THE ERZINCAN, TURKEY EARTHQUAKE OF 13 MARCH 1992

A FIELD REPORT BY EEFIT

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SYNOPSIS

The city of Erzincan is situated in a deep alluvial basin in a mountainous region of eastern Turkey, a few kilometres from the highly active North Anatolian Fault. Prior to the recent earthquake, Erzincan had a rapidly growing population of 92,000, with a further 58,000 residing in the surrounding villages. The city has been completely rebuilt since being destroyed by a major earthquake in 1939; much of the building and infrastructure dates from the last ten years.

On 13 March 1992 an earthquake having a surface wave magnitude of 6.8 struck Erzincan, giving rise to peak horizontal accelerations in the centre of the city of $0.5g$ in the east-west direction and $0.4g$ in the north-south direction. The earthquake caused 394 fatalities in Erzincan city and a further 150 in the surrounding villages. Around 5,500 buildings collapsed or were damaged beyond repair.

Five engineers from the UK based Earthquake Engineering Field Investigation Team (EEFIT) spent eight days in Erzincan shortly after the earthquake. This report sets out their findings, which are summarised below.

1. While many of the collapses occurred in mid-rise reinforced concrete structures, masonry buildings and traditional forms of construction also fared badly in the earthquake. Most of the failures were due to well understood deficiencies in design and construction. No significant shortcomings in codes of practice were evident, but there is clearly a major problem with enforcement. Buildings in Erzincan are likely to be typical of modern structures in most parts of Turkey, making the design faults noted in this report particularly important.

2. The majority of damaged structures collapsed along an axis oriented east-west. This suggests a greater intensity of shaking in the east-west than in the north-south direction, a finding which agrees with strong motions recorded in the city centre.

3. It was not possible to identify any surface fault break due to the earthquake. There were some instances of ground cracking, liquefaction and landslides but very few foundation failures.

4. From the surveys carried out in Erzincan and the surrounding villages, it is not possible to identify a simple amplification effect in the frequency range of interest due to the deep sediments in the Erzincan Basin. Further investigation of this topic would be valuable, but is made difficult by the lack of borehole information within the basin.

5. The modern construction in Erzincan is probably among the least seismically vulnerable in eastern Turkey; a similar magnitude earthquake occurring in an area with a greater predominance of traditional building types could cause significantly greater damage and loss of life.

CONTENTS

1. INTRODUCTION

At 19:18 local time on Friday 13 March 1992 the city of Erzincan in eastern Turkey was struck by a large near-field earthquake, having a surface wave magnitude of 6.8 and a focal depth of 27 km. The earthquake caused considerable loss of life, with an estimated 394 fatalities in Erzincan city and a further 150 in the surrounding villages. Around 700 people were seriously injured and 1400 suffered minor injuries. The time of occurrence may have prevented the casualty rate from being considerably higher, since many collapsed structures were office buildings which would have been fully occupied a few hours earlier. At the time of the earthquake, many people were praying in mosques, which generally suffered remarkably little damage.

It is estimated that about 5500 buildings collapsed or will have to be demolished, making about 20% of the 180,000 inhabitants of the region homeless. The health sector in Erzincan suffered particularly badly, with major collapses at all three hospital sites causing loss of life and leaving the affected area without medical facilities when they were most needed. Industrial buildings also suffered serious damage; the resulting monetary loss will seriously hamper the regional economy's recovery over the next few years.

There were many reinforced concrete (RC) structures that survived with limited structural damage, or some non-structural damage. Nevertheless, this earthquake follows a recently observed worldwide trend, in that most of the casualties were due to the collapse of RC buildings. This is the first such event of importance in Turkey, with the possible exception of the 1976 Çaldiran earthquake, and therefore its effects should be studied carefully in order to draw lessons for the improvement of RC construction in Turkey and worldwide.

1.1 The EEFIT Mission

Shortly after the earthquake, the UK-based Earthquake Engineering Field Investigation Team (EEFIT) mounted a mission to Erzincan. The team arrived in Turkey on Sunday 29 March and spent seven days in the affected area. The EEFIT team consisted of:

- Giovanni Vaciago (the team leader), a geotechnical engineer from High Point Rendel, based in Bolu, Turkey;
- Martin Williams (the editor of this report), a lecturer in structural engineering at the University of Oxford;
- Antonios Pomonis, a research assistant at the Martin Centre for Architectural and Urban Studies, University of Cambridge;
- Steve Ring, a lecturer in structures and geotechnics at the University of Bath;
- Edmund Booth, a structural engineer from Ove Arup and Partners, who travelled to Erzincan as part of the French AFPS mission (AFPS, 1992), and worked closely with the EEFIT team.

Financial support for Antonios Pomonis, Martin Williams and Steve Ring was provided by the Science and Engineering Research Council. The team spent most of its time in the field, but also held a number of extremely useful fact-finding meetings with academics at the Middle East Technical University and staff of the Directorate of Disaster Affairs, both in Ankara.

1.2 Contents of the Report

This report presents the findings of the EEFIT mission. Chapter 2 gives background information on the topography, population and economy of the affected region. In Chapter 3 a brief account of the seismology of the earthquake is given. Chapters 4-6 concentrate on observations of damage in and around Erzincan. Firstly, types and causes of damage are identified and a number of case studies are discussed. This is followed by a description of the distribution and extent of damage, including the results of a detailed photographic survey around the strong motion instrument. In Chapter 7 a number of other aspects of the earthquake are discussed, including the performance of lifelines and the organisation of the relief operation. Chapter 8 briefly outlines the reconstruction programme. Conclusions of the work are given in Chapter 9.

1.3 Background to EEFIT

EEFIT is a UK-based group of earthquake engineers, architects and scientists who seek to collaborate with colleagues in earthquake-prone countries in the task of improving the earthquake resistance of both traditional and engineered structures.

The principal activity of EEFIT is conducting field investigations following major, damaging earthquakes, and reporting to the local and international engineering community on the performance of civil engineering and building structures under seismic loading. A preliminary reconnaissance mission is carried out within a few days of an earthquake and detailed survey or follow-up visits are then arranged as appropriate.

EEFIT was formed in 1982 as a joint venture between universities and industry. It has the support of the Institution of Civil Engineers through its society SECED (the British section of the International Association for Earthquake Engineering), and of the Institution of Structural Engineers. It is advised by a number of British engineers experienced in the field of earthquake engineering. Funding for its missions has come from the Science and Engineering Research Council and other research and industrial sources.

EEFIT has investigated earthquakes in Liège, Belgium (1983), Chile (1985), Mexico (1985), San Salvador (1986), Loma Prieta, California (1989), Newcastle, Australia (1989), Romania (1990), Manjil, Iran (1990), Luzon, Philippines (1990), Sicily (1990), Erzincan, Turkey (1992) and Roermond, Holland (1992). EEFIT reports are available or in preparation for all these events and can be obtained from the secretary of EEFIT at the address on the back cover.

2. THE AFFECTED AREA

2.1 Location and Topography

Erzincan is situated in eastern Turkey, approximately 600 km due east of Ankara. The prefecture of Erzincan is a highly mountainous region with an area of 11,900 km^2. The city sits in a basin measuring 50 km east-west by 15 km north-south, at an altitude of 1200 m above sea level, surrounded on all sides by mountains rising to heights in excess of 3000 m. Originally Erzincan was situated close to the River Euphrates, but after being devastated by an earthquake in 1939, the city was completely rebuilt a few kilometres to the north of the railway. Figure 2.1 shows the topography of the Erzincan region, indicating the major population centres, roads, railways and rivers. The 1500 m contour corresponds approximately to the edge of the basin. Figure 2.2 shows a street plan of the city, which has a modern, grid-iron layout, divided into 21 districts. The number of buildings in each district is shown in brackets. Winters in the region are severe, with heavy snows for around four months of the year and temperatures as low as -30°C. In March, at the time of the earthquake, there was still snow on the ground and temperatures were well below zero.

2.2 Population and Economy

Turkey's population is estimated to be around 57 million people, a population density of 71 people per km^2, with an expected growth rate of 1.6% per annum. The national economy grew by 5.5% annually between 1982 and 1990, though the actual per capita income growth was only about 3.5%. In 1988 Turkey's GDP was US\$72.5 billion, with an average per capita income of approximately \$1400 per annum (Economist, 1990). The immediate monetary loss from this earthquake, excluding the cost of rescue, emergency services and loss of business or employment, is estimated as \$0.4 billion (Bogaziçi University, 1992), equivalent to about 0.5% of GDP. A preliminary estimate by the Ministry of Public Works and Resettlement put the overall cost of the earthquake at \$6 billion.

The Erzincan prefecture has a population of 300,000, split roughly equally between urban and rural areas. The prefecture is divided into eight provinces, of which Erzincan and Uzumlu were the most severely affected by the earthquake. The demographic characteristics of these two provinces according to the 1990 population census are given in Table 2.1.

	Erzincan Province	Uzumlu Province
Population	149,837	29,859
Major town (pop.)	Erzincan (91,772)	Uzumlu (17,314)
No. of villages (pop.)	88 (58,065)	28 (12,275)
Area (km^2)	1756	410
Density (people/km^2)	85	72

Table 2.1 Population Distributions in Erzincan and Uzumlu Provinces

The local economy is based primarily on agriculture and animal husbandry, the Erzincan Basin being one the few fertile areas in a very barren region, and some associated industries (e.g. textiles, sugar processing, animal feed production), the largest of which employs 600 people. Local government and utilities also play a significant role in the economy. For example, the prefecture highways department employs 1500 and the PTT (telecommunications company) employs over 500. In addition, there is a substantial military presence in the area.

2.3 Geology and Ground Conditions

The principal geological feature of the affected area is the Erzincan Basin (Figure 2.3). Formation of the basin was initiated by a pull-apart motion between two divergent segments of the North Anatolian Fault; subsequently development is complex and incompletely understood (Barka and Gülen, 1989). The centre of the basin is filled with alluvial plain deposits, consisting of silts, sands and gravels. These comprise rather more loose material than the alluvial fan deposits which predominate around the edges of the basin. Prior to 1939 Erzincan was situated in the alluvial plain area, but following the 1939 earthquake the city was relocated in an area of alluvial fan deposits. The deepest recorded borehole in the basin, at 250 m, failed to reach the bottom of the sediments, whose thickness is thought to vary between 500 and 3500 m (Bogaziçi University, 1992).

2.4 Principal Building Types

The city of Erzincan was destroyed by a magnitude 7.9 earthquake in 1939 and has subsequently been completely rebuilt slightly north of its previous location. It therefore contains a large number of modern buildings and relatively few traditional ones. Plates 2.1 and 2.2 show views of the city centre prior to the earthquake (reproduced from the book "Erzincan '90," 1990). The predominant structural types in the city are in situ reinforced concrete (RC) frames of up to six storeys, mostly with unreinforced masonry infill panels; and low rise unreinforced brick masonry structures, often with an RC ring beam, a timber roof and clay tiles; some of the houses use an RC slab instead of a timber roof. Steel structures are extremely rare. There is a certain amount of lightweight timber frame housing, mostly single-storey buildings clad with wire mesh and plaster, which were erected very shortly after the 1939 earthquake, and also some traditional Turkish housing of the types described below. Figure 2.4 shows the approximate distribution of building types within the city.

In the rural areas there is a wide variety of traditional building types, covering virtually the whole spectrum of traditional Turkish housing. Among the most common are: *himis* houses, comprising single-storey timber frames with infill mostly of adobe bricks; rubble stone masonry houses with very heavy, flat roofs consisting of soil compacted on timber joists; and adobe brick houses, often with timber *hatils*, or tie-beams, running around their perimeter. Some of the more accessible villages now contain many more modern brick or concrete block masonry structures, with RC houses also increasing in number.

Figure 2.1 Topography of the Erzincan region

Map legend:

- Major road
- Railway
- River
- Major town
- Large village ●
- N. Anatolian Fault
- Provincial boundary

Labels on map: UZUMLU, Mertekli, Altinbasak, Suleymanli, Uzumlu Province, Erzincan Province, Eksisu, Mollakoy, Akyazi, Yogurtlu, Epicentre, Gecitkoy, ERZINCAN, Euphrates, Yaylabasi, Mecediyeh, Isikpinar, Hashasi, Erzincan (pre-1939), Ulalar, Cukurkuyu/Yalnizbag, Vartansah/Berkisor, Davarli, Calabzur/Cazanfer, Bahceli, Erzincan Province, Kemah Province

Contour labels: 2000, 1500, 1250, 1000

5 km scale

N

Figure 2.2 Street plan of the city of Erzincan, showing the number of buildings in each district

Figure 2.3 Simplified geological plan of the Erzincan Basin (after Ministry of Energy and Natural Resources, 1981)

Legend:
- Alluvial fan deposits
- Alluvial plain deposits
- Salt playa
- Volcanic rocks (early quaternary)
- Clay, sand, conglomerates (late tertiary)
- Limestone (late tertiary)
- Older rocks

N.E. Anatolian Fault

N. Anatolian Fault

Ovacik Fault

Uzumlu

Erzincan

N

0   10 km

Figure 2.4 Approximate distribution of building types in the city of Erzincan

Figure labels (as they appear in the map):

- 500 m
- N
- RC (1-4 storeys)
- RC prefab. (single storey)
- RC prefab. (single storey)
- Brick masonry (1-2 storeys)
- RC (1-4 storeys)
- Brick masonry (1-2 storeys)
- RC (1-4 storeys)
- RC (single storey)
- Mixed
- Timber prefab. (single storey)
- Adobe or brick masonry (single storey)
- RC (1-4 storeys)
- Mixed
- Timber prefab. (single storey)
- RC (3-6 storeys)
- Mixed
- Brick masonry (1-2 storeys)
- Brick masonry (1-2 storeys)
- Timber prefab. (single storey)
- Brick masonry (1-2 storeys)
- RC (1-4 storeys)
- Adobe or brick masonry (single storey)

Plate 2.1
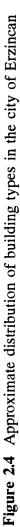Erzincan before the earthquake: the main east-west avenue, looking west. This road is part of the E23 national road linking Ankara and Sivas to the west with Erzurum and the Armenian border in the east. In the foreground on the left is the main bus terminal.

Plate 2.2
Erzincan before the earthquake: the city centre and the main north-south avenue, looking north. Many important buildings are located on or near this road, including the telecommunications centre and the state hospital.

3. SEISMOLOGY OF THE EARTHQUAKE

3.1 Tectonics of Turkey and the Erzincan Region

Turkey, one of the most seismically prone countries in the world, is part of the active tectonic area of the Middle East. The tectonics of this region are extremely complex, encompassing areas of very high seismicity such as the North Anatolian Fault, the Caucasian region and the Zagros Mountains in Iran, next to very inactive areas such as central Turkey and most of the Arabian peninsula. The main feature of this complex area is the northwards movement of the Arabian Plate, causing a large release of seismic energy along its border with the Eurasian plate.

Figure 3.1 shows the major fault systems in Turkey and the earthquakes of magnitude ≥ 6.0 occurring this century, compiled from Ambraseys (1988) and Barka and Kadinsky-Cade (1988). Due to the northwards movement of the Arabian plate the main body of Turkey (the Anatolian Block, bounded by the North and East Anatolian Faults) is forced to move westwards with a slight counterclockwise rotational motion, while the Northeast Anatolian Block is pulled eastwards and the south-eastern part of Turkey is compressed. Meanwhile, the western part of the country is indirectly affected by the movement of the African plate, relative to Eurasia. This complex behaviour has resulted in more than 60 earthquakes of magnitude 6.0 or greater this century, with the 1939 Erzincan earthquake of magnitude 7.9 being the largest.

As can be seen from Figure 3.1, many of the large earthquakes in Turkey this century have occurred along the North Anatolian Fault, one of the longest faults in the world. The fault extends for about 1500 km from the Karliova junction in eastern Turkey to the Aegean coast. Barka and Kadinsky-Cade (1988) suggest that the total relative displacement varies from 40 km in the highly active region around Erzincan, to 15 km in the Sea of Marmara. A number of westward-migrating sequences of earthquakes have occurred along the fault, most recently between 1939 and 1967, starting from Erzincan and reaching the region of Bolu in the west.

The area around Erzincan is illustrated in more detail in Figure 3.2, showing the junctions of the North Anatolian Fault with the Northeast Anatolian Fault at the north-west end of the basin and with the Ovacik Fault to the south-east. This region has suffered major damaging earthquakes for thousands of years, eighteen in the last millennium having peak intensities of VIII or greater (Bogaziçi University, 1992). The most recent of these was the December 1939 earthquake of magnitude 7.9, which ruptured a zone of about 240 km, starting from the Erzincan basin and extending westwards. The maximum slippage reached 7 m near the epicentre (about 15 km north-west of Erzincan), decreasing to less than 2 m at the western end of the fault rupture. The earthquake caused an estimated 33,000 deaths and the destruction of 140,000 homes.

In the segment of the North Anatolian Fault that extends south-east from Erzincan towards Karliova, there have been three damaging earthquakes this century, in 1949 (Yedisu; M_s = 6.8), 1966 (Bingol; M_s = 6.8) and 1967 (Pulumur; M_s = 6.0). More recently, a small (M_s = 4.8) earthquake struck Erzincan in November 1983, causing some minor structural damage.

3.2 Magnitude and Epicentral Location

The 1992 Erzincan earthquake was caused by the right-lateral strike-slip movement of the North Anatolian Fault. As can be seen in Figure 3.2, its epicentre was close to that of the 1939 event, and very near to the city of Erzincan. The physical parameters of this earthquake, along with the largest aftershock which followed 46 hours later, are summarised in Table 3.1.

	Main Shock	Main Aftershock
Occurrence date	Fri. 13 March 1992	Sun. 15 March 1992
Occurrence time (local)	19:18:39.9	18:16:24.2
Magnitude	$M_s = 6.8$ $(m_b = 6.2)$	$M_s = 5.8$ $(m_b = 5.5)$
Focal depth	27 km	21 km
Maximum reported intensity	IX	VIII
Preliminary epicentre coordinates	39.71° N, 39.61° E	39.53° N, 39.93° E

Table 3.1 Physical Parameters of the 1992 Erzincan Earthquake and Main Aftershock (source: NEIC monthly listings, US Geological Survey)

3.3 Strong Motion Parameters

Strong ground motions were recorded by a Kinemetrics SMA-1 accelerometer located in the meteorology station, about 0.75 km north-west of Erzincan city centre and an estimated 5 km from the epicentre. The station, a well built two storey RC frame house, was completely undamaged during the earthquake. The accelerometer was installed in the northern part of the ground floor and bolted to the floor. According to discussions with local and foreign experts, it is unlikely that the instrument had any malfunction during the earthquake.

Digitised acceleration histories for the main shock were provided by the Turkish Directorate of Disaster Affairs; these are shown in Figure 3.3, while acceleration, velocity and displacement response spectra for 5% damping are plotted in Figure 3.4. The east-west component is the strongest, with a maximum acceleration of $0.5g$. The spectrum for this record shows peaks at periods of approximately 0.2, 0.3 and 0.7 seconds, with the 0.3 second component predominant. The peak spectral amplification factor for 5% damping is 2.70, comparable with those observed during many Californian earthquakes. The north-south record is slightly less strong, and its strongest cycle is at a noticeably lower frequency than the rest of the signal, giving spectral peaks of roughly equal magnitude at periods of 0.3 and 0.95 seconds.

Table 3.2 summarises the main ground motion parameters. In addition to the peak values from the strong motion record, a range of other parameters are shown, including RMS acceleration, peak and mean response spectral accelerations and released energy. These will be referred to in Section 6.2, when correlations between strong motion parameters and observed damage are discussed.

Strong Motion Parameter	N-S	E-W	Vert.
Peak Ground Accn. (g)	0.390	0.492	0.244
Peak Ground Velocity (cm/s)	108.5	78.1	24.7
RMS Acceleration (g)	0.100	0.109	0.059
Energy (m^2/s^3)	10.01	11.83	3.93
Significant duration (s)	9.1	9.0	11.4
Duration > 0.05g (s)	8.0	8.0	8.0
Duration > 0.1g (s)	4.7	4.7	7.4
Predominant period (s)	0.95	0.31	0.16
PRSA (g)	0.905	1.605	0.690
MRSA between 0.1 and 0.3 s (g)	0.655	1.064	0.533
MRSA between 0.3 and 0.5 s (g)	0.783	1.064	0.495

Table 3.2 Peak Ground Motion Values Recorded at Erzincan Meteorology Station

Notes on Table 3.2:
- *RMS Acceleration* is the root mean square acceleration over the significant duration of the strong motion.
- *Energy* released is defined as the integral of the acceleration squared.
- *Significant duration* is the time over which 90% of the record's energy was released.
- *PRSA* is the peak response spectral acceleration at 5% damping.
- *MRSA* is the mean response spectral acceleration at 5% damping in the period range shown; different ranges correspond to different building heights.

In order to give a better idea of the magnitude of the ground motion, some parameters of the Erzincan record are compared with those of several other near-field earthquakes in Table 3.3 (data from Hudson, 1988). In most respects the Erzincan record appears unexceptional for a near field earthquake, but the peak velocity of the record is extremely strong, one of the highest peak horizontal velocities ever recorded.

In addition to the motions recorded in Erzincan, the earthquake also triggered instruments in Tercan, 80 km to the east, and Refahiye, 65 km west of Erzincan. These records are shown in Figures 3.5 and 3.6 respectively. The magnitudes of the accelerations recorded at Tercan are very small and the reliability of this record must be open to question.

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Region	Gazli	Tabas	Montenegro	Michoacan	Nahanni	San Salvador	Erzincan
Country	Uzbekistan	Iran	Yugoslavia	Mexico	Canada	El Salvador	Turkey
Year	1976	1978	1979	1985	1985	1986	1992
Fault dist (km)	10	3	28	0	0	4	5
Mechanism	Reverse	Thrust	Thrust	Strike-Slip	Reverse (?)	Thrust	Strike-Slip.
Magnitude, M_s	6.8	7.4	7.0	8.0	6.8	5.6	6.8
MPHGA (g)	0.62	0.86	0.39	0.21	0.91	0.57	0.45
PVGA (g)	1.22	0.61	0.21	0.13	1.76	0.40	0.25
MPHGV (cm/s)	62.2	110.1	32.6	21.3	43.1	70.9	93.3
Sig. Dur. (s)	7.1	17.5	12.5	45.0	8.0	3.9	9.1
RMSA (g)	0.22	0.35	0.18	0.06	0.38	0.47	0.11
Energy (m^2/s^3)	57	136	42	90	50	26	22

Table 3.3   Comparison of the Erzincan Parameters with some Well-Known Near-Field Earthquake Records

Notes on Table 3.3:
- *MPHGA* is the mean of the two peak horizontal ground accelerations.
- *PVGA* is the peak vertical ground acceleration.
- *MPHGV* is the mean of the two peak horizontal ground velocities.
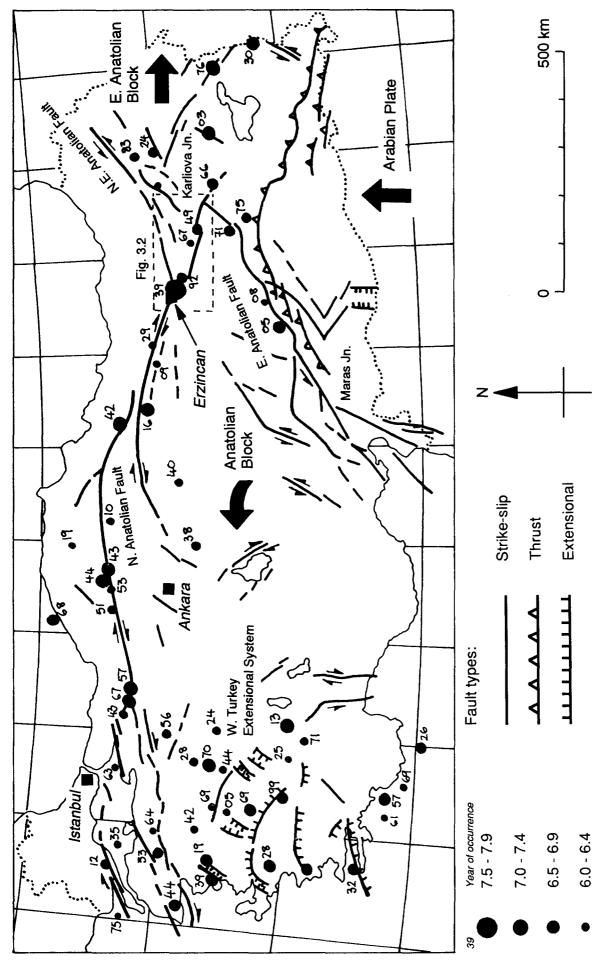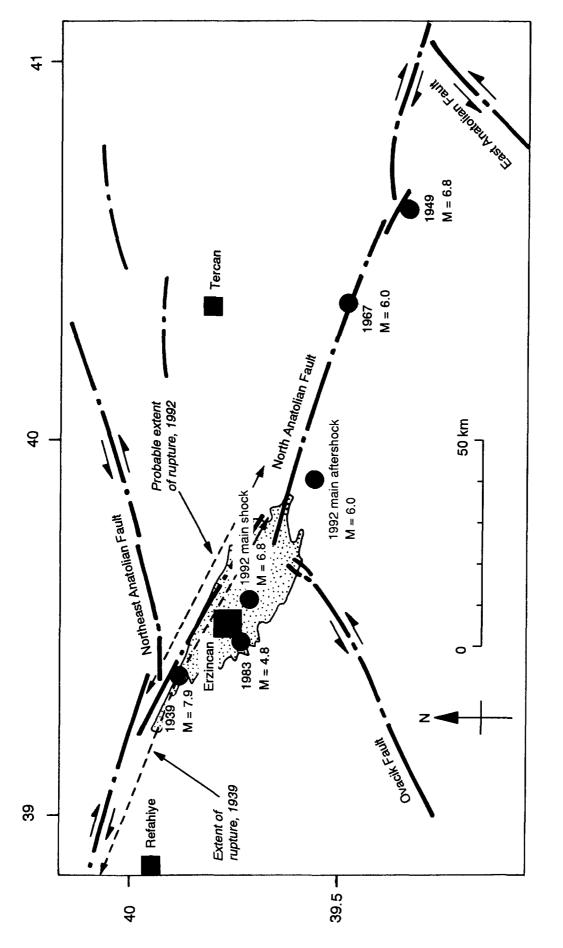- The energy quoted here is the sum of the two horizontal values.

Figure 3.1 Major fault systems of Turkey, with earthquakes of magnitude greater than 6.0, 1899-1992 (after Ambraseys, 1988, Barka and Kadinsky-Cade, 1988)
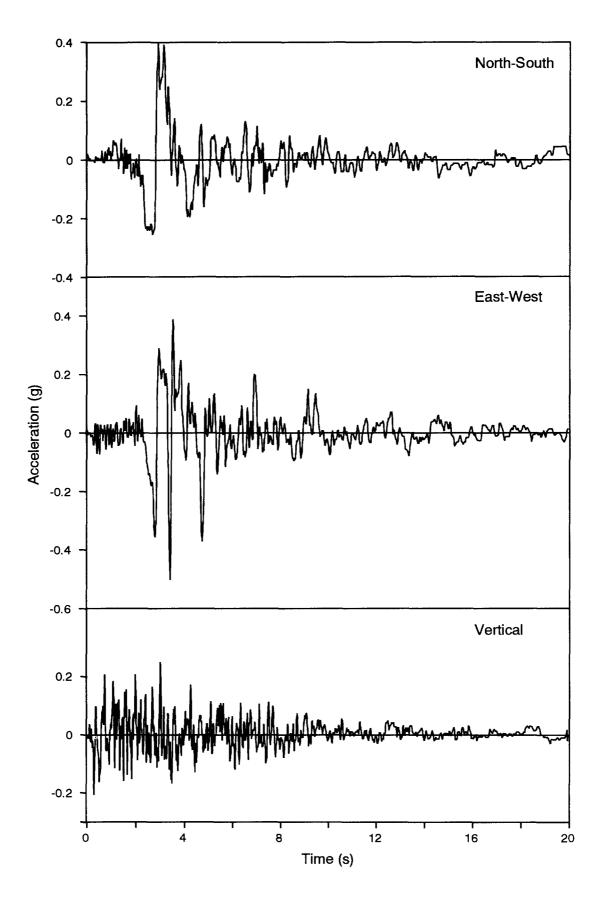
Figure 3.2 Fault systems and significant recent earthquakes in the Erzincan region

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Figure 3.3 Acceleration time histories for the main shock (Erzincan station)
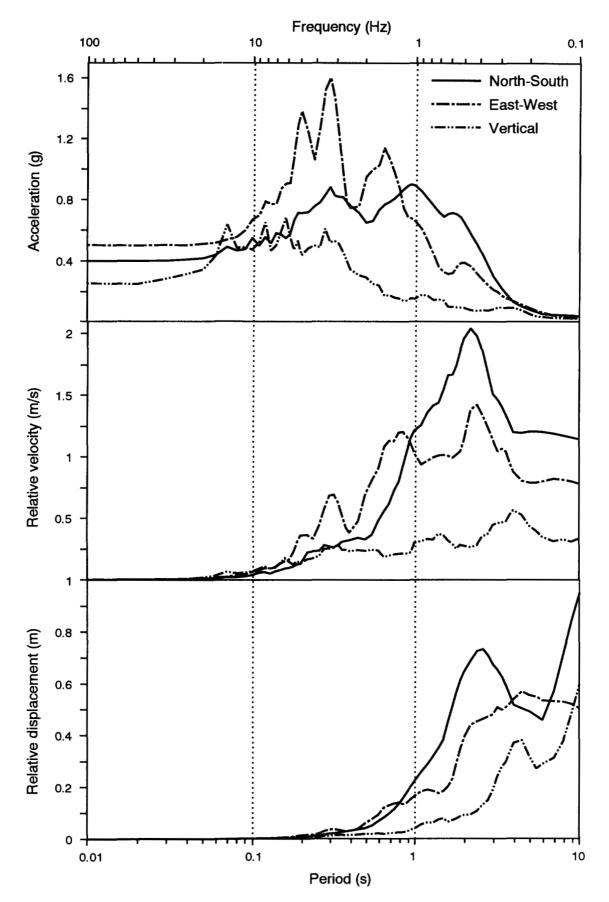
Figure 3.4 Acceleration, velocity and displacement response spectra (5% damping) for the main shock
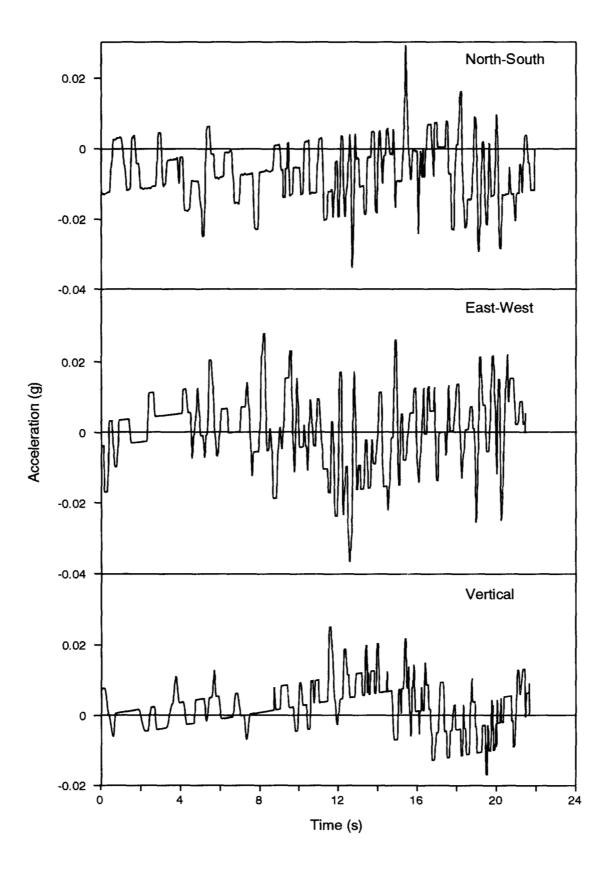
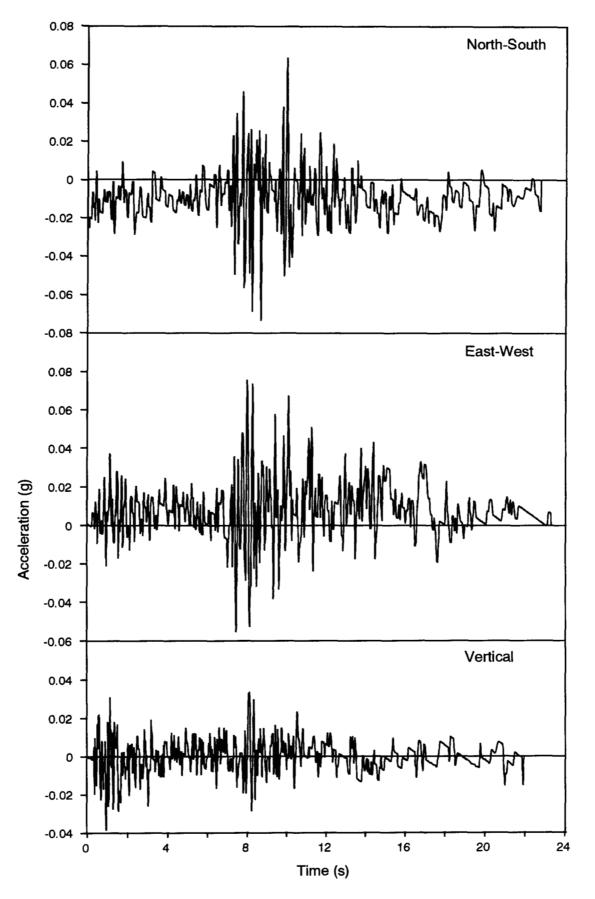**Figure 3.5** Acceleration time histories for the main shock (Tercan station)

Figure 3.6 Acceleration time histories for the main shock (Refahiye station)

4. TYPES OF DAMAGE

In this chapter the principal modes of failure observed in buildings and civil engineering works in and around Erzincan are discussed with brief reference to examples. Fuller case studies of individual structures are presented in Chapter 5. The locations of most of the structures in Erzincan referred to in Chapters 4 and 5 are identified in Figure 4.1. Structures located in the surrounding villages can be located from Figure 2.1.

Early reports suggested that nearly all the serious damage in Erzincan city was to RC structures, but the EEFIT team also observed significant damage levels in brick masonry buildings. Low-rise timber frame houses suffered relatively little damage. Of the traditional housing, timber framed *himis* houses were damaged but usually possessed sufficient ductility to remain standing, while rubble stone or adobe houses suffered more severe damage.

4.1 Reinforced Concrete Structures

4.1.1 Mid-rise buildings

The building type that suffered the most damage was mid-rise (three to six storeys) RC framed structures (residential, office, schools, hospitals, industrial and public buildings). The EEFIT team arrived in Erzincan sixteen days after the earthquake, by which time many of the completely collapsed structures had already been cleared. Nevertheless there were numerous uncleared sites and partially collapsed buildings from which a lot of information could be obtained. Additionally, much information on the cleared buildings was provided by Professor Çetin Yilmaz of the Middle East Technical University.

The most common faults in design and construction were:

- stiffness discontinuities, mainly in the form of soft storeys at ground floor level (Plate 4.1);
- weak column-strong beam design (Plate 4.1);
- frames distributed in only one horizontal direction, or being very weak in one of the horizontal directions; lack of structural redundancy (Plate 4.2);
- inadequate number and size of stirrups, and in some instances complete lack of confining reinforcement (Plate 4.3);
- short columns, especially in buildings with semi-basement and industrial structures with windows spreading between columns, and also in some hospital buildings (Plates 4.3);
- lack of uniformity between openings from floor to floor or from side to side of a building; the most striking examples were the collapsed nursing school in the state hospital and the neighbouring operations clinic, which was severely damaged (see Section 5.1);
- insufficient cover to reinforcement - rusted reinforcing bars were frequently visible, especially near crucial joints; this deficiency was noted in both damaged and intact structures, e.g. Plate 4.4;
- lack of horizontal load-resisting systems such as shear walls, which were used only in a few of the major structures. Plate 4.5 shows a typical RC frame under construction, without any shear walls;
- mass eccentricities in plan, such as the inclusion of an additional storey over only half of the plan area (Plate 4.6);

- poor quality concrete; honeycombed or segregated concrete, with rounded aggregates, was frequently observed. According to local engineers, the average compressive strength of concrete used in the region was only 10 N/mm^2 (Bogaziçi University, 1992). This is less than half the minimum strength of 23 N/mm^2 required both by the Turkish code and by the Uniform Building Code for buildings in highly seismic zones (UBC, 1991).
- there was some evidence of inadequate repair to minor structural damage caused by the 1983 earthquake (Plate 4.7). While the team did not see any instances of further damage to inadequately repaired buildings, this is obviously undesirable.

The types of structural collapse observed can be split into three main categories:

- Most of the collapses started from the ground floor (*bottom-top* collapse). In the worst cases this collapse mode can cause the complete failure of the structure above, leaving very few voids and hence only a small chance of survival for its occupants (*pancake* collapse). The cause of this type of collapse is usually either a soft ground storey (either left open for parking or used for shops, with very few infill walls) or the existence of a series of short columns at semi-basement level. Many of the collapsed buildings had completely lost their first two floors, but preserved an increasing volume of voids in the upper storeys. Complete pancake collapses were relatively few (Plates 4.8 and 4.9).

- Cases of *torsional failure* were also observed, especially in corner buildings. This mode may be caused by: a significant number of openings in street facing facades; eccentric positioning of staircases; variations in the amount of openings from floor to floor; or a combination of these factors (Plate 4.6).

- *Mid-storey* collapses, with the bottom and top floors surviving, were observed in just a few cases. Plate 4.10 shows a large building that lost its second floor, the floors above and below standing relatively intact. This collapse may be due to pounding between adjacent structures with different dynamic characteristics and different floor levels, or due to a short column effect.

Only one instance was noted of a building in which the top storey had collapsed but the others had remained standing (Plate 4.11). Top-down collapse usually involves structures of more than six storeys, very few of which have been built in Erzincan (Pomonis, 1992). No instances of basement collapse were observed.

In addition to collapses, a very large number of buildings suffered out-of-plane failure of cladding. While this does not cause collapse and is repairable, it creates a considerable hazard for people who have evacuated buildings, it is costly to repair and it causes a temporary loss of use of the building.

Not all reinforced concrete buildings were structurally damaged. Three examples of RC buildings that performed well are discussed below.

(a) Government apartment block, Inonu district (Location 7)
An apartment block was situated next to the strong motion instrument in Erzincan (plate 4.12). It was used as housing for government employees. It is understood the structure was designed by a government department in Ankara and was completed in 1990.

The apartments consisted of three storeys and a basement. The building was regular in plan with two pairs of apartments at each level to either side of a central stair core. The superstructure consisted of a reinforced concrete frame with blockwork infill while the basement exterior walls appeared to be in solid concrete.

The building was inspected both internally and externally. The internal inspection revealed the following. No damage was observed or reported in the basement. Access could not be gained to the ground floor apartments, but no serious damage was reported. Minor cracking in the plaster could be seen at the junction between infill and frame at first floor level. Externally, there was evidence that the western gable end was starting to separate at roof level, a near universal condition in Erzincan. Some separation of infill and frame could be seen at ground floor level at one location, but this did not appear serious. There was clear evidence of relative movement between the ground floor slab and surrounding backfill.

Overall, the apartment block had survived the earthquake in very good condition. The only obvious design defect was the lack of adequate restraint to infill blockwork at one gable end, noted above. The block was within the area of the detailed survey by EEFIT which established an intensity (based on the performance of single storey traditional houses) of about VIII; moreover, the 5% damped spectral acceleration measured at the adjacent strong motion instrument was 1.0 to 1.5g in the period range 0.2 to 0.3 seconds (the likely first period range of the building). It is therefore clear that the building was strongly excited by the earthquake.

(b) New Town Hall, Inonu district (Location 8)
This building was structurally complete but unoccupied. Finishes were present and most of the fitting out was complete. Figure 4.2 shows a sketch ground plan of the Town Hall and Plates 4.13 and 4.14 give views. Generally, a regular beam and column frame was provided, with main rectangular blocks split by expansion joints. The auditorium (Plate 4.14) was however irregular in elevation.

No structural damage was noted anywhere in the building. However, there was extensive cracking of finishes at the joints between infill and frame, and in the auditorium the infill showed signs of separating completely from the frame. At the time of EEFIT's inspection, plaster finishes were being hacked off and replaced and it was apparently the intention to open the building for occupation in the near future without structural modification.

(c) Concrete arch building, Ataturk district (Location 9)
A sports hall building (Plate 4.15) consisted of concrete arches 800mm by 300mm in cross section with a span of about 50m and a rise of about 6m. The concrete structure appeared undamaged though there was some spalling in the render to the arches, some window glasses had cracked and there was more serious cracking in the block end gable wall.

4.1.2 Low-rise RC housing
Reinforced concrete was occasionally used for low-rise housing in Erzincan and in the more accessible surrounding villages. On the whole, the behaviour of RC houses was good, with damage limited to cracking of infill panels. However, a number of complete collapses were observed, mainly in buildings having a soft ground floor used for storage or parking, e.g. Plate 4.16.

A four-storey RC framed house in the village of Ulalar that collapsed completely was surveyed by the team. As can be seen in Plate 4.17, the building lost its ground floor and most of its first floor. The top storey covered only half the area of the lower floors, the remainder being used as a balcony. This created a sizeable mass eccentricity. Further problems were caused by a soft ground floor, which was open for parking and was slightly higher than the floors above, and a frame much stiffer on its east-west than on its north-south axis.

4.2 Masonry Buildings

While a large number of brick or concrete block masonry buildings were damaged by the earthquake, there were relatively few total collapses. The number of failure modes observed in masonry buildings was much fewer than in RC structures. Many houses suffered out-of-plane failures of one or more walls, though frequently the presence of an RC ring beam prevented complete collapse. Severe shear cracking initiated at window and door openings was also common, though again this rarely led to complete structural failure. A detailed study of the behaviour of brick masonry houses in Uzumlu is given in Section 5.4.

Plate 4.18 shows the damage to the school building in the village of Mecediyeh, about 15 km north of Erzincan. This was a single storeyed stone masonry structure with cement mortar, RC ring beam and timber framed roof clad with clay tiles. The northern gable end was made out of large and irregularly shaped stones, with inferior mortar. The gable end, due to its heavy weight, fell first, inducing the collapse of the rest of the wall and the timber roof structure.

4.3 Timber Houses

The imported, lightweight timber houses which were constructed in the immediate aftermath of the 1939 earthquake performed very well, suffering only moderate damage at worst, and in most instances incurring only light, superficial damage.

4.4 Traditional Housing

The EEFIT team spent a considerable amount of time surveying villages in the Erzincan region. While damage levels varied considerably from village to village (see Section 6.3), it is possible to draw the following general conclusions about the performance of the various structural types.

There were a large number of adobe brick houses, often with timber hatils (Plate 4.19) or an RC ring beam, and lightweight timber roofs. These generally fared quite badly in the earthquake, with damage due to shear (Plate 4.20) frequently leading to complete collapse (Plate 4.21). In houses where the roof joists protruded well beyond the adobe wall or were supported by timber props just inside the walls, cases of total collapse were much reduced.

An older form of construction uses adobe bricks or rubble stone walls, with a very heavy, flat roof made of soil compacted onto timber joists. In some villages these are still used for human habitation, but in many instances they now house livestock. These buildings also suffered very high levels of damage in many villages.

The most successful of the traditional construction types were the himis houses, with timber frames and adobe brick infills. Many of these were severely shaken, causing spalling of plaster, revealing the infill adobe blocks. On occasions himis houses were observed leaning badly (Plates 4.22, 4.23), but complete collapse was comparatively rare, due to the flexibility of the timber frame.

4.5 Performance of Gable End Walls

Failures of gable end walls were widespread. Three types of gable end were noted; their performance was quite different. Where the gable end extended vertically to eaves level without lateral restraint (Plate 4.24) failure was almost universal, probably exceeding 80%. Vertical gables which were restrained by the roof purlins (Plate 4.25) suffered substantially less damage, though cracking around the purlin can be seen in the plate. The third type of gable end was provided with a pitched end to the roof which bore onto the end wall; no failures were observed with this arrangement.

4.6 Bridges

The small number of bridges in the area generally suffered little damage, the exception being one concrete slab highway bridge on the road to Kemah, to the south-west of the city (Plates 4.26-4.28), which suffered subsidence and slip of embankment fill material and serious cracking to the embankment walls, which pushed out onto the adjacent columns causing damage to column heads and relative movement between deck segments. Inspection revealed poor construction details such as the lack of any bearings at the column heads. There was also evidence that previous subsidence had occurred and been improperly repaired, suggesting that not all the damage was due to the earthquake. This bridge was closed to traffic. The next bridge along this road suffered rather less damage, with some spalling at the crossheads, while a bridge approximately 15 km from the centre of Erzincan suffered no visible damage.

The main road bridge over the River Euphrates directly south of Erzincan is shown in Plate 4.29. It was a concrete bridge, apparently in good condition, with one river span and two abutment spans. The river span had a central portion supported on halving joints. One abutment supported its deck by a simple steel roller bearing; the other was fixed. A separate pipebridge ran adjacent to the road bridge. No damage or evidence of movement could be seen on either bridge, except for some minor soil movements at the north abutment of the pipebridge. The bridges are in an area of softer fluvial deposits just south of the original site of Erzincan before its destruction in the 1939 earthquake. The absence of damage on such a potentially dangerous site is noteworthy.

A four span concrete bridge takes the main north south road through Erzincan over the railway, at the southern end of town (Plate 4.30). Simple sliding joints were provided at the tops of the pier crossheads and at the north abutment, with the south abutment fixed. A timber and polystyrene vertical layer a few centimetres thick separated the bridge deck from the north abutment. There were no signs of movements at the abutments, but the top and bottom of 3 out of 4 of the columns on the central pier had vertical cracks up to 2mm wide. The north and south piers seemed undamaged. The bridge was open to unrestricted traffic.

4.7 Roads, Embankments and Other Geotechnical Aspects

There are two major traffic routes through the basin, the main highway from Ankara in the west to Erzurum in the east (the only main road in/out of the basin), and the associated rail route. The highway has to climb several hundred metres at either end of the basin and relies on following the natural terrain, cut into the hill side, with steep embankments both above and below. Once on the basin floor both are carried on embankments until the outskirts of the city.

The road system suffered relatively few problems in the earthquake. Inspection of the main highway coming over the western pass showed some recent falls onto the road. However, the cut and natural slope in the area are extremely steep, with no reinforcement or netting to stabilise the slopes or contain any loose scree. While the recent rock falls may have been triggered by the earthquake, it is likely that such events occur periodically anyway. More worrying are the steep highway embankments, which showed signs of damage due to the regular traffic of heavy goods vehicles. In sections the road surface had longitudinal cracks extending to the centre of the carriageway, with voids in the sub base, extending down the embankment slope. These problems may have been the result of dynamic compaction of granular fill material due to the earthquake.

An embankment failure (Plate 4.31) was noted in a road about 0.5 km east of Eksisu along the edge of the basin. The embankment material comprised fill about 6m high; an approximate sketch section is shown in Figure 4.3. The area is noted on a hydrogeological map dated 1981 as being a zone of springs, and is on the boundary between marshy ground and rock at the edge of the basin. The failure is likely to have been associated with liquefaction.

At Eksisu, where a spring is tapped commercially, a line of cracking in the soil was noted extending about 100m along a filled area at the edge of the basin forming a car park. The line of cracking was in the direction and supposed position of the Northern Anatolian Fault. The cracks were a millimetre or so wide but showed no lateral displacement. A similar line of cracking in soft soil was noted over a much longer length on high ground just above Davarli, again in the general location and orientation of the North Anatolian fault. Barka (Bogaziçi University, 1992) has associated this cracking with the underlying tectonic fault movements. No surface expressions of the fault with significant lateral movements were observed by the EEFIT team and it is understood that none have been found by others.

The airport on the eastern edge of the city was operational very shortly after the earthquake, implying that the runway was undamaged.

Most building foundations consisted of spread footings of about 3 m depth into stiff, sandy soils. No damage to foundations was observed by the team. One failure was reported where a building's foundations were undermined by an open excavation for an adjacent building under construction.

4.8 Industrial and Telecommunications Facilities

While there were no catastrophic structural collapses, many industrial facilities suffered sufficient damage to put them out of action for several months, a significant blow to the recovery of the local economy. As well as structural problems, there were many instances of machinery shifting during the earthquake due to inadequate fixity at the supports, though in most cases the equipment seems to have survived without serious damage.

Of the RC structures, the Sumerbank Textile Factory (discussed in more detail in Section 5.2) suffered severe damage to many beam-column joints and shifting of machinery from its supports. The management suggested that the factory would be closed for at least three months while undergoing repairs. Even more serious damage due to inadequate joint strength was reported at the flour mill to the west of the city, probably requiring complete demolition. In this case poor floor connections caused major damage to the internal machinery. A large meat processing plant just south of the River Euphrates consisted of a range of RC buildings (offices, warehouses, cold stores etc.), of which only the cattle sheds suffered severe damage (Plate 4.22). Since these were extremely poorly constructed RC frames, with honeycombed concrete and exposed, corroded reinforcing bars much in evidence (Plate 4.4), the high level of damage was not surprising.

The Erzincan Sugar Factory, on the southern edge of the city, is the only multi-storey steel framed structure in the city. The basic structure stood up to the earthquake well, though there was some secondary damage due to the collapse of a 36 m high storage silo (Plate 4.23). There was also substantial non-structural damage to windows and masonry infill panels, many of which failed out-of-plane. The mechanical equipment was undamaged.

The team visited a range of buildings owned by the PTT, the Turkish telecommunications company, most of which had suffered little or no structural damage. The main communications tower (Location 12) survived intact, as did the modern digital exchange equipment. However, there were some problems due to shifting of the older, mechanical switching racks. Conflicting accounts make it hard to gauge the severity of these problems, but it seems that local telephone lines were inoperative for about a day after the earthquake, with long-distance links not restored for three days.

4.9 Earthquake Resistant Design Procedures

4.9.1 Description of Turkish Earthquake Code

The current Turkish earthquake code (Turkish Government, 1975) is dated July 1975. It provides for an equivalent static calculation of lateral forces, on similar lines to chapter 23 of the Californian Uniform Building Code (UBC, 1991). Thus, the lateral strength of a structure is calculated as a function of the seismic zone of the site, the natural period of the structure, the building type, the importance of the building and the nature of the foundation soils. Ductile moment resisting frames require the least lateral strength while bearing wall or "box systems" and braced steel frames require the most. Unusually, the lateral resistance of moment frames depends on the nature of the partition walls; buildings with vertically and horizontally reinforced walls require a lower lateral strength than those with unreinforced, prefabricated concrete or sparse partition walls. The seismic base shear for a low rise moment frame with reinforced walls of "normal importance" in the most seismic zone (which

includes Erzincan) is 6% of the weight, expressed as a working load, equivalent to about 10% as an ultimate load. By comparison, UBC requires a working base shear of 9.2% for low rise ductile moment frames in the most seismic zone of the USA. The Turkish code requirement increases to 16% working (about 22% ultimate) for the least ductile structure. Buildings taller than 75m or with an irregular load bearing system must be designed using an "appropriate and rigorous dynamic analysis" for which no further advice is given.

Separate chapters are given for concrete, steel, timber, masonry (with and without concrete floors) and adobe buildings. These are now briefly discussed.

(a) Concrete buildings
The provisions for concrete structures, though less rigorous than current US or New Zealand requirements, are nevertheless quite comprehensive. They require ductile detailing of beams, columns and beam-columns joints which are not out of line with current international standards. Some detailing rules are given for infill walls. Storey drifts (relative deflections between storeys) must be restricted to 1/400 times the storey height, under code specified forces, unless special provision is made to prevent deflection induced damage to non-structural attachments. This is a somewhat more stringent requirement than the current UBC limit on drift.

The main features not included which are currently contained in many seismic codes for concrete structures are as follows.

(i) A requirement for the flexural strength of a column in a ductile moment frame to exceed the strength of beams framing into it.

(ii) A requirement for shear strength of frame members to exceed shear corresponding to plastic hinge formation.

(iii) A requirement for transverse confining steel at the highly stressed edges of ductile shear walls.

(b) Steel structures
The requirements are minimal, covering such matters as minimum slenderness of bracing members and connections between partitions and frames. They are similar to the UBC requirements extant in 1975, although current international requirements for steel buildings (including those of the UBC) are much more comprehensive. In particular, current standards required that columns must be stronger than the beams framing into them and connections must develop the strength of the members they connect. Current standards additionally specify more stringent measures for column-beam joint zones and for bracing members in braced frames.

It is understood that draft revisions to the steel provisions of the Turkish seismic code are in preparation, but they have not yet been adopted.

(c) Masonry structures

Detailing rules are given for the minimum thickness of wall, which in the superstructure must exceed 500mm for stone walls, 300mm for block and 1 brick length for brick. The minimum strengths of stone, block, brick and mortar are specified. There is a general requirement to maximise horizontal and vertical symmetry. Rules are given on the maximum size of door and window opening and the associated requirements for lintels. There is a height limitation of 2 storeys above ground in the most seismic zone, which includes Erzincan.

(d) Adobe buildings

These are restricted to single storey buildings not higher than 2.7m. The walls above ground must be 450mm thick. The requirements for masonry generally apply in addition.

(e) Timber buildings

Some fairly simple detailing rules are given for timber buildings, which are not allowed to exceed 2 storeys above ground.

4.9.2 Discussion of Turkish Code Provisions in Relation to Damage at Erzincan

As discussed above, the 1975 Turkish code provides comprehensive design rules for simple buildings which, at any rate for low rise buildings, are considered adequate to provide a considerable degree of seismic resistance. Some of the provisions need updating to conform to more recent standards and the level of lateral strength requirement is somewhat below that currently adopted in California for similar levels of seismicity. However, there was no evidence that code compliant structures failed in Erzincan because of this. On the contrary, there is a great deal of evidence that lack of conformity to the 1975 Turkish code in concrete structures led to serious damage or collapse. Particular features that can be cited are inadequate transverse steel in columns, beams and beam-column joints, inadequate anchorage of steel, low concrete strength and inadequate design of infill walls.
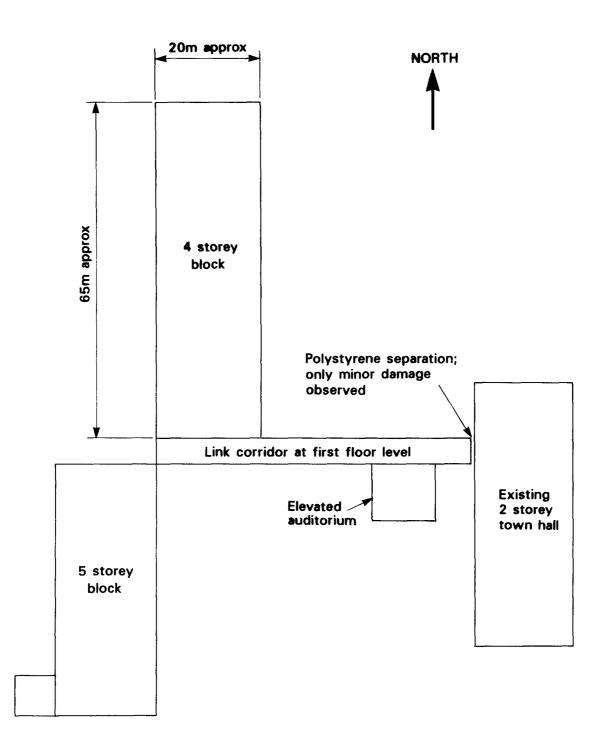
Figure 4.1 Locations of structures illustrated in the plates of Chapters 4 and 5

Figure 4.2 Sketch plan of new town hall, Erzincan. *Location 8.*
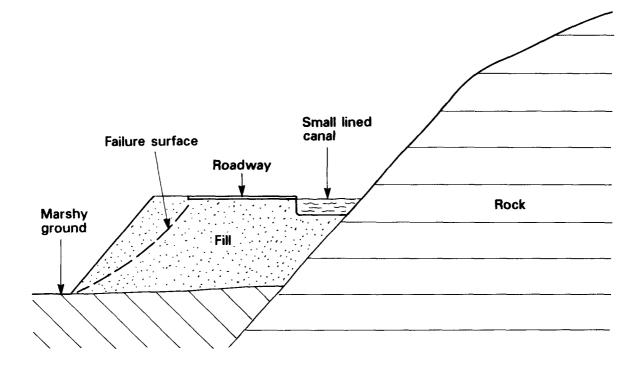
Figure 4.3 Sketch section through road embankment failure near Eksisu.

Plate 4.1
Detail of three storey RC frame apartment building with masonry infill panels and a soft storey at ground floor level, still under construction. Note the poor reinforcement detailing and the slenderness of the column in comparison with the perimeter beam. A similar building which collapsed completely can be seen in the background. *Location 1.*

Plate 4.2
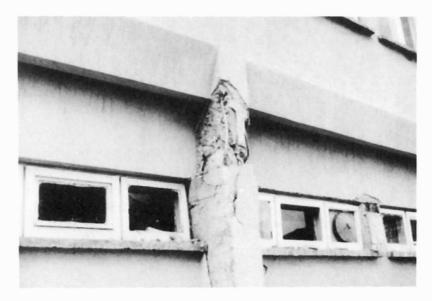Detail from a collapsed RC building in which the columns were distributed in one direction only. *Location 2.*

Plate 4.3
Inadequate column reinforcement. The longitudinal bars are made of mild steel and there is no lateral reinforcement. This plate also illustrates the short column effect.

Plate 4.4
Roof of cattle shed at meat processing plant. Insufficient cover and poor compaction has resulted in corrosion of the reinforcement. Other sections of this roof collapsed completely (see Plate 4.22).

Plate 4.5
Typical RC frame under construction, showing lack of shear walls. Numerous such buildings were under construction in the city at the time of the earthquake. Generally, those which had not yet been infilled suffered less damage, due to the lower total weight of the structure. *Location 3.*

Plate 4.6
Four storey RC frame which has lost its bottom two floors. Torsional loads are introduced by the top floor covering only half the plan area of the structure, and the staircase located at the back corner. *Location: Ulalar.*

Plate 4.7
Poorly repaired structure, the original damage probably caused by 1983 earthquake. *Location 4.*

Plate 4.8
Bottom-top collapse, starting from ground floor level, with all the floors above surviving. Notice the detached entrance staircase.

Plate 4.9
Bottom-top collapse of four storey RC frame with double basement. The structure has split at a movement joint, with the near half suffering a complete pancake collapse, while in the other half the top two storeys have survived. *Location 5.*

Plate 4.10
Mid-storey collapse of a
five storey apartment
building.

Plate 4.11
Building with top floor
collapse. *Location 6.*

Plate 4.12
Government apartment
block, next to strong motion
instrument. *Location 7.*

Plate 4.13
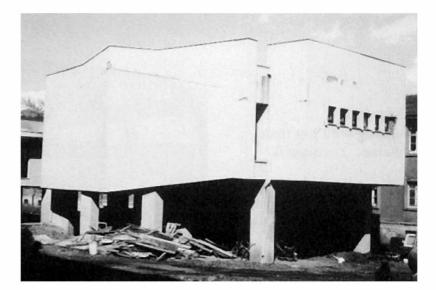New town hall, Erzincan.
Location 8.

Plate 4.14
New town hall, Erzincan,
the elevated auditorium.
Location 8.

Plate 4.15
Concrete arch structure.
Location 9.

Plate 4.16
Collapsed two storey RC frame with basement and ground floor used for storage, without most of the infill. *Location: Çukurkuyu.*

Plate 4.17
Collapsed four storey RC frame building. The ground floor was a soft storey used for parking (see also Plate 4.6). *Location: Ulalar.*

Plate 4.18
Roof collapse of a single storey stone masonry school building. *Location: Mecediyeh.*

Plate 4.19
Traditional adobe house reinforced by timber hatils at the top and bottom of the windows. *Location: Altinbasak.*

Plate 4.20
Single storey adobe house with RC ring beam, under construction. The load-bearing walls are on the verge of failure due to shear. *Location: Yalnizbag.*
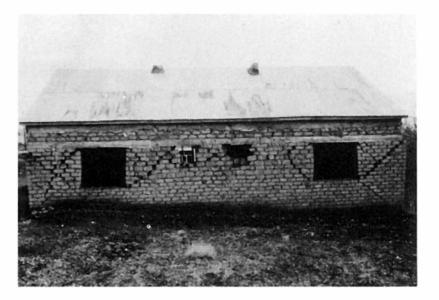

Plate 4.21
Collapse of a single storey adobe house with concrete ring beam. The roof was heavier than that in Plate 4.20, as it was clad with clay tiles. *Location: Yalnizbag.*

Plate 4.22
Severely damaged himis house. Two single storey concrete block masonry houses next to it collapsed. *Location: Calabzur.*

Plate 4.23
Shattered himis building, twisted and leaning, but still standing. *Location: Altinbasak.*

Plate 4.24
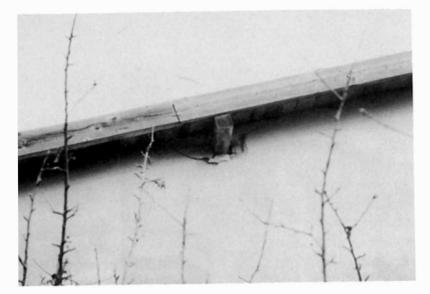Failure of unrestrained gable ends. *Location 10.*

Plate 4.25
Gable end supported by
roof purlins. *Location 11.*

Plate 4.26
Badly damaged highway bridge on road to Kemah. Notice the lack of bearings between the crossheads and main beams. The nearest column is badly bent.

Plate 4.27
Severe cracking of embankment wall of the same bridge.

Plate 4.28
Detail of column head of the same bridge, showing poor construction quality and damage due to pounding between adjacent deck segments.

Plate 4.29
Main road bridge over
Euphrates river south of
Erzincan.

Plate 4.30
Bridge taking main North-
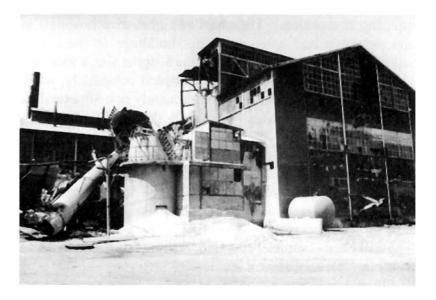South road over the railway
to the south of the city.

Plate 4.31
Failure of road embankment
near Eksisu.

Plate 4.32
Collapse of the roof of an RC frame cattle shed at the meat processing plant. The columns are considerably more slender than the roof beams, and the standard of construction is very poor (see also Plate 4.4).

Plate 4.33
Erzincan sugar factory. The 36 m high silo collapsed, causing some secondary damage; otherwise the structure stood up well.

5. CASE STUDIES

5.1 The Hospitals

Hospitals are among the most crucial buildings in the aftermath of a disaster. It is therefore vital that they survive strong earthquakes and other natural disasters without major disruption. Unfortunately, the health sector in Erzincan was badly affected by the earthquake, seriously hampering its role in the recovery operation. Because of their strategic importance, the EEFIT team visited all three hospital sites in the city. Although clearing of collapsed structures had already commenced, it was possible to gather a considerable amount of useful information about these critical buildings.

5.1.1 The state hospital
The state hospital consists of six major buildings (Figure 5.1), occupying an area of about 45 acres in the centre of the city; the dates of construction of the various buildings are shown on the figure. The hospital is used mainly by government employees. The nursing school was the only building to collapse, the operating clinic being seriously damaged and probably requiring demolition. The chest and obstetrics hospitals suffered less damage, as did the administration and emergency unit buildings in the centre of the hospital yard. Of the buildings immediately adjacent to the hospital site, a four storey RC frame apartment building with shops at ground floor level collapsed completely, while a similar building, but without an open ground floor, survived with mainly non-structural damage. A range of single storey structures were undamaged or slightly damaged.

The nursing school, pictured before the earthquake in Plate 5.1, was built before the introduction of the 1977 Turkish earthquake code. The building had five storeys and a semi-basement, with the layout shown in Figure 5.2. As can be seen in Plate 5.2, the western wing collapsed completely during the earthquake, while the eastern wing remained standing, with some structural damage. When EEFIT visited the site, 21 days after the event, the collapsed wing had been completely cleared, making it difficult to comment with certainty on the causes of failure. Nevertheless, a number of important aspects of the structural behaviour could be identified:

- The transverse wall separating the two wings is a shear wall. However, it is unlikely that any of the internal longitudinal walls acted as a shear wall as there are numerous door openings leading to rooms on either side of the central corridor.
- The columns are quite slender for a structure of this size and importance.
- The amount of openings in the longitudinal direction is very large, especially in the ground floor. Although the amount of openings in each of the upper floors is the same, the positions vary from floor to floor, creating significant non-uniformities in the cladding stiffness. In many instances, non-structural elements can significantly strengthen the structural frame, but in this structure the non-uniformity of the layout and the large number of openings is likely to have minimised any such beneficial effects.
- The stairwell is located on the northern side of the building, which may have resulted in a significant torsional eccentricity.
- The building's long axis is parallel to the fault that caused the earthquake.
- At the third floor of the eastern wing there were positions in which the central columns were just between window openings, creating a short column effect. However, at

semi-basement level there was a concrete shear wall spreading to the foundation in both wings, without forming any short column. (The short column effect, which is quite common in RC framed buildings, was more commonly found to occur at semi-basement level, where the basement windows tended to cover the whole span between columns in order to provide the maximum possible lighting.)

The operations clinic suffered badly, with significant damage to the columns and most of the infill masonry having deep diagonal cracks. The most interesting aspect of this building is that the distribution of cladding is exactly reversed between the southern and northern facades (Plates 5.3 and 5.4), causing significant torsional effects. The location of the staircase on the northern side may have exacerbated this problem. The orientation of the clinic's long axis is also parallel to the North Anatolian Fault.

5.1.2 The insurance hospital
This is probably the largest civilian hospital in Erzincan prefecture in terms of bed capacity, and is the only hospital in the region used by non-government employees. It is located at the western edge of the city, near the Erzurum-Sivas highway, occupying an area of about 25 acres. The hospital consists only of two five storey buildings with semi-basement, connected by a common concourse. The structure (prior to the earthquake) is pictured in Plate 5.5, and illustrated in plan in Figure 5.3.

The western wing of this large building collapsed completely, with the remainder suffering some structural damage. The long axis of the collapsed wing ran parallel to the North Anatolian Fault. The amount of openings was quite large, but the columns were stronger than those in the nursing school. The insurance hospital, like the nursing school, has a semi-basement with RC perimeter walls. The site was visited 22 days after the event and unfortunately had been completely cleared, apart from the semi-basement (Plate 5.6). Inspection of the surviving wing suggested that the structure had very strong infill panels, made of solid brick, but was vulnerable to short-column failure at the top of the basement.

5.1.3 The military hospital
This very large structure is located on the northern outskirts of the city, close to the North Anatolian Fault. As shown in the approximate plan of Figure 5.4 and in Plate 5.7, the main structure is six storeys high with a semi-basement having RC perimeter walls. A number of one to three storey extensions have been added to the north of the original structure.

During the earthquake, the western end of the main structure, where several re-entrant corners and balconies were located, collapsed completely, while the rest of the west wing lost its first floor (Plate 5.8). The ground floor and second to fifth floors were severely damaged, but remained standing, while the first floor was completely crushed by the weight of the four floors above it (Plate 5.9). As can be seen in Plate 5.7, the collapsed floor is the only one in which the number of window openings differs significantly between the east and west sides of the building. It is also possible that some pounding from the low-rise extensions to the north occurred.

The almost identical eastern wing, located on the opposite side of the central tower, survived with severe structural damage. Extensive damage was also noted inside the structure, with many walls and columns crushed or collapsed and a large amount of scattered debris.

Immediately after the earthquake, the army devoted a lot of manpower to the rescue of people trapped in the building. A military spokesman told EEFIT that there were no fatalities in the hospital, which had around eighty occupants at the time of the earthquake.

5.1.4 Concluding remarks

A few common features emerge from these three buildings:

- All were five to six storey RC framed structures with a large amount of window openings and a semi-basement with RC perimeter walls.
- All had a long, thin shape with aspect ratios between three and four, and the long axis oriented east-west, parallel to the North Anatolian Fault.
- All three, being very large structures, lost some of their wings, with the remaining structure suffering rather less damage. All the collapsed wings were on the western side of the structure.

The damage to these structures thus follows a general trend observed in Erzincan, in which the greatest damage occurred in the east-west direction. Directionality of damage is discussed further in Section 6.1.

5.2 Sumerbank Textile Factory

The state-owned Sumerbank Textile Factory, located just to the south of the city centre (Location 16), is one of the largest industries in the region, with an annual turnover of around $5 million and a workforce of 600. The site consists of two large, single-storey RC buildings housing the textile spinning machines, a mid-rise RC generator building and some light steel and RC storage buildings.

The main machine hall, constructed in the 1950s, consists of a series of pitched-roof bays, with the approximate cross-section shown in Figure 5.5. Most of the bays were reinforced by horizontal cross-beams, but occasionally these were omitted to allow for movement at the expansion joints. During the earthquake severe damage was induced at the column-heads where this discontinuity occurred (Plate 5.10). There was also moderate or minor damage to many other columns (Plate 5.11) and major cracking of masonry infill panels. The damage level was noticeably less at the southern end of the building, where some shear walls and double columns were present. Following the 1983 earthquake, attempts had been made to strengthen the beams and joints in this structure using external steel fixtures (see Plate 5.11). These alterations were insufficient to impart significant moment capacity to the joints.

The hall houses very new spinning machines, commissioned less than a year before the earthquake. These long machines are highly sensitive to misalignment, and must be positioned to a high tolerance on axial straightness in order to operate properly. However, the specification for the machines contained no earthquake performance requirements. The machines were well restrained by concrete castings at their ends, but had only vertical support elsewhere. During the earthquake, the machines shifted off their intermediate supports, causing them to bow slightly. At the time of the EEFIT visit, no attempt had been made to straighten or restart the machines, but the management seemed confident that they were not seriously harmed. Of more concern was the risk of damaging the machines while repairing the surrounding structure, since moving them to another part of the site during the repair

operation was considered virtually impossible. The adjacent hall, built in the 1960s, was occupied by old, disused machinery. This was a similar structure which suffered slightly worse damage than the older hall, with many column heads completely sheared through (Plate 5.12).

Of the remaining structures, the most interesting were two very similar storage sheds, consisting of a light steel frame, open at the sides and with steel sheet cladding on the roof. The column bases were restrained by just four small bolts, as shown in Plate 5.13. The lack of any longitudinal bracing meant that the structures relied entirely on the moment resistance of the joints to resist horizontal loads. The only major difference between the two structures was their orientation. The shed with its long axis running east-west (i.e. in the direction of the strongest ground motion) collapsed completely, falling to the west (Plate 5.14) while the other shed, oriented north-south, survived with no visible damage (Plate 5.15). This suggests that the frames were designed primarily for lateral loading, with insufficient consideration given to loads applied along the axis of the structure.

The financial cost of the earthquake to the factory is substantial. While no buildings require demolition, major repairs are needed. The management suggested that the factory would be inoperative for three months, but repairs had still not commenced seven months after the earthquake. This delay resulted in further damage from aftershocks, making the original repair plan unfeasible.

5.3 Erzincan City Centre

A number of major structures were located at the crossroads of the main east-west and north-south highways in the very centre of the city (Location 17). The mid-rise RC framed hotel buildings which occupied two of these corner sites collapsed completely and were cleared before EEFIT reached Erzincan. This section concentrates on the block immediately south-east of the intersection, shown in Figure 5.6. All of the buildings on this site are RC frames. The building on the north side of the site consists of shops at ground floor level with offices above. To the south east is a PTT office building, while the south west side of the block is occupied by single storey shop structures. All of the buildings look out onto a courtyard in the centre of the block. Both of the four storey buildings suffered major damage.

The northernmost structure is a four storey RC frame with basement. On the street side there are shop fronts at ground level, with the cantilevered upper storeys infilled by masonry panels on the first floor and a proprietary cladding system on the upper two floors (Plate 5.16). All the masonry panels on the facade at first floor level were totally or partially collapsed, showing classic diagonal cracking patterns (Plate 5.17). In plan this building has an L-shape, with the two legs separated by an expansion/construction joint, as shown in Figure 5.6. The earthquake caused this joint to open up throughout all accessible levels of the building (Plate 5.18).

Internally, the building is arranged around a central well, with offset floor levels, and flights of stairs between floors spanning across the well. Shear walls inset from the ends of the well were weakened by circular cut-outs just below the soffit of the roof, and suffered severe cracking and buckling of reinforcement in this area (Plate 5.19). The standard of construction was poor, with minimal confinement steel and token cover to the main reinforcement. A poor

design feature of the building was the row of windows just below roof level on the south face, introducing both asymmetry and short columns into the design. All the short columns in this area showed signs of distress, and the shear failure of a nearby stub wall emanated from this feature.

As can be seen from Plates 5.20 and 5.21, the PTT building suffered the complete loss of its first floor. The upper floors had moved south and west relative to the ground and basement levels, by approximately 0.5 to 1.0 m in each direction. All internal and external walls consisted of infill masonry or glazed panels. Lateral stiffness of the structure relied entirely on the beam column connections, which were inadequate to cope with the motion induced by the earthquake.

The remaining construction on this block is single storey with basement, and opens onto the common courtyard with the PTT building. This construction suffered only superficial damage to the facades of the retail units and some damage to the internal services, such as burst water pipes.

5.4 Cooperative Housing Estate, Uzumlu

This housing estate, located to the east of Erzincan, just north of the main Erzincan to Erzurum highway, was still under construction at the time of the earthquake. It is likely that work on the houses had ceased during the winter months and was about to recommence. The houses were near to completion and would probably have been occupied by the end of the summer of 1992. The estate consists of 21 largely identical, two storey houses of load-bearing hollow brick masonry in cement mortar, with a concrete ring beam and RC floor slabs, a timber framed roof and clay tiles (Plates 5.22 and 5.23). Vertical concrete members were not used. An approximate plan of the second floor is shown in Figure 5.7.

The estate is of particular interest as the buildings were arranged in lines running down the hillside, with the degree of damage increasing steadily from one end to the other. This makes it possible to see clearly the sequential stages in the collapse of one of the most common building types in the world. The layout of the buildings and the assigned damage degrees are shown in Figure 5.8. Clearly the damage became progressively more severe towards the southern part of the estate, these houses situated on deeper sediments, and also being the most recently built, the mortar possibly not yet fully cured.

The failure mechanism, shown in Plates 5.23 to 5.27, is typical of unreinforced brick masonry. Damage starts by the development of diagonal shear cracks at a re-entrant corner on the ground floor (Plate 5.23). Subsequent plates show the spread of damage through the structure, right up to complete collapse.

Two types of damage survey were performed on the housing estate. Firstly, each structure was assigned a damage degree (shown in Figure 5.8) and the cumulative damage statistic were used to estimate the intensity using the Parameterless Scale of Intensity (PSI) developed at the Martin Centre, University of Cambridge. This procedure is described in more detail in Chapter 6 and Appendix A. The results are summarised in Table 5.1.

Damage degree	No. of houses	% of total	% (cumulative)
D5	3	14.3	14.3
D4	1	4.8	19.1
D3	2	9.5	28.6
D2	5	23.8	52.4
D1	0	0	52.4
D0	11	47.6	100.0

Table 5.1 Damage Statistics for Cooperative Housing Estate, Uzumlu

About 20% of the estate suffered a damage degree D4 or greater (partial or complete collapse) and the average damage degree was 1.67. The PSI intensity using the vulnerability functions shown in Appendix A, obtaining the mean value from all five damage degrees, is 9.8, which is somewhat higher than an MSK intensity of VIII. If the estate were to be split into two parts (northern part: houses 1 to 10; and southern part: houses 11 to 21) then the PSI values for each part would be 7.0 and 11.5 respectively. This implies an expected response spectral acceleration in the 0.1 to 0.3 seconds period range (5% damping) of 8-10 m/s^2 (75% range) in the southern part. (This subdivision of the data should be treated with some caution as it reduces the sample size below the value of 20 which is considered a minimum for the use of the PSI scale - see Appendix A.)

Secondly, an investigation was made of the loss of indoor space in each of the four houses that suffered collapse. Many casualty estimates are based on a crude classification of collapsed buildings into a single category, giving only a very rough idea of the life loss inside them. However, it is clear that buildings classified as collapsed may affect their occupants to widely differing degrees (compare Plates 5.26 and 5.27). The primary factor causing casualties within a collapsed structure is the loss of indoor space; the higher the loss indices, the less likely is an occupant to be able to free himself from the rubble.

House no.	Ground floor			First floor		
	W_1	W_2	W_3	W_1	W_2	W_3
14	0.5	0.3	0.25	0.5	0.3	0.25
18	1	1	1	0	0	0
19	1	1	1	0.25	0.2	0.15
20	1	1	1	1	0.8	0.7

Table 5.2 Loss of Indoor Space Indices for Collapsed Houses

Three indices of loss of indoor space were determined for each floor of the collapsed houses on the estate: index W_1 describes the space loss in plan; index W_2 describes the space loss in section; and index W_3 describes the volume loss (Okada et al, 1991; Coburn et al, 1992). Table 5.2 shows that the four houses that collapsed lost very different amounts of indoor space, even from floor to floor. All the houses suffered higher losses in the ground floor than in the first floor, suggesting that the collapse was initiated at ground floor level, a conclusion which is in agreement with observations of the less severely damaged houses.

5.5 Four Storey Apartment Blocks, Fatih District, Erzincan

This estate (Plate 5.28, Location 18) consisted of two apparently very similar types of apartment, which had however performed rather differently. There were about eighteen older apartments of the first type (Plate 5.29) to the south of the site. They had four above ground storeys and a semi-basement about half of which was above ground level. The structure consisted of concrete frames with clay tile infill; some apartments had their major axes oriented east-west and others north-south. There was little evidence of damage externally in any of these older apartments; internally there was extensive severe cracking to the infill panels (but no collapses were noted) and hairline cracking could be seen in some beams and columns. The occupants were not sleeping in these apartments but used them during the day for washing etc.

To the north were eight more recent apartments (Plate 5.30) of similar construction to those discussed above. Two of the apartments with their long axis in a north south direction had collapsed and had been demolished; two had suffered a failure in their semi-basement (Plate 5.31). There was less apparent damage in the blocks oriented at right angles.

The structural difference between the two types of blocks was that the older blocks were provided with concrete walls between basement and ground floor level (Plate 5.32), whereas in the newer blocks the concrete frame continued to foundation level and was infilled with concrete blockwork. Plate 5.33 shows an adjacent block under construction with this detail. The newer (and presumably cheaper) construction practice had apparently resulted in the creation of a stiff but weak storey with columns whose shear strength was less than their flexural strength. This detail should therefore be avoided.

5.6 Six Storey Building, Bahcelievler District, Erzincan

A dimensional survey was carried out on a six storey residential block with a single basement level in the Bahcelievler district of Erzincan (Location 19). It was structurally complete, but not fitted out; the roof slab had been cast but the shuttering had not been struck. Blockwork infill walls appeared mainly complete but no finishes had been applied.

Figure 5.9 shows the building plan, based on a site survey using only a tape measure. The vertical structure started at basement level and continued essentially undiminished to roof level. Some reduction in column sizes were noted at upper levels, but a detailed survey was not carried out, due to the unsafe state of the building. Storey heights were 3m, except for the ground floor and basement which had storey heights of 3.6m. The infill blockwork comprised 180mm thick hollow clay tile, except in the basement, where hollow concrete block was used. The columns at ground floor level had about 1% vertical and 0.3%

transverse steel, with no indication of closer spacing at column ends. The shear wall (gridlines 2, D-E) appeared to have about 0.2% steel vertically and 0.15% horizontally in each face, with no evidence of special confinement to the vertical edges. These estimates are based on the exposed steel visible on the surface of some of the members.

The building appeared to be perilously close to collapse. The major damage was at ground floor level, with none visible in the basement and little at higher levels (though a detailed inspection was not made). The central shear wall was very heavily damaged with the concrete reduced essentially to rubble and the reinforcement clearly visible. There was severe spalling at many of the tops and bottoms of the ground floor columns between gridlines B and H (which provided the resistance to east-west motions); little damage could be seen in the other columns which provided north-south resistance. The stair slab had fractured at ground floor level, where the single mat of steel had pulled completely out of the concrete. There was also heavy damage to the stair slab at first floor level.

It can be seen from Figure 5.9 that the structure was reasonably uniform in plan; an analysis by EEFIT has shown that the ratio of torsional to translational stiffness conformed with the limits required for regular structures in the Japanese Building Standard Law (IAEE, 1988). The vertical regularity was also reasonable, although the ground floor height was 20% greater than that of upper floors, and the building damage was concentrated at ground floor, indicating a weak storey at that level. The vertical regularity was therefore checked by performing a response spectrum analysis of the building, using the 5% damped spectrum recorded in Erzincan. The storey height to storey drift ratio in the ground floor obtained from this analysis was found to be 90% of the average ratio for all floors. This compares with a minimum ratio of 60% specified in the Japanese Building Standard Law for regular structures.

For reinforced concrete structures up to 31m in height with a minimum degree of ductility and lateral strength and which conform to the regularity limits referred to above, the Japanese Building Standard Law specifies that if either of the following inequalities are satisfied, "phase 2" checks for resistance to an extreme earthquake are not required. The equations are based on observations of building performance in damaging Japanese earthquakes (Aoyama, 1981).

Buildings well provided with shear walls:

$$\frac{\Sigma\,2.5A_w + \Sigma\,0.7A_c}{0.75\,ZWA_i} \geq 1 \tag{1}$$

Buildings mainly relying on beam-column frames:

$$\frac{\Sigma\,1.8A_w + \Sigma\,1.8A_c}{1.0\,ZWA_i} \geq 1 \tag{2}$$

where A_w = area of shear walls in the direction of seismic force being considered (mm^2)
A_c = area of columns (mm^2)
Z = zone factor (= 1.0 for the most seismic area of Japan)
W = weight of building above the level under consideration (N)
A_i = vertical distribution factor in Japanese code (= 1.0 at ground floor level).

The present building satisfies both regularity and height requirement, but is unlikely to conform to the minimum ductility requirement of the Japanese code. Nevertheless, it is instructive to calculate the ratios of equations 1 and 2 at ground floor level. The zone factor was taken as 1 on the basis that both Japan and this area of Turkey are regions of high seismicity. Since the columns are very much stiffer in one direction that the other, only the columns aligned with the direction under consideration were included in the calculation of A_c. In the east-west direction, (the likely direction of maximum shaking) the ratio was calculated at 0.55, and in the north-south it was 0.78. This was based on equation 2; equation 1 gave less favourable answers.

Similar ratios were calculated for the southern end of the new Town Hall (Plate 4.13) in Erzincan, a five storey building which was structurally complete but unoccupied at the time of the earthquake. The building also had a moment resisting reinforced concrete frame with limited concrete shear walls and infill blockwork. The building appeared to have suffered no structural damage but there was extensive cracking to finishes and infill walls. The ratios calculated from equation 2 were 0.79 east-west (the likely direction of maximum excitation) and 0.51 north-south. These values would not in themselves be unacceptable in Japanese practice but would necessitate a further check to be carried out.

A very simple calculation has therefore demonstrated for a severely damaged but still standing building that inadequate provision of seismic resisting structure was probably provided in the direction of maximum shaking. A building with no structural but extensive non-structural damage has a provision of lateral resisting structure in the direction of maximum shaking which, by this crude calculation, is 44% greater. The simple formula therefore appears to have some merit, provided it is combined with simple but conservative rules for ensuring vertical and horizontal regularity, perhaps based on those in Eurocode 8 (1988), provision of minimum main and transverse steel (based for example on the Turkish seismic code) and limits on beam dimensions to encourage "strong column/weak beam" structures. It is suggested that such checks may be useful for rapid assessment of the seismic adequacy of low to medium rise buildings. Such simple rules may be more effective than more sophisticated measures which could be misapplied, and can certainly be helpful as independent supplementary checks.
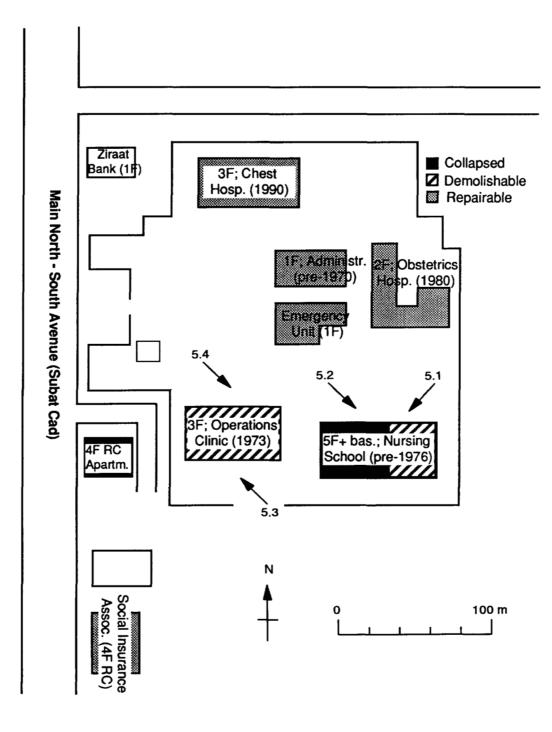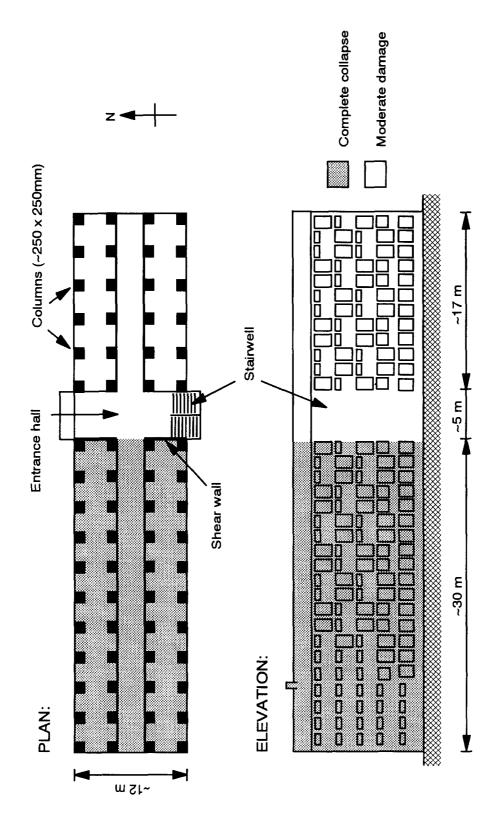
Figure 5.1 Layout of buildings at the Erzincan state hospital (arrows indicate viewpoints of plates)
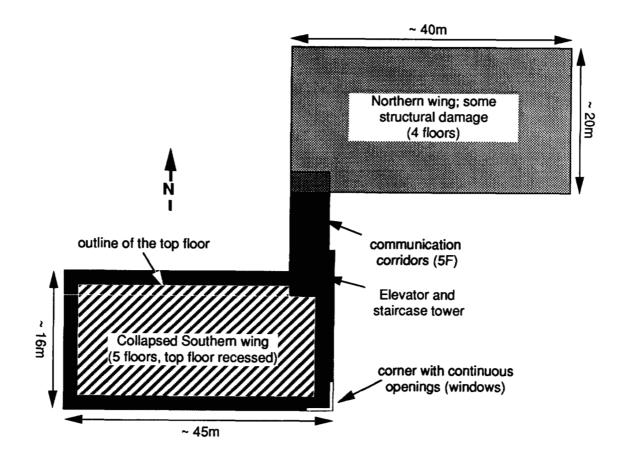
Figure 5.2 Some structural details relating to the collapse of the nursing school, Erzincan state hospital. *Location 13.*

Figure 5.3 Approximate plan view of the insurance hospital. *Location 15.*

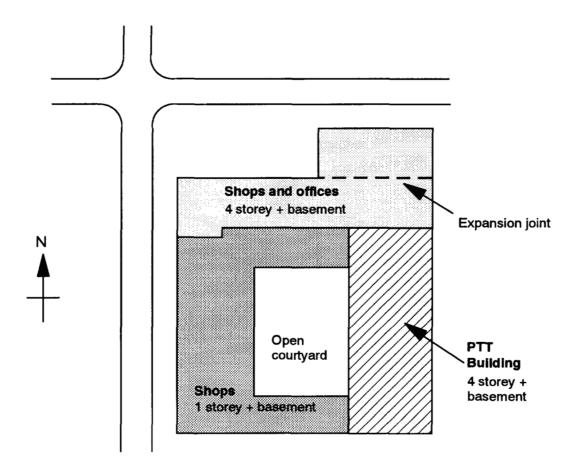**Figure 5.4** Approximate plan view of the military hospital.

Figure 5.5 Approximate section through the main machine hall, Sumerbank Textile Factory. *Location 16.*

Figure 5.6 Layout of buildings on south-east corner block of Erzincan city centre (Kizilay). *Location 17.*

Figure 5.7 Approximate second floor plan of typical house on the Cooperative Housing Estate, Uzumlu

Figure 5.8 Layout of buildings on the Cooperative Housing Estate, Uzumlu, showing damage levels

Figure 5.9 First floor plan of six storey apartment block, Bahcelievler district. *Location 19.*

Plate 5.1
The nursing school in the state hospital, before the earthquake. Notice the irregular distribution of openings from floor to floor. The northern façade of the building is identical apart from the entrance canopy. *Location 13.*

Plate 5.2
The same side of the nursing school after the collapse and debris clearance. *Location 13.*

Plate 5.3
Operations clinic, state hospital, southern façade. *Location 14.*

Plate 5.4
Operations clinic, state hospital, northern façade. Notice the identical location of damage to the infill walls at ground floor level. *Location 14.*

Plate 5.5
The insurance hospital, Erzincan before the earthquake, looking northwest. *Location 15.*

Plate 5.6
The insurance hospital after the earthquake and debris clearance, looking eastwards. *Location 15.*

Plate 5.7
The military hospital before the earthquake, looking north. The hospital is situated on the northern outskirts of Erzincan, very near the North Anatolian fault.

Plate 5.8
Western half of the military hospital after the earthquake. The end section has collapsed completely while the remainder has lost its first floor. The eastern half was severely damaged but remained standing.
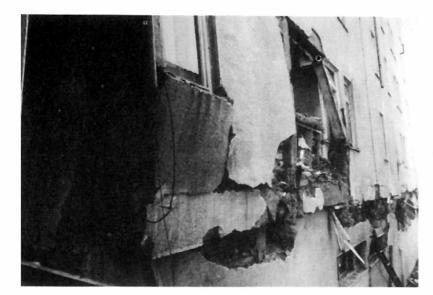
Plate 5.9
Detail of the military hospital, showing the complete crushing of the first floor.
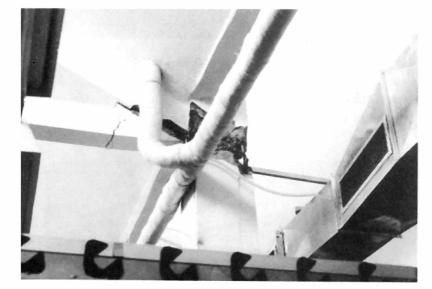

Plate 5.10
Sumerbank textile factory, main machine hall. Severe damage to column head adjacent to span where crossbeams are absent. The ducting was flexibly attached and mostly survived well, but the pipework suffered several ruptures. *Location 16.*
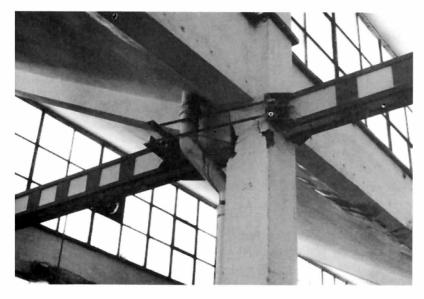

Plate 5.11
Sumerbank textile factory, main hall. Column heads across which the crossbeams were continuous suffered less damage. Note the attempt at external reinforcement of the beams and the joint. *Location 16.*
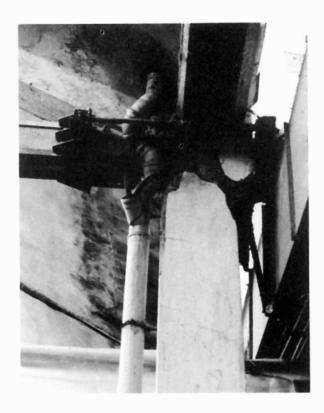
Plate 5.12
Sumerbank textile factory, secondary machine hall. Complete shear failure of a column head. *Location 16.*
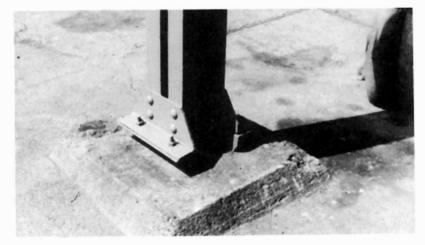

Plate 5.13
Sumerbank textile factory. Detail of column footing in steel storage sheds. When one of the sheds collapsed, the footing bases remained intact but the bolts connecting them to the columns failed. *Location 16.*

Plate 5.14
Sumerbank textile factory. The east-west oriented storage shed collapsed due to the weak connections and lack of longitudinal bracing. *Location 16.*

Plate 5.15
Sumerbank textile factory. The north-south oriented shed suffered no visible damage, the frame being adequate to resist the strong east-west ground motion. *Location 16.*
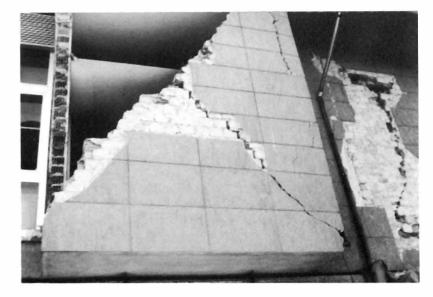

Plate 5.16
City centre. General view of shop and office building showing facade finishes and damage. *Location 17.*

Plate 5.17
City centre. Diagonal cracking of cantilevered masonry panel. *Location 17.*

▲ Plate 5.18
City centre. Opening of movement joint. *Location 17.*

▲ Plate 5.19
City centre. Failure induced by openings in reinforced concrete wall. *Location 17.*

Plate 5.20
City centre. Mid-storey collapse of PTT offices, other floors severely damaged. *Location 17.*

Plate 5.21
City centre. View of PTT offices from the internal courtyard. *Location 17.*

Plate 5.22
General view of the Cooperative Housing Estate, Uzumlu, looking southwards from the top of the estate. Notice that the first few houses, whose roofs are already finished, are completely undamaged. The significant slope of the site is also visible. The collapsed buildings are at the bottom of the estate, presumably in deeper soil.

Plate 5.23
Cooperative Housing Estate, Uzumlu. View of house no. 12, one of the first to exhibit slight damage in the form of diagonal shear cracks at the right hand corner of the ground floor.

Plate 5.24
Cooperative Housing Estate, Uzumlu. House no. 21. Severe damage at the ground floor corner.

Plate 5.25
Cooperative Housing Estate,
Uzumlu. House no. 18.
Complete collapse of the
ground floor while the
second floor remains
standing.

Plate 5.26
Cooperative Housing Estate,
Uzumlu. House no. 14.
Collapse of both floors
(only in the western part).

Plate 5.27
Cooperative Housing Estate,
Uzumlu. House no. 20.
Complete collapse of both
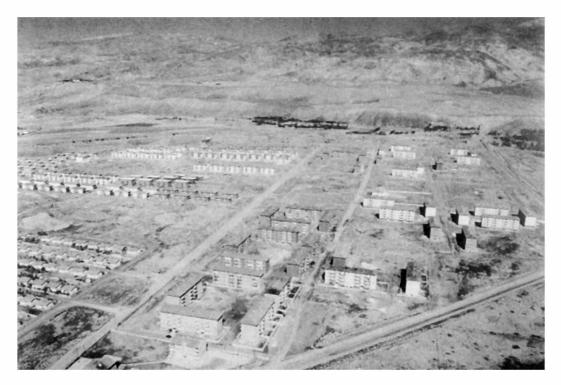floors in all parts of the
building.

Plate 5.28
Aerial view of housing estate in Fatih district, before the earthquake. *Location 18.*
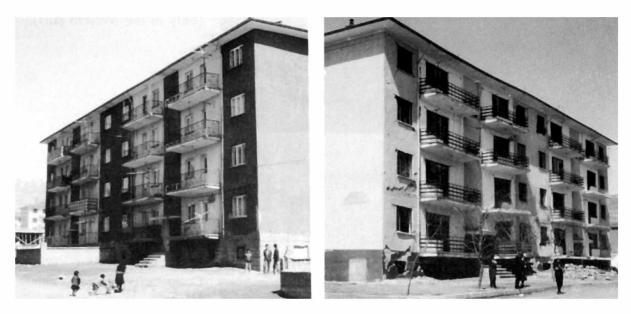

Plate 5.29
Older apartment block in
housing estate of Plate 5.28.
Location 18.

Plate 5.30 Recent
apartment block in housing
estate of Plate 5.28.
Location 18.

Plate 5.31
Weak storey failure in recent apartment block. *Location 18.*

Plate 5.32
Concrete wall at semi-basement level in older apartment block. *Location 18.*

Plate 5.33
Concrete block infill at semi-basement level in apartment under construction. *Location 18.*

6. EXTENT AND DISTRIBUTION OF DAMAGE

6.1 The City of Erzincan

6.1.1 Damage statistics

Data published by the Ministry of Public Works and Resettlement shortly after the earthquake showed that, prior to the earthquake, Erzincan contained a total of 28,000 buildings. Of these, 2,168 (7.7%) were destroyed by the earthquake, 3,290 (11.7%) were classified as moderately damaged and 4,131 (14.7%) suffered light damage. (These early figures are now thought to be underestimates.) The number of fatalities was 394. These figures are rather lower than would have been expected on the basis of previous earthquakes in Turkey during this century (Pomonis, 1992). The main reasons for this are thought to be:

- The buildings in Erzincan have all been constructed quite recently, since the city was devastated by the 1939 earthquake, and therefore are likely to be significantly less vulnerable than those in most cities in eastern Turkey.
- The focal depth (~27 km) was significantly greater than in most recent Turkish earthquakes.

While the relatively low number of buildings destroyed is encouraging, it should be remembered that many collapsed buildings were multi-storey RC structures. Collapse of one of these major structures is likely to cause a much greater loss of life than collapse of a low-rise masonry structure, and the cost of replacement will also be considerably higher.

Figure 6.1 shows the distribution of building damage by district (data provided by the Turkish Directorate of Disaster Affairs). The three central districts of Karaagaç, Kizilay and Inönü, where many of the major mid-rise office and hotel buildings are located, all suffered very high levels of damage, with around half of their buildings moderately damaged or collapsed. High levels of damage are also apparent in the outlying districts of Yavus Selim, on the north-west of the city, and Fatih to the east. Both of these districts contain a large number of newly constructed, multi-storey housing complexes, built in response to the recent population boom in the city. On the whole, districts containing predominantly low-rise buildings suffered much less severe damage.

6.1.2 North-south transect through the city

A simple damage survey was made to try to establish whether ground motions in Erzincan had varied systematically with distance from the edge of the basin. The survey was conducted along a north south line across the city, just to the west of city centre (Figure 6.2). Based on a hydrogeological survey map dated 1981, it appears that the depth of sedimentary material varied considerably along the survey line, Figure 6.3. (It should be noted that this section is deduced from very limited survey information and is thus based on a certain amount of conjecture.) Two types of damage were recorded, as follows.

- The percentage of boundary walls between 1m and 1.5m in height which had collapsed. The boundary walls were all to domestic properties adjoining the line of the survey and were all aligned in a north south direction.

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- The percentage of chimneys on buildings not exceeding 3 storeys in height which had collapsed, and were visible from the line of the survey. Chimneys were rated as 50% collapsed if they were still standing but obviously damaged.

The intention was to adopt a measure of damage which was easily recorded and was sufficiently abundant to provide some statistical measure of confidence. Generally, the type of housing and the nature of the boundary walls appeared similar throughout the survey, so it was hoped that a reasonably uniform damage measure would result.

District	Length (m)	State (fallen or standing)	Chimneys	Boundary Walls		
				Adobe	Stone	Block
Kazim Karabekir	550	F	58	-	-	26
		S	127	-	-	309
		F/(F+S)	31%	-	-	7%
Yenima-halle	580	F	28½	28	-	7
		S	95½	5	-	401
		F/(F+S)	22%	84%	-	2%
Inonu (1)	600	F	31	-	5	51
		S	80	-	330	381
		F/(F+S)	28%	-	1½%	12%
Inonu (2)	435	F	16½	20	44	53
		S	63½	49	194	200
		F/(F+S)	20%	29%	18%	21%
Kizilay	435	F	15	0	12	11
		S	22½	15	40	188
		F/(F+S)	40%	0	23%	6%
Hocabey	465	F	21½	-	-	261
		S	46½	-	-	255
		F/(F+S)	32%	-	-	51%
Taksim	300	F	6	0	0	83
		S	28	14	22	48
		F/(F+S)	18%	0	0	63%

Table 6.1 Results of Damage Survey along North South Transect Through Erzincan

The results of the survey are shown in Table 6.1 and are plotted in Figure 6.4. The results are not conclusive; the chimney survey shows no significant variation of damage with distance from the north edge of the basin while the boundary fence survey suggests some increase in motion at the southern end of the survey line. It is possible that more reliance can be placed on the chimney survey because the collapse of one chimney is unlikely to destabilise others, whereas toppling of one section of wall may bring down other lengths with it. Certainly, the absence of trend in the chimney damage suggests that any tendency of the motions to increase with distance from the edge of the basin was low. This finding is in agreement with the damage data presented in Figure 6.1, tending to support the conclusion of absence of basin effect.

6.1.3 Directionality of motion

A survey was made of boundary walls to an estate of about 6 two-storey apartment blocks of similar construction at the southern edge of the town (Location 20). The boundary walls were in concrete block and about 1.5m high. 75% of the 120m length of walls running in a north south direction had overturned while only 10% of the 245m length of east west walls had overturned. Since overturning is caused by motions perpendicular to the plane of the wall, it appears that east west motions were significantly stronger at this point than north south motions, at least at the relatively high frequencies likely to damage these low walls. This is consistent with expectations in the vicinity of an east west running fault experiencing a strike slip displacement. It is also consistent with the strong motion record, and with findings given elsewhere in this report (e.g. Section 5.1).

6.2 Detailed Survey Around the Strong Motion Instrument

A detailed damage survey was carried out around the strong motion recording station near the city centre. This work forms part of a programme aimed at relating observed building damage to the recorded earthquake ground motion. Damage distributions are used to assess the earthquake intensity according to the PSI scale using a range of vulnerability functions developed at the Martin Centre, University of Cambridge. By carrying out numerous such surveys around strong motion stations, it is possible to assess the degree of correlation between the PSI value and various ground motion parameters (Spence et al, 1992). The procedure is described in more detail in Appendix A.

6.2.1 Details of the surveyed area

The strong motion instrument in Erzincan was located in the meteorology station in the Inönü district, about 0.75 km north-west of the city centre (see Figure 6.5); the ground in the surveyed area slopes gently. The station is a well built two-storeyed RC frame house with a basement in the southern part, and was undamaged during the earthquake. Next to the station in the southern direction there is a three storeyed RC frame apartment building with basement of very good construction that also survived the earthquake with only minor damage. Immediately to the north of the station is one of the larger mosques in Erzincan, which also survived the shaking without any significant damage. The remainder of the area around the station is mainly covered with single storeyed timber frame or himis dwellings, dating from the post-1939 reconstruction period. In the last two decades the number of masonry and RC frame buildings in the area has gradually increased, especially to the south of the station, towards the town centre.

The survey covered a total of 125 buildings, which can be divided into the construction types listed below and shown in Plates 6.1 to 6.4:

- *Timber Frame:* There were 39 single-storey buildings made of timber frame covered by diagonal timber laths (sheathing) with a vapour barrier, a 2mm mesh and 5mm render. These usually rested on reinforced concrete foundations and had a concrete walled semi-basement for storage. These houses were built during the reconstruction after the 1939 earthquake (Plate 6.1).
- *Himis:* There were 35 one-storeyed timbered buildings, having infill mostly of adobe bricks covered with plaster, without a semi-basement (Plate 6.2).
- *Block Masonry:* 28 houses of one or two storeys were made of solid brick or concrete block masonry, with RC ring beam and timber roof with clay tiles; some houses had an RC slab instead of a timber roof (Plate 6.3).
- *Reinforced Concrete Frame:* 23 structures were RC frames of two or three storeys, with unreinforced horizontally perforated brick infill walls (Plate 6.4). Unlike the other building types, the RC structures showed wide variations in layout and quality of design.

6.2.2 Results of the survey

An assessment of the damage suffered by each building was carried using the MSK intensity scale damage grading, ranging from D0 for undamaged to D5 for collapsed buildings. A more detailed description of the various damage categories can be found in Appendix A. For this assessment, photographs were taken of each building, allowing a careful assessment of the damage degree to be carried out later.

Figure 6.6 shows the damage distributions obtained from this analysis. Clearly the damage levels in the area around the strong motion station were not as severe as in the nearby city centre. There was only one collapsed (D5) building and four classified as severely damaged (D4) among the 125 surveyed. The collapse occurred in a single storey confined solid brick masonry building (i.e. masonry with some vertical RC columns at the corners), used as a marble workshop. The damage to the timber buildings was light and as expected for the level of recorded motion. The damage to the RC framed buildings is much lighter than in the city centre or in the outskirts where many residential complexes are situated. There are three possible reasons for this:

- In the surveyed area there were no soft storeyed buildings, which are considerably more vulnerable to seismic damage. As can be seen in Figure 6.6, the proportion of RC buildings in the surveyed area rated D3 or worse was 26%. If the surveyed buildings had all had shops on the ground floor, as in the city centre, we might have expected this figure to reach 40-50% (Pomonis, 1992); this is close to the value experienced in the city centre.

- The RC buildings in the high damage areas were mid-rise (four to six storeys) as against one to three storeys in the surveyed area.

- The area in the centre of the city is likely to have a somewhat deeper soil profile than near the station. This may have contributed to a further increase of the motion in the 0.3 to 0.6 seconds period range, over the recorded value of 1.06g.

By fitting the damage distributions to the vulnerability functions shown in Appendix A, the PSI intensity in the surveyed area was found to be 9.6, equivalent to an intensity somewhat higher than VIII on the MSK scale.

In order to relate the experienced ground motion to the observed damage in the surveyed area, a detailed analysis of the main shock record was carried out, giving a range of parameters with which correlations could be attempted; these parameters have already been presented in Table 3.2.

In Figure 6.7 peak horizontal ground acceleration (PHGA) and mean response spectral acceleration (MRSA) in the period range 0.1 to 0.3 seconds are plotted against PSI value for the fifteen surveys carried out to date, together with the corresponding regression lines. Of the two ground motion parameters shown, the MRSA (0.1 to 0.3 s), with a correlation coefficient R^2 of 0.84, shows a slightly better correlation with the PSI scale than does the PHGA, for which the R^2 value is 0.82. Other parameters, such as RMS acceleration and effective peak acceleration were also found to correlate well with the PSI scale.

The data points from the Erzincan survey presented here are indicated by arrows on Figure 6.3. Obviously, the Erzincan record is a very strong one, almost the strongest of the fifteen records for which such a survey has been carried out. While both points lie above the relevant regression lines, which indicate a PSI value of 11 to 12 corresponding to these accelerations, the level of agreement is comparable to that achieved in other surveys.

6.3 Rural Areas

A total of thirteen villages in the region around Erzincan were visited, with locations ranging from remote mountain sites to the north, through the foothills of the mountains (very near to the North Anatolian Fault), to the central, flat part of the basin. The purposes of these visits were to investigate damage out of the city, which had scarcely been reported, and to compare the extent of damage in villages located on high, rocky ground with those located in the flat part of the basin, on deep sediments. The villages visited are listed below and their locations are shown in Figure 2.1.

To the west of Erzincan:
- Calabzur and Cazanfer (10 km from Erzincan, very near the North Anatolian Fault);
- Çukurkuyu and Yalnizbag (6-8 km from Erzincan, at the edge of the basin);
- Vartansah and Berkisor (8 km from Erzincan, in the flat part of the basin);
- Hashasi and Ulalar (4 km from Erzincan, in the flat part of the basin).

To the east of Erzincan:
- Uzumlu (21 km from Erzincan, at the northeastern edge of the basin);
- Suleymanli and Altinbasak (20 km from Erzincan, in the flat part of the basin).

North of the North Anatolian Fault:
- Davarli (12 km to the west of Erzincan, almost exactly on the North Anatolian Fault);
- Mecidiyeh (15 km north of Erzincan, in the mountains, altitude 2300 m).

A detailed damage survey carried out in the villages of Çukurkuyu and Yalnizbag is described below. This is followed by more descriptive accounts of damage levels in the other villages visited by the team.

6.3.1 Damage survey in the villages of Çukurkuyu and Yalnizbag

These two villages are situated very close together in the foothills of the mountains on the northern edge of the basin, about 7 km from the centre of Erzincan. Their populations are 2734 and 1274 respectively, and the total residential building stock is estimated to be in excess of 300 houses. Damage in both villages was quite severe, with about 70% of buildings in Yalnizbag rated D3 or worse. The Prefecture of Erzincan reported 5 and 21 fatalities in Çukurkuyu and Yalnizbag respectively (0.65% of the combined population).

These villages, being near the capital of the province, are quite different from the more remote settlements, in that a significant proportion of their building stock consisted of one to three storey RC frames. The masonry houses had adobe or concrete block walls, set in cement mortar, nearly always with an RC ring beam, and were one or two storeys high. Himis housing was not common in these villages. Buildings used as animal sheds or for storage purposes were of much weaker construction, usually adobe bricks, with mud mortar and heavy timber log joists, resting on the walls.

A survey was carried out of RC and adobe masonry buildings located on either side of the road that runs through the villages; the weaker buildings not used for human habitation were excluded from the survey. A survey in this location provided a particularly good opportunity to compare the performance of RC and masonry buildings since the majority of buildings had heights ranging from one to three storeys, and hence had similar natural periods. (In Erzincan most of the RC buildings that suffered serious damage were four to six storeys high, while the non-RC houses usually had only one or two storeys. In the area around the strong motion instrument, the non-RC buildings were predominantly timber framed or himis, making a comparison between RC and masonry very difficult.)

Damage distributions for the two construction types surveyed are shown in Figure 6.8. Due to time limitations, it was only possible to survey a total of 30 buildings (16 RC and 14 adobe masonry). The observations made below may therefore not be fully representative of the behaviour of buildings in the villages as a whole, and should be treated with caution.

Most of the RC houses behaved well, suffering only non-structural damage of level D1 or D2 (fine or wide cracks in the infill panels). However, there were also two completely collapsed RC houses, one of which is shown in Plates 4.16. These two buildings had soft ground floors, used for storage and parking respectively.

The masonry houses surveyed were mostly single storey, with walls made of adobe bricks or hollow concrete blocks, set in cement mortar, with lightweight timber roofs, covered by corrugated sheet (Plates 4.20 and 4.21). Overall, the masonry houses suffered more severe damage, with an average damage degree of 2.5 for masonry compared to 1.4 for RC frames. However, unlike the RC buildings surveyed, while several of the masonry buildings showed high damage levels, none suffered complete collapse.

The PSI intensity, taking the mean of the values determined for the two building types, is equal to 8.9, which is equivalent to an MSK intensity of VIII. Referring to the regression shown in Figure 6.7, this value implies a mean response spectral acceleration in the 0.1 to 0.3 seconds period range (5% damping) of between 0.5 and 0.7g. Both the PSI value and the MRSA are considerably lower than those obtained in the vicinity of the meteorology station (9.5 and 0.9g respectively).

6.3.2 Other villages on the west side of the basin

The village of *Calabzur* is situated on a hillside 500 metres south of the North Anatolian Fault. The building types are quite different from those in the two large villages discussed above. The predominant building type is himis but in recent years concrete block masonry has become increasingly common. Most of the concrete block masonry is set in cement mortar and has an RC ring beam at the level of the tops of the windows. All the houses in this village are single storey, with timber roof joists supporting galvanised corrugated iron sheets or clay tiles.

Although about 75% of the buildings suffered damage level D3 or worse, with many of the houses destroyed, there were no fatalities. Most of the collapsed masonry houses had suffered failure of one or more walls, but in most cases the roof remained standing due to the presence of an RC ring beam. Those that did collapse completely did not trap or crush their occupants since the roof structure was very light. Of the himis houses, many were shattered, with all the plaster shaken out revealing the infill adobe bricks, and some were leaning badly, but the timber frame usually retained sufficient strength to prevent complete collapse (see Plate 4.22).

Animal husbandry, mainly sheep, was the main source of income in the village. Most of the households had animal sheds, built from adobe blocks with heavy roofs of compacted earth on timber joists, many of which collapsed, killing the animals within. Villagers estimated that they lost about one third of their livestock in the earthquake.

Cazanfer is situated just 500 metres south of Calabzur and has a similar setting. The building characteristics and damage here were very similar to Calabzur.

Vartansah and *Berkisor* are located 7 km north-west of Erzincan, on the lower, flat part of the basin. Their population is probably included in the 2734 of Çukurkuyu, which is nearby. The predominant building type is adobe masonry of one or two storeys, with two timber hatils around the walls below and above the windows; some houses have an RC ring beam. Himis houses are also quite common. These villages suffered rather less damage than those on higher ground. In particular, the damage to the traditional adobe houses was less severe than in nearby Çukurkuyu. There were no major collapses in either village.

The villages of *Hashasi* and *Ulalar* are located just to the west of Erzincan. Ulalar is a large village with a population of 4000 (including several smaller villages in its surroundings) and is located on roughly the same ground conditions as the central part of Erzincan city. Damage in both villages was less severe than in Çukurkuyu and Yalnizbag. However, in Ulalar there were seven deaths and around 100 injuries. A four-storey RC framed house in Ulalar which collapsed during the earthquake is shown in Plates 4.6 and 4.17.

Outside the basin to the south-west, the town of Kemah, 40 km from Erzincan, suffered severe damage to one school building and one government building. Beyond Kemah, numerous landslides and building collapses have been reported as far south-west as Iliç, 80 km from Erzincan, possibly due to some form of channelling along the Euphrates valley (Hencher, 1992).

6.3.3 Villages on the east side of the basin

Uzumlu is the second largest town in the area affected by the earthquake, with a population exceeding 17,000. It is located in the foothills of the mountains, 20 km east of Erzincan and 5 km north of the E23 Sivas-Erzurum national road. The houses are similar to those in the western part of the basin, with himis and adobe masonry with timber hatils being widespread among the older buildings, while concrete block masonry and RC frames with brick infill are more common among the new buildings.

Unlike the western part of the basin, where villages on high ground were seriously damaged, very little damage was observed in the old part of Uzumlu, with only 15% of the buildings rated D2 or worse (equivalent to an MSK intensity of just over VI). This is in spite of the fact that Uzumlu lies very close to the North Anatolian Fault. It is likely that the damage level is related to the low sediment depth in the foothills. As discussed in Section 5.4, a detailed survey at the new Cooperative Housing Estate nearer to the flat part of the basin yielded an estimated PSI value of 9.8, corresponding to an MSK intensity higher than VIII.

The EEFIT team were unable to visit the village of *Karakaya*, 5 km to the east. Reports suggest that this village was much more severely damaged than the old part of Uzumlu, with 22% of its buildings suffering damage degree D3 or worse.

Suleymanli and *Altinbasak* are situated next to each other on the flat part of the basin, south of Uzumlu, and have a combined population of 2230. Unlike Yalnizbag, there are very few RC buildings. Most of the old houses were either himis or adobe masonry, often with the old style heavy, flat roofs made of soil compacted onto timber log joists, which rest on the walls with little or no overhang. Most of the adobe houses had at least two timber hatils around the perimeter of the building.

Damage in these villages was quite severe, though not as high as in some of the villages to the west of Erzincan. Government statistics state that 22% of the houses were destroyed, a figure which agrees closely with EEFIT's assessment. According to Altinbasak's muftar, the amount of houses that are beyond repair is actually 40%. In terms of human casualties, Altinbasak was one of the most severely affected villages, with 20 fatalities (1% of the population). A further 500 people were trapped under the rubble, but were rescued by local people within one hour (local anecdotal reports). Considerable damage was observed on the asphalt road that links Suleymanli with Altinbasak, which was said to have happened during the earthquake. The railway line that runs through Altinbasak has also suffered some damage due to liquefaction.

A few kilometres south-east of Altinbasak is the village of *Mertekli*, which according to government statistics suffered even more damage (32% of houses destroyed) although the loss of life was only 0.4% of the population. This village was not visited by the EEFIT team.

6.3.4 Villages to the north of the North Anatolian Fault

Davarli, a small village of 200 people situated at the western edge of the Erzincan basin, lies almost exactly on the North Anatolian fault. The houses are predominantly adobe masonry with timber hatils in the walls. There are just 42 houses, of which 32 (76%) suffered damage degree D3 or worse, with just a few timber frame houses surviving. During the earthquake the whole population, except a few children, was inside the mosque, which suffered extensive damage, but did not collapse. There were therefore no human casualties in this village.

The village of *Mecidiyeh* is situated 15 km north of Erzincan, in the Sipikor mountains, at an altitude of 2300 metres. The village is located near the Erzincan-Çayirli road, which is usually closed during the winter months. According to the 1990 census, it has a population of 202 people, but its winter population is around 100. The predominant building type in Mecidiyeh is stone masonry, which was scarcely seen in the Erzincan basin. More recent houses are made with concrete blocks and cement mortar, but without timber hatils or concrete ring beams. Almost all the houses are one storey high. Damage in the village was not as serious as in the basin (about 20% of buildings lost one wall in an out-of-plane collapse).

6.4 Discussion: The Basin Effect

One objective of the studies outlined in this chapter was to assess whether the deep sediments in the Erzincan basin caused a significant modification of the ground motion as it was transmitted up from the bedrock. The main findings can be summarised as follows.

Erzincan is situated quite near the northern edge of the basin, and the limited data available suggest that sediment depth may vary quite considerably across the city (Figure 6.3). However, no obvious basin effect was discernible within the city itself, where the damage distribution could largely be explained by the distribution of building types and heights.

To the west of the city, damage appeared to be more severe in the mountain villages close to the North Anatolian Fault than in the flat part of the basin. This could be taken as implying that high frequency amplitudes were reduced due to the sediments, or simply that damage was related to proximity to the fault.

To the east, observed damage levels increased towards the centre of the basin, though nowhere were they as severe as in the villages close to the fault on the west side. On the basis of very limited evidence, it seems that damage in mountain villages a short distance north of the fault was much less severe than in the basin.

PSI intensities obtained from the detailed damage surveys carried out by the EEFIT team are summarised in Table 6.2. It should be remembered that the large discrepancy between the north and south halves of the Uzumlu housing estate may be due to factors other than sediment depth (see Section 5.4). Nevertheless, it is noticeable that by far the highest intensity was achieved on a site near (but not at) the edge of the basin, with an intermediate sediment depth, while the lowest intensities were on rocky sites.

No attempt has been made to model the soil response computationally because of lack of data on sediment depths and properties. However, analyses carried out for another recent near-

field earthquake (EEFIT, 1991) suggest that amplifications in the low period range typical of low-rise housing (0.1 to 0.3 seconds) are greatest for quite shallow soils, while deeper sediments are likely to cause amplifications in a higher period range, which would only be significant for taller buildings.

Survey	Location	PSI value
Meteorology station	Close to city centre - deep sediments	9.6
Çukurkuyu and Yalnizbag	To the west, near fault - edge of basin	8.9
Cooperative Housing Estate, Uzumlu	To the east, near fault - edge of basin	9.75
	• Northern half - shallow sediments	7.0
	• Southern half - deeper sediments	11.5

Table 6.2 Summary of PSI Intensities Calculated from Damage Surveys

It is difficult to draw firm conclusions from this rather disparate evidence. Clearly, the effect of the deep sediments is not a simple amplification of ground motions, as occurred at Mexico City, for example (EEFIT, 1986). Any basin effect which does exist is an extremely complex one, its predominant feature possibly being amplification in the period range affecting low-rise buildings in the relatively shallow sediments near the edge of the basin, with perhaps even some attenuation in the areas of deeper sediments.

Figure 6.1 Distribution of building damage in Erzincan by district, all construction types (data provided by Ministry of Public Works and Resettlement)
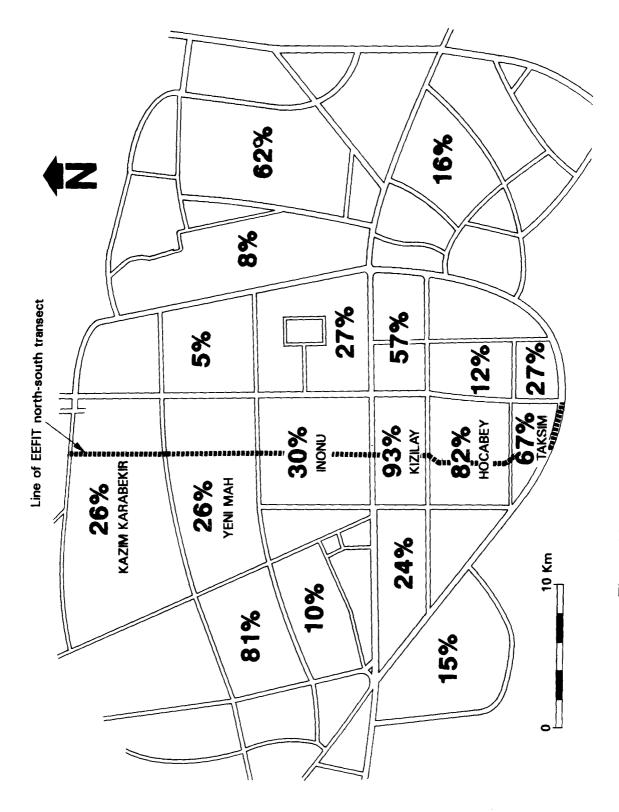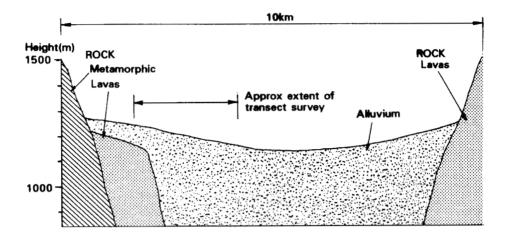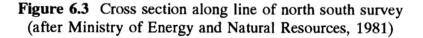
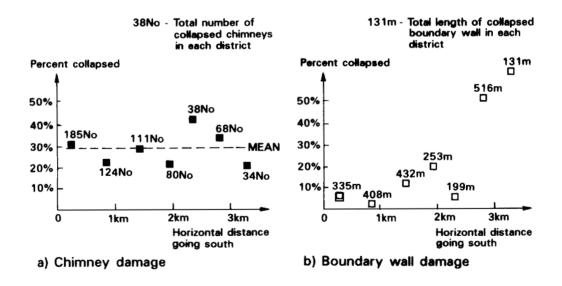Figure 6.2 Line of north south survey through Erzincan

Figure 6.3 Cross section along line of north south survey (after Ministry of Energy and Natural Resources, 1981)

a) Chimney damage

b) Boundary wall damage

Figure 6.4 Variation of damage along north south survey line

Figure 6.5 Surveyed area around the strong motion instrument

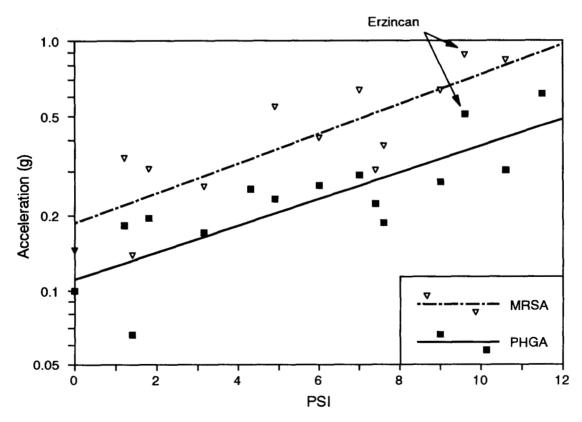**Figure 6.6** Non-cumulative damage distributions in the vicinity of the strong motion instrument

Figure 6.7 Correlation of peak horizontal ground acceleration (PHGA) and mean response spectral acceleration (MRSA) for periods of 0.1 to 0.3 seconds, with PSI value
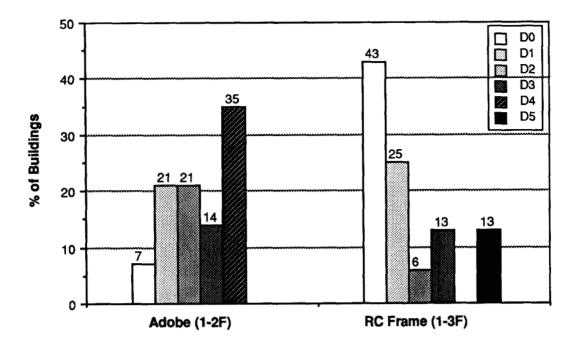
Figure 6.8 Non-cumulative damage distributions for adobe masonry and RC frame buildings in the villages of Çukurkuyu and Yalnizbag

Plate 6.1
Damage survey around
meteorology station.
Typical timber frame house,
the main building type
during the post-1939
reconstruction. They
suffered little damage.

Plate 6.2
Damage survey around
meteorology station. Himis
timbered houses with adobe
infill are one of the most
common traditional Turkish
building types in rural
areas, but their number is
gradually decreasing due to
scarcity of timber material.
These houses performed
reasonably in the
earthquake.

Plate 6.3
Damage survey around
meteorology station.
Typical solid brick masonry
building with lime mortar,
found mostly in the
southern part of the
surveyed area. The loss of
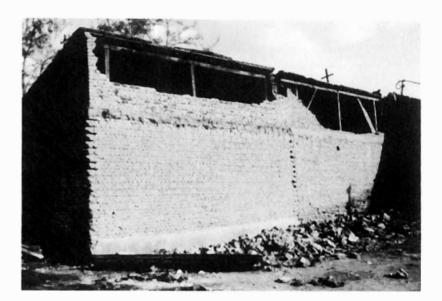a gable wall was the most
common damage.

Plate 6.4
Damage survey around meteorology station. Typical residential RC framed building. Except for one very poorly designed building, these structures survived the earthquake with light damage. However, 750-1000 metres away many such buildings collapsed, mostly due to soft ground storeys. None of the 23 buildings in the surveyed area had a soft storey.

7. OTHER ASPECTS OF THE EARTHQUAKE

7.1 Distribution of Casualties

As stated previously, the total death toll in Erzincan city was 394, with a further 150 fatalities in the surrounding rural areas. The number of injuries is estimated at 2,100, of which 688 required hospitalisation. These figures are low in comparison with other earthquakes in Turkey this century, but agree well with projections made by the Turkish Directorate of Disaster Affairs of 399 deaths, 728 serious injuries (Ergunay, 1992).

Obviously, information on how the casualties were distributed among the different building types in the city would be extremely useful. Unfortunately, due to the chaos caused by a major earthquake and the urgent nature of the relief operation, no such data are kept by the authorities. From interviews with relief workers and organisers, it was possible to gather a limited amount of information on the distribution of casualties in Erzincan. Most of this relates to the collapse of individual large buildings:

- The Hotel Roma, a mid-rise RC structure, contained between 25 and 30 people when the earthquake struck. The building collapsed, killing 13 of its occupants, 12 of whom were on the ground floor.

- In the nursing school at the state hospital there were about 50 student nurses inside the west wing of the building when it collapsed; 22 of these died.

- In the insurance hospital it was reported that 21 people lost their lives. EEFIT was unable to obtain concrete evidence of the exact numbers of people killed or rescued.

- In a three storey RC building in Yalnizbag village, three out of five occupants were killed, all of them on the top floor. The survivors reportedly suffered only minor injuries.

Thus the fatality rate in the collapse of these large RC structures was around 50% in each case. In the first two days after the earthquake (14 and 15 March), between 200 and 300 trapped people were rescued from the rubble of collapsed buildings, less than the total number killed in the city. Allowing for the fact that some occupants may have been able to walk away with little or no injury, these figures support the hypothesis that collapsed RC structures in Erzincan caused the death of about 50% of their occupants.

7.2 Performance of Lifelines

All transportation links with Erzincan suffered little disruption due to the earthquake, with the worst damage to roads being minor cracking and rockfalls, which were easily repaired. This state of affairs could be regarded as rather fortuitous in view of the excessively steep embankments noted on several vital road links (see Section 4.7). The railway line suffered some localised damage due to liquefaction, while the airport was virtually unaffected.

Telecommunications suffered some short-term problems, with local lines out of action for one day and long distance lines for three days. A few emergency lines were set up about ten

hours after the earthquake, but since there was no more substantial backup equipment the city was cut off for some time after the earthquake.

Electric power is supplied to Erzincan from a hydroelectric station 120 km to the south. No serious damage was incurred by power supply facilities during the earthquake. Most losses of electrical power were caused by emergency shut-downs to minimise the fire risk, rather than by failure of equipment or power lines, with supplies restored within two days. The main transformer station on the eastern edge of Erzincan suffered light damage but remained functional.

Some serious difficulties were experienced with the water supply, despite the fact that no damage to water mains was reported. The problems were caused by failures of pipe connections to damaged buildings, which took up to a week to repair.

7.3 Emergency Relief Operation

Search and rescue (SAR) and emergency response information is hard to gather. However, the team was able to talk to several relief workers and officials, from whom it was possible to build up a reasonable picture of the aftermath of the earthquake.

A crisis committee was formed, consisting of the governor of the city, the provincial representative of the Ministry of Public Works and the head of the local police. For the first four days all available manpower concentrated on search and rescue operations and medical treatment for the injured. During the first couple of days the operations were coordinated from the street, as officials were reluctant to enter buildings due to the risk of collapse.

A total of ten international SAR teams worked in Erzincan. However, as in previous earthquakes, they were unable to reach the affected area sufficiently quickly to play a major role in lifesaving. Nevertheless, the local rescue teams and doctors reported that the international teams' experience and organizational methods were useful. An emphasis on training before the disaster is therefore seen as an essential part of the strategy of international SAR teams. The local teams struggled in the first few days, due to their lack of experience or expertise in dealing with collapsed reinforced concrete buildings, this being the first major earthquake in Turkey to cause damage to large numbers of such buildings. There was a small number of fires in Erzincan. The fire squads were reportedly insufficiently experienced in dealing with SAR work.

The army has a significant presence in the region, its installations occupying a large area on the northern edge of the city. The army participated extensively in the SAR activities, but was seriously hampered by the collapse of the military hospital, which was its first priority in the immediate aftermath of the earthquake.

The response and solidarity from neighbouring provinces was very quick. Hundreds of ambulances were sent to Erzincan in the hours following the disaster; these were extremely useful, allowing the injured to be transported to safer areas. About 800 injured people were sent to Erzurum (200 km east of Erzincan), with an additional 1300-1500 treated locally.

Probably the most serious failure in terms of disaster preparedness was the inadequate construction of the hospital facilities. All three hospitals in Erzincan suffered major structural collapses, causing considerable loss of life and severely disrupting their ability to deal with the large number of medical emergencies. The most intact of the three was the state hospital in the city centre, but even here the collapse of the nursing school dominated the efforts of the hospital staff in the first few hours after the earthquake.

80% of the casualties referred to the state hospital arrived during the first 24 hours. In the first hours after the earthquake the hospital treated about 300 people for burns caused by overturned tea pots etc. The majority of these were children carried by their parents, often themselves injured. The staff reported that they were not able to respond well to the unexpectedly high number of burns cases.

Other problems at the state hospital included lack of power and water supplies. The hospital has a backup generator, but this was located inside a damaged building which staff were understandably reluctant to enter, and so was not brought on line. The lack of clean water meant that, for the first few days, many operations were performed without adequate sanitation. The hospital did not have a proper mobile unit, so most of the operations were carried out in the open air, or in tents installed after the first day. For several days there was also insufficient blood for transfusions.

8. RECONSTRUCTION AND RECOVERY PROGRAMME

8.1 Short Term Measures

During the first week, no demolition was carried out because of the possibility of trapped occupants still being alive. After that, collapsed and severely damaged buildings began to be cleared rapidly.

20,000 tents and 500 tonnes of food were distributed in the affected area within three days of the earthquake. Nevertheless, there were numerous complaints that people whose houses were not damaged had snatched tents, before genuinely homeless people were able to claim them. Several weeks after the earthquake many families whose houses were relatively lightly damaged continued to camp out because of fear of further aftershocks.

It was estimated that 1344 families in Erzincan lost their homes in the earthquake. Six sites within the city were set aside for the provision of temporary accommodation for the homeless. Two weeks after the earthquake the erection of 2000 temporary prefabricated shelters provided by Azek Tepe, a company affiliated to Ankara University's Department of Engineering, was well underway. The manufacture of these prefabricated houses was commissioned by the United Nations in 1991, in response to the Kurdish refugee crisis that affected Southern Turkey at the end of the Gulf War. It was therefore somewhat fortuitous that the housing could be provided so quickly after the earthquake.

8.2 Long Term Reconstruction

The long-term reconstruction programme in Erzincan is being financed by a World Bank loan of $285 million over the period 1992-95. The loan covers demolition, reconstruction, rehabilitation and repair of a range of buildings in urban and rural areas, together with training and studies to reduce earthquake vulnerability. Work to be undertaken includes:

- approximately $30 million worth of new housing;
- construction of three new hospitals, providing a total capacity of 475 beds;
- 400 government offices and related buildings;
- over 100 km of roads;
- rehabilitation of 3,700 shops and offices;
- repairs to water supply and sewage treatment plant.

Reconstruction of Erzincan and the surrounding villages is now in progress. In the city, substantial efforts are being made to strengthen and repair a large number of moderately and heavily damaged structures, while in the villages many new houses are being built (Murakami, 1992). Rebuilding programmes are based on surveys carried out by Middle East Technical University (METU) and Istanbul Technical University, who make decisions on whether buildings should be repaired or replaced. Detailed design work is put out to tender, with guidelines and checking by METU and other universities. Construction is supervised by a team of government engineers, who again refer to METU for expert advice.

One of the first projects to be started was a $7 million contract for the rebuilding of schools and public buildings, for which construction commenced in September 1992. By mid-

November approximately 3,000 buildings were under construction by 20 different contractors. A second substantial phase of construction, including housing, shops and businesses, is due for completion by the winter of 1993/94. Although efforts have been made to control the quality of the reconstruction, these have met with mixed success.

Those whose houses have been destroyed have the choice of buying a new home from the government or rebuilding themselves (subject to the same controls as the government-built housing). For either option, the government will provide twenty year interest-free loans, making the new houses virtually free, since Turkey has a very high inflation rate.

9. CONCLUSIONS

1.   The earthquake had a surface wave magnitude of approximately 6.8 and was centred very close to the city at a depth of about 27 km. Accelerations of up to $0.5g$ were recorded in the city centre, together with very high horizontal velocities. Motions were more intense in the east-west than in the north-south direction, consistent with a strike-slip fault mechanism for the earthquake. The associated fault break was on the North Anatolian Fault; a fault length of about 40 km has been inferred, but there is no direct evidence for this.

2.   From observations of building damage, it was possible to infer a greater magnitude of shaking in the east-west than in the north-south direction, a finding which agrees with the measured strong motions in Erzincan. Many damaged structures visited by the team had fallen to the west, or suffered more severe damage at their western end.

3.   The damage and loss of life in the earthquake were broadly in line with government projections, although considerably lower than might have been predicted by simple extrapolation from previous Turkish earthquakes this century. The building stock in Erzincan is all relatively modern (post-1939). The city is thus probably the least seismically vulnerable in eastern Turkey; a similar magnitude earthquake occurring in a high density area elsewhere in Turkey could cause significantly greater damage.

4.   In the city of Erzincan, the most serious damage was to RC structures having three or more stories; around 80 such buildings suffered complete or partial collapse, accounting for about 75% of the total loss of life. The earthquake was the first to affect new buildings resulting from the recent Turkish population and construction boom. Buildings in Erzincan are likely to be typical of modern structures in most parts of Turkey, making the design faults noted in Chapter 4 particularly important.

5.   While poor design and construction were often observed, no major deficiencies in codes of practice were evident. The principal need is therefore improvement of the understanding and enforcement of existing codes, rather than major revisions.

6.   In the rural areas, the behaviour of masonry buildings was as expected. The use of timber hatils or concrete ring beams substantially improved performance. Timber framed and himis houses mostly survived quite well due to the ductility of the timber, while rubble stone masonry houses fared much less well. Low-rise RC housing only suffered serious damage when soft storeys were present.

7.   From the surveys carried out, it is not possible to identify a simple amplification effect due to the deep sediments in the Erzincan Basin. There is some evidence that shallow sediments near the edge of the basin caused greater amplifications than deep sediments, but this is not conclusive.

8.   Surveys carried out in and around Erzincan have made valuable additions to the Martin Centre seismic damage database. The correlation obtained for this earthquake between observed damage and a number of strong motion parameters was consistent with findings from other earthquakes.

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APPENDIX A: THE PARAMETERLESS SCALE OF INTENSITY (PSI)

The PSI is a damage-based seismic intensity scale which has been developed at the Martin Centre, University of Cambridge, as part of a programme of work aimed at evaluating earthquake damage potential for various building types, and at establishing a range of useful strong motion parameters for use in seismic risk assessment. Vulnerability functions based on the PSI scale are now available for most common building types. The Martin Centre has collected an extensive database of damage surveys around strong motion stations after earthquakes, which serve as a link between the PSI scale and actual recorded ground motion (Spence et al, 1992).

A building by building survey is carried out in the region around the triggered strong motion instrument. This extends up to a maximum radius of 500 m around the station, so long as the geological and topographic conditions do not change significantly. Information recorded for each building includes:

- construction type;
- damage degree;
- number of storeys;
- condition and any other observations about the building.

Damage degrees are defined using the MSK scale, in which structures are classified in a range between D0 for undamaged to D5 for collapsed. Specific definitions of damage degrees for masonry and RC frame buildings are shown in Table A1. In cases where structural and non-structural elements show differing damage, the higher damage degree is assigned.

At the end of the survey, damage distributions are produced for each construction type that has a sample of more than 20 buildings; these can then be fitted to the PSI vulnerability functions. Vulnerability functions for various structural types are shown in Figure A1 (Coburn et al, 1990). These curves show the expected damage level in relation to the PSI scale and the corresponding mean response spectral acceleration in the period range of 0.1 to 0.3 seconds. The area between each curve corresponds to a damage degree, while the curves themselves express the vulnerability in cumulative terms (meaning $\geq$ D1, $\geq$ D2, etc.). The MSK'81 scale definitions for each intensity level are also shown for comparison.

Data collected from numerous such surveys show that the PSI value correlates well with a number of measured strong motion parameters, including the peak horizontal ground acceleration (PHGA), the effective peak acceleration (EPA) and the mean response spectral acceleration (MRSA) in the period range 0.1 to 0.3 seconds (Pomonis, 1992). These correlations are discussed further in Chapter 6 of this report.

	Unreinforced Masonry Buildings	
	Load bearing walls:	*Non-load bearing walls:*
D1	Fine cracks (< 3 mm), plaster dislodged	Cracks < 10 mm
D2	Cracks between 3 and 10 mm	Wider cracks spreading diagonally or dislodging of wall
D3	Wider cracks spreading diagonally, dislodging of wall or partial corner failure	Partial collapse, usually at top of gable wall
D4	Partial collapse (significant leaning of structural wall or loss of one or more corners)	Total collapse of one or more walls
D5	Structural collapse (loss of one or more structural walls and more than half of roof or floor system)	N/A

	RC Frames or Reinforced Masonry Buildings	
	Structural elements:	*Infill panels:*
D1	No damage	Small boundary cracks
D2	Cracks < 10 mm, usually near joints	Severe cracking, usually diagonal
D3	Severe cracking, some loss of concrete	Collapse or severe crushing of the infill panels
D4	Buckling of column reinforcement, large loss of concrete	N/A
D5	Complete or partial collapse of building	N/A

Table A1   Assignment of Damage Degrees to Masonry and RC Buildings

Figure A1 PSI vulnerability functions for various structural types