airport, Atoyac, El Cayaco, Tonalapa and Teacalco) were inspected by the New Zealand reconnaissance team. There was slight damage at the airport, and some cracking of poorer brick at Atoyac, but we found no damage at the other accelerograph sites. Six other towns containing accelerograph sites (La Villita, La Union, Papanoa, El Suchil, El Paraiso and Coyuca) were visited

FIGURE 15. PROTOTYPE ACCELEROGRAPH STATION AT THE LAVA SITE AT UNAM.

but we did not locate the instruments. Neither did we see significant damage in any of these towns.

All the array instruments were nominally sited on bed-rock. However, there was considerable variation between the sites we visited. The Teacalco site was on a rocky outcrop in volcanic country. The Atoyac site was next to a church on a rounded hill-top where the soil conditions could not be determined because there was a paved surface. We suppose the presence of crumbly granite rock like that outcropping in nearby hill-tops. The ridge at the Ixtapa airport where instruments were sited was of similar material. The Caleta de Campos site was on the seaward end of a broad ridge of weathered argillite. The El Cayaco instruments were sited on similar soil on flat ground.

Several instruments were located on soils not typical of their areas. Firm ground or rock sites were chosen where possible so as to limit site variability and thus to simplify studies of the propagation of earthquake waves. However, most structures in many of these towns were built on sandy soils. For example, the Ixtapa-Zihuatanejo instrument was on a rocky ridge outcrop at the airport, while the nearby terminal building which suffered slight damage and the extensively damaged hotels at Ixtapa about 10 kilometres away were built on sandy soils. Similarly there were no array instruments on the alluvial deposits in the delta of the Rio Balsas where the damaged structures in the Lazaro Cardenas area were built. The sites had been chosen to suit the needs of seismologists interested in the generation and propagation of earthquake motion rather than the needs of engineers interested in earthquake effects on structures. The records from the rock sites often gave little indication of the long period motions which affected structures built on flexible soils.

The array produced records from 15 stations. Records from 2 sites had not been retrieved at the time of our visit, and 3 instruments did not function. The principal

aftershock was recorded by most instruments. Of the 3 instruments which failed to produce records, one had been flooded, one had a tape transport malfunction and one suffered multiple triggerings which filled the cassette before the earthquake. Overall the array performance was good. It appears that thorough laboratory testing before installation had eliminated most initial faults with the instruments, and most had been installed too recently for faults to have developed in the field.

A list of peak accelerations recorded by the Guerrero array instruments during the main shock and principal aftershock, from reference [10], is given in Table 4.

For the main shock, peak accelerations of 0.12g to 0.17g were recorded at 5 array stations, 4 of which were sited above the underground rupture area. Peak accelerations exceeding 0.1g were recorded at 2 of these stations during the largest aftershock, with a maximum acceleration of 0.2g. A non-array instrument at Zacatula [14], situated on compacted clay in the delta zone of the Rio Balsas, 9 km from the steel mill at Lazaro Cardenas, recorded a peak acceleration of 0.27g, higher than any of the rock-based instruments of the Guerrero array. Teacalco, the array station closest to Mexico City, produced a peak acceleration of 0.05g, only slightly higher than the strongest rock records obtained in Mexico City. A record recovered after the team visit from the accelerograph site at El Paraiso produced the strongest peak accelerations from the array. The accelerations were 0.64g for the North-South component and 0.48g for the East-West component for the magnitude 7.5 aftershock of 21 September 1985. By comparison the same accelerograph recorded only 0.12g for this main shock [15]. The spectra in the epicentral region were generally broad-band between about 0.1 and 3 Hz, with no predominant frequency. They contained high frequency motion as well as the long period motion which dominated Mexico City's records.

The strongest 5% damped absolute acceleration spectrum for the processed records from the coast zone was for the NS component recorded on a non-array instrument on the clay soils at Zacatula. This spectrum peaks at 1.03g at a period of 0.34 seconds, and from a small scale plot appears similar in strength and shape to the El Centro 1940 NS component, the record most widely used as a design accelerogram for seismic regions around the world. Spectra on similar soils at the nearby Sicartsa steel mill in the 25 October 1981 earthquake peaked at 1.33g at 0.30 seconds period (NS) and 1.16g at 0.34 second period (EW) [14].

The response spectra for the records from the rock-mounted instruments of the Guerrero array are much weaker than the spectrum of the Zacatula record. The strongest 5% damped rock spectrum, the NS component from La Union, peaked at 0.5g at 0.3 second spectral period. The spectra of the records obtained from above the rupture zone at Caleta de Campos, La Union and La

TABLE 4. PEAK ACCELERATIONS FROM GUERRERO ARRAY STATIONS.
(from references [10] and [15])

	Peak Accelerations (cm/sec ²)					
	Ms 8.1 Mainshock			Ms 7.5 Aftershock		
	N-S	E-W	Vert.	N-S	E-W	Vert.
Caleta de Campos	138	141	89	-	-	-
La Villita	125	122	58	33	40	20
La Union	166	148	129	48	76	58
Zihuatanejo	103	161	105	120	130	90
Papanao	162	117	84	200	200	150
El Suchil	102	81	51	80	50	40
Atayao	53	59	60	80	74	73
El Cayaco	41	48	24	60	43	21
Coyuca	42	35	19	40	50	30
El Ocotito	49	54	21	-	-	-
La Venta	18	21	16	13	18	14
Cerro de Piedra	27	15	13	10	10	10
Las Mesas	22	18	19	31	22	19
Teacalco	49	24	27	15	11	18
Xaltianguis	25	18	20	-	-	-
El Paraiso	120	90	73	180	480	640
(UNAM rock site)	32	35	22	-	-	-

Villita (fig. 16) are generally similar in strength to the 1952 Taft N21E component, although somewhat stronger at periods longer than about one second. The Taft record was obtained from a large magnitude earthquake but on different soils from any Mexican record. The Taft soil column consisted of 40 feet of alluvium over 2000 feet of consolidated gravel, sand and clay. The instrument site was at an epicentral distance of 43 km from the magnitude 7.7 Kern County earthquake. The Caleta de Campos, La Union and La Villita spectra are only about half the strength of the El Centro 1940 NS component, which was recorded on several thousand feet of alluvium at an epicentral distance of 9 km from the magnitude 6.7 Imperial Valley earthquake of 1940.

The acceleration history plots of the Zacatula, Caleta de Campos, La Union and La Villita records are high-frequency in appearance, but have the suggestion of an underlying component with a period of several seconds, especially the La Villita record. The duration of strong shaking was fairly long, approximately 40 to 50 seconds for the Zacatula and La Villita (fig. 17) records, although it was only 25 to 30 seconds for the La Union (fig. 18) record.

The Zacatula and La Villita records show very clearly two bursts of energetic shaking, the first starting about 10

seconds into the record and the other after about 40 seconds. The first burst of strong motion is of higher amplitude and somewhat longer duration than the second burst. Seismologists have interpreted these two bursts of energy as two distinct ruptures within the overall event, the first near the north-western end of the rupture zone near Caleta de Campos, and the other further south in the La Union region.

The strength of shaking recorded over the rupture zone was surprisingly moderate for a magnitude 8.1 event. The focal depth, approximately 20 km, may have moderated the epicentral shaking. Shallower Californian-type events of similar magnitude are likely to produce much stronger near-source shaking. However, the moderate accelerations in the epicentral region cannot be interpreted as a general feature of large subduction zone earthquakes. The magnitude 7.8 Chilean earthquake of 3 March 1985 produced peak horizontal accelerations of up to 0.67g. The recorded accelerations are consistent with the extent of damage observed in the epicentral region, which was also surprisingly light. Near the rupture zone, we observed damage mainly to longer period structures, again somewhat surprising given the broad band nature of the accelerograms. Amplification of long period components by the soft alluvial soils of the Rio Balsas delta seems to have occurred, judging by the nature of the

F19852F3 LA UNION COMPONENT N00E

MICHUACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME

ACCELERATION RESPONSE SPECTRUM

Damping value = 5 percent of critical

F19852F4 LA VILLITA COMPONENT N00E

MICHUACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME

ACCELERATION RESPONSE SPECTRUM

Damping value = 5 percent of critical

F19852F1 CALETA DE CAMPOS COMPONENT N00E

MICHUACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME

ACCELERATION RESPONSE SPECTRUM

Damping value = 5 percent of critical

FIGURE 16. NORTH-SOUTH COMPONENT OF THE 5% DAMPED ACCELERATION RESPONSE SPECTRA FROM ABOVE THE RUPTURE ZONE AT (a) LA UNION, (b) LA VILLITA AND (c) CALETA DE CAMPOS.

F19852F4 LA VILLITA COMPONENT N00E

MICHOACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME
 BAND-PASS FILTER TRANSITION BANDS ARE 0.100-0.250 HZ AND 24.5-25.5 HZ
 ● Peak acceleration: 1176 mm/s/s

F19852F4 LA VILLITA COMPONENT N90E

MICHOACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME
 BAND-PASS FILTER TRANSITION BANDS ARE 0.100-0.250 HZ AND 24.5-25.5 HZ
 ● Peak acceleration: -1191 mm/s/s

FIGURE 17. ACCELERATION HISTORIES OF THE HORIZONTAL COMPONENTS RECORDED AT LA VILLITA.

damage. The amount of damage was much less than expected for a magnitude 8.1 earthquake. Even poorly constructed dwellings in most of the towns and villages of the epicentral region survived unscathed.

Accelerograms from the epicentral region revealed several features which were very significant for Mexico City. The long duration in the source region, caused by the length of rupture and the multiple events, was an important factor. This duration of strong shaking was increased in the soft soil zone of Mexico City. Over the long propagation path of the earthquake waves to Mexico City, considerable dispersion (that is the separation of short-period and long-period waves) occurred. Waves of different frequencies propagate with different velocities. Thus the duration of the shaking lengthens as

the wave packets disperse with increasing propagation distance. The duration of shaking in the soft soils of the lake-bed zone increased considerably from that of the dispersed incoming wave train because there were multiple reflections within the lacustrine deposits. Wave reverberation lengthened shaking duration to several minutes in places.

The rich long-period content at the source was also important in determining the motion in Mexico City. High-frequency waves die out rapidly with distance as they propagate, while long-period waves decay much less rapidly. This behaviour is illustrated by comparing the spectra from Teacalco (fig. 19), the array station closest to Mexico City, with those from the coastal region. The Teacalco spectral accelerations are similar to those of the other records at long periods, but much

F19852F3 LA UNION COMPONENT N00E

MICHOACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME
 BAND-PASS FILTER TRANSITION BANDS ARE 0.100-0.250 HZ AND 24.5-25.5 HZ
 ⊕ Peak acceleration: 1634 mm/s/s,

F19852F3 LA UNION COMPONENT N90E

MICHOACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME
 BAND-PASS FILTER TRANSITION BANDS ARE 0.100-0.250 HZ AND 24.5-25.5 HZ
 ⊕ Peak acceleration: -1473 mm/s/s,

FIGURE 18. ACCELERATION HISTORIES OF THE HORIZONTAL COMPONENTS RECORDED AT LA UNION.

F19852F2 TEACALCO COMPONENT N00E

MICHOACAN EARTHQUAKE 1985 09 19 1317 UNIVERSAL TIME
 ACCELERATION RESPONSE SPECTRUM
 Damping value = 5 percent of critical

FIGURE 19. NORTH-SOUTH 5% DAMPED ACCELERATION RESPONSE SPECTRUM FROM TEACALCO, THE ARRAY STATION CLOSEST TO MEXICO CITY.

reduced for periods less than one second. As in the epicentral region, the long-period motions did not have much effect on short-period structures. There was little structural damage to low-rise buildings in Mexico City.

6.2 Strong Motion Records in Mexico City

The shaking was recorded in Mexico City at strong-motion accelerograph stations located in each of the soil zones of the city (fig. 5): the lake-bed zone (soft deep clays), the transition zone, and the hill zone (stiff soils and rock, including lava flows). These different soil types respond differently in earthquakes, producing both different intensities and different dominant frequencies in the ground motions. In the 19 September 1985 earthquake, the soft soil zone experienced considerably higher intensities than the hill zone. In Ciudad Universitaria in the hill zone, the intensity reached perhaps MMV. Much of the transition zone experienced MMVI. The intensities were MMVIII in much of the compressible clay zone, reaching MMIX in parts of it.

At the time of the team's visit, records were listed as having been retrieved from 15 sites in Mexico City, as follows [7]:

- 4 from Ciudad Universitaria (relatively hard ground, hill zone)
- 1 from Tacubaya Seismological Observatory (relatively hard ground)
- 1 from Los Viveros de Coyoacan (regular ground conditions, transition zone)
- 1 from Secretaria de Comunicaciones y Transportes (SCT) compound (soft clay of standard thickness, lake bed zone)
- 2 from Central de Abastos (soft clay of greater thickness, lake bed zone)

2 from Lago de Texcoco (soft clay of greatest thickness, lake bed zone)

2 from Tlahuac (soft ground)

1 from ground site adjacent to Madin dam (hard ground)

1 from upper part of Madin dam

Response spectra had been published from 3 of the Ciudad Universitaria records, and from Tacubaya, Los Viveros, SCT and Central de Abastos (locations are given in fig.s 4, 5, & 7). Computer tapes of ten of these records are available from the Physics and Engineering Laboratory, DSIR.

On the hard ground, the maximum horizontal accelerations were between 1 and 4% of gravity (Table 5). The acceleration spectra of the records from these sites were strongest at short periods. However, even at these sites there was considerable long-period content in the spectra. At the Ciudad Universitaria lava site [16] (fig. 20), there was a stronger 2-second period content than there was at many sites in the epicentral region, which is very unusual for this type of ground. The Tacubaya record [17], also obtained on hard ground but considerably nearer the centre of the city, showed similar characteristics, strongest at short periods but still containing significant long-period components. The records from the soft soil sites were considerably different in nature, characterised by dominant periods of 2 seconds or greater and reflecting the greater intensities experienced there.

The motion recorded at the SCT site in the soft clay zone was much amplified from that at the rock sites, with an acceleration of 0.17g in the east-west direction and with a nearly sinuoidal appearance (fig. 21) [18]. The response spectra were sharply peaked at 2.1 seconds, but with other peaks at both

TABLE 5. "FREE-FIELD" PEAK GROUND MOTIONS RECORDED IN MEXICO CITY.

	Acceleration (cm/sec ²)			Velocity (cm/sec)			Displacement (cm)		
	NS	EW	Vertical	NS	EW	Vertical	NS	EW	Vertical
HILL ZONE									
UNAM Patio	32	35	22	10	9	8	6	8	7
Tacubaya	34	33	19	14	10	8	12	9	8
TRANSITION ZONE									
Viveros	44	42	18	11	12	6	9	7	7
LAKE BED ZONE									
SCT	98	168	37	39	61	9	19	22	8
Abastos Frigorifico	81	95	27	25	38	9	15	19	8

F19851F2 Idel Patio COMPONENT S00E

Michoacan Earthquake 1985 09 19 1319 UNIVERSAL TIME

ACCELERATION RESPONSE SPECTRUM

Damping value = 5 percent of critical

F19851F2 Idel Patio COMPONENT N90W

Michoacan Earthquake 1985 09 19 1319 UNIVERSAL TIME

ACCELERATION RESPONSE SPECTRUM

Damping value = 5 percent of critical

FIGURE 20. 5% DAMPED ACCELERATION RESPONSE SPECTRA FOR THE LAVA SITE AT UNAM IN THE HILL ZONE: (a) NORTH-SOUTH AND (b) EAST-WEST COMPONENTS.

shorter and longer periods (fig. 22). The north-south component also had a sinusoidal appearance, again with a dominant 2.1 second spectral peak, but with a maximum amplitude of only 0.09g. Resolving the horizontal components in various directions gave a picture of directionality of the horizontal motion. There was a maximum acceleration of 0.20g in the N60E direction, but only 0.08g in the S30E direction orthogonal to it (fig. 23), and with little correlation apparent between components. It is not known whether this directionality is a feature of the incoming excitation or is caused by the geometry of the old lake bed zone. (Reference [18] gives the directions of strong and weak motions as S60E and N30E respectively. However, taking south as positive for the north-south component, as indicated by the

labelling S00E, gives the directions quoted above).

There were many cycles of strong motion, with 0.1g in the N60E direction first exceeded 16 seconds from the beginning of the record and last exceeded about 43 seconds into the record. The peak ground accelerations on the soft soils of Mexico City were only moderate, similar in amplitude to those recorded in the epicentral region; but, because of their long period, they were associated with large ground displacements. The SCT records showed a peak displacement of 212 mm for the EW component (fig. 21). The long duration of long-period large ground displacements was a major cause of the damage in Mexico City.

F19851F1 Secretaria de Comunicaciones y Transportes COMPONENT N90W

Michoacan earthquake 1985 09 19 1319 UNIVERSAL TIME

HIGH-PASS FILTER TRANSITION BAND IS 0.100-0.250 HZ

⊕ Peak values: acceleration -1700 mm/s/s, velocity -614.1 mm/s, displacement 212.23 mm

FIGURE 21. N90W COMPONENT OF THE ACCELERATION, VELOCITY AND DISPLACEMENT HISTORIES OF THE GROUND MOTION RECORDED ON THE SOFT CLAYS AT THE SCT CENTRE IN THE LAKE-BED ZONE.

The 5% damped acceleration response spectrum of the SCT EW component reached a peak of $1g$ at a period of 2.1 seconds. This is almost the same peak spectral acceleration as the El Centro NS component, but the peak in that record occurred at a period of 0.55 seconds, so El Centro displacements are much smaller (for near sinusoidal motions the response displacements are proportional to the square of the period). The spectral peak at 2.1 sec is sharp enough to make a good approximation of the displacement response available. It is 1.1 m.

To summarise, the SCT accelerograms showed that the ground moved nearly sinusoidally, with a peak acceleration of 0.2 g and 212 mm displacement. Ground motions with this amplitude of displacement for such a long duration have never been recorded before. The amplitude exceeded half these values for most cycles over an interval of 27 seconds. For a 5% damped structure of 2.1 seconds period at this site, that is for a structure with its natural period tuned perfectly to that of the soil, the ground motion would have caused a maximum response at the top of the structure of about $1g$ horizontal acceleration and more than 1 m displacement, with the vigorous response lasting through many cycles.

The $1g$ spectral acceleration response at 2.1 seconds period is the result of a "double resonance" of the soil layers and buildings. The ground motion records at this site differ from most ground acceleration records, which usually have a

high frequency noisy appearance similar to the appearance of the coastal region records. The regular, almost sinusoidal, long period motions are characteristic of the roof responses of tall structures. The approximately 30 m depth of soft sand and clay layers to the first hard layer at the SCT site responded in a manner similar to the response of a 10-20 storey building to the same rock motion. The long duration of the excitation allowed the motion in the soil layers to build up resonantly in response to the 2.1 second period components in the rock excitation. A double resonance occurs when a building tuned at the same period responds to the soft soil motion.

The SCT instrumentally recorded motion that was more structurally damaging than any previously recorded motion. Within the SCT compound a number of structures suffered partial collapses, typically at about the 6th floor of 10 storey structures. The taller SCT tower, 15 stories plus microwave antennas, suffered virtually no damage apart from that caused by a fire in an electrical thunderstorm following the earthquake. The large response displacements in structures at this site doubtless contributed to their failures. The number of cycles of yielding these structures experienced far exceeds the number specified by any design code. The New Zealand code requires 8 half cycles without significant degradation; but these buildings were subjected to about 24 half cycles of yielding.

F19851F1 Secretaria de Comunicaciones y Transportes COMPONENT S00E
 Michoacan earthquake 1985 09 19 1319 UNIVERSAL TIME
 ACCELERATION RESPONSE SPECTRUM
 Damping value = 5 percent of critical

F19851F1 Secretaria de Comunicaciones y Transportes COMPONENT N90W
 Michoacan earthquake 1985 09 19 1319 UNIVERSAL TIME
 ACCELERATION RESPONSE SPECTRUM
 Damping value = 5 percent of critical

FIGURE 22. 5% DAMPED ACCELERATION RESPONSE SPECTRA OF THE HORIZONTAL COMPONENTS RECORDED AT THE SCT CENTRE: (a) NORTH-SOUTH AND (b) EAST-WEST COMPONENTS.

The SCT accelerograms are extremely important records obtained from a zone of major damage; but the SCT compound is approximately 6 km from the centre of the main damage zone in the heart of the city. Previously there had been accelerographs in the central city, located in the National Lottery Tower on the corner of the Reforma and Juarez Avenue, near the collapsed and burnt out Regis Hotel, but these had not been maintained because of a lack of funding and so did not function properly. Similarly there had once been an instrument at Nonoalco adjacent to the Tlatelolco area north of the city centre, where the Nuevo Leone apartment building collapsed with large loss of life and the Foreign Ministry tower was badly damaged. Instruments at both of these sites would have produced records as valuable as the SCT centre records.

There are lessons for us in this failure to obtain records from sites. The New Zealand strong motion earthquake accelerograph network operated by PEL is presently in jeopardy, threatened by funding problems similar to those experienced in Mexico. It seems easier to obtain funding for the initial purchase of an accelerograph than for vital ongoing servicing.

Fortunately, despite the disappointment of obtaining no strong-motion records in the central city, records from previous earthquakes have shown that the characteristic periods of the soils in the city centre area are similar to those displayed by the SCT records during this earthquake (fig. 24) [19]. Hence the SCT records are probably representative of the motions throughout the heavily damaged central city region.

F19851F1 Secretaria de Comunicaciones y Transportes COMPONENT N60E

Michoacan earthquake 1985 09 19 1319 UNIVERSAL TIME

HIGH-PASS FILTER TRANSITION BAND IS 0.100-0.250 HZ

⊕ Peak values: acceleration 1891 mm/s/s, velocity 619.2 mm/s, displacement 220.99 mm

F19851F1 Secretaria de Comunicaciones y Transportes COMPONENT S30E

Michoacan earthquake 1985 09 19 1319 UNIVERSAL TIME

HIGH-PASS FILTER TRANSITION BAND IS 0.100-0.250 HZ

⊕ Peak values: acceleration -858 mm/s/s, velocity 302.3 mm/s, displacement 131.11 mm

FIGURE 23. THE HORIZONTAL COMPONENTS OF THE GROUND MOTIONS AT THE SCT CENTRE RESOLVED INTO THE N60E AND S30E DIRECTIONS TO SHOW THE STRONG DIRECTIONALITY OF THE MOTION.

FIGURE 24. SPECTRAL VELOCITY RATIOS FOR RECORDS FROM NONOALCO-TLATELOLCO (NT) IN THE LAKE-BED ZONE AND UNAM, THE NATIONAL UNIVERSITY (NU) IN THE HILL ZONE FROM PREVIOUS EARTHQUAKES (from [19]).

In other parts of the lake-bed zone, there were even longer predominant periods than those recorded at the SCT centre. For example, two ground mounted accelerographs at the Central Produce Market (La Central de Abastos) [20], located 7 km south of the airport on deeper soft soil, showed dominant periods of 3.5 to 4.0 seconds (fig. 25), although peak accelerations were only 95 cm/sec/sec free-field and 80 cm/sec/sec in the basement of a one-storey office building in the EW direction (168 cm/sec/sec at SCT) and 81 cm/sec/sec free-field and 69 cm/sec/sec in the basement for the NS component (98 cm/sec/sec at SCT) (fig. 26). Because of the longer dominant periods, the maximum ground displacements were very similar to those at the SCT (19 cm free-field and 25 cm basement, compared with SCT's 21 cm in the EW direction and 15 cm free-field with 25 cm basement displacement against the SCT's 17 cm in the NS direction). The records were of very long duration. The free-field accelerograph remained triggered for 1 1/2 minutes and the building basement instrument for 5 minutes. (The peak values given above are taken from reference 20. The plots are from data refiltered at PEL, DSIR, so show different peak values).

Despite the similarity of maximum displacements and the probability that the duration of shaking at the Central de Abastos was longer than at the SCT centre, there was very little damage in this area. Most construction was low rise, with natural periods far shorter than the predominant shaking at 3.5 seconds. Hence

the structures did not undergo the type of resonant response associated with the damage at the SCT centre and in the central city.

Low-rise buildings also generally escaped major damage in the central city area. Generally the earthquake affected only those buildings that had natural periods in the same range as the dominant periods of the ground vibrations, or periods which moved into this range upon softening of the structure. Low buildings and very tall buildings (e.g. Latin-American tower, PEMEX tower) escaped major damage principally because their response frequencies did not match energetic frequencies in the earthquake motion.

It is not always easy to predict the effective periods of buildings in Mexico City. Many of the seriously damaged buildings would not have started their response to the earthquake with periods of about 2 seconds. They were built with unreinforced masonry infill panels, tightly fitted, which would have stiffened them appreciably. But, even at low stress levels, brick panels cracked and the infill fell out. Period lengthening associated with increased flexibility increased response by forcing a shift into more energetic part of the spectrum of this earthquake.

Most of the damage occurred to 7 to 15 storey buildings. New Zealand buildings of this height would not normally have periods as long as 2 seconds, at least not until considerable yielding had occurred. The

F19851F4 C de Abastos Oficina COMPONENT S00E

Michoacan Earthquake 1985 09 19 1319 UNIVERSAL TIME

ACCELERATION RESPONSE SPECTRUM

Damping value = 5 percent of critical

F19851F4 C de Abastos Oficina COMPONENT N90W

Michoacan Earthquake 1985 09 19 1319 UNIVERSAL TIME

ACCELERATION RESPONSE SPECTRUM

Damping value = 5 percent of critical

FIGURE 25. 5% DAMPED ACCELERATION RESPONSE SPECTRA FROM THE OFFICE SITE AT LA CENTRAL DE ABASTOS ON VERY THICK SOFT CLAYS: (a) NORTH-SOUTH AND (b) EAST-WEST COMPONENTS.

Mexican preference is for more flexible framing and thus for longer period buildings, once the stiffness contribution of the infill panels is removed. Displacements in the soft soils in which buildings were founded would further lengthen the vibration periods.

The Mexico City experience that, as medium rise buildings soften, they move into more energetic parts of the earthquake response spectrum, differs from our experience that the response diminishes with softening. The response spectral peak for most earthquakes on stiff to medium soils is at periods in the 0.2 to 0.7 second period range, as exemplified by El Centro record. Yielding considerably lengthens the effective vibration periods of buildings, so, for these earthquakes, the onset of yielding moves the effective period out of the range

of most energetic excitation, and this affords protection. However, the long natural periods of the soil in Mexico City made the softening mechanism a liability rather than an advantage. Softening of the stiffness of buildings of about 6 to 15 storeys moved them into the energetic part of the excitation, rather than out of it.

The long natural periods of the soils of Mexico City are recognised in the seismic loadings code used there [21]. Reduced seismic design resistance for structural periods beyond 0.6 to 0.8 seconds, allowed in the New Zealand loadings code, which is similar in this respect to most codes, is not allowed in the code for Mexico City. The maximum level of seismic resistance in the soft soil zone must be provided for structures with periods between 0.8 and 3.3 seconds. Unfortunately, although the long-

TABLE 6. MAIN PEAKS OF ACCELERATION SPECTRA OF MEXICO CITY RECORDS.

	Peak no.	5% Damped Response Spectrum		Fourier Spectrum
		SA _{max} (mm/sec ²)	Period (sec)	Period (sec)
HILL ZONE				
UNAM Patio EW	1	1300	0.95	2.05
	2	1140	0.70	1.32
	3	1070	1.2	2.73
	4	882	2.1	1.64
	5	803	1.6	0.92
	6	723	0.525	0.80
	7	641	2.6	
	8	641	2.8	
	9	588	0.48	
	10	563	0.34	
NS	1	1180	1.7	2.56
	2	1010	0.9	1.95
	3	923	2.1	1.74
	4	852	2.6	0.91
	5	762	1.3	1.37
	6	702	0.65	
	7	634	0.55	
	8	591	0.40	
	9	561	0.44	
Tacubaya EW	1	976	0.75	
	2	840	0.85	
	3	817	0.44	
	4	799	0.55	
	5	725	1.3	
	6	638	2.6	
	7	600	0.24	
	8	595	3.0	
	9	578	1.0	
	10	492	1.7	
NS	1	1150	0.80	
	2	817	1.3	
	3	790	0.37	
	4	679	0.65	
	5	676	0.44	
	6	662	2.6	
	7	546	0.33	
	8	541	0.24	
	9	533	0.28	
TRANSITION ZONE				
Viveros EW	1	1570	0.70, 0.75	2.64
	2	1440	0.525	0.90
	3	1390	0.575	0.54
	4	969	0.42	0.80
	5	949	2.2	0.53
	6	918	2.6	
	7	913	2.4	
	8	823	3.0	
	9	798	1.6	
NS	1	1640	0.575	1.32
	2	1550	0.80	3.03
	3	1130	1.3	0.58
				2.00
				0.54
LAKE BED ZONE				
SCT EW	1	9850	2.0	2.02
	2	7120	2.6	2.56
	3	3500	0.70	2.34
NS	1	6010	2.1	2.05
	2			2.64
Abastos Office EW	1	3430	4.0	3.91
	2	2980	1.3	
	3	1970	0.8	
NS	1	4130	3.6	3.64
	2	2960	1.3	

F19851F4 C de Abastos Oficina COMPONENT N90W

Michoacan Earthquake 1985 09 19 1319 UNIVERSAL TIME

HIGH-PASS FILTER TRANSITION BAND IS 0.100-0.250 HZ

⊕ Peak values: acceleration 806 mm/s/s, velocity -378.1 mm/s, displacement 187.06 mm

FIGURE 26. ACCELERATION, VELOCITY AND DISPLACEMENT HISTORIES FOR THE EW COMPONENT OF THE OFFICE SITE RECORD AT LA CENTRAL DE ABASTOS.

period character of the shaking in this earthquake was anticipated and catered for in the Mexico City code, its amplitude was not. The strength of the long-period shaking, in terms of maximum ground acceleration, was about three times that envisaged in the code, which had been formulated on the basis of assessments of the maximum ground accelerations in the 1957 earthquake. The duration of strong shaking also greatly exceeded expectations.

Table 5 is a summary of the strong-motion data from the digital accelerographs in Mexico City. It gives the maximum ground accelerations, velocities and displacements. Table 6 lists some of the peaks of the response spectra.

The free-field and basement accelerographs in Mexico City gave sufficient coverage of each of the major ground condition classes to quantify the role of the soft soils in contributing to the damage in Mexico City. More operational accelerographs at previously instrumented sites in the central city region would have been valuable. The major deficiency in strong-motion recording in Mexico City and in the epicentral region was the almost total lack of structural response accelerographs, except on a few dams. Sets of free-field, basement, mid-height and roof recorders in buildings, or at least a minimum of basement and roof response instruments, interconnected to trigger together with common time-marks, would have provided much information about structural response, including the actual response accelerations and displacements in structures.

Records, had they been available from low-rise, medium-rise and high-rise structures would have demonstrated differences in response amplitudes for these classes of structure. Response records would have made it possible to track changes in the effective natural periods of structures as they softened through yielding or the loss of "non-structural" panels. Numbers of cycles of yielding in of various structures could have been identified. The role of soil-structure interaction could have been studied quantitatively. There would have been new information about torsion effects. Structural response records may have helped to determine causes for some unexplained features of the damage observed in Mexico City, such as the prevalence of mid-height collapses. In the absence of recorded response motions, it is difficult to tell whether these failures are related to some feature of the earthquake excitation or to some property of the structure.

7. SEISMIC DESIGN PROVISIONS IN MEXICO

7.1 Code Status and Implementation

Under the 1917 Constitution Mexico became a Federal Republic comprising 31 states and a Federal District which contains a major part of the capital, Mexico City. Mexico City contains over one sixth of the country's population and a large proportion of the engineered structures. The States have complete autonomy on local matters and this includes the responsibility for formulating building codes including seismic provisions. In contrast, the

Federal District is effectively under the control of the President of the United States of Mexico through his appointee, the Chief of the Federal District. The trend has been for seismic provisions to be developed for the Federal District and then be adapted by individual States although it would appear that not all States have or enforce seismic regulations.

The authority responsible for drafting codes and for issuing construction and occupation permits in the Federal District is the Federal District Department.

Responsibility for complying with code provisions is usually placed with the registered engineer or architect who is awarded the construction licence, and thus department engineers rarely check computations and drawings except in special circumstances [22].

A great deal of freedom has thus resulted in the design and supervision of construction of privately owned buildings.

This had led to a tendency for building codes to be regarded by Mexican engineers more as guidelines rather than rigid regulations. In legal disputes however ignorance or violation of the code requirements has usually greatly weakened the designer's position [22].

7.2 Comparison between Design Spectra and Spectra Derived from Motions Recorded During 19 and 20 September 1985 Earthquake

A comparison between the design spectra from the 1977 regulations ($\mu=1$) and spectra derived from motions recorded during the September 1985 earthquake is presented in fig. 27.

FIGURE 27: A COMPARISON BETWEEN THE DESIGN SPECTRA FROM THE 1977 REGULATIONS ($\mu=1$) AND SPECTRA DERIVED FROM MOTIONS RECORDED DURING THE SEPTEMBER 1985 EARTHQUAKE.

The spectrum recorded on the hard ground site, VIV, compares well with the code spectrum for Zone I (hard ground). It is apparent however that the 5% damped spectrum obtained at the SCT exceeds the design spectra for Zones II (transition) and III (soft soil) by approximately 200 and 300% respectively. When it is considered that many high rise buildings could have conceivably been designed for 25% of the spectral ordinates shown in fig. 27 (i.e. $\mu=4$) it is apparent that minimum design levels would have been exceeded by at least 8 times in the soft soil areas of the city. As already discussed a ductility factor of 4 could be used with very little attention to detailing for ductility.

It is of interest to note that although many structures were severely damaged when subjected to this level of overload, the great majority of structures situated in the soft and transitional soil zones of the city escaped with little or no apparent damage.

7.3 History of Development of Seismic Design Provisions in the Federal District (Mexico City)

The first regulations relating to seismic design in Mexico City were established in 1942. This was a working stress document allowing a 33% increase in allowable stress for load combinations including earthquake [22]. These regulations were only applicable to buildings 16m or greater in height, those structures with an aspect ratio greater than two and for buildings where crowds could be expected to congregate. A uniform horizontal acceleration was defined varying between 0.01 and 0.10g depending on the importance or intended usage of the structure. For general office buildings an acceleration of 0.025g was prescribed. An extract from the 1942 code may be found in reference 23.

Following the 28 July earthquake, which caused severe damage in Mexico City, emergency regulations [23] were drafted which remained operable until a new code was introduced in February 1966. The 1957 regulations were applicable for all new buildings less than 45m in height while buildings of greater height were required to be subjected to special study for which no special recommendations were given. Three different subsoil conditions in the Valley of Mexico were recognised, namely Lake bottom, Transition Zone and Hilly Zone (fig. 5) and buildings were classified in terms of usage and structural type. Seismic design coefficients of between 0.05 and 0.2g were applicable depending on building classification and location, and a triangular variation in horizontal acceleration over the height of the building was defined. A typical reinforced concrete framed office building without walls would have for example attracted a seismic coefficient of between 0.07 and 0.1g depending on location. Accidental eccentricities equal to 5% of the maximum building dimension were allowed for. This applied to all buildings except warehouses for which 7% was substituted. A two stage design procedure was required for structures having filler walls which could

be considered to contribute to structural rigidity. Firstly an analysis assuming the walls were present was required, followed by a check of the structure assuming failure of the walls. In each instance seismic design coefficients appropriate to the different design assumptions were allowed. The emergency regulations were, like the 1942 provisions, based on working stresses. Up to twice the allowable stresses applicable for static loads for concrete and up to 90 percent of the yield stress for mild steel reinforcement could be used. Structural steel structures could be designed for stresses up to 1.67 times the allowable static load stresses. Limit state type design methods were provided for with load factors of 1.2 and 1.0 for flexure and 1.5 and 1.3 for axial compression for reinforced concrete and structural steel respectively. Maximum relative displacements resulting from the code load distribution calculated using gross section properties were limited to 0.002 times the difference in elevation. Separate studies were required to determine separations to avoid pounding between adjacent buildings.

The provisions for earthquake resistant design introduced in 1966 [25] made several changes to the emergency regulations issued eight years previously. Many engineers who had applied the emergency regulations felt for various reasons that typical buildings were being unduly penalised because some vulnerable non typical structures had been subjected to high acceleration in 1957 [26]. The general feeling was that the design of typical buildings could be liberalised provided due attention was given to such factors as relative rigidities, pounding of neighbouring buildings and of apparently minor structural details [26]. The changes in the 1966 regulations to a certain extent reflected these views and included; the incorporation of the Transition Zone into the Lake Zone leading to the definition of only two soils zones, a zone of high compressibility and a zone of low compressibility; amendment of the base shear coefficients resulting in large reductions in base shear for important structures, such as telephone exchanges, hospitals etc, located in the Lake Zone; the requirement for a two stage design to account for the uncertain behaviour of infill walls was dispensed with even though unreinforced masonry infill walls satisfying certain requirements were still permissible. When using working stress methods of design for combinations of load including seismic the allowable stresses (assuming static load conditions) were permitted to be increased by 50% for timber, structural or reinforcing steel and by 33% for concrete and masonry. Three methods of analysis were introduced. For predominantly shear wall type structures satisfying given limits for aspect ratio and structural layout, a simplified method was allowed. Under this method, no check of lateral displacements, torsions or overturning moments was required. For other structures, either an equivalent static analysis or a spectral mode response analysis was permissible. Under the equivalent static analysis, a triangular

distribution of acceleration over the height of the structure was assumed. For a spectral modal analysis design, spectra were defined. There was a requirement to consider displacement, i.e. P- Δ effects, in the design of all structural members. Drift limitations in the 1966 regulations were as follows; 0.002h (h = storey height) where non-structural elements were not separated; 0.003h where non-structural elements in important structures situated in the zone of compressibility were separated and 0.004h for such structures elsewhere. For other structures, e.g. office buildings where non-structural items were separated, no limitations were imposed. The requirements for separation between neighbouring structures were quantified. In the zone of high compressibility, for example, the separation was required to be not less than the maximum lateral displacement calculated using the equivalent static load distribution at any level plus 0.006 times the elevation of that level.

The 1977 seismic loadings code [27] current until the earthquake on the 19th September 1985, differed from its predecessors in several respects. These differences included requirements for ductility, and the use of design load values based on a probabilistic assessment of seismicity. Structures were classified into three importance groups in a similar fashion to that set out in the 1966 code. Structures were also classified in accordance with their structural form i.e.:-

Type 1 : Structures in which lateral forces are resisted at each level by specified structural systems or combinations of systems.

Type 2 : Tanks

Type 3 : Retaining Walls

Type 4 : Other structures

Typically design spectra were given covering the three different ground conditions namely; firm ground, Zone I, transition soils, Zone II, and soft soils, Zone III.

The spectra defined were elastic spectra being reduced to design values through the use of a reduction factor based on an assessed ductility factor listed in the code. It is important to note that ductilities of up to 4 could be assumed under the provisions of this code with no special requirements for detailing for ductility. A ductility factor of 6 was possible if some ductile detailing similar to that set out in Appendix A, ACI 318 [27] was carried out. It would appear however that many engineers accepted the higher load levels resulting from the use of an additional detailing required for a ductility of 6. The code is essentially written in terms of ultimate strength design methods and the use of a load factor of 1.1 is implied by the level of seismic base shear coefficient.

Under the provisions of the code one of three analysis methods was to be used, the

choice depending upon the height and complexity of the proposed structure. Structures with height less than 60 m could be analysed using either the equivalent static or spectral modal response methods. For higher structures, a spectral modal response analysis was required. A simplified analysis could be used for predominantly wall type structures satisfying several aspect ratio requirements and with height not greater than 13 m. A capacity design type approach was not required.

Three limit state requirements relating to prevention of damage due to excessive drift were included. The first limited drift to 0.008h except where non-structural elements were fully separated from the structure when the limit was increased to 0.016h. Deformations were calculated using the reduced design spectra scaled up by the full ductility factor. The second required provision for glass panes to move in window frames and the third provided limits for separation at boundaries or at joints in structures. Separations at boundaries were required to be at least the calculated horizontal displacement plus 0.001, 0.0015 and 0.0020 times the height for Zones I, II and III respectively.

Following the severe shaking experienced during the earthquakes of 19 and 20 September 1985, several deficiencies in the 1977 provisions were recognised and amendments were made in the emergency regulations promulgated on 18 October 1985.

The emergency regulations set new seismic coefficients for Zones II and III which are respectively in the order of 35% and 70% greater than results obtained from application of the 1977 provisions. In addition, the implied ductility factors have been reduced from 6.0 to 4.0 and from 4.0 to 2.0 as applicable. The increase in design load when detailing requirements are ignored is therefore more pronounced. Drift limitations calculated using these increased loadings remain unchanged.

Flat slab type structures proved very flexible and in recognition of this the stiffness contribution for the slab was reduced by reducing the width of the "beam" strip and taking half of the column stiffness in the equivalent frame analysis.

It is intended that these emergency regulations be applied to all new construction and also to all repair work that is undertaken.

7.4 Material Codes

In the Federal District, materials codes have been established for Reinforced Concrete, Masonry, Timber and Steel. These tend to follow accepted international practice, e.g. ACI 318 for reinforced concrete [27]. In general however, there seems to be no special requirements for seismic detailing except for those set out in the 1977 code for use with a ductility factor of 6.0.

A seismic design manual for buildings [28] is available which sets out recommendations

on confinement and other aspects of detailing for reinforced concrete structures appropriate to a choice of ductility factor of either 6 or 4. It is uncertain however what legal standing, if any, these recommendations have.

8. FOUNDATIONS

8.1 Soil Conditions

Mexico City has been built over two quite different foundation zones with a transition zone between them (fig. 5). According to Marsal [4] the original development of the City occurred in the western region of Lake Texcoco where the clays are highly plastic, compressible soft soils with natural moisture contents (the most characteristic parameter of the soils) which vary from about 100 to 400 percent. A harder sandy clayey layer about 3 m thick lies at depths from 25 to 35 m while a further hard sandy layer lies at a depth of about 50 m. The water table is commonly one metre below the surface. The geological profile is given in fig. 28.

Later the City growth extended westwards over the foothills of the Sierra de las Cruces where hard soils and lava predominate. In between these two zones a transition zone where clayey layers of lacustrine origin alternate with erratically distributed sandy alluvial deposits [4].

8.2 Common Foundation Types

Foundation types are chosen to suit the type and size of building. Types include rafts, compensated rafts, friction piles, and end bearing piles to upper or lower firm layers. Some examples of a compensated raft giving relief to piles were observed. It was reported that up to 5 or 6 storey buildings have compensated raft foundations.

Most major structures are founded on end bearing piles taken down to the upper hard layer. This renders the building vulnerable to settlements due to loading of the stiff clay and accounts for many out-of-plumb buildings in Mexico City.

8.3 Settlement

The extraction of water from aquifers under the City and the presence of fill and construction loads have caused considerable sinking of the City [4].

General settlement and differential settlement of significant proportions is evident in many buildings. This is particularly true of the older masonry structures of up to 4 storeys. Even so, some of the more modern buildings show signs of settlement and tilting under gravity loads.

8.4 Lateral Loads

It appears that designs allow for lateral loads to be transmitted by passive pressure on foundation walls, or by pile flexure. No clear indication was given as to the way in

FIGURE 28: MEXICO CITY: TYPICAL SOIL PROFILE FOR THE NORTH-WEST DIRECTION.

which adjacent buildings affect this.

8.5 Effects of September 1985 Earthquakes

The following effects were observed:

- Differential settlements of up to 500 mm were numerous. These were usually in the short direction of buildings but in one case, in the long direction. The effect was observed in buildings of 6-12 storeys.
- Uplift of the foundation slab or pile cap combined with settlement of the ground surrounding piled structures. Relative displacements were generally 150-200 mm. Slender structures were the worst affected.
- One case of complete pile pull-out with resulting building over-turning was reported.
- Buckling of ground slabs, pavements, etc. as evidence of relative displacements of buildings at ground level.

In many cases, structural failure prevented foundation failure by acting as a fuse. Low rise buildings, presumably on rafts, compensated rafts or spread footings, suffered little damage. In cases observed of differential settlement, it was difficult to determine whether such movement took place as a result of the earthquakes.

Observed damage to superstructures (described below) frequently showed the worst effect at 4 - 5 storeys up. This is clear evidence of a combined super-structure/substructure response - the influence of soil-structure interaction was very significant.

9. STRUCTURAL DAMAGE

9.1 General

The dominant construction material in Mexico for engineered structures is reinforced concrete. There are only a few steel structures. Consequently, most of the damaged structures are of reinforced concrete; but there were nevertheless steel-framed building failures some of which are discussed in this report. For non-engineered structures (eg: single-storey dwellings) the observed damage was surprisingly slight despite the fact that they are typically constructed of baked bricks and mortar with a plaster finish, or adobe, with minimal lateral load resistance. Even in the city of Lazaro Cardenas, which is located on the coast less than 30 km from the earthquake epicentre, the damage to housing was minimal whereas some structures of two or more storeys had collapsed or were severely damaged.

9.2 Damage in Mexico City

Mexico City covers an area in excess of 1000 sq.km. Within this area damage to buildings was confined to an area of approximately 65 sq.km with a zone of approximately 25 sq.km where collapse or severe damage to buildings occurred. These two zones are shown in Fig. 2. The area of extreme damage or collapse however encompasses the central city zone in which many of the Government buildings are located and so it was the very heart of the city which was hit. As an indication of the extent of damage the following data was obtained from the office of Secretary General of Building Construction on 16 October 1985 (this data was being revised daily as the number of post-earthquake inspection reports filed increased and as circumstances changed).

Category	Not to be used*	Usable if repaired	Usable
Public buildings	68	27	39
Schools	372	115	800
Health centres	10	15	28
Cinemas and theatres	38	18	43
Buildings and apartments	449	643	652
Sports centres	3	1	7
Markets	8	5	99
Bridges		1	

(* "Not to be used" does not necessarily mean collapsed or damaged - some structures were so classed either because the loss of potable water supply rendered them unsuitable for occupancy, eg: schools, or because they could have been threatened by the possible collapse of an adjacent structure. In the schools category, 25 were classed for reconstruction although not all 25 had been damaged).

Although many structures were damaged, it is important to keep this in perspective. There are more than 1.5 million structures in Mexico City.

The observed affects of the earthquakes provided clear examples of almost every known cause of poor seismic performance, especially in multi-storey construction.

These included:

- Inadequate member strength and lack of ductile capability.

Numerous instances of complete disintegration of reinforced concrete columns were evident, due to the opening up of column ties and loss of confinement as a result of removal of concrete cover due to hinging or shear and the unfolding of 90° hooks to ties. As a result, longitudinal bars buckled, core concrete fractured and the gravity load carrying capacity at the column was lost (fig. 29). In beams, similar effects were observed in 'hinge' regions. The use of 90° hooks to column ties and stirrups appears to be almost universal in Mexico. Several examples of apparent compression failures in columns of were also observed. Fig. 30 illustrates one example of beam column joint failure leading to partial collapse of a reinforced concrete office building and figs. 31 and 32 show damage resulting from apparent column failures.

- Pounding of adjacent structures.

Provision of a seismic separation gap between adjacent structures, particularly in the case of newer construction, was common. However, in many instances, the clearance was inadequate and impact occurred. The consequence of impact was especially severe when floors were at different heights, so that the floors of one structure struck the columns of its

FIGURE 29: BUCKLING OF LONGITUDINAL BARS, FRACTURE OF CONCRETE CORE OF COLUMN AND LOSS OF CONFINEMENT DUE TO UNDOLING OF 90 DEGREES HOOKS TO COLUMN TIES. NOTE BUNDLED BARS IN CORNERS OF COLUMNS.

FIGURE 30: EXAMPLE OF BEAM - COLUMN JOINT FAILURES LEADING TO PARTIAL COLLAPSE OF A REINFORCED CONCRETE OFFICE BUILDING ON CALLE DE TANALA, MEXICO CITY.

FIGURE 31: COLLAPSE OF THE HOTEL DE CARLO, MEXICO CITY DUE TO COLUMN FAILURES. NOTE THE LOSS OF TWO STORIES AT THE FRONT AND PROBABLY THREE OF THE REAR.

FIGURE 32: COLLAPSE OF TWO STORIES AT THE BANK OF MEXICO, MEXICO CITY. THE BUILDING WAS UNDER CONSTRUCTION AT THE TIME OF THE EARTHQUAKE. NOTE THAT THE CONCEPT OF THE STRUCTURAL SYSTEM WAS NOT ONE OF WEAK BEAM - STRONG COLUMN.

FIGURE 33: IMPACT DAMAGE AND COLLAPSE OF STRUCTURE AS A RESULT OF POUNDING OF ADJACENT FLEXIBLE BUILDINGS, NOTE THAT THE FLOOR SLABS DID NOT CORRESPOND.

neighbour. One such example is shown in fig. 33. In some instances it was the intent of the designer to provide a seismic gap. However, a lapse in quality control either during construction or subsequent to completion meant that the gap was not maintained. For example, several cases were observed where formwork had been left in place effectively nullifying the gap. Appropriate flashing of the gap to prevent the accumulation of debris in the clearance during the working life of the facility had not been installed in other instances.

One problem peculiar to Mexico City is large scale settlements which have, and still are, occurring in buildings located on the soft clay soils. The settlement is typically non-uniform over the extent of any one site with the result that very few structures still stand in their as constructed vertical alignment. The relative tilt in adjacent structures had reduced the seismic gap for some buildings prior to the earthquake, thus increasing the potential for collision.

Another failure mode related to the absence of effective seismic separation gaps was that caused by the propping action of lower height adjacent structures. The lower portion of the

taller structure was stiffened by the propping but the upper levels above prop height continued to respond with full flexibility. Failure generally occurred in these upper levels. Many of the older low rise masonry or adobe structures were built with no seismic gap. However, the characteristics of this earthquake were such that these short period structures were not strongly excited. Damage did occur to some of these low rise structures when a flexible frame structure impacted against them. The characteristics of failures of this type were diagonal tension cracking extending at an angle of approximately 45° from the point of impact across the facade of the lower structure to ground level. In one case, the cracking was observed uninterrupted through three adjacent buildings.

- Torsional eccentricities introduced by building configuration.

Torsional eccentricity was responsible for the poor performance of some buildings. Given the population density of Mexico City, the temptation is to develop any plot of land regardless of its shape. Consequently, it is not uncommon to see triangular (in plan) structures occupying the land bounded by two streets meeting at an acute angle. To suit a retail occupancy, the two street frontage faces would be open frame with glazing and the third back wall probably a shear-wall formed by a concrete frame with brick infill. The result of this configuration is a large eccentricity between the centre of stiffness and the centre of mass. Several structures were severely damaged or collapsed because of the high ductility demand on the columns remote from the main boundary walls (these columns typically had only nominal ties with right angle hooks). A notable exception was the steel framed National Lottery building which was undamaged externally despite its triangular plan form and glass curtain wall on two faces (fig. 68).

- Weak column-strong beam frames

Numerous instances of column failure in shear, torsion or flexure resulting in the partial collapse or collapse of buildings were observed. Many of these failures were caused by the use of deep spandrel beams or unseparated infill panels which had the effect of shortening columns and increasing shear forces. The detailing of the columns could not match the requirement for ductile behaviour.

- Weak flat slab-column junctions

Several failures due to inadequate shear capacity in flat slabs adjacent to columns were observed. The waffle slab in various forms is a common slab type used in Mexico City. A typical failure is a partially collapsed building is shown in fig. 34. Fig. 35 shows an example of a collapse of an eleven storey flat slab structure. The columns remain standing, but separated from the slabs

FIGURE 34: TYPICAL FAILURE OF WAFFLE FLAT PLATE FLOOR DUE TO FLEXURAL - TORSIONAL - SHEAR FAILURE ADJACENT TO COLUMNS. SECRETARIAT OF WATER SUPPLY BUILDING, MEXICO CITY.

they once supported by shear failures in the junction between slab and column.

- Additional loading

Some structures, due to a change in occupancy, were carrying imposed loads in excess of those assumed in design. This may have contributed to collapse or damage.

- Poor steel-to-concrete bond

The team was surprised to see numerous instances where concrete had debonded completely from the reinforcing steel especially in regions where hinging had occurred and also the ease at which some structures were being demolished by hand with sledge hammers (fig. 37). One case was reported of a full beam cage being recovered intact and in another, reinforcing bars being removed apparently undamaged from a slab under demolition. No obvious reasons for this behaviour was established and further investigation is clearly needed.

9.3 Damage to buildings in the epicentral region including: Cd Guzman, Chilpancingo, Acapulco, Ixtapa, Lazaro Cardenas and Playa Azul.

Cd Guzman was one of the few cities in which adobe construction was severely damaged. It has been estimated that approximately 40% of the buildings in the city were of adobe type construction and of these, some 75% were damaged to some extent. Damage was generally to the front rooms of houses which opened directly on to the street. Several villages around Guzman were also severely hit, but it was interesting to note that in many others, structures of similar construction survived almost completely undamaged.

FIGURE 35: SHEAR FAILURES AROUND COLUMNS IN AN 11 STOREY STRUCTURE RESULTED IN CATASTROPHIC COLLAPSE. NOTE THE COLUMNS STILL PARTIALLY STANDING. BUILDING ON AV OAXACA, MEXICO CITY.

FIGURE 36: BUILDING, REPORTED TO HAVE BEEN DESIGNED FOR OFFICE USE, WHICH WAS A CLOTHING FACTORY AT THE TIME OF THE EARTHQUAKE. NOTE THE HEAVY LOADING OF FABRICS AND PACKAGING MATERIALS.

FIGURE 37: CIRCULAR COLUMN REINFORCING CAGES FROM A BUILDING UNDER DEMOLITION ON AV INSURGENTES, MEXICO CITY. THE CONCRETE CAN BE EASILY REMOVED FROM OUTSIDE AND FROM WITHIN THE REINFORCING CAGE.

Chilpancingo which has been severely hit in previous earthquakes was also affected by these earthquakes. Cracking was observed to the church tower in the square. There was also some damage to the Departmental Buildings around Main Square. This damage was generally non-structural or to elements which had not been adequately separated.

It was reported that there was no damage in Acapulco.

At Ixtapa, a tourist resort, none of the nine or so high-rise hotels escaped damage in some form or other and many had been closed. The Sheraton, a 14 storey structure suffered damage between the main building and an attached atrium structure even though some attempt had been made by the designers to allow each structure to act independently. Some suffered extensive structural and non-structural damage while a few including the Dorado Pacifico

suffered considerable non-structural damage.

Damage to buildings at Lazaro Cardenas (approximately 30 km south of the epicentre) was reported to be mainly to structures of two or more stories. Several structures including schools collapsed.

A number of hotels were badly damaged in Playa Azul, a resort area to the north of Lazaro Cardenas. In one hotel, three separate, three storey wings of a four wing complex partially collapsed due to column failures. The failures were due in the main to the existence of a soft first storey caused by infill walls being present only in the upper floors. The lack of damage to single storey, adobe construction so close to the epicentre was of particular interest.

9.4 Case Studies

The following are reports on individual buildings studied by team members. Due to lack of information and in-depth studies, some of the comments lessons and conclusions are inclusive. It is believed however that the information gathered is worthwhile and instructive in the form presented.

CASE STUDY NO.1

Building: Hotel de Paseo
Corner Napoles and Reforma,
Mexico City

Principal Features: Superstructure: 17 storey
r.c./masonry in infill frame
(adjacent sides). Glazed
street frontages (see fig.
38). Set back forms 'L'
shape above 6th floor.
Square plan below 6th floor.
Highly irregular.

Foundations: Not ascertained.
Second firm layer at 37m
depth.

Earthquake Performance: No significant damage
apparent from external
inspection in spite of
irregularities and damage to
similar nearby buildings.
Building not occupied.

Reinstatement: Apparently minor.

Lessons/Questions: Did foundations provide
protection? Did site escape
major ground motion?

Comments/Conclusions: Earthquakes are fickle.
Irregularities in themselves
are not causes of collapse.
Response of irregular
structures, including
foundation interaction
requires further study.

CASE STUDY NO.2

Building: DDF Water Supply Secretariat.
Corner Calz de Tlalpan and
Juan A Mateos, Mexico City

Principal Features: Superstructure: 10 storey
r.c. waffle slab
construction. Masonry infill
on adjacent walls. In frame
glazing (see fig. 39).

Foundations: Not ascertained.
Piles to second firm layer at
46 m depth.

Earthquake Performance: - Collapse of 8 levels in
the first bay on both street
frontages. Levels 1 and 2
remained intact (fig.s 40 and
41).
- Clear evidence of torsional
response.
- Low reinforcement levels in
slabs for diaphragm action.
- Computer floor pedestals
(glued to floor) all standing

FIGURE 38: HOTEL DE PASEO, MEXICO CITY.
VIEW FROM THE STREET SHOWING
SET-BACK.

after earthquake.

- Lightweight glazed partitions to Level 2 computer room.
- Considerable damage to unbraced suspended ceilings: rockfibre panels laid into very light exposed aluminium T-rail, hung on wires. Unrestrained areas of ceiling largely intact. Attachment of lightweight partitions to ceiling framing appeared to have contributed to ceiling damage.
- Computer floor panels removed but pedestals glued to floor slab all standing. Panels and computer equipment reported to have been undamaged.

Reinstatement: Will require demolition and a new building.

Lessons/Questions: Eccentricity clearly contributed to collapse though base columns were not severely distressed. Would better diaphragm integrity have prevented collapse?

Comment/Conclusions: Diaphragm integrity is important in controlling interstorey drifts, especially at the extremities of asymmetrical structural systems.

FIGURE 39: SECRETARIAT OF WATER SUPPLY, MEXICO CITY - CNR JUAN A MATEUS AND CALZ DE TLALPAN, PLAN AND SECTION

FIGURE 40: SECRETARIAT OF WATER SUPPLY BUILDING - VIEW FROM REAR SHOWING THE DAMAGE TO BRICK INFILL PANELS AND THE COLLAPSE OF THE TOP 8 STOREYS OF THE FIRST BAY OF THE BUILDING ADJACENT TO THE STREET.

FIGURE 41: SECRETARIAT OF WATER SUPPLY BUILDING - CLOSE UP VIEW OF FLEXURAL-TORSIONAL FAILURES OF THE WAFFLE SLAB ADJACENT TO THE COLUMNS.

CASE STUDY NO.3

Building: Hotel Continental (formerly Hilton),
cnr Reforma, Insurgentes and
Roma, Mexico City

Principle Features: Superstructure: 15-storey
r.c. 'frame' in two seismically
separate narrow blocks. Glazing
between slabs/shallow spandrels.

Foundations: Piles (?).
Second firm layer at 40m
depth.

Earthquake Performance: - Collapse of all levels of
one block above the 6th level
due to impact with adjacent
modern building (fig. 42).
- Severe damage to other
block with loss of glazing
and masonry.

Reinstatement: Both blocks will require
demolition and replacement
with modern construction of
greater integrity and
resilience.

Lessons/ Questions: Would collapse have occurred
without impact? Would a
modern building of greater
integrity survived the
impact?

Comments/ Conclusions: Weak columns had
insufficient shear strength
to withstand high impact
forces. New Zealand
designers should take care in
providing separation to avoid
impact at and beyond design
level earthquake. Considerable
ground deformation along the Reforma
frontage of the badly damaged
block which is still
standing. This hotel was
extensively damaged in the

1979 Petatlan earthquake, and
repaired afterwards. It was
also damaged in 1957, soon
after its construction, when
it was the Hilton.

CASE STUDY NO.4

Building: Hotel Jena
Av. Morelos and Reforma,
Mexico City

Principle Features: Superstructure: 12 storey
symmetrical r.c./masonry
infill. Glazing plus
spandrels at street front.

Foundations: R.C. piles
probably to upper firm layer.
Second firm layer at 42m
depth.

Earthquake Performance: Partial collapse of roof
level. Damage to masonry
shear walls. Glass breakages
at street level. Evidence of
considerable movement of
ground floor relative to
pavement. Buckled
reinforcing at pile head
indicated lifting of
superstructure. No serious
tilting.

Reinstatement: Foundations suspect but
capable of patch repair on
perimeter. Reinstall roof,
with mainly cosmetic repairs
elsewhere.

Lessons/ Questions: Lack of ductile detailing at
between pile and pile cap
causes very local buckling of
reinforcement at top of pile.
Clear evidence that piles
were required to resist
uplift, even though building
was not severely damaged.
What is the influence of
subsoil strata and foundation
type on building response?

FIGURE 42: HOTEL CONTINENTAL (FORMERLY HILTON) MEXICO CITY. COLLAPSE OF 15
STOREY BLOCK ABOVE SIXTH LEVEL DUE TO IMPACT WITH ADJOINING
BUILDING.

Comments/
Conclusions: Building is not seriously damaged visibly but foundation integrity requires checking. Access would be expensive and difficult, further reinforcing the philosophy of keeping damage out of foundations when they are not accessible.

CASE STUDY NO.5

Building: Security & Social Services Building, Mexico City

Principle Features: Superstructure: 15 storey r.c. symmetrical with curtainwall glass front. Infill r.c./masonry infill walls at each end. Modern construction.

Foundations: Piles (?). Second firm layer at 38m depth.

Earthquake Performance: Generally very good with little if any permanent displacement. Evidence of movement of ground floor relative to pavement due to a combination of rocking and movement of ground. 10% loss of curtain wall glass, almost certainly due to impact with adjoining similar (but older) building. Clear evidence of impact of this older building (15 storeys) with its 5 storey neighbour (fig. 43).

Glazing and mullions at first level lost and buckled, due to downward movement of the curtain wall system relative to the pavement. Structural damage unknown but foundation may have broken its back.

Reinstatement: Restore cladding, repair masonry infill panels and any structural damage. Work was proceeding in October 1985.

Lessons/
Questions: Curtain wall glazing was lost either due to lack of separation or more probably due to impact forces. Glazing framing should be seismically separated from the ground.

Comments/
Conclusions: Building was not seriously damaged. Impact caused most of the damage, but relative movement of the building during rocking and the ground caused some.

CASE STUDY NO.6

Building: Office Building, No.57 Varsovia, Mexico City

Principle Features: Superstructure: 10-storey private building approx.

FIGURE 43: SECURITY AND SOCIAL SERVICES BUILDING, MEXICO CITY. VIEW OF 15 STOREY BUILDING SHOWING LOSS OF GLASS FROM CURTAIN WALL. NOTE THE IMPACT DAMAGE RESULTING FROM POUNDING WITH THE ADJOINING 5 STOREY BUILDING.

18mx12m in plan. Reinforced concrete shear walls in sides and at rear. Fully glazed front with balcony over street.

Foundations: Piles (?). Second firm layer at 34m depth.

Earthquake Performance: No evidence of structural damage. Glazing fully intact. Loss of marble facade at entrance. Cosmetic damage to partitions in upper floors. A good performer.

Reinstatement: Cosmetic only. Almost completed by October 1985.

Lessons/
Questions: The good behaviour of the r.c. walls begs the question - why not use them more often in place of infill masonry? Did these walls stiffen the structure and prevent resonant response?

Comments/
Conclusions: This building should be held up as an example of good conceptual design for Mexico City conditions. The owners would be pleased that they

spent the additional money on reinforced concrete walls rather than masonry infill panels.

Masonry infill buildings of this height did not generally perform well. It may be that they 'softened' with initial response and were then more in tune with the 2 second period of ground motion.

CASE STUDY NO. 7

Building: Office Building, No 43 Av. Sinaloa, Mexico City, constructed 1982,

Principal Features: Superstructure: 16 storey r.c. frame and shear wall building, 24 x 30m in plan. Shear walls at sides full depth, full height. Frame and shear walls around lift core in transverse direction. Carpark extends beyond rear face of building below level 4 and has a masonry infill wall stabilising it at the rear (fig. 44). Building is seismically separated from its neighbours. Curtain wall glazing at front. Spandrel and flush glazing at rear. Modern construction, much more solid than usual. Concrete members are of NZ proportions. The lower four levels are for carparking and include double storey heights at the front.

Foundations: Piles (?). Second firm layer at 38m depth.

Earthquake Performance: Excellent. Some minor structural and non-structural damage. No displacement visible. Interior shows minor signs of movement at partition junctions. Suspended ceilings intact. Computers on wheels undamaged. Two bookshelves on 7th floor (2mx400) toppled.

Reinstatement: Minor cosmetic work only required. No structural implications.

Lessons/Questions: Did the building experience strong ground motion? Reinforced concrete shear walls perform well in Mexico City.

Comments/Conclusions: A good performer, of higher than usual design and construction quality. Long period motion is kind to these buildings.

FIGURE 44: OFFICE BUILDING NO. 43, AV SINALOA, MEXICO CITY, PLAN AND SECTION.

CASE STUDY NO. 8

Building: Apartment Building, Cnr Salamanca and Cozumel, Mexico City

Principal Features: Superstructure: 11 storey reinforced (?) concrete frame with infill masonry. Intermittent glazing and projecting balconies on street frontages. 'L' shaped in plan (fig. 45).

Foundations: Piles (?). Second firm layer at 34m depth.

Earthquake Performance: Satisfactory at first sight. However, collapse of a substantial part was imminent and saved by steel grilles at ground floor level. The corner column at ground level disintegrated completely, revealing woefully inadequate reinforcement (fig. 46). The 'settlement' of the upper floors at this position caused cracking in the spandrels which also helped to prevent collapse.

NOTE:
All dimensions are
approximate

FIGURE 45: APARTMENT BUILDING CNR SALAMANCA AND COZUMEL, MEXICO CITY. PLAN OF GROUND FLOORS AND TYPICAL FLOOR.

Some minor spalling of infill masonry and masonry facings evident but generally little other damage.

Reinstatement: Problematical. Collapsed column suggests other inadequacies in design and/or construction. A possible candidate for demolition depending on other structural damage, cost of repairs, ingenuity of owners and code requirements.

Lessons/Questions: Is lack of reinforcement due to bad design or construction, or both? Eccentricity contributed to concentration of energy at this column.

Comments/Conclusions: A good example of a building which looks to have survived well until closer inspection is made. There could be numerous cases in New Zealand following a major earthquake which would pose difficulties

FIGURE 46: APARTMENT BUILDING CNR SALAMANCA AND COZUMEL, MEXICO CITY. SEVERE DAMAGE TO CORNER COLUMN DUE TO ECCENTRICITY. COMPLETE COLLAPSE PREVENTED BY A SUBSTANTIAL STEEL GRILLE AND STIFFER STRUCTURE ABOVE THE BOTTOM STOREY. NOTE THE WIDE SPACING OF COLUMN TIES AND INADEQUATE DETAILING.

in quickly assessing the need for demolition or strengthening.

CASE STUDY NO. 9

Building: Apartment Building, No. 43 Amsterdam, Mexico City

Principal Features: Superstructure: 11 storey r.c. shear wall and frame. Shear walls at sides, spandrel/glazing front and rear. 'U' shape in plan. Brick infill spandrel walls at rear are discontinued at ground level (see fig. 47).

Foundations: Piles (?). Second firm layer at 34m depth.

Earthquake Performance: Looks undamaged from street. R.c. shear walls again served the building well. Framing perpendicular to the shear walls was not robust and considerable damage to

FIGURE 47: APARTMENT BUILDING NO. 43 AMSTERDAM, MEXICO CITY. TYPICAL FLOOR PLAN.

FIGURE 48: APARTMENT BUILDING NO. 43 AMSTERDAM, MEXICO CITY. REPAIRS UNDERWAY TO GRAVITY COLUMNS. CONTINUITY THROUGH FLOORS AND CONFINEMENT PROVIDED BY STIRRUPS IS SUSPECT.

FIGURE 49: APARTMENT BUILDING NO. 43 AMSTERDAM, MEXICO CITY. REPAIRS UNDERWAY WITH UNREINFORCED MASONRY ELEMENTS BEING REPLACED WITH 100MM OF REINFORCED CONCRETE. CONTINUITY OF WALLS THROUGH THE FLOORS IS SUSPECT.

spandrels, interior finishes, lift shaft resulted. Gravity columns were sorely tested and these were being strengthened at October 1985 (fig. 48).

Reinstatement: Extensive repairs to finishes, lift shaft and spandrels were being carried out. In addition, unreinforced masonry 'gravity' elements were being replaced with 100mm thick r.c. walls whenever this was possible. (Continuity of these walls through the floors was suspect (fig. 49)). Lengthy evacuation and costly repairs.

Lessons/ Questions: Again, r.c. shear walls proved their worth, but the lack of a reliable system in the perpendicular direction resulted in damage. Compare with 57 Varsovia (Case study 6) which had r.c. shear walls in both directions.

Comments/ Conclusions: This building is a high quality apartment block. Unfortunately the conceptual design in the transverse direction, and for gravity did not match the wise choice of r.c. shear walls for front-to-back stability.

CASE STUDY NO. 10

Building: Head Office Mexican National Railways, Cnr Insurgentes and Alvarado, Mexico City

Principal Features: Superstructure: Large 15 storey r.c. (?) frame building, 'Y' shape in plan (fig. 50). Stone faced spandrels on long faces, stone faced infill masonry walls on the three end walls. 3-level podium connecting the extremities - open at ground level - solid infill wall faced with stone for levels 2 and 3. Building appeared to have no seismic breaks.

Foundations: Piles (?). Second firm layer at 34m depth.

Earthquake Performance: Very good. Not inspected internally but showed few signs of damage. Stone facing had fallen off all three end walls and at the corners of the podium. No apparent glazing damage or permanent distortion. Building in use at October 1985 (fig. 51).

Reinstatement: Repairs to stone faced walls. Relatively inexpensive.

Lessons/ Questions: How did this building survive so well with its unusual plan shape?

Comments/ Conclusions: The good performance of this building would be worth closer study. Lack of response measurements would make conclusions difficult. Perhaps the ground at this site did not experience the same accelerations as elsewhere, but it is well inside the area of major damage.

CASE STUDY NO. 11

Building: Head Office, STC Metro, Av Delicias, Mexico City

Principal Features: Superstructure: 9 storey in-situ reinforced concrete. R.c. frame, infill shear walls at ends, r.c. frame in long direction. In-frame glazing. Notable for its reliance on frame action rather than a mixture of frame and infill (in long direction) (fig. 52).

Foundations: Piles (?). Second firm layer at 46m depth.

Earthquake Performance: No significant permanent displacement, or evidence of movement relative to ground. Every pane of glass shattered (fig. 53). Damage to reinforced infill in shear wall, opening up a gap between it and surrounding frame. Beam hinges at underside of all beams, but worst on floors 3, 4, and 5. Bars buckled and concrete spalled. Stair walls damaged due to lack of separation. Lift shaft enclosure destroyed beyond recognition for same reasons. No damage reported to ceilings or services. Empty at October 1985.

Reinstatement: Problematical, even though from the street, the building seems to require reglazing only. Shear walls require extensive repair. Lift shaft needs rebuilding. Repairs to beams difficult to accomplish effectively since bars were badly buckled.

Lessons/ Questions: Separate non-structural elements, particularly lift shafts and glazing. Use effective stirrups (135 degree hooks) at closer spacing to give greater ductility to beam hinges. Although not sufficiently ductile to satisfy codes such as the New Zealand code, the beams yielded before the

FIGURE 50: HEAD OFFICE MEXICAN NATIONAL RAILWAYS CNR INSURGENTES AND ALVARADO, MEXICO CITY. TYPICAL FLOOR PLAN.

FIGURE 51: HEAD OFFICE MEXICAN NATIONAL RAILWAYS, MEXICO CITY. VIEW OF BUILDING FROM STREET.

columns. R.c. frames are not the best form of construction for medium rise buildings in Mexico City?

Comments/
Conclusions:

This is a prestige office building, with good quality appearance and finish. However it was rendered unserviceable by the earthquake. Structurally it was outwardly similar to many NZ buildings. Lack of separation of its non-structural elements and lack of proper ductile detailing

accounted for almost all the damage. With some attention to such details it would have performed much better and survived virtually unscathed.

In many ways, a classic illustration of why NZ codes call for separation of non-structural elements. Also a pointer to the merits of shear walls for Mexico City buildings.

Worthy of analysis in detail.

FIGURE 52: HEAD OFFICE STC METRO AV DELICIAS, MEXICO CITY. PLAN, ELEVATION AND DETAILS.

FIGURE 53: HEAD OFFICE STC METRO AV DELICIAS, MEXICO CITY. VIEW OF FRONT FACADE OF BUILDING NOTE THE COMPLETE LOSS OF GLASS.

FIGURE 54: OFFICE BUILDING 102 SALAMANCA, MEXICO CITY. PLAN AND DETAILS.

FIGURE 55: OFFICE BUILDING 102 SALAMANCA, MEXICO CITY. VIEW OF THE BUILDING TAKEN FROM THE STREET, VIRTUALLY NO EVIDENCE OF DAMAGE.

CASE STUDY NO. 12

Building: Office Building
102 Salamanca (Near corner
Salamanca/Oaxaca), Mexico
City

Principal Features: Superstructure: 12 storey
steel frame, rectangular
plan with one basement level.
Plan 36m by 12m (fig. 54).
Steel trusses at 4m centres
support solid concrete
floor. These provide some
transverse framing action but
there are infill masonry
walls at each end, faced with
travertine.

Longitudinal stability is
provided by steel truss beams
spanning between the columns
at 4m centres. Curtain wall
glazing, neatly detailed on
both long faces for full
height. Structure by
Rosenbleuth (?). Constructed
around 1950-1960.

Earthquake Performance: Foundations: Piles (?).
Exceptionally good.
Virtually no evidence of
damage (fig. 55). Small
signs of impact with
adjoining 2 storey building.
Nearby buildings of lesser
quality and older
construction suffered
extensive damage and/or
collapse.

Reinstatement: None required.

Lessons/Questions: Did this building respond to
ground motions? Very few
signs of significant inter-
storey drifts or incipient
buckling of steel truss
beams. (Compare Pino Suarez
Towers). Clearly a well-
designed building even though
the ductility of the truss
beams is questionable.

Comments/Conclusions: A pity that no instruments
were in this building to
measure response. It would
lend itself well to analysis
because of its simple,
symmetrical form.

CASE STUDY NO. 13

Building: Office Building for Ministry
of Foreign Affairs
Cnr. Flores Magnon and Lazaro
Cardenas, Mexico City

Principal Features: Superstructure: 20 storey
r.c. and steel frame
designed by E Rosenblueth in
1964. Coupled r.c. infill
frame shear walls at ends
plus shear walls adjacent to
lift shaft provide lateral
stability. Longitudinal
framing is groups of three
close spaced r.c. columns at

7.2m centres with r.c. beams.
Plan area 40m x 18m (fig.
56). Floor slab solid r.c.
100mm with 30mm non-
structural topping supported
on steel trusses spanning 18m
and at 1.8m centres. Trusses
approximately 750 deep with
100x100x12 double angle
chords. Shear walls are
faced in travertine. R.c.
columns on long walls are
wide with beams set back to
give sheer glazing in
between.

Foundations: R.c. piles to
first firm layer (25m).
Second firm layer is at 40m
depth. (Some settlement has
caused tilting of the
building.)

Earthquake Performance: Generally excellent. This
building is adjacent to the
heavily damaged Tlatlolo
area and to all outward
appearances is not damaged
(fig. 57). However, it was
vacant for two reasons: (a)
repairs to shear walls,
ceilings, partitions, stair
walls, lifts as a result of
the earthquake, (b) it is
proposed to attempt levelling
of the building and underpin
it to prevent further
settlement. Earthquake damage
is minor. Some marble
facings came off at corners
of stair walls, some ceiling
tiles fell, portraits swayed
markedly and occupants of the
20th floor could not stand
during the earthquake.

Earthquake Performance: The heaviest damage was at
the 5th floor where repairs
to the shear wall columns
were underway. Repairs
consisted of the placement of
steel angles at the corners
which penetrate the floor
slab. These angles are laced
together by steel flats
placed hot to provide a
shrink/confining effect. The
original columns have 3 No.
25mm bars in each corner with
12mm stirrups at 200mm
centres. Plaster tell-tales
have been placed on columns
to detect further movement.

Bookshelves on the 18th floor
toppled in the long
direction.

Mechanical equipment in the
basement was undamaged.

Personnel and files were
relocated in the basement,
and an adjacent church and
kindergarten.

Reinstatement: Minor due to earthquake as
described above. Correction
of consolidation settlement

FIGURE 56: OFFICE BUILDING MINISTRY OF FOREIGN AFFAIRS CNR FLORES MAGNON AND LAZARO CARDENAS, MEXICO CITY. TYPICAL FLOOR PLAN AND DETAILS OF REPAIR TO DAMAGED SHEAR WALL.

FIGURE 57: OFFICE BUILDING MINISTRY OF FOREIGN AFFAIRS MEXICO CITY. VIEW OF THE BUILDING TAKEN FROM THE STREET SHOWING LITTLE OR NO EVIDENCE OF EARTHQUAKE DAMAGE.

in addition to earthquake repairs are said to cost NZ\$500,000.

**Lessons/
Questions:**

Well designed buildings of integrity have a good chance of survival with minimal damage. Shear walls behave well, as do stiff longitudinal frames.

Why is damage greatest at the 5th floor? Instrumentation of major buildings such as this would help improve our understanding of the response and behaviour of the building frames and their components.

Conclusions: Mexico City has very difficult foundation conditions which make it hard to guarantee a lack of settlement problems.

The interaction of the subsoils, the long piles and the structure warrant close study.

CASE STUDY NO. 14

Buildings: Pino Suarez Tower Complex Av. Pino Suarez (At Pino Suarez Metro Station), Mexico City

Principal Features: Superstructure: A complex of 5 steelframed towers with 3-storey podium detailed above Pino Suarez Metro Station. Storey heights were 14,22,20,22 and 14 arranged symmetrically, and along the line of the Metro. Each tower is 4 bays by 2 giving a plan area 22m x 12m approximately. All columns are box type with trusses

FIGURE 58: PINO SUAREZ TOWER COMPLEX, AV PINO SUAREZ, MEXICO CITY. TYPICAL FLOOR PLAN AND ELEVATION OF 22 STOREY TOWER.

FIGURE 59: PINO SUAREZ TOWER COMPLEX, MEXICO CITY. VIEW SHOWING DEBRIS OF COLLAPSED 22 STOREY AND 14 STOREY BUILDING.

FIGURE 60: PINO SUAREZ TOWER COMPLEX, MEXICO CITY. VIEW OF THE REMAINING 14 STOREY AND 22 STOREY TOWERS AT THE NORTHERN END OF THE COMPLEX OF FIVE BUILDINGS.

supporting in-situ r.c. floor and providing lateral framing in the short direction. One bay of 'K' bracing in long direction (fig. 58).

In-situ (?) concrete cladding and spandrels with in-frame windows.

Foundations: Heavy shear walls around Metro. Piles under (Second firm layer at 49m depth).

Earthquake Performance: Collapse of one 22 storey tower above podium (fig. 59). Near collapse of its neighbour. Total collapse of the southernmost 14 storey tower due to collapse of 22 storey tower on top of it. Remaining 14 storey and 22 storey towers relatively undamaged to outward appearances. (Not inspected) (fig. 60).

Southernmost remaining tower tilted southward about 1m permanently over 50m height (fig. 61) with incipient buckling failure of corner column temporarily repaired (fig. 62). Fracture of concrete cladding, external glazing, partitions and glazing, stairwell and ceilings.

Fortunate that few people were in this building complex at the time of the earthquake.

"Rock-fibre" ceiling tiles in concealed light steel one way rail, unbraced. Light

FIGURE 61: PINO SUAREZ TOWER COMPLEX, MEXICO CITY. NEAR COLLAPSE OF CENTRE 22 STOREY BUILDING WITH 1M TILT SOUTHWARD DUE TO INCIPIENT BUCKLING FAILURE OF CORNER COLUMN.

FIGURE 62: PINO SUAREZ TOWER COMPLEX, MEXICO CITY. INCIPIENT BUCKLING FAILURE OF CORNER COLUMN OF CENTRE 22 STOREY BUILDING JUST ABOVE BUTT WELD. THE BEAMS FRAME INTO THE COLUMN JUST BELOW THE BUTTWELD. NOTE THE TEMPORARY STRUT TO STRENGTHEN THE COLUMN TO THE RIGHT OF THE PHOTOGRAPH.

FIGURE 63: PINO SUAREZ TOWER COMPLEX, MEXICO CITY. VIEW LOOKING DOWN INSIDE A BOX COLUMN SECTION RECOVERED FROM THE COLLAPSED 22 STOREY TOWER SHOWING LACK OF FUSION OF THE FILLET WELD TO THE DIAPHRAGM PLATE. THE BACKING BARS TO THE COLUMN BUTT WELD CAN BE SEEN IMMEDIATELY ABOVE THE FILLET WELD.

fittings separately hung. Negligible damage where no collapse of floor.

No apparent damage to stairs.

Reinstatement: Demolition of southernmost 22 storey block, evaluation of remaining 14 and 22 storey blocks prior to repair and re-occupation. Salvage of furniture was a major exercise, and removal of bodies was continuing as the debris of the collapsed towers was cleared.

Lesson/
Questions: What effect did stiff foundations/podium have? We observed local buckling at columns and weld failure (lacked penetration) at third storey level of collapsed 22 storey tower, two storeys above podium (column 450 x 600 with 15 mm plate). There was severe hinging in the truss beams from the bracing action at infill panels. High flange width to plate thickness ratio (i.e. insufficient wall thickness) in box columns: 600mm x 500mm column had 12mm plate thickness (fig. 63).

Comments/
Conclusions: A number of buildings of this general type performed very well in the earthquake. Contributing factors to the collapse of Pino Suarez Towers were certainly the length and strength of the earthquake, behaviour of the podium/foundation and the lack of weak (ductile) beam - strong column concept.

Research into this collapse would be most illuminating and throw light on foundation and superstructure response. A pity that no instruments were included in the towers to record response of the various blocks.

CASE STUDY NO. 15

Building: Apartment Building
No. 1 Piazza Villa de la Madrid, Mexico City

Principal Features: Superstructure: Above average, modern 18 storey r.c. frame building with some infill walls. Reasonably regular plan except for 7 storey car park annex coupled to tower. This incorporated automobile lifts. R.c. floors (slab and beam). Curtain wall glazing to three sides with infill masonry predominantly on one face (fig. 64). Suspended ceilings on 600 mm grid.

Foundations: Piles. 38m to second firm layer.

Earthquake Performance: Very little damage to all outward appearances (fig. 65). However, building swayed and hit its neighbour causing shear cracks in columns fanning out from point of impact. Minimal damage to glazing, light-weight partitions and ceilings. However, brick partitions around stairwell were demolished and all stairs slipped partly off their seatings requiring expensive repair.

Tentative evidence of torsional response of the building was shown in the column cracking, although inter-storey drifts were not large.

Serious structural implications in the shear cracks in columns especially in view of very light and ineffective confining steel present.

Impact caused by reduction of seismic gap due to tilting of building next door under consolidation settlement.

Reinstatement: A feature of interest, reinstatement was proceeding apace at time of inspection. Building evacuated and partitions, ceilings and floor coverings removed. Damaged columns were being strengthened with additional corner bars, stirrups at 100mm centres and concrete,

all placed around the original cross section. Continuity of main bars was achieved through penetrations in the floor at each level. Concrete was site mixed on the floor of the apartments. Some columns repaired by shrink fitting of stirrups and angles placed in grooves in the original cover concrete.

Repairs cost US\$1,000,000 and were clearly being done as a matter of urgency to reopen these prestige apartments.

Lessons/ Questions:

Without impact, this building would almost certainly have survived well in a zone of great damage to adjacent buildings. Damage to columns showed up the weaknesses in these elements, even though their proportions were generous.

Many Wellington buildings would have a similar appearance to this building after a major earthquake. i.e. outwardly not seriously damaged, but requiring extensive structural reinstatement, in turn requiring evacuation and demolition of finished elements - all costly. Interstorey drifts were large enough to damage stair walls and stair seatings, but curtain wall glazing fared well, suggesting that even "non-seismic" detailing has some capacity to tolerate displacements without glazing damage.

What was the response of this building? How would a building detailed to New Zealand codes have fared?

Comments/ Conclusions:

This building did not have the qualities it would need if designed to the New Zealand code but performed well considering the length and strength of the ground motion.

Impact of one building and its neighbour is an important thing to avoid. Do we in New Zealand pay enough attention to it? Or is the gap apparently too expensive a piece of real estate?

CASE STUDY NO. 16

Building: Secretariat of Transport and Communications (SCT) Main office building on Lazaro Cardenas Street, Mexico City

FIGURE 64: APARTMENT BUILDING NUMBER 1
PIAZZA DE LA MADRID MEXICO CITY.
PLAN OF AND SECTION THROUGH
BUILDING.

Principal Features: 17 storey H shaped plan office building, with penthouse for communications equipment and plant (fig. 66).

Earthquake Performance: Relatively little damage and apparently not structurally damaged (fig. 67). Configuration suggests shear walls, but this not positively established as drawings were not available.

Stone claddings applied to end walls of each wing were undamaged.

Apparently no damage to glazing or aluminium curtain wall with individual glass

FIGURE 65: APARTMENT BUILDING NUMBER 1
PIAZZA VILLA DE LA MADRID MEXICO
CITY. VIEW OF THE APARTMENT
BUILDING FROM THE STREET. NOTE
THE ADJOINING BUILDING WHERE
POUNDING OCCURED.

pane sizes approximately 900 x 900.

A number of the 1200 x 900 ceiling panels had apparently fallen out or been damaged with no provision for movement at columns.

Stairs were damaged at every floor level with cracking between the stair intersection and the adjoining lift shaft.

FIGURE 66: SCT MAIN OFFICE BUILDING LAZARO CARDENAS, MEXICO CITY. PLAN AND ELEVATION OF TOWER.

Local air extract units were located above toilets at each floor level. This plant was operating and appeared undamaged. At the penthouse level a small air handling unit, cast iron plumbing risers and the main electrical distribution room appeared undamaged. The air handling unit was not bolted to the floor and not resiliently mounted but did not appear to have moved despite damage to adjacent brick walls.

Temporary communications equipment on the roof indicated that some problems

FIGURE 67: SCT MAIN OFFICE BUILDING LAZARO CARDENAS, MEXICO CITY. VIEW OF 17 STOREY BUILDING FROM STREET.

had occurred. However a tall unstabilised electrical cabinet had apparently remained upright.

No observed damage to light-weight partitions.

Fully glazed partitions between entrance lobby and first floor offices on one wing. Approximately 100 x 50 aluminium box section joints. Both glass 75 x 12 fin stiffened silicone glued intermediate mullions. Several glass fractures mainly at head of glazing indicating insufficient movement provision.

Damage to tiled brick partitions especially at junctions with stair flights where no provision for movement.

Reinstatement: Minor

Comments/ Conclusions: A good performer generally, especially when contrasted with collapsed buildings on the adjacent site, and known ground motions at SCT station.

FIGURE 68: PRISMA BUILDING CNR PASEO DE LA REFORMA AND AVE JUAREZ, MEXICO CITY. PLAN AND ELEVATION OF BUILDING AND DETAILS OF COLUMN AND CLADDING.

FIGURE 69: PRISMA BUILDING CNR PASEO DE LA REFORMA AND AVE JUAREZ, MEXICO CITY. VIEW OF BUILDING FROM PASEO DE LA REFORMA. NO DAMAGE APPARENT.

CASE STUDY NO. 17

Building: Prisma Building (Lottery Building),
Cnr Paseo De La Reforma and Juarez, Mexico City

Principal Features: 26 storey, steel perimeter frame, triangular plan with cantilever portion at triangle points (fig. 68). Basement 10m deep with piles extending approximately 20m below that. Compensated foundation. Perimeter columns fabricated from steel plate.

Earthquake Performance: Performed well with no damage apparent (fig. 69). One lift was not working (earthquake related?) and lift ride suggested some misalignment in lift rails. Building occupants reported that response during earthquake was frightening. Pavements damaged around building with indications of settlement relative to building basement.

Reinstatement: None required.

Lessons/Questions: Apparently well conceived symmetrical building which performed extremely well. Flexibility of structure may have placed it outside the predominant resonance characteristics of the deep soft soils at this particular site.

CASE STUDY NO. 18

Building: Playa Azul Hotel, Playa Azul Pacific Coast (Epicentral region), Mexico

Principal Features: 4 wings - 3 stories high (fig. 70). Generally moment resisting frames although infill brick walls positioned non uniformly throughout structures without separation. One wing had been strengthened after a previous earthquake.

Earthquake Performance: 3 of the wings performed extremely poorly. Columns proved inadequate to carry loads once concrete was lost from the hinge regions. Stiff spandrel beams created a soft storey requiring all energy dissipation to occur in non ductile columns (fig. 71).

Shear failures occurred in many of the ground floor shear walls.

The wing which had been strengthened (by casting new concrete on the outside of

the ground floor columns thus increasing their size) suffered only minor structural damage and some distortion (fig. 71)

Reinstatement: At least three of the wings will require demolition.

Lessons/Questions: Column hinge mechanisms should be avoided unless such hinges can be shown to have sufficient ductility (fig. 72)

CASE STUDY NO. 19

Building: Hotel Dorado Pacifico, Ixtapa - Pacific Coast, Epicentral Region

Principal Features: 12 storey reinforced concrete frame atrium building with some structural walls. Infill masonry partitions. Unusual shape (fig. 73).

Earthquake Performance: No apparent damage to frame (fig.s 74 & 75).

Infill masonry walls badly damaged even though most isolated from the frame using several layers of fibre board.

Stairs not isolated between consecutive floors, cracking observed at stair/landing junction.

Diagonal shear cracking of cantilever beams in stair wells.

Reinstatement: Considerable rebuilding, redecorating required due to failure of infill walls. Estimated repair costs reported to be approximately US\$1 million

Lessons/Questions: Notwithstanding unusual shape the structure performed well. Fibreboard isolation between infill walls and frame proved ineffective.

CASE STUDY NO. 20

Building: Cardiology Building, Central Medical Hospital, Mexico City

Principal Features: 9 storey reinforced concrete frame structure, two bays wide by 5 long separated by a seismic gap (approx 150mm) (fig. 76). Circular spirally reinforced columns. Built early 1960's?

Foundations: Not known

Earthquake Performance: Still standing but the 150mm seismic separation now nearly 1.0 m at the top (fig. 77). Most of deflection in 3rd

FIGURE 70: PLAYA AZUL HOTEL, PLAYA AZUL, PACIFIC COAST. VIEW OF A TYPICAL WING.

FIGURE 71: PLAYA AZUL HOTEL, PACIFIC COAST. VIEW OF STOREY COLLAPSE AND FAILED COLUMNS. THE FIRST FLOOR OF THE WING ON THE RIGHT CORRESPONDS WITH THE GROUND FLOOR OF THE WING ON THE LEFT. NOTE THE GOOD PERFORMANCE OF THE STRENGTHENED COLUMNS DAMAGED DURING A PREVIOUS EARTHQUAKE.

FIGURE 72: PLAYA AZUL HOTEL, PACIFIC COAST. FAILURE OF COLUMN AT A PLASTIC HINGE DUE TO INADEQUATE CAPACITY OF COLUMN ONCE HINGES HAD FORMED. NOTE WIDE SPACING OF COLUMN TIES INADEQUATE CONFINEMENT AND POOR WORKMANSHIP.

FIGURE 73: HOTEL DORADO PACIFICO IXTAPA, PACIFIC COAST. TYPICAL LOWER FLOOR PLAN.

FIGURE 74: HOTEL DORADO PACIFICA IXTAPA, PACIFIC COAST. GENERAL VIEW OF THE HOTEL FROM THE BEACH. NO APPARENT EARTHQUAKE DAMAGE.

FIGURE 75: HOTEL DORADO PACIFICO IXTAPA, PACIFIC COAST. INSIDE OF ATRIUM LOOKING TOWARDS THE LIFT INSTALLATION.

FIGURE 76: CARDIOLOGY BUILDING CENTRAL MEDICAL HOSPITAL, MEXICO CITY. GENERAL VIEW OF BUILDING SHOWING DAMAGE TO THE END WALL HOUSING THE STAIRS. NOTE THE BULGE IN THE END WALL DUE TO PERMANENT DEFORMATION OF THE FRAME TOWARDS THE VIEWER.

FIGURE 77: CARDIOLOGY BUILDING CENTRAL MEDICAL HOSPITAL, MEXICO CITY. VIEW OF SEISMIC SEPARATION SHOWING PERMANENT DEFORMATIONS IN THE FRAMES IN A LONGITUDINAL DIRECTION. GAP HAD MOVED FROM 150MM TO ABOUT 1000MM.

FIGURE 78: CARDIOLOGY BUILDING CENTRAL MEDICAL HOSPITAL, MEXICO CITY. HINGING IN INTERIOR LONGITUDINAL BEAMS SHOWING INADEQUATE CONFINEMENT OF MAIN BARS.

FIGURE 79: CARDIOLOGY BUILDING CENTRAL MEDICAL HOSPITAL, MEXICO CITY. VIEW OF STAIR WELL SHOWING SHATTERED MASONRY WALLS AND FRACTURED STAIRS LITTERED WITH DEBRIS. EVEN SO PATIENTS WERE EVACUATED BY THE STAIRWAYS.

FIGURE 80: CARDIOLOGY BUILDING CENTRAL MEDICAL HOSPITAL, MEXICO CITY. INTERIOR VIEW SHOWING OFFICES LITTERED WITH FALLEN CEILING TILES, OVERTURNED FURNITURE AND THE CONTENTS OF DRAWERS AND FILING CABINETS.

storey where beam hinges show shear failures and buckling and fracture of flexural steel (fig. 78). The stirrup spacing was 300mm. Infill masonry walls shattered and stairs broken and littered with debris (fig. 79). Offices littered with fallen ceiling tiles, overturned furniture and contents of spilled drawers and filing cabinets (fig. 80). Light weight cladding basically, intact except for glazing. Glazing in opening sashes fared better.

Lesson/
Questions:

Confinement provided by spiral reinforcement in circular columns designed to the static load provisions of ACI 318 ensures good seismic

Comments:

performance of the column (fig. 81). This has been noted in previous earthquakes (eg. Olive View Hospital, San Francisco). The use of circular columns undoubtedly prevented the complete collapse of the building.

The loss of shear resistance and lack of confinement in beam hinges softens critical stories concentrating damage which may lead to a soft storey collapse. Where not detailed for separation glazing performs better in opening sashes.

It was amazing that no lives were lost and that patients were evacuated successfully down the debris strewn

FIGURE 81: CARDIOLOGY BUILDING CENTRAL MEDICAL HOSPITAL, MEXICO CITY. VIEW OF SEISMIC SEPARATION SHOWING PERMANENT DEFORMATIONS IN THE FRAMES IN A LONGITUDINAL DIRECTION. GAP HAD MOVED FROM 150MM TO ABOUT 1000MM.

FIGURE 82: CARDIOLOGY BUILDING CENTRAL MEDICAL HOSPITAL, MEXICO CITY. FREE STANDING LIQUID OXYGEN TANK WHICH SHOWED NO SIGN OF MOVEMENT. IT WAS NOT FIXED TO THE GROUND FLOOR SLAB.

stairs. An adjacent free standing vertical liquid oxygen tank showed no sign of movement it was not fixed to the ground floor slab (fig. 82).

CASE STUDY NO. 21

Building: 90 Durango, Offices of the Archbishop of Mexico, Cnr Durango and Cordoba, Mexico City

Principal Features: Superstructure: 11 Storey RC frame with in-situ concrete infill panels and exterior frames in one direction, deep precast spandrels in the other direction not separated from the structure, significant change in structural form with floor slab stepped back for three floors above the second floor. Connected to adjacent four storey building by bridges (fig. 83). Large windows stiffened with glass fins to setback storeys on Durango facade (fig. 84).

Foundations: Piles, basement for parking and building services.

Earthquake Performance: Deep spandrel panels interacted with the columns causing shear cracking in the reduced column lengths (fig. 85). The fixings to the precast spandrels failed in a number of cases allowing the spandrels to fall outwards. The spandrels were left hanging from the reinforcing connecting them to the slabs (fig. 86).

Glass was shed from the Durango facade apart from the three top floors (fig. 84).

Damage was evident at some beam column joints.

Damage was caused to the corners of columns where relatively thin reinforced concrete walls framed in without separations (fig. 87). 90° hooks in column ties opened up at column corners (fig. 88). Plastered brick partitions without separations from the frame were damaged.

Precast concrete stair treads spanning between walls failed due to differential movement of the supporting walls. Lightweight ceilings incorporating light fittings were little damaged.

Reinstatement: Possible but likely to be expensive.

**Lessons/
Questions:**

Separation of rigid elements, including glass, from the frame is essential otherwise serious damage can occur to both. Fixings for precast units need to be designed and also need to be ductile.

**Comment/
Conclusions:**

A modern building which suffered severe damage due to inadequate structural concept and poor detailing.

CASE STUDY NO. 22

Building: Medical Centre, Durango, Mexico City

Principal Features: Slender 12 storey reinforced concrete frame structure two bays wide and four bays long with deep spandrels (fig. 89).

The building suffered considerable damage in the 14 March 1979 earthquake in Mexico City. Structural damage was reported [30], as columns in several frames being badly cracked in the first three stories and beams being also cracked by shear and flexure. The solution adopted for repairing and strengthening the building was to repair the badly cracked columns with expansive mortar and strengthen them with external structural steel plates at the first levels (fig. 90); repair the cracks in the beams using epoxy resin; strengthen and stiffen the building in the short direction with a structural steel grid at each of the two external frames (fig. 91) with bolted connections to the spandrels (fig. 92); in addition the slab was reinforced to act as a diaphragm.

**Earthquake
Performance:**

Apparently good as building was in use after the earthquake. No damage or movement of the structure could be detected at joints between the original structure and the strengthening system.

**Lessons/
Questions:**

A relatively simple solution which is apparently architecturally acceptable.

The bolted connection between the original structure and the structural steel grid (fig. 92) appeared minimal but was adequate at least for the September 1985 earthquake.

FIGURE 83: 90 DURANGO CNR DURANGO AND CORDOBA STREET, MEXICO CITY. PLAN AND ELEVATION FROM CORDOBA.

FIGURE 84: 90 DURANGO CNR DURANGO AND CORDOBA, MEXICO CITY. VIEW OF DURANGO ELEVATION. NOTE THE STEPBACK FLOORS FROM SECOND FLOOR AND LOSS OF GLAZING APART FROM TOP THREE FLOORS (EXCEPT ONE PANEL).

Comments/ Properly designed repairs and
Conclusions: strengthening systems
together with good
workmanship can ensure good
earthquake performance even
in buildings severely damaged
in previous earthquakes.

FIGURE 85: 90 DURANGO CNR DURANGO AND CORDOBA, MEXICO CITY. DEEP SPANDREL PANELS NOT SEPARATED FROM COLUMNS CAUSED SHEAR CRACKING IN REDUCED COLUMN LENGTHS.

FIGURE 86: 90 DURANGO CNR DURANGO AND CORDOBA, MEXICO CITY. FAILURE OF THE CENTRAL FIXING NIBS TO THE PRECAST SPANDRELS ALLOWED THE SPANDRELS TO FALL OUTWARDS. THE SPANDRELS WERE LEFT HANGING FROM THE REINFORCING CONNECTING THE SPANDRELS TO THE FLOOR SLABS.

FIGURE 87: 90 DURANGO CNR DURANGO AND CORDOBA, MEXICO CITY. DAMAGE CAUSED TO CORNERS OF COLUMN DUE TO THIN WALLS BEING CAST IN WITHOUT SEPARATIONS. NOTE THE SHEAR CRACKING IN THE COLUMN DUE TO SHORTENED COLUMN LENGTH.

FIGURE 88: 90 DURANGO CNR DURANGO AND CORDOBA, MEXICO CITY. OPENING UP OF 90 DEGREES HOOKS IN COLUMN HINGE. THE TIE ABOVE HAS FRACTURED.

FIGURE 90: MEDICAL CENTRE DURANGO, MEXICO CITY. VIEW OF BUILDING FROM DURANGO.

FIGURE 89: MEDICAL CENTRE DURANGO, MEXICO CITY. PLAN AND ELEVATION FROM DURANGO.

FIGURE 91: MEDICAL CENTRE DURANGO, MEXICO CITY. STRUCTURAL STEEL PLATE STRENGTHENING TO COLUMN.

FIGURE 92: MEDICAL CENTRE DURANGO MEXICO CITY. BOLTED CONNECTION BETWEEN STRUCTURAL STEEL GRID AND SPANDREL AT FLOOR BOLTS.

10. NON-STRUCTURAL DAMAGE TO BUILDINGS IN MEXICO CITY INCLUDING BUILDING SERVICES

10.1 General

This section summarises the behaviour of non-structural elements and of building services.

Two general comments can be made:

- (a) Very few specific steps had been taken to limited damage to secondary elements by way of designed separation from the structure, or independent bracing.
- (b) The value of New Zealand requirements for separation was demonstrated in many examples where brittle secondary elements had not been separated and had suffered severe damage.

10.2 Ceilings

The observed variety of ceiling types was similar to the range seen in New Zealand, but there were few timber framed ceilings.

The most common suspended ceiling consisted of a lightweight metal grid with lightweight ceiling panels of rock-fibre or similar material. The wire hung rail system was sometimes exposed and sometimes concealed, but there were no examples of the use of effective bracing. As a rule, light fittings were independently wire hung to finish flush with the suspended ceiling.

Heavier ceilings were also used, examples being insitu plaster on mesh, plasterboard sheets and fibrous plaster tiles.

Ceilings had been damaged in several ways:

- a) By collapse of the adjoining or supporting structure.
- b) By damage to panels and/or rails that had been restrained by the adjacent elements.
- c) By panels falling out, although damage to the rail system was slight. This was common.

10.3 Partitions

Single skin brick partitions are most common, especially in older buildings. The brickwork is plastered with an applied decorative finish of paint or tiles. Stone facings are used over brick partitions in feature areas such as foyers.

Damage to these partitions was frequent, arising from lack of separation. Damage took the form of crushing of brickwork at junctions with the structure, diagonal shear cracks and cracking at weak sections, such as around doors and other openings. In more severe cases, brick panels had collapsed, but even where damage to the brickwork appeared slight finishes of plaster, tiles or stone had spalled off.

Demountable partitions are also used. These generally survived undamaged, except when there were structural failures.

Timber framed partitions are uncommon.

10.4 Stairs

Insitu concrete stairs are usual, but steel is also found. There was usually no positive separation at floor landings and damage had occurred in many cases as a result of this. Even where structural damage to stairs was slight, architectural finishes were damaged where movement had occurred between the stairs and adjacent elements.

10.5 Windows and Glazing

Damage to window systems, and particularly to glazing, was widespread in the severe damage zone. There were also many anomalous situations in which glazed curtain walls were undamaged or only slightly damaged, despite apparent deficiencies in the structural form of the building.

Some observations were:

- a) Seismic sub-frames for windows are not used.
- b) In many cases of both curtain wall systems and conventional windows, the tolerance in the rebate between sheets of glass and window frames had evidently been sufficient to accept interstorey drift.
- c) Glazed sashes were sometimes less damaged than fixed glazing.
- d) Steel strap connectors between curtain wall framing and the structure probably admitted some movement.
- e) Glass breakage was high in building facades which alternated glazing with stiffer spandrel panels.
- f) In the two observed examples of glass fin reinforced glazing there was some damage to the large sheets of glass but the fins were intact.
- g) No examples were seen of silicone rubber "stuck on" glazing.

10.6 Exterior Cladding

Apart from glazing, the dominant cladding was plastered brickwork, used as infill panels without any separation.

Other cladding examples were:

- a) Some slab and tile facings applied to brick or insitu concrete. Slabs were broken or detached where movement had occurred.
- b) Brittle insitu plaster on mesh panels in a steel framed structure. No separation was provided and damage was extensive.
- c) Lightly reinforced insitu and precast concrete elements, unseparated from the structural frame. One building containing such elements was severely damaged.

- d) Thin sheet metal cladding, in strip form, lightly attached to brick infill panels. Large areas of cladding were detached although brick walls appeared undamaged.

10.7 Building Services

Surprisingly few examples of failure of building services were reported or observed. There was negligible evidence of seismic restraint of items of building plant, pipework or ducting. Air handling and electrical equipment appeared to have been unaffected by the earthquake even though plant items were sometimes not bolted down. A large vertically mounted oxygen storage tank in the Central Medical Centre was undamaged although the building enclosure was shattered. The absence of significant damage probably results from the predominant long period characteristics of the earthquake.

Lifts had remained operative in some buildings even where there was obvious damage to the lift shaft enclosure, but apparently not to the guide rails.

Mexico City does not have a reticulated gas system; instead tankers supply gas to storage tanks commonly seen on building roofs. On the roof of one office building, an unfixed gas tank had rolled over and damage to gas systems was reported, but damage was evidently not extensive.

11. DAMAGE TO CIVIL ENGINEERING STRUCTURES

11.1 Mexico City

In general, civil engineering structures performed well in the earthquake. There were no reports of damage to steel towers, underpasses and retaining walls while only one bridge was slightly damaged. The bridge was made serviceable within a day or so of the earthquake. However, surface settlement of up to 300mm did occur in the fills behind some retaining walls.

The reinforced concrete telecommunications tower at the central telephone exchange suffered some damage to beams and beam-column joints (fig. 93) and to the covered walkways to adjacent buildings. The tower appears to be plumb and is still in use. It is not known whether the microwave antennae were damaged or needed to be realigned.

Two 40m tall reinforced concrete multi-cell grain silos, full of grain at the time of the earthquake, were severely damaged or collapsed. The walls of the damaged silos failed in compression and yielded in tension just above the hopper cones. They had a permanent deformation of about one metre. The other silo installation collapsed completely also above the hopper cone after pounding by an adjacent stiff grain cleaning house. There was a separation between the structure but they were connected together by a roof slab at the top. It was reported that the silos were designed to ACI 313 [31] with accelerations of 0.08g.

FIGURE 93: REINFORCED CONCRETE TELECOMMUNICATIONS TOWER AT THE CENTRAL TELEPHONE EXCHANGE, MEXICO CITY.

11.2 Epicentral Region

The most widespread earthquake damage was the settlement of the approach fills at bridge abutments. Some two weeks after the earthquake this had been repaired by the use of plant-mix. Apart from this, there was little sign of movement or damage to bridges in the region. Two notable exceptions were a bridge on Highway 200 near Tecpan and twin bridges over the Rio Balsas servicing the port of Lazaro Cardenas.

The bridge near Tecpan is reinforced concrete with five spans and is on a 24 degree skew. The spans are simply supported on elastomeric bearings and restrained laterally by reinforced concrete shear keys. The spans are not tied together. Under the action of longitudinal forces the spans moved in the direction of the skew and shattered the shear keys.

The twin bridges on the Rio Balsas are reinforced concrete simply supported spans. A number of the piers suffered major damage due to hinging and loss of confinement. The bridge showed evidence of longitudinal movement, and damage to the lugs restraining the spans transversely. In addition, the approach fills had settled and suffered tension cracking longitudinally. The damage was sufficient to close one bridge to vehicles while only

one unladen vehicle per time was permitted on the other (fig. 94).

The almost completed grain handling and storage facilities at the Port of Lazaro Cardenas were severely damaged. At the time of the earthquake the silos did not contain grain, however shear cracks were evident in the walls of one of the silos. The five storey structure above the silos partially collapsed and lost a complete storey due to column shear failure (fig. 95). In addition, the reinforced concrete conveyor support structure to the grain unloading facility on the adjacent wharf collapsed. Collapse appeared to start at the free end (fig. 96) due to inadequate anchorage of the column bars into the wharf deck. The twisting caused torsion cracking of the spine beam and failures of the beam at cut-outs in the slab. Failures of the cantilever columns in some cases involved complete disintegration due to inadequate column ties and the use of 90 degree hooks. The large structural steel mobile grain elevator on the wharf was restrained by angles welded to the rail but was undamaged (fig. 97).

At the container terminal the container storage pavement slabs settled 200-250 mm relative to the piled wharf structure (fig. 98). The construction joints in the concrete pavement opened and three large sand boils formed, one about 10 m across with some gravels up to 40 mm diameter mixed with the sand (fig. 99).

There are two hydro-electric stations on the Rio Balsas each with rock fill dams. At La Villita where accelerations of 0.45g were recorded on the crest of the dam, longitudinal tension cracks up to 25 mm occurred.

Similar cracking occurred in previous earthquakes and is due to differential settlement as the core is founded on bedrock and the shoulders of the dam on gravels. Further up the river some minor cracking occurred on the crest of the Infiernilla Dam. At the time of the earthquake the crest was being raised and the accelerograph was resited on an intermediate level berm. Peak ground accelerations of 0.34g were recorded.

No damage was reported either in the underground power station nor in the yards, but two of the six generators tripped out. These were quickly put back on line.

11.3 Case Studies

The following are reports on individual civil engineering structures studied by team members. Due to lack of information and in-depth studies, some of the comments, lessons and conclusions are inconclusive and must remain as questions. It is believed however that the information gathered is worthwhile and instructive in the form presented.

CASE STUDY NO. 1

Structure: Jose Maria Morelos Dam and La Villita Power Station near mouth of Rio Balsas.

Principal Features: The earth-filled dam is approximately 50m high by 380m long, founded on 70m thick alluvial deposits.

The high voltage switch yard is adjacent to the west end of the dam crest.

The power station is housed in a steel framed building below the crest on the west bank and consists of four 76 megawatt units, completed in 1973.

Earthquake Performance: Peak ground motion was approximately 0.15g on rock.

Mechanical and electrical equipment in the station were undamaged by the earthquake. Protective relays disconnected the station from the power grid during the earthquake.

The dam settled and displaced upstream as shown in fig. 100.

Longitudinal cracks up to 150mm wide in the road along the dam crest.

Some 0.5m high concrete walls along the down stream side of the crest toppled.

Reinstatement: Will require filling of cracks in road pavements and putting back toppled low walls.

Lessons/Questions: High voltage equipment can be designed to withstand earthquakes and protective relays can work.

Settlement and horizontal displacement of earth embankments is to be expected.

Comment/Conclusions: Settlement/displacement records and monitoring show effects of earthquakes since dam was built (fig. 100).

The El Infiernillo Dam and Power Station located about 30 km upstream also performed well. It was being made higher at the time of the earthquake and was not visited. The Dam is about 150m high by 380m long. The power station is located in the rock face of a hillside at the discharge of 1km long penstocks. The station includes four 160 Mw and two 180 Mw units and was

FIGURE 94: TWIN BRIDGES ON THE RIO BALSAS AT LAZARO CARDENAS. THE SEVERELY DAMAGED BRIDGE ON THE LEFT WAS RESTRICTED TO PEDESTRIAN TRAFFIC WHILE THE BRIDGE ON THE RIGHT WAS LIMITED TO UNLADEN VEHICLES.

FIGURE 95: GRAIN HANDLING FACILITIES AT THE PORT OF LAZARO CARDENAS. THE INSTALLATION WAS ALMOST COMPLETE BUT DID NOT CONTAIN GRAIN AT THE TIME OF THE EARTHQUAKE. NOTE THE FIVE STOREY RIGID STRUCTURE MOUNTED ABOVE THE SILOS WHICH LOST ITS BOTTOM SOFT STOREY DUE TO COLUMN FAILURE.

FIGURE 96: GRAIN HANDLING FACILITIES AT THE PORT OF LAZARO CARDENAS. REINFORCED CONCRETE CONVEYOR STRUCTURE WITH CANTILEVER COLUMNS WHICH COLLAPSED FROM THE FREE END DUE TO INADEQUATE ANCHORAGE INTO THE WHARF DECK SLAB.

FIGURE 97: GRAIN HANDLING FACILITIES AT THE PORT OF LAZARO CARDENAS. THE STRUCTURAL STEEL GRAIN ELEVATOR MOUNTED ON RAILS ON THE WHARF WAS UNDAMAGED IN THE EARTHQUAKE. IT WAS RESTRAINED FROM LONGITUDINAL MOVEMENT BY ANGLES WELDED TO THE RAILS.

FIGURE 98: CONTAINER TERMINAL PORT OF LAZARO CARDENAS SHOWING SETTLEMENTS OF THE PAVEMENT SLABS OF 200-250MM RELATIVE TO THE PILED WHARF STRUCTURE. NOTE THE FAILURE OF THE DOWELS CROSSING THE CONSTRUCTION JOINT.

FIGURE 99: CONTAINER TERMINAL PORT OF LAZARO CARDENAS SHOWING OPENING OF THE CONSTRUCTION JOINTS TO THE CONCRETE PAVEMENT SLABS DUE TO LIQUEFACTION AND THE SAND BOIL WHICH FORMED. FLOOD LIGHTING TOWER COLLAPSED DURING THE EARTHQUAKE DUE TO BRITTLE FRACTURE STARTING FROM THE FILLET WELD AND RUNNING THROUGH THE PARENT PLATE OF THE PIPE SECTION.

FIGURE 100: JOSE MARIA MORELOS DAM AND LA VILLITA, POWER STATION ON RIO BALSAS. SETTLEMENT/DISPLACEMENT RECORDS FROM JUNE 1974 TO SEPTEMBER 1985.

constructed in the mid 1960's.

Earthquake Performance: Silos brimful at time of earthquake.

CASE STUDY NO. 2

Structure: Grain silos, Charimera de Mexico Flour Mills, Av. Cipres

Principal Features: Superstructure: Honeycomb group of 5 x 2 no. r.c. silos, 40m high, approx 6.4m diameter. 165mm wall with single layer of light reinforcement.

Foundations: Driven end bearing piles to 30m depth. (Second firm layer is 26m deep at this site.)

Combined tension/compression/shear failure of walls resulting in fracture at approx. 8m height (fig. 101). Severe tilting above this level, loss of grain but no total collapse (fig. 102). Damage worse at east end of group. Buckling/fracture of steel in many places. Complex unusable and close to collapse. Evidence of horizontal displacement of foundation pad.

Reinstatement: Demolish and rebuild. (Plans are for a lower complex, possibly out of Mexico City).

FIGURE 101: CHARIMERA DE MEXICO FLOUR MILLS. FAILURE OF SILO WALLS SHOWING BUCKLING AND FRACTURE OF REINFORCING STEEL AND TILTING OF THE SILOS ABOVE THE FRACTURE.

FIGURE 102: CHARIMERA DE MEXICO FLOUR MILLS. FURTHER VIEW OF THE FRACTURE IN THE WALLS OF THE SILO.

Removal of grain brought problems attempts to drain it from holes at the base caused further tilting as grain at top stayed up.

Lessons/
Questions:

Would these silos have survived if they had had two layers of linked reinforcement? (They survived the 1979 earthquake.)

Comments/
Conclusions:

Engineers at site expressed dismay at failure, especially after surviving the 1979 earthquake. (Site is out of "major damage zone" and well towards the 'hills'). Perhaps they were not full in 1979. The long strong shaking of the 1985 earthquake was clearly too much for the non-ductile construction.

CASE STUDY NO. 3

Structure:

Grain silos, El Pilar Flour Mill, 597 Ferro Carril de Hidalgo

Principal
Features:

Superstructure: Honeycomb cylindrical reinforced concrete silos in a group of 5 x 2 no. approx. 6.42m diameter, 40m high, 165mm wall with single layer of reinforcement. Adjacent cleaning house approximately 10m square in plan and 36m high r.c. frame/wall construction (fig.s 103-104).

Foundations: Piles to 42m (?).

Earthquake
Performance:

Spectacular collapse of silos due to impact with cleaning house. Structures

FIGURE 103: EL PILAR SILOS. PLAN AND ELEVATION OF SILOS AND CLEANING HOUSE.

FIGURE 104: EL PILAR SILOS. GENERAL VIEW OF THE INSTALLATION.

were linked with dynabolts which all pulled out allowing cleaning house to sway away from silos (allegedly 2m at top!) and then crash into silo complex on return.

Silos were brimful at time of earthquake and were totally destroyed virtually to ground level (fig. 105). Cleaning house only slightly damaged but some evidence of foundation up-lift. Some cracks in walls, but of minor significance. Flour sifters in an adjoining structure were toppled.

This complex is well out of the "damage zone".

Chemical analysis of concrete samples indicated high voidage with assessed strengths of 10 to 15 MPa.

Reinstatement: Rebuild silos with adequate separation or proper linkage to cleaning house. Major expense required plus loss of stock and production at flour mill.

Lessons/Questions: It pays to separate structures effectively. A small detail can give rise to disaster. Would silos have survived if they had been reinforced with two layers of steel with cross ties; if impact had not occurred?

Conclusions: Keep up vigilance on building separation in NZ. Continue to detail for ductility by confining concrete core. Study of similar structures in NZ would be interesting.

CASE STUDY NO. 4

Structure: Bridge on Highway 200 Near Tecpan, Pacific Coast

Principal Features: 5 simply supported reinforced concrete spans of about 20m each supported in elastomeric bearings and reinforced concrete solid piers. The beams are restrained laterally by 200mm square RC shear keys at the pier caps but not tied together longitudinally with seismic links. The bridge has a skew of 24 degrees.

Earthquake Performance: The approach fills settled at the abutments but no apparent damage occurred to piers or abutments. Under the action of the longitudinal forces the spans moved along the line of the support with offsets of up to 200mm at the kerbs and gaps between the spans of up to 200mm.

FIGURE 105: EL PILAR SILOS. VIEW FROM ABOVE THE COLLAPSED SILOS SHOWING SILO BASES AND SPILT GRAIN.

(Figs 106 & 107). The bridge had suffered transverse movement in previous earthquakes as the offset indicated by the centre line marking was about 100mm whereas the offsets at the kerb was up to 200mm. Obviously many of the shear keys were destroyed or badly damaged by the lateral movement of the beams on the elastomeric bearings.

Reinstatement: Depressions due to settlement of the fill at the abutments had been brought to grade using plantmix. Elastomeric bearings need replacement and new shear keys provided. While vehicles could still use the bridge with a large 200mm gaps between spans these need bridging with plates. The offsets in the kerbs were not large enough to require the spans to be restored to their original positions.

Comments/Conclusions: Simply supported bridge spans with large skews act as wedges and induce high loads on shear keys. Seismic links between spans should be provided to control relative movement between spans.

FIGURE 106: BRIDGE ON HIGHWAY 200. OFFSETS IN THE KERBS OF UP TO 200MM DUE TO MOVEMENT ALONG THE LINE OF THE SUPPORT DUE TO LONGITUDINAL FORCES ON THE SKW DECK UNITS.

FIGURE 107: BRIDGE ON HIGHWAY 200. GAP OF ABOUT 200MM ON THE LINE OF THE 24 DEGREE SKEW ABOVE A PIER. NOTE THE OFFSET IN THE CENTRE-LINE MARKING OF ABOUT 100MM.

CASE STUDY NO. 5

Structure: Twin bridges over the Rio Balsas, Port of Lazaro Cardenas

Principal Features: Superstructure: Five prestressed concrete simply supported spans of about 24m (fig. 94).

Substructures: Short single stem reinforced concrete circular piers with large pile caps, and deep tapered pier caps, apparently designed and constructed in the 1980s.

Earthquake Performance: Severe damage occurred to many of the short pier stems due to hinging and the loss of confinement (fig. 108). Simple circular stirrups were used which were ineffective when the concrete cover was lost. The spacing of the stirrups was large and estimated at 300mm. As a result the main longitudinal bars buckled severely. Most of the damage occurred at

the top of the pier stems just below the stiff caps. There was some evidence that hinging occurred at levels in the stem where some of the main longitudinal bars were terminated. The shear keys at the ends of the pier caps were cracked in many instances.

Longitudinal movement also occurred causing impact damage at the kerbs and paint scratching on the pipe handrail joints of about 75mm. In some cases the pipe handrails had come out of the sleeve joints and were misaligned (fig. 109).

The approach fills settled and cracked longitudinally while the concrete slab protection armouring the abutment slopes moved up to 300mm at construction joints (fig. 110). The movements followed the slope of the embankment. Settlement of about 300mm occurred in the approach fills at the abutments (fig. 111).

FIGURE 108: TWIN BRIDGE OVER THE RIO BALSAS. VIEW SHOWING CONSTRUCTION OF BRIDGES AND SEVERE HINGING DAMAGE TO THE STEM OF THE SECOND PIER ALSO SIMILAR DAMAGE TO SEVERAL OTHER PIERS. NOTE THE CRACKING OF THE SHEAR KEYS AT THE ENDS OF THE PIER CAPS AND THE MOVEMENT OF THE CONCRETE SLAB PROTECTION TO THE APPROACH FILLS AT THE ABUTMENTS.

FIGURE 109: TWIN BRIDGES OVER THE RIO BALSAS. VIEW SHOWING LONGITUDINAL MOVEMENT IN THE PIPE HANDRAIL OF THE BRIDGE. THE PIPE HAS SLIPPED OUT OF THE SLEEVE AND IS MISALIGNED BY ABOUT HALF THE PIPE DIAMETER.

FIGURE 110: TWIN BRIDGES OVER THE RIO BALSAS. VIEW SHOWING SETTLEMENT AND LONGITUDINAL CRACKING OF THE APPROACH FILL ON THE PORT OF LAZARO CARDENAS SIDE AND THE OPENING UP OF THE CONCRETE PROTECTION SLABS AT CONSTRUCTION JOINTS.

FIGURE 111: TWIN BRIDGES OVER THE RIO BALSAS. SETTLEMENTS OF ABOUT 300MM OCCURRED IN THE APPROACH FILLS AT THE ABUTMENTS. THE ARTICULATED SLABS OF THE ABUTMENTS CONNECTING THE DECK SLAB AND THE ROADWAY ON THE APPROACH FILL MARKED WELL.

The damage to the piers was sufficient to close one of the bridges to vehicles while only one unladen vehicle per time was permitted on the other. The bridge closed to vehicles was used by pedestrians (fig. 94).		CASE STUDY NO. 6	
Reinstatement:	Possible. Piers can be repaired and proper confinement provided to the stems, shear keys can be replaced, approaches and abutment protection can be upgraded.	Structure:	Grain handling facilities, Port of Lazaro Cardenas
		Principal Features:	Large group (28 No) of tall reinforced concrete grain silos, with a five storey structure above the silos (fig. 95), rail loading facilities, wharf structures for shipping and associated unloading/loading equipment and conveyors, and electrical installation (fig. 112).
Lessons/ Questions:	Adequate confinement of longitudinal bars in pier stems is vital for good seismic performance.	Earthquake Performance:	It was reported that the facility was to have been officially opened on the day of the earthquake. The silos did not contain grain at the time of the event.
	Did the stiff pier caps respond in the vertical direction to bring the hinges up close to the top of the stem? Was there hinging also in the piles below the pile caps?		Major damage occurred to the five storey structure above the silos which lost a complete storey due to column failure (fig. 95). The structure appears quite stiff over the top four storeys with infill panels.
Comments/ Conclusions:	Obtain more details of the damage and an analysis of this bridge would be desirable. Articulated approach slabs connecting the deck and the approach fill worked well in this case (fig. 111).		There was some evidence of cracking in the walls of the silos.
	The research programmes carried out in New Zealand and the recommendations produced for ductile pier stems are vindicated by the seismic performance of these two bridges.		There was no apparent damage to the wharf structure proper nor to the large circular piles/caissons. The steel mobile ship loader/unloader was also undamaged (fig. 97). A reinforced concrete tower structure at the junction of the high and low level conveyor system and housing the transformers and

FIGURE 112: GRAIN HANDLING FACILITIES PORT OF LAZARO CARDENAS. GENERAL VIEW OF THE FACILITIES WITH THE SILOS ON THE LEFT, WHARF IN THE FOREGROUND STEEL MOBILE LOADED/UNLOADER AND CONNECTING CONVEYOR STRUCTURES AND SERVICE BUILDING IN BETWEEN.

FIGURE 113: GRAIN HANDLING FACILITIES PORT OF LAZARO CARDENAS REINFORCED CONCRETE STRUCTURE ON THE WHARF HOUSING ELECTRICAL INSTALLATION SHOWING FAILURE OF COLUMN AT A BEAM-COLUMN JUNCTION DUE TO LACK OF COLUMN TIES ABOVE THE DEEPER BEAM ON THE LEFT. NOTE THE USE OF BUNDLED BARS IN THE CORNERS OF THE COLUMN.

FIGURE 114: GRAIN HANDLING FACILITIES PORT OF LAZARO CARDENAS. FAILURE OF CANTILEVER COLUMN AT FREE END OF CONVEYOR SUPPORT STRUCTURE DUE TO INADEQUATE ANCHORAGE OF COLUMN BARS.

electrical switchrooms was severely damaged. It suffered beam-column joint failures as well as column hinging since it was inherently a weak column-strong beam structure. One failure occurred due to inadequate confinement and lack of column ties above the soffit of the deeper beam (fig. 113).

The lower level conveyor support structure consisting of reinforced concrete short cantilever columns and spine beam collapsed completely (fig. 96). Collapse appeared to commence at the free end of the structure due to inadequate anchorage of the column bars into the wharf deck slab (fig.s 114 & 115). The structure was parallel to the wharf and the river and at right angles to the earthquake propagation. In addition, some column failures involved complete disintegration of the column due to loss of confinement from the failure of light column ties with 90 degree hooks (fig. 116). Classical torsional cracking and failures occurred in the spine beam at cut-outs in the slab (fig.s 117 & 118).

There was some evidence of poor workmanship in concrete and reinforcing steel placing.

Reinstatement: The five storey structure above the silos will need to be rebuilt, possibly elsewhere. In itself, it is

FIGURE 115: GRAIN HANDLING FACILITIES PORT OF LAZARO CARDENAS. VIEW OF BOTTOM OF COLUMN IN FIG 114 SHOWING FAILURE OF BAR ANCHORAGE INTO DECK SLAB OF WHARF. NOTE THE BRITTLE TYPE FAILURE OF THE BARS AT THE TOP LEFT OF THE PICTURE.

FIGURE 116: GRAIN HANDLING FACILITIES PORT OF LAZARO CARDENAS. VIEW OF COMPLETELY FAILED CANTILEVER COLUMN DUE TO THE USE OF 90 DEGREE HOOKS AND SMALL DIAMETER COLUMN TIES. NOTE THE WIDE SPACING OF TIES AND THE USE OF BUNDLED BARS TO THE CORNERS OF COLUMNS.

FIGURE 117: GRAIN HANDLING FACILITIES PORT OF LAZARO CARDENAS. CLASSICAL TORSIONAL CRACKING AND FAILURE OF SPINE BEAM AT CUT-OUT IN SLAB.

FIGURE 118: GRAIN HANDLING FACILITIES PORT OF LAZARO CARDENAS. CLOSE-UP VIEW OF FAILURE OF SPINE BEAM.

a challenging demolition task. The shear cracking in the unloaded silos is ominous and considerable investigation will be necessary to arrive at acceptable repairs.

The low level conveyor structure and the structure of the junction of the conveyor systems will need to be demolished and rebuilt.

Lessons/
Questions:

Earthquakes search out the weakest links in the chain. If the anchorage of the end column had not acted as a fuse the conveyor structure still could have collapsed from the lack of confinement at hinge positions. Unless columns are protected they

must be ductile for adequate seismic performance.

Comments/
Conclusions:

A simple capacity design approach, better detailing and workmanship would have produced a structure capable of surviving.

12. DAMAGE TO MAIN FRAME COMPUTER INSTALLATIONS, STORAGE RACKS AND MISCELLANEOUS STRUCTURES

12.1 Main Frame Computer Installations

There was little or no reported damage to main frame computer installations in Mexico City. Facilities, such as the Federal Electricity Commission computer installed in the basement of an office building on the edge of the damage zone, was reported to be operational after the earthquake.

The SPERRY UNIVAX 1200 computer installation controlling the operation of the Sicartsa Steel Mill in Lazaro Cardenas was down for 5 days following the earthquake. It was reported that the damage to the computer was the reason why the mill was out of operation for this length of time. Little or no earthquake damage was caused to the actual production facilities.

Earthquake damage to the installation included tiles to the computer floor flipping out allowing the unrestrained cabinets and consoles of tape drives, controls, printers etc to overturn into the resulting openings in the floor. Considerable damage was done to such items. In addition racks of spares, disc storage racks and the standby battery racks overturned, causing some damage and acid spillage. The air conditioning unit was undamaged.

In order to get the computer operational, it was reported that it was necessary to fly in spares from the United States.

12.2 Storage Racks

Conventional racking systems performed poorly either overturning, buckling or collapsing.

Extensive failure of storage racks occurred in the stores and spare parts warehouse of the Sicartsa Steel Mill where about 60% of the racking collapsed (fig. 119). The

warehouse was about 100m by 150m and contained 75,000 different items totalling several million in all. Racking containing heavy items was particularly severely damaged but racking with walkways over the top connecting racking bays and containing small items suffered no damage apart from dislodgement of contents.

12.3 Transmission Towers

The only damage to electrical transmission towers occurred to several towers close to the sea near Lazaro Cardenas reported to have been damaged from the effects of the tsunami.

12.4 Micro-wave Towers

Micro-wave installations in the SCT tower in Mexico City were undamaged.

The main micro-wave tower to the central exchange in Mexico City, a reinforced concrete structure, had hinging in the beams and in the beam-column joints (fig. 93). It remained operational and the damage appears to be repairable. Significant damage was caused to the walkways linking the tower to adjacent buildings. The relative movements were reported to be 200mm.

13. PERFORMANCE OF LIFELINE SYSTEMS

Most engineering studies after a major earthquake focus on damaged structures. However, the performance of many other engineering systems is of vital importance, especially the so-called lifeline systems: water supply, electricity, gas, road and rail transport, medical facilities, sewerage disposal, telephone systems, radio and television, airports and harbours among others. While in Mexico, the reconnaissance team gathered considerable information on the effects of the earthquake on these systems.

FIGURE 119: SPARE PARTS WAREHOUSE SICARTSA STEEL MILL LAZARO CARDENAS. COLLAPSE OF CONVENTIONAL STORAGE RACKS AND AN ILLUSTRATION OF THE RESULTING CHAOS.

13.1 Water Supply

In the weeks after the earthquake, disruption to the water supply to Mexico City was one of the most obvious effects of the earthquake on lifeline systems. There was severe damage to water mains in the central city area in the zone of building damage, and also in the south-east section of the city on flexible soils which had predominant ground periods exceeding 3 seconds but where there had been little building damage. The supply had been reduced by 7.6 cumecs, resulting in a complete loss of supply to 4 suburbs and severe curtailment in 6 others. About 60 to 70 per cent of the normal supply is pumped from wells, and the rest is brought into the valley through an aqueduct system. Four branches of the aqueduct had been severely damaged. A program of detection and repair of leaks had localised 900 faults corresponding to 55 primary mains and 740 secondary lines, of which 280 had been repaired by 29 September. A recovery of 4.7 cumecs of the lost 7.6 cumecs had been achieved, restoring half the regular supply to two suburbs and better than this to the rest, including complete recovery to two. Repairs were hindered by people altering valves to re-establish their own supply, and because some leaks showed up only after others had been repaired and the pressure increased.

Various emergency supplies had been established. Communal supplies were set up with 424 pipes running continuously which delivered 20 million litres a day, together with portable tanks. Purified drinking water was distributed in bottles and cartons (400,000 litres per day), along with 200,000 litres of soft drinks in the affected areas. The railways contributed with 20 tank wagons delivering 600,000 litres daily.

As a sidelight, the severity of this earthquake was compared to earlier earthquakes on the basis of damage to water mains (which are affected directly by the ground motion), rather than by building damage because in previous earthquakes there were few buildings susceptible to long-period shaking. In this century, only the 1911 earthquake possibly rivalled that of 1985.

13.2 Electricity Supply

Immediately after the earthquake, 40 per cent of the 3,200,000 customers in Mexico City had lost their electricity supply. The capacity was reduced to 1500 MW, compared to the peak demand of 3600 MW, although the main problem was with failures in the distribution system rather than with electricity generation. Fortunately most of the service was restored rapidly. Only 1 per cent of the consumers were affected long-term, although this was still a significant number amounting to 34,000.

Most of the electricity supply for Mexico City comes from outside the valley of Mexico, principally from hydro-electric generation from dams on the Rio Grijalva 1,000 km south-east of Mexico City and from

the Infiernillo and La Villita dams on the Rio Balsas in the epicentral region over 300 km south-west of Mexico City. There were no long-term problems with the electricity generation, although the immediate capacity was reduced considerably by tripping out of several of the generators at La Villita and Infiernillo. There was no damage to the powerhouses or switchyards in the earthquakes.

The cause of the disruption of supply to most consumers was that use of 270 of the 700 high-voltage primary distribution lines was lost, but 60% were recovered immediately as their failure resulted from the tripping of circuit breakers. By 7.30pm on the Friday evening, 36 hours after the main shock, 263 of the 270 feeders had been restored, only for another 88 feeders to be lost for a total of 95 in the large aftershock of 19:38 on September 20. All the 88 new failures were retrieved within 24 hours.

Supply was lost from 8 substations, three 230 kV and two 85 kV lines, and two 85 kV cables. Different sources quoted losses of 4km of high tension lines and 14km of low voltage lines (according to the Comision Metropolitana de Emergencia) or 10km of high voltage and 30km of 220 and 127 V medium voltage lines (according to the Compania de Luz Y Fuerna, the regional electricity distribution authority). A major problem was the collapse of 600 power poles and damage to 300 transformers. Many of the transformers were attached inadequately to their supporting poles and fell under the severe vibration. An additional connection linking the transformers to an overhead cross-arm on the poles has been designed since the earthquake. Other transformers were lost when the buildings housing them collapsed. Many of the electricity wires were dragged down or snapped by falling building debris. Metal clad underground cables fractured in tension from large amplitude ground movements, with the thickness of the metal piping being reduced from 1/4 inch to 1/8 inch and 1/2 inch to 1/4 inch in some cases. There were no problems with a modern underground system of cross-linked thermosetting polyethylene cabling in the central city area.

The restoration of the electricity services required mobilization of considerable manpower. Initially there were 300 crews of 5 men, increased later to 600 crews when others were brought in from outside Mexico City. The crews worked in three shifts around the clock. Two hundred vehicles such as cranes and basket-trucks to restring the wires required.

A feature of the performance of the electricity distribution system in this earthquake was the lack of damage to normally vulnerable switchyard equipment either at the power houses in the epicentral region or at the substations in Mexico city. If there had been earthquake detectors to override the circuit breaker relays, there would have been very few problems with the delivery of the supply from the switchyards. Perhaps there was insufficient high-frequency content in the

FIGURE 120: MEXICO CITY. TANKER TRUCK DELIVERING LPG TO STORAGE TANKS OF A BUILDING IN A CITY STREET.

ground motion to affect the switchyard equipment in this earthquake. Also, most of the Mexico City sub-stations were outside the zone of strong ground motion.

13.3 Gas Supply

In Mexico City, LPG gas is delivered to storage tanks in buildings and apartments by tanker truck (fig. 120). Consequently, there is no underground gas distribution system and there were no fires fed by gas from leaking or ruptured gas mains. The only serious fire reported was due to a leak in the gas storage tank in the St Regis Hotel. This fire spread to an adjacent department store and an office building occupied by a government department.

13.4 Road and Rail Transport

There was very little disruption to road or rail transport by the earthquake. There were temporary closures in the 36 hours after the earthquake on the access routes to Ciudad Guzman and Lazaro Cardenas. There was minor damage to a few bridges and small rockslides in road cuttings north of Lazaro Cardenas. Three weeks after the earthquake the route from Ixtapa to Ciudad Altamirano was blocked by a military guard because of large rockfalls. The twin access bridges to the port at Lazaro Cardenas were extensively damaged. Road causeways in the port area were washed away by the tsunami but quickly replaced.

In Mexico City, slight damage to only one road-bridge was reported. Many streets were blocked by rubble, or suffered cracked pavements, but there were readily available alternative routes because of the flat terrain. One of the major problems in using cars in Mexico City in the weeks after the earthquake was greater lack than usual of parking space. Several parking buildings in the central city area collapsed, roadside parking was disrupted by repairs to water mains, and empty lots were being used for the temporary storage of rubble.

The structures of the Metro commuter rail system in Mexico City performed extremely well in the earthquake. After an initial short delay for safety checks, all lines were in operation, apart from the closure of the Pino Suarez and one other underground station whose entrance ways were affected by the collapse of buildings.

13.5 Telecommunications

The telephone system was seriously damaged in the earthquake. The system comprised 1,700 long distance circuits, of which 1,200 linked with Canada and the United States and 500 linked to the rest of the world, and 25,000 in Mexico City.

Equipment and staff were housed in 68 buildings. One of these, the six storey Victoria Exchange, built in 1965, collapsed. Three more suffered severe damage, principally to concrete masonry walls and to reinforced concrete frames. A further ten were slightly damaged.

Equipment was damaged or destroyed when structures housing it collapsed or were damaged. It also suffered in relatively undamaged buildings where battery racks failed and rows of switching equipment cabinets fell like dominoes (fig. 121). All the damage was attributable directly to intense shaking. No fires occurred, and there were no short-circuits.

The Mexico City telephone system was a centralised one, dependent on two principal exchanges. It was critically affected by the failure of one of them, the Victoria Exchange, where 10 of the 24 operators in the building at the time of the earthquake were killed, and by damage at the other, the San Juan Exchange. Due to the complete failure of the Victoria Exchange, it was not possible to maintain operator assisted telephone communications in the aftermath of the earthquake. Other internal services were maintained with difficulty, and most of the services abroad were either curtailed or lost entirely.

FIGURE 121: MEXICO CITY. TELECOMMUNICATIONS SWITCHING CABINETS WHICH FELL OVER LIKE DOMINOES DUE TO INADEQUATE FIXINGS AND RESTRAINT.

At the main micro-wave tower, a reinforced concrete structure adjacent to an exchange building to which it was connected by walkways, there was significant damage because the connections did not provide for the 200mm of relative movement that occurred in the earthquake.

Large ground deformations caused difficulties with ducted cables, but, except at connection boxes, damage was trivial.

Emergency service restoration was effected using equipment destined for new installations which was, at the time of the earthquake, in store, and serviceable material salvaged from damaged installations. Also used were two United States supplied and installed temporary digital exchanges. In all, some thirty five tonnes of equipment was brought into the city in the first week after the earthquake. Foreign links were restored by establishing emergency lines to undamaged exchanges at Monterrey and at Guadalajara. Within two weeks there was a facility functioning, capable of handling 20% of the normal volume of long distance traffic. At the time of the teams visit, this had been improved to 50%.

Proposed for the rehabilitation of Mexico City's telephone system are new exchanges to decentralise the system and thus to reduce its dependency on the two principal exchanges. Scheduled for commissioning in January or February of 1986 were three additional exchanges to be linked to the micro-wave system. Each new exchange will have optic fibre line links with restored Victoria and San Juan exchanges. New codes and procedures are being devised to ensure, so far as it is possible to do that, buildings and their foundations will be adequate to resist the kinds of earthquake excitation that Mexico City experiences. In addition new seismic requirements and installation procedures for equipment will be introduced. The new system will be much less vulnerable to earthquake attack than was the old one.

13.6 Mexico City Airport

The asphalt airport runways were all undamaged. Taxiways and concrete aprons suffered minor damage due to the opening of some joints. Airport buildings were undamaged, apart from cracking in reinforced concrete elements. The airport remained fully functional except for a short period after the earthquake while the facilities were checked for damage. The principal difficulties directly attributable to the earthquake were with communications and with fuel supplies. The general failure of much of Mexico City's telephone system created difficulties, especially in the post earthquake traffic volume.

Communications proved to be the biggest operational constraint due to failure of the links through the microwave tower at the Ministry of Communications and Transport (SCT Centre). As a result, it was only possible to communicate with aircraft for 15 minutes prior to their arrival for a considerable period after the earthquake.

An underground line ruptured near the airport's apron area and caused the loss of some 60,000 litres of fuel. The Airport's fuel tank farm, with a capacity of 12,000,000 litres in six tanks and with a supply line from a refinery, was virtually undamaged. Aircraft were supplied with fuel by tankers until the underground lines were repaired.

A free unloading and refuelling service was supplied to all relief aircraft bringing in supplies and equipment as aid from other countries.

During the emergency there was a large increase in the volume of traffic through the airport. There were relief flights, and flights to cope with an exodus of illegal immigrants from South American countries and an influx of Mexicans from overseas anxious about family and friends. The workload for airport staff and the

hours worked increased dramatically for this reason and also because there were failures in the water reticulation, drainage and communication systems, and because minor damage at the airport needed attention. Airport staff were accommodated in the Holiday Inn Hotel, adjacent to the airport, from where they could be on duty within 5 minutes of a call or an emergency arising.

14. THE RESPONSE OF EMERGENCY SERVICES

14.1 Civil Defence and Rescue

No Civil Defence organisation such as we know it in New Zealand operated in Mexico before the earthquake. There were provisions for the President to assume extra-ordinary powers in the case of a major emergency such as the earthquake. The army and navy, a number of government departments and large private organisations had their own emergency plans which were put into action. The major problems in the initial stages were with overall coordination and lack of communications to determine the true extent of the disaster. The response was improvised reasonably well.

The President initiated the Federal Government response at 7.45 am on the morning of the earthquake, giving instructions to the Ministry of Defence and the Department of the Federal district to initiate the first phases of rescue and emergency response.

Metropolitan and National Commissions of Emergency were set up. There were 50,000 workers available in the public sector. However, a major problem with implementing the government response was that many government buildings suffered heavily in the earthquake. Among departments badly affected and requiring rehousing were the Secretariats of the Navy, Communications and Transport, Trade and Industry, and Social Welfare.

The emergency response included rescuing 3,300 people from buildings, attending to 30,000 in hospital and first aid posts, and locating 17,000 missing persons. Housing was found in shelters and camps for 33,000 homeless, who required food, clothing and medical attention. There was a daily distribution of 80 tons of fruit and vegetables, 20,000 food rations, 1,500 tons of groceries 200,000 litres of milk and 15 tons of tortillas. A health monitoring program was set up in the camps and shelters in coordination with the W.H.O. to prevent the outbreak of epidemics of contagious diseases, and to maintain the best-possible sanitary conditions.

Although there was no pre-determined plan for them to do so, radio stations quickly became a focus for organising assistance where required. Initially their reporters called in from radio-cars, and later a listeners call-in system was set up. There was little public awareness of how to react in the emergency, with nothing even like the minimal instructions given on the back of New Zealand telephone directories.

The army was greatly involved in the rescue work, in demolishing dangerous structures in the early stages, and providing a security cordon in badly damaged areas. Later on squads of workers from PEMEX, the Mexican Petroleum organisation which is one of the world's largest corporations, and their heavy equipment were well in evidence in the demolition work.

There were various techniques for demolishing buildings, and for rescuing those trapped or for recovering bodies. The Mexicans themselves preferred to lift the rubble away piece by piece from the top while some of the foreign rescue teams tunnelled from the side or top.

There was at least one televised difference of opinion between a French and an American team resulting in blows. The Mexicans were strongly opposed the use of explosives, as they wanted every opportunity to recover bodies. Demolition using explosives was finally introduced about 6 weeks after the earthquake to speed progress, with the newspaper headlines "MANANA DINAMITA".

During the rescue phase, there were various aids to locating trapped people. Perhaps the most successful and efficient was sniffer dogs. More sophisticated techniques like the "electronic eye" or "snake", a video camera on the end of a flexible cabling which could be directed down inaccessible spaces in the rubble, and heat sensing devices were also used.

Helicopter cranes were not used. There were doubts about their operability at the altitude of Mexico City, but the major drawback was that overpressures and vibration could trigger further collapses of weakened buildings and building elements. In addition, they would make conditions for the trapped and for workers unacceptable from dust raised from the building debris.

The Mexican people themselves responded magnificently to the disaster, with assistance to those worse affected than themselves. They were also very appreciative of the generous foreign aid, although there were problems coordinating it. Some of the early rescue teams arrived while communications, both within Mexico City and to the rest of the world, were a problem. There were difficulties finding accommodation, with many hotels damaged. There were also problems in co-ordinating and supervising rescue efforts by the foreign rescue teams.

Problems with supplies that were not required have already been mentioned. Excess blood plasma had to be stored in the airport catering service cold stores, and more than a year's supply of swimming pool chlorine tablets to treat the water supply arrived from an enterprising American firm which had set up a public donation scheme to purchase their product.

There were problems in the early stages in providing assistance for foreign scientific and engineering study teams. We were requested to delay our arrival for two weeks after the earthquake by which time

the rescue phase was completed. When the team arrived, the Mexicans were well organised, with reports on the damage and strong-motion records prepared by the University Institute of Engineering, and the DDF providing technical guides, interpreters and transport to inspect damage sites. The team was treated very hospitably and information was forthcoming from virtually all the organisations we contacted.

14.2 Medical Services

At the time of the earthquake there were 24 hospitals in Mexico City. Of these, one was a general hospital and the rest either specialised in a medical sense, for example by offering maternity care (8 hospitals), or were established for a particular group of people, for example, civil servants. Among the specialist hospitals were 5 accident and emergency hospitals, with, between them, a total of 800 beds. There were also ambulance services, most of them attached to the principal hospitals, and a number of small health centres and clinics. Ambulance officers were trained drivers but had no paramedical skills.

The central medical hospital, the biggest facility in Mexico City, was severely damaged in the earthquake and was unusable. Its patients were transferred elsewhere. Other hospitals were also damaged and suffered varying degrees of operational impairment.

Mexico City's accident and emergency hospitals have experience in coping with large influxes, gained as a result of events such as the annual Independence day celebrations when, each 15 September, some 300 people are hospitalised - while a much smaller number than the 30000 injured in the earthquake, it is comparable with the 1500 actually admitted to hospital - after the earthquake. Within 6 hours of the earthquake, some 75% of the injured had been treated. Before midnight on the day of the earthquake, some seventeen hours after the event, the proportion treated reached an incredible 98%. Fifty ambulances were working immediately after the earthquake. Later the number was increased to 127.

Twenty-seven mobile units, each with a vehicle, a doctor, a nurse, a social worker and a dentist were established and assigned tasks in the main earthquake affected areas in cooperation with rescue teams. These units were all operating within 24 hours after the earthquake, the crisis period for medical services. They were supported by ambulance services for transporting victims who needed further care and treatment.

Those 5% of the injured to be hospitalised were accommodated in one of the five accident and emergency hospitals from which all former patients had been transferred so that beds were available.

In order to prevent the spread of disease, it was necessary to recognise and to isolate persons with infections.

There was an adequate supply of dressings

and of most medical supplies; but a shortage of anaesthetics developed. Emergency supplies were flown in from the United States of America.

Catering problems beset the hospital administration services, which were obliged to arrange for feeding large numbers of victims, medical staff and volunteer workers in conditions that were not ideal. The potable water supply failed, and there were difficulties with contaminated water supplies from tankers which aggravated problems in hospital and kitchens, but these were overcome.

Throughout the first day after the earthquake, medical effort was directed entirely to emergency treatment of casualties. Thereafter, as the need to provide first aid diminished, emphasis shifted to disease prevention and associated extraordinary problems like refuse disposal in difficult conditions. There were homeless needing care and feeding in hostel accommodation, with the ever present vital need for cleanliness and for precautions against infection. Finally, three or four weeks after the earthquake, and with the pressure on medical services greatly diminished, people with earthquake generated psychiatric difficulties were treated in a coordinated operation involving group meetings and therapy.

Handling and disposal of corpses removed from the rubble of collapsed buildings was a very important operation. Bodies recovered were put in individual plastic bags for removal to a collecting point which was established at the IMSS Baseball Park. Here a doctor and a lawyer were on duty, and the dead were held briefly for identification by relatives. During the first 24 hours after the main shock, bodies were held there for 2 hours. Later this holding time was reduced to 1 hour. Family wishes for disposal of identified dead were respected. Bodies unidentified in the holding period were then removed for burial. The medical emergency was, on the whole, well handled. De-briefings have shown however, there is room for improvement, and it is proposed that training will be given to doctors, especially to surgeons, to fit them better for emergency work, that paramedical skills will be required of ambulance officers, that ambulance vehicles will be equipped with radios, and systems for intercommunication between hospitals and medical facilities will be improved by the provision of radios as an alternative to the fallible telephone system.

14.3 Radio and Television Response

There was no emergency plan to use radio or TV to inform the population of what was happening or what to do and where to go for assistance in the event of a disaster. In the first few hours, the mobile radio cars broadcast events as they happened while they were trapped with the commuter traffic (from 7.20am on the Thursday morning when the first tremor occurred); for example, which buildings had collapsed, which roads were blocked and where emergency services

were required. The radio and TV stations pooled resources and broadcast a common programme and late on Friday the Police and Civil Authorities were using this media to advise the public where to get help and what was being done in response to the disaster. Radio also acted as a "go-between" for those wanting to give assistance, in any form, (eg. a woman offering to drive a bulldozer), would ring in and their offer would be broadcast. Apparently the response was phenomenal - like a telethon for two weeks. After about two weeks, TV and radio had programmes on how to repair damage and where to seek proper advice.

In general the response was spontaneous rather than planned and there was nothing but praise for radio announcers who worked long hours under trying conditions. One TV channel was broadcasting a live new programme when the studios were destroyed and several staff killed.

In all cases, emergency power was required for broadcasting.

14.4 Fire Services

Within 24 hours following the earthquake about 200 fires were reported. The worst fire started from ruptured gas lines of the collapsed Hotel Regis and spread to the adjacent apartment store and office building and hampered rescue operations.

There was no major conflagration as in San Francisco 1906, Tokyo 1922 or Napier 1931. Presumably due to the type of construction, absence of wooden buildings. Also, most domestic gas is LPG 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 otherwise may have been saved.

The effect of the failure of the fire station headquarters buildings and the consequent delay in organising fire services, may have had a serious effect in terms of saving human life. The lack of water due to broken mains and distribution pipes in and around collapsed buildings could also have adversely affected fire fighting ability.

15. LESSONS FOR NEW ZEALAND

The principal aim of the reconnaissance was to investigate whether there were lessons for New Zealand from Mexico's experience.

From the evidence gained on the performance of structures in both Mexico City and the epicentral region there is nothing that would suggest inadequacies in current New Zealand seismic design philosophy and design procedures. Rather the reverse would be true in that if capacity design procedures and the detailing provisions of New Zealand materials codes had been adopted to produce 'tough' structures, we are of the view that they would have

performed well in the earthquake.

However, there are some areas where it would be prudent to review the New Zealand approach. These include;

1. Soil-structure interaction. The interaction of foundations and structure needs further attention from designers and researchers.
2. There were numerous instances of pounding of adjacent buildings due to large displacement responses imposed on the structures and even though quite liberal seismic separations were provided. In New Zealand, normal practice should always be to allow for foundation rotation in addition to the prescribed deformation of the structure. The usual New Zealand practice of providing for movement of elements in all but the stiffest structures was fully vindicated (fig. 122).

FIGURE 122: MEXICO CITY. HINGING IN STAIRS AND LANDINGS DUE TO LACK OF SEPARATION FROM FLIGHTS AFTER INFILL PANELS HAD FAILED.

Micro-zoning studies of New Zealand cities are required, particularly in areas situated on soft marine or deep alluvial deposits, to ensure that they are not prone to resonance problems such as those demonstrated in Mexico City and suspected to have occurred in other damage zones.

Continued commitment to the New Zealand strong motion accelerograph programme

including, upgrading, extension and modernisation is required to ensure we obtain records as in Mexico City and the epicentral region, of the damaging ground motion in the next major New Zealand earthquake. In addition we need to ensure that we instrument low, medium and highrise buildings to obtain the structural response to such ground motions in order to maximise the benefits of the strong motion records. The lack of structural response records in Mexico city was one of the disappointing features of the 1985 Mexico earthquakes.

Redundancy of life-line systems should be investigated in New Zealand and steps taken to ensure protection, as far as is practicable, against problems, such as those which severely restricted communication after the collapse or severe damage to international telephone exchanges.

There are a number of Civil Defence lessons which need to be considered by New Zealand authorities including communications, intelligence gathering, locating and rescuing of people trapped in buildings, targeting of international aid and the assistance and provision of information to foreign reconnaissance teams seeking lessons from the earthquake.

16. CONCLUSIONS

The severe damage in the September 1985 Mexican earthquakes generally occurred where structures with poor seismic resistance were built on ground which strongly amplified the incoming earthquake motions.

Higher than expected ground accelerations occurred in the soft soils of Mexico City while the 60 seconds duration of severe shaking was particularly damaging.

The characteristics of the earthquake, especially its dominant period, were such that buildings were attacked selectively. One and two storey adobe and brick houses were largely unaffected, except in a very few towns.

The earthquake highlighted the need for soil-structure interaction to receive more attention from designers and researchers. Many buildings suffered damage at about mid-height, for example, from the fourth to the eighth floor in a typical twelve storey building, probably as a result of interaction of foundations and structure.

As in many previous earthquakes the need was demonstrated for 'tough' structures capable of ductile response under repeated cycles of earthquake loading. The team concluded that the earthquake completely supported the capacity design procedures and stringent detailing requirements of New Zealand seismic design codes.

New Zealand type capacity design and detailing procedures for reinforced concrete and steel structures were not used in Mexico. There was evidence of inadequate detailing practice such as the almost universal use of right angle hooks

to column ties which resulted in the early collapse or partial collapse of such members due to lack of confinement. The earthquake also demonstrated once again that confinement provided by spiral reinforcement ensures good performance of the columns. As an example, The spiral reinforcement in the circular columns of the Cardiology block at the Central Medical Hospital in Mexico city prevented the complete collapse of the building.

A feature of building practice in Mexico was the wide spread use of flat plate type frame structures due to the advantages of buildings up to about 10 to 12 storeys. These buildings as a group performed badly. The earthquake demonstrated tragically, the extreme difficulties of proportioning and detailing such structures to ensure good performance and the prevention of combined flexural torsional and punching shear failures.

Large displacement responses and some structures in Mexico city caused pounding of structures even when quite liberal seismic separations were provided between buildings. The effect was compounded by differential settlement and foundation rotations which occurred in a number of cases. The lack of separation between elements with different responses caused considerable damage and made the evacuation of buildings more difficult in the case of damaged stair flights and landings. The experience would suggest a re-evaluation of the adequacy of the requirements of NZS4203 for structural separation between elements of the same structure and between adjacent structures and whether foundation rotation in soft soils should be included.

Compressible filler materials used in structural separations should be tested to ensure that they crush as intended and do not transfer more than nominal or calculated forces to the structure.

Reinforced concrete shear walls are not widely used in buildings in Mexico. There were however a number of examples of good seismic performance of such structures even where thickness to span ratios were very much less than for New Zealand practice.

There was some evidence of low standards of construction including low cement contents and high water cement ratios in concrete, the use of poor quality and badly graded concrete aggregates in Mexico City and poor workmanship in welding of structural steel. On the other hand, building materials such as cement and reinforcing steel were reported to readily comply with recognised international standards.

Change of occupancy of buildings leading to much higher overall loadings than the design loadings was apparent in some severely damaged buildings in Mexico City. Change of occupancy needs to be monitored by authorities particularly in larger cities.

Microzone studies of New Zealand cities, particularly those areas situated on soft marine or deep alluvial deposits, are required to ensure that they are not prone

to resonance problems as demonstrated in Mexico City and suspected to have occurred in other damage zones.

Excellent digital accelograph records were obtained in the various soil zones of Mexico City but unfortunately there were no structural response records from buildings. Such records, had they been available for low, medium and high-rise buildings, would have helped to determine causes for some unexplained features of the damage observed in Mexico City. The Mexico experience confirms the importance of continued commitment to the New Zealand strong motion accelograph programme, including its extension and modernisation, to ensure we obtain records of damaging ground motion and structural response in the next major New Zealand earthquake.

Redundancy of life-line systems should be ensured as far as possible to protect against the problems which severely restricted communications after the collapse or severe damage to the international telephone exchanges in Mexico City.

The importance and length of the rescue phase following a major earthquake in large modern cities like Mexico City was once again highlighted. Greater emphasis and priority needs to be placed on developing techniques in New Zealand for the location and rescue of people trapped in collapsed buildings.

This should include the training of sniffer dog teams, reported as being so successful in Mexico City, the study of methods of dismantling of collapsed structures for rescue of trapped people and the use of quick methods of demolition of dangerous building. This could involve the use of explosives where conventional demolition methods and the possibility of progressive collapse may mean unacceptably high risks and lead to unnecessary casualties.

International aid needs to be co-ordinated and priorities for aid determined rapidly otherwise aid can easily be misdirected. The setting of priorities require good communications and intelligence gathering within the affected region. The 1985 Mexico earthquakes showed the media whether local or international, cannot be expected to fill this role.

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