THE VRANCEA, ROMANIA
EARTHQUAKES OF 30-31 MAY 1990

A FIELD REPORT BY E.E.F.I.T.

By:

Antonios POMONIS
The Martin Centre for Architectural and Urban Studies
University of Cambridge, Department of Architecture

Dr. Andrew W. COBURN
Cambridge Architectural Research Ltd.

and Dr. Steve LEDBETTER
University of Bath

November 1990

EEFIT
Earthquake Engineering Field Investigation Team
Institution of Structural Engineers
11, Upper Belgrave Street
London SW1X 8BH, U.K.

CONTENTS

1. INTRODUCTION

1.1 The effects of recent earthquakes in Romania

Two strong, intermediate depth earthquakes on May 30 and 31, 1990 occurred in the seismically active region of Vrancea of Northeast Romania. The effects of these earthquakes were the strongest in Romania since the devastating March1977 earthquake that caused the death of more than 1500 people. This is the fourth time that very strong damaging earthquakes have occurred in the Vrancea region since 1940. The seismic parameters of all these events are summarised in Table 1 (Radu, et. al. 1991).

Table 1: Seismic parameters of the major earthquakes in Vrancea

Date (Y/M/D)	Occurr. Time (Local Time)	Epicenter (^{0}N - ^{0}E)	Depth (km)	Seism. Mom. M_0 (dyn.cm)	Moment Magn. (M_W)	Richter Magn.
1940/11/10	3:39:07	45.80 - 26.70	133	5.1 * 1027	7.8	7.4
1977/3/4	21:21:56	45.78 - 26.78	93	2.5 * 1027	7.5	7.2
1986/8/30	23:28:37	45.53 - 26.47	133	1.0 * 1027	7.3	6.9
1990/5/30	13:40:06	46.03 - 26.89	89	0.47 *1027	7.1	6.7
1990/5/31	3:17:49	45.83 - 26.89	79			6.1

It is notable that the main shock of May 30th was quite strong and somewhat shallower than all the other major events. This earthquake caused considerable building damage and the human casualties were 8 deaths, 75 seriously injured and 221 lightly injured (Adevarul Newspaper, 31 May). Fortunately there were not any major building collapses and damage to civil engineering works was not extensive. Damage to industrial facilities was more common but no major collapse incidents were reported. A large number (at least 38) of strong motion records were obtained by Romania's Building Research Institute (INCERC) during the main shock and about 20 during the aftershock. It is hoped that their analysis will shed new light, regarding intermediate depth earthquakes and their effects on soft alluvial soil deposits.

Two of the deaths occurred in Bucharest in the district of Colentina, more than 90 km from tre epicentre, when the heavy plasterboard of a large 11-storey apartment block collapsed along the expansion joint (seismic gap), due to pounding between the two separate parts of the structure (Plates 1 and 2). As a result of this pounding the cement-lime plasterboard was crushed, detached and fell on the ground killing the two people trying to evacuate a ground floor shop that unfortunately had its entrance along the seismic gap line (Plate 3). The occurrence time of the event in mid-day contributed to this fatality occurrence. Intermediate depth earthquakes propagate long period waves at long distances, that are significantly amplified in areas of soft alluvial deposits (as in a large part of Bucharest) and affect mostly high rise buildings built on them. In Colentina and other parts of Bucharest (Plates 4 and 5) but also closer to the epicentral area (Plate 6), this type of damage was very common. It is very fortunate that no other similar incidents happened in Bucharest where a large part of the 1.5 million population lives in similar multi-storey reinforced concrete apartment buildings.

1.2 Background to EEFIT and its mission to Romania

The Earthquake Engineering Field Investigation Team of the United Kingdom (EEFIT) is a group of civil, structural, geotechnical and earthquake engineers as well as architects, planners, and scientists. The aim of the team is to collaborate with colleagues in earthquake-prone countries with the aim of improving the understanding of the effects that earthquakes can have upon the built environment as well as to contribute to the advance of earthquake related research. The long term aim is the mitigation of the effects of earthquakes through improvements in the seismic design of all types of structures and the improved response and preparedness to future earthquakes. To this end, EEFIT organises field investigations in the immediate aftermath of major, damaging earthquakes and publishes reports of its findings on the performance of buildings and civil engineering works.

EEFIT was formed in 1982 as a joint venture between universities and industry. It has the support of the Institution of Structural Engineers and the Institution of Civil Engineers through its society SECED (the British section of the International Association for Earthquake Engineering). EEFIT members have investigated the earthquakes in Liege, Belgium (1983), Chile (1985), Mexico (1985), San Salvador (1986), Loma Prieta, California (1989), Newcastle, Australia (1989), and Philippines (1990).

Shortly after the earthquakes in Romania EEFIT dispatched a team of three members that worked in collaboration with Romania's Building Research Institute (INCERC) for one week. The EEFIT team consisted of: Andrew Coburn, an architect, planner with experience in earthquake protection and vulnerability studies from Cambridge Architectural Research Limited, Steve Ledbetter a civil engineer from Bath University, School of Building Science, at present chairman of EEFIT and Antonios Pomonis a structural engineer with earthquake experience at present with University of Cambridge, Department of Architecture, The Martin Centre for Architectural and Urban Studies. The team arrived in Bucharest on 14 June 1990 and returned to the UK on 21 June. The days of Friday and Saturday were spent with investigations in Bucharest and meetings with the members of INCERC and from Sunday until Wednesday a field trip to the Vrancea region was organised with the help of engineer Emil-Sever Georgescu of INCERC. A mini-bus was rented from Romania 's Tourism Organisation that allowed the team to visit the towns of Buzau, Focsani, Valenii de Munte and Ploiesti that suffered some damage and had stations that recorded both events. The three EEFIT members, were joined and aided in this trip by two Romanian engineers (Emil-Sever Georgescu and Olga Stancu) and an interpreter, all staff of INCERC. Detailed damage surveys were carried out in the vicinity of the strong motion observation stations in Buzau, Focsani and Valenii de Munte while further investigation were carried out around the stations in the town of Ploiesti and in Mirinescu street (Bucharest). During the meetings in INCERC active participation and guidance were given by Dr. Horea Sandi, head of INCERC's Earthquake Engineering and Seismology Division.

1.3 The contents of this report

Romania has just emerged from a long period of political isolation, and knowledge on the seismicity, the effects of the 1977 earthquake, the development of the building industry, and other related issues is somewhat limited in Western Europe. Considering the limited extent of damage, and lack of general information about the country, this EEFIT report will discuss the seismicity of Romania, as well as issues related to the types of residential buildings constructed in Romania during the post-war period and their seismic vulnerability. Chapter 2 presents and analyses the seismicity in Vrancea and other parts of Romania. Chapter 3 briefly discusses the urban geography in relation to the seismic zonation of Romania. Chapter 4 describes the residential building typology and discusses the seismic vulnerability of masonry and reinforced concrete structures through the findings of the damage surveys carried out by INCERC after the 1977 earthquake. Chapter 5 debates on the relationship between seismic vulnerability and strong ground motion and presents the results and conclusions from the three damage surveys carried out by EEFIT. Finally in Chapter 6 some conclusions are listed. It must be pointed out that the report is only about residential, office and public buildings and does not cover industrial facilities and civil engineering works.

2. SEISMOTECTONICS AND SEISMICITY OF ROMANIA

Romania has a surface area of 237,500 km^2 (97% of U.K.) and population of 23 million people (population density 97 people per km^2, or 40% of Britain's). The most important geographical feature of the country is the Carpathian Mountains that form a curve which separates the country into two distinct geographical and cultural zones . The highest peak of these mountains reaches 2544 metres, with at least 35 other peaks exceeding 2000 metres. The mountains spread from the centre of the Northern border of the country southeastwards for 280 kilometres at which point a sudden turn of almost 90 degrees occurs due to the collision between the African and Eurasian plates. As a result the rest of the Carpathians spreads westwards for a further 320 kilometres almost parallel to the Bulgarian borders and the flow of the Danube river. These two parts of the mountain chain are called Oriental and Meridional Carpathians (the latter are often called Transylvanian Alps).

Figure 1 shows the map of Romania with the seven geographical regions superimposed on the seismic zoning map used until the 1990 earthquakes. This zoning map is at present under revision.

Not surprisingly Romania's seismotectonics and seismicity are closely related to this spectacular geomorphological feature. The Oriental Carpathians with their northwestern extension in Eastern Czechoslovakia (Tatra mountains) form the border between two distinct parts of the large Eurasian Plate. These are the Inter-Alpine subplate (covering all the Balkan peninsula in the Near East, Italy and Northwestern Africa in the West, Turkey and the Caucasian region in the Middle East) and the East European plate (covering Moldavia, Ukraine, Russia and Siberia). The Inter-Alpine subplate is further divided in two distinct lithospheric formations within Romania's territory that are separated by the Carpathian mountains. In the SE of the mountains is the Black Sea subplate (also called Moesic subplate) that moves towards the NW, subsiding underneath the Carpathians. In the North and West the rest of the country is covered by the largely aseismic Transylvanian plateau. As a result of this complex tectonic formation considerable crustal movements are observed in Romania. In the zone of the Oriental Carpathians a crustal uplifting of the order of more than 5 mm per year is occurring, while a much smaller rate is observed along the Meridional Carpathians. On the other hand in Transylvania, right behind the mountains a subsistence of about 2 mm per year is occurring in the zone parallel to the Oriental Carpathians. Similarly a subsistence of the same order is registered around the Black Sea coast.

Because of these crustal movements the seismic activity of Romania is considerable with several distinct seismic zones. These zones are of course closely related to the seismotectonic morphology briefly described above. There are basically 5 zones of varying seismic activity: Vrancea, Fagaras, Transylvania, Banat and Dobrogea zones. Vrancea is by far the most seismically active zone of Romania with 97% of the total seismic energy release in the period of 1091-1979.

2.1 Seismicity in the Vrancea region

This zone is around the curvature of the Carpathians where the submerging of the Moesic subplate takes place. The name of this seismic zone is taken from the county of Vrancea that is located in the region. In Vrancea earthquakes of moderate to large magnitude ($6<M<7.5$) with a focal depth ranging between 75 and 175 kilometres occur frequently. The actual area is relatively small, about 8000 km^2 (roughly half the size of Northern Ireland). A few shallow focus earthquakes of smaller magnitude ($M<5.5$) occur within this zone, especially towards its southern part. The average annual energy release in Vrancea is one of the highest in the Eurasian plate borders, but fortunately the large focal depths make these events less destructive. The occurrence of the two earthquakes in May 1990 that have magnitudes larger than 6.3 is certainly having an effect on the overall seismic hazard of the Vrancea region, and it is useful to reappraise here the seismicity of this zone that is of paramount importance to Romanian engineers. Reliable information on the magnitude of the Vrancea earthquakes is available for the period 1900-1981 (for $M>6$) and until present for $M>5.9$.

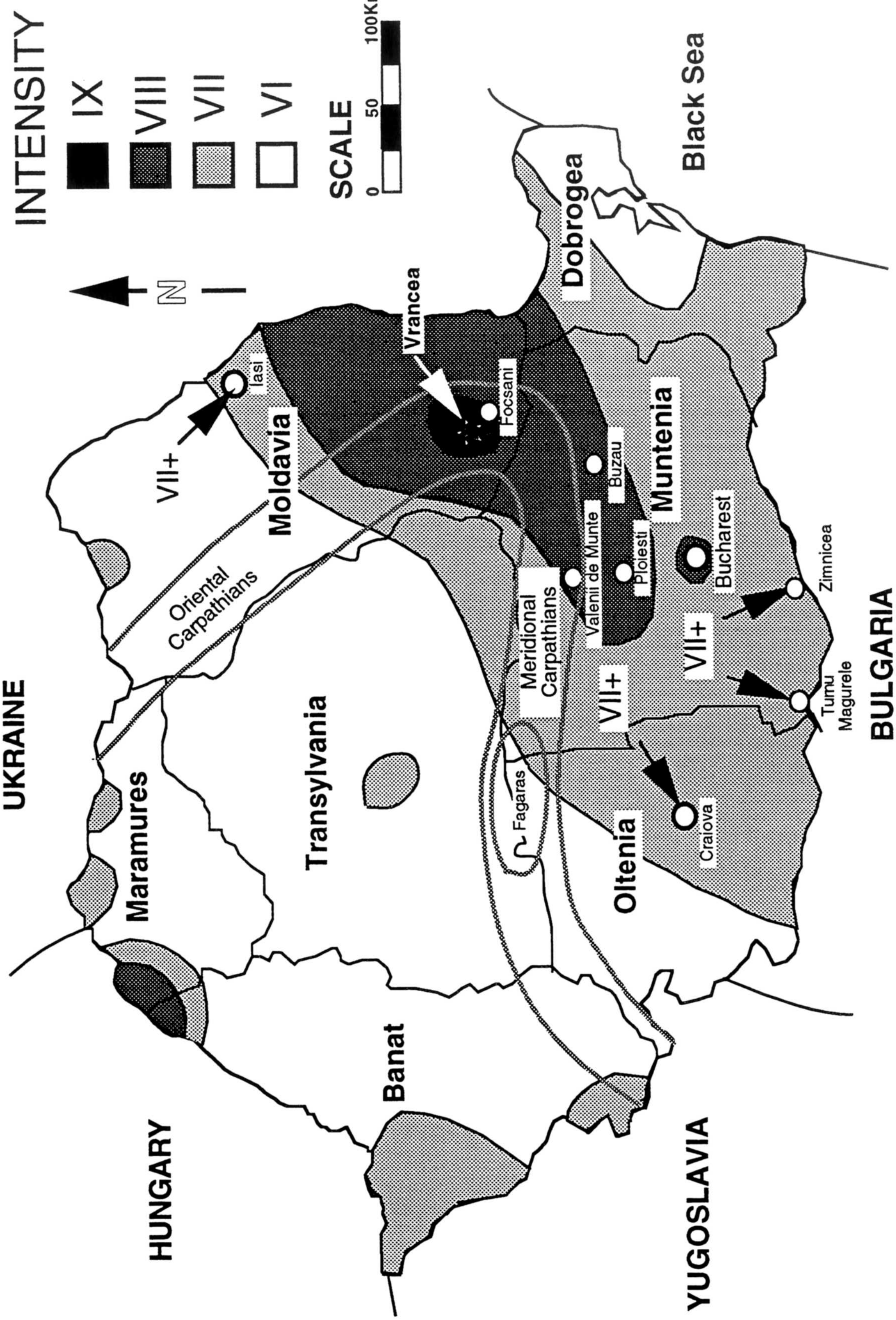

Figure 1: Map of Romania , showing the epicentral location of the 1990 earthquake (main shock) as well as the towns where damage surveys were carried out by EEFIT.

In Table 2 the earthquakes have been divided in five magnitude classes and average annual probability of occurrence and return periods (in years) were calculated using the seismicity data for each magnitude class (by simply dividing the number of events in each class with the number of years). The distribution of focal depths is also shown. Nevertheless this annual probability of occurrence is not stable throughout the 90 year period, especially for the events of M=6-6.4 and 6.5-6.9 because most of the former ones occurred in the first half of the century, while most of the latter ones occurred in the second half of the century. In order to illustrate this temporal distribution, the seismicity of consecutive 25 year periods was also studied (67 periods). The results are shown in the second part of Table 2. The maximum, average, standard deviation and minimum number of events for each magnitude class, that occurred in any of these 25 year periods is shown. Notice the significant difference between maximum and minimum number of events for moderate magnitudes. Finally using the average number of events the mean annual probability of occurrence and return periods where recalculated for each magnitude class. The most significant difference between the two methods is for the events of magnitude 6.5 to 6.9.

Table 2: Seismicity in Vrancea during the 1900-1990 period

Magnitude	5.0 - 5.4	5.5 - 5.9	6.0 - 6.4	6.5 - 6.9	7.0 - 7.4
No. of events	53 (1900-81)	24 (1900-81)	6 (1900-90)	5 (1900-90)	2 (1900-90)
Mean Annual Probability of Occurrence	0.646	0.293	0.066	0.055	0.022
ReturnPeriod (years)	1.72	3.42	15.17	18.20	45.50
Mean Depth (km)	124	125	89	107	121
Stand.Dev. (km)	37	27	17	31	12
25 year period					
Maximum	30	9	4	4	1
Average	18.54	7.21	1.70	0.64	0.60
Standard Dev.	7.72	1.21	1.41	1.50	1.02
Minimum	11	5	0	0	0
Mean Annual Probability of Occurrence	0.742	0.288	0.068	0.026	0.024
Mean Return Period (years)	1.35	3.47	14.69	38.94	41.88

Furthermore the Gutenberg - Richter cumulative frequency law , for M>4.9, was calculated using the data now available for the whole 1900-1990 period. The result is shown in Figure 2. Using this relationship the estimated mean annual probability of occurrence for each of the magnitude classes studied in Table 2 is as follows: 0.597; 0.234; 0.092; 0.037 and 0.024. We thus notice that this equation is underestimating the probability of occurrence of events with M<6, and overestimates that of events of magnitude 6.0 to 6.4. Nevertheless for the most important events of M>6.4, the estimation is not significantly different from that of the previous two methods.

In order to further elucidate the seismicity patterns of Vrancea region, it is of course important to look at the seismicity during the pre-1900 period. As aforementioned earthquake catalogues of Romania with data starting as early as 1091 are available (Balan et. al., 1982). Maximum observed seismic intensity (I_0) is the most reliable parameter for this period. The events that are known to have caused $I_0 \geq VII$ in the last 900 years are all listed in Appendix A. As we see there have been 20 events with I_0=VII, 10 with I_0=VII+, 14 with I_0=VIII, 4 with I_0=VIII+ and 3 with I_0= IX (the symbols of VII+, VIII+ are equivalent to intensity degree between the two intensities, in other words 7.5 and 8.5). This data gives a return period of about 43 years for $I_0 \geq VIII$, and 113 years for $I_0 \geq VIII+$. In Appendix A the magnitudes that are in brackets are estimated macroseismically (M_I), using a formula that converts the earthquake magnitude from the maximum observed intensity (Balan et. al., 1982). In the case of the intermediate depth earthquakes in Vrancea this is:

$$M_I = 0.56\ I_0 + 2.18$$

Focal depth is unknown for all the pre-1900 events and "i" stands for intermediate focal depth (in the case of Vrancea zone 75-175 km). All other magnitudes except otherwise stated are surface wave magnitudes (M_S).

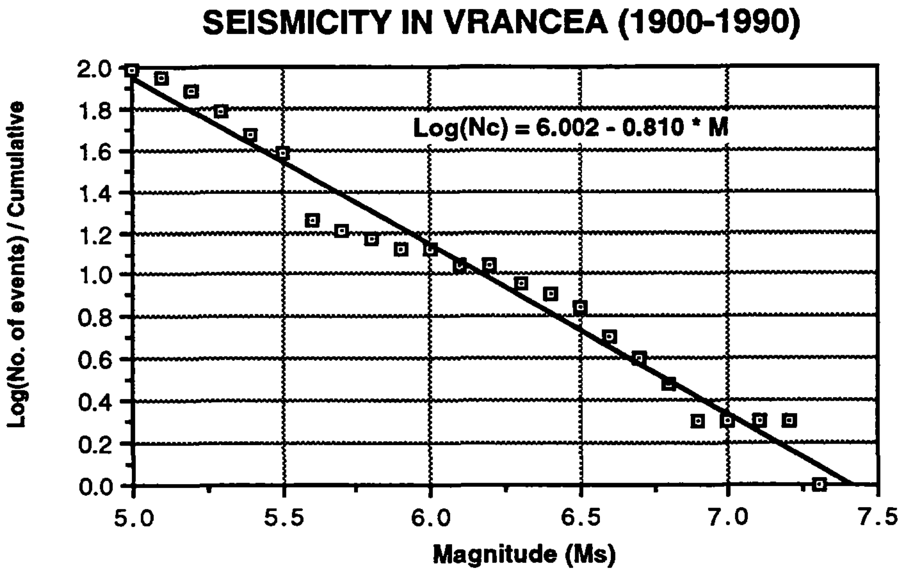

Figure 2: Gutenberg-Richter cumulative frequency law for Vrancea (1900-1990)

However this catalogue is complete only for the post-1471 period for $I_0 \geq$VIII, and only for the post-1800 period for $I_0 \geq$VII. A study of the intensity recurrence for consecutive 24 year periods (e.g. 1471-1494 ; 1472-1495 and so on) was therefore done for all the intensities that exceed VII, for the respective periods of catalogue completeness (498 periods for $I_0 \geq$VIII and 168 periods for VII$\leq I_0$ <VIII) in order to estimate the seismic hazard more accurately. The results are summarised in the lower part of Table 3. Maximum, minimum and average number of events in any 24 year period are given. Using the average number of events, the mean annual probability of occurrence and mean reccurrence period were also calculated. We thus see that the mean reccurrence period for $I_0 \geq$VIII is actually 34.2 years and for $I_0 \geq$VIII+ is only 82.8 years. The difference in the estimation of hazard between the two methods is quite small but not insignificant (upper and lower part of Table 3).

Table 3: Seismic Intensities in Vrancea

Intensity	$I_0 \geq$ VII	$I_0 \geq$ VII+	$I_0 \geq$ VIII	$I_0 \geq$ VIII+	$I_0 \geq$ IX
No. of events	19(1799-1990)	9(1799-1990)	16(1471-1990)	6(1471-1990)	3(1471-1990)
Mean Ann. Prob. of Occurrence	0.100	0.047	0.031	0.012	0.006
Return Per. (years)	10.00	21.11	32.44	86.50	173.00
24 Year Per. Maximum Average Stand. Dev. Minimum	 5 2.143 1.342 0	 3 0.994 0.703 0	 3 0.702 0.769 0	 1 0.290 0.454 0	 1 0.145 0.352 0
Mean Ann. Prob. of Occurrence	0.089	0.041	0.029	0.012	0.006
Mean Return Per. (years)	11.20	24.14	34.18	82.83	165.67

Another important feature of the seismicity in Vrancea is the fact that there are distinguishable periods of higher seismic activity during which earthquakes causing $I_0 \geq VII+$ always occurred in every century since 1091. There are 5 such periods in the years 1-8; 16-30; 37-46; 68-71; 77-96 of every century. The events that occurred in each of these periods are summarised in Table 4. This of course does *not mean that in future all the strong earthquakes in Vrancea will occur exactly in the same periods.* Furthermore it must be repeated that the catalogue is believed to be complete only since 1471 for I≥VIII and only since 1800 for I<VIII.

Table 4: Periods of increased seismicity in every century for the period 1471-1990

Intensity	Period 1-8	Period16-30	Period37-46	Period68-71	Period77-96
I_0=VII+	3 (1606;1701; 1908)	2 (1620;1829)	2 (1637;1945)	1 (1868)	2 (1986;1990)
I_0=VIII	2 (1604;1605)	1 (1327)	3 (1446; 1545;1838)	3 (1170;1471; 1569)	5 (1196;1679; 1681;1790;1977)
I_0=VIII+	–	2 (1230;1620)	1 (1738)	–	1 (1590)
I_0=IX	1 (1802)	1 (1516)	1 (1940)	–	–

The *Poisson probability* distribution P (κ ; λ) gives the probability of occurrence of κ events (κ=1 to n) if the mean expected number of events within a certain time period is λ, and is equal to:

$$P(\kappa;\lambda) = \sum_{\kappa=1}^{n} \frac{e^{-\lambda}\lambda^{\kappa}}{\kappa!}$$

where:

λ = the average number of events to occur within an examined time period (e.g. 50 years), in other words the fraction between the time period and the mean recurrence time and

κ = number of events likely to occur during the examined period (in the case of $I_0 \geq$ VIII it ranges from 1 to 6 events for a 50 year period, see Table 3).

This formula can be used in relation to the results of Table 3, in order to predict the occurrence of events causing intensity greater than VII. For the most important large magnitude earthquakes (M≥7) that usually cause maximum intensity of VIII or larger (depending on their focal depth) we see from Table 3 that the mean return period is 34.2 years, with a standard deviation of 31.2 years. Using this we can find that λ = 1.463 for a 50 year period. Thus we obtain a 76.8% probability of occurrence of one up to six such events within any 50 year period. The last such event occurred 13 years ago (in1977) which means that the probability for the 1990-2027 period is higher than 80%. If the occurrence pattern mentioned in Table 4 is to be repeated, the **2016 - 2030** period seems to be the most likely.

2.2 Seismicity in other parts of Romania

In terms of eathquake risk the second most important zone of seismicity in Romania is the **Fagaras zone** (2.9% of the energy release), that is in the centre of the Meridional Carpathians. The name of the zone is taken from that of the high mountains that cover this part of the country (most of the highest peaks in the Carpathians are located there). Seismicity is occurring in a relatively small area of about 2000 km^2 that is mountainous. The difference between this zone and that of Vrancea is that the earthquakes are of shallow focus. The maximum expected magnitude in the zone is defined macroseismically as 6.5 (I_0=VIII) using the formula (Balan et. al., 1982):

$$M_l = 0.66\, I_0 + 1.23.$$

The largest earthquakes in this zone have occurred in April and May 1571 (two events of I_0=VIII; M=6.5; H<60 km), December 1746 (I_0=VIII; M=6.5; H<60 km), February 1832 (I_0=VII; M=5.9; H<60 km) and January 1916 (I_0=VIII; M=6.5; H<60 km). This seems to suggest a recurrence period of about 170 years for the larger events, with the last one 74 years ago. An earthquake of M=5.2 occurred in April 1969 with maximum intensity VI and focal depth of 10 kilometres. There are no other earthquakes from this zone mentioned in the latest Romanian catalogues. The Fagaras zone fortunately is one of the zones with the lowest population density in Romania and no fatalities have been reported due to the aforementioned earthquakes. Nevertheless the seismic risk of the zone should not be underestimated because shallow earthquakes of M=6-6.5 occurring near to towns have a considerable potential of destruction. The town of Brasov (popul. 300,000) is situated only 50 kilometres from this earthquake source, while the town of Pitesti (popul. 150,000) is at a distance of 65 kilometres. Two smaller towns (Fagaras and Cimpulung) are located at even smaller distances from this zone. A lot of important industrial facilities are also located in the area near those four towns.

The other three zones of seismic activity in Romania are of less importance from the point of view of damage potential since no earthquake larger than 5.5 has ever been reported (all three zones together contributed 0.1% to the total seismic energy release in the period 1091-1979). Seismicity in **Transylvania** is quite low with maximum expected magnitude of the order of 5.5 (defined macroseismically). Eartqhuakes of M=4.5-5.5 occur in various parts of this large geographical zone especially in the counties of Sibiu, Satu Mare and Maramures. Eight events of considerable intensity ($VI<I_0<VIII$) have been reported in the 1523-1990 period. It is noteworthy that all of these events occurred before 1900, and six happened during the 19^{th} century. The zone of **Banat** is situated in the SW of the country around the town of Timisoara. The maximum magnitude in the zone is 5 and most of the events are shallow. There have been 7 earthquakes of considerable intensity reported in the last 200 years with the latest one in May 1959 (I_0=VII+; M=4.6; H=5km). Finally the zone of **Dobrogea** is situated along the Black Sea coast where an earthquake of magnitude 5.2 (I_0=VI+; M=5.2; H=11km) has occurred in November 1981. No other earthquake is known to have occurred in this zone. The formula used to estimate the magnitude of the events in these three zones is (Balan et. al., 1982):

$$M_I = 0.60\, I_0 + 0.52.$$

2.3 Strong motion records in Romania

After the 1977 earthquake a decision was taken to expand the strong motion observation network of Romania, that until then comprised only a few stations. As a result during the May 1990 earthquakes at least 29 instruments were triggered in various towns especially in the East and South of the Carpathians and in addition 9 instruments recorded the motion in various locations of the capital Bucharest. About half of these 38 stations were also triggered during the aftershock of 31^{st} May, mostly in the eastern part of the country, registering smaller peak accelerations (except in Focsani). In Figure 3, all the stations that were known to have recorded the two events, are shown, along with the value of peak ground acceleration (PGA) during the main shock and the aftershock. Unfortunately details about the 9 Bucharest records were not yet available.

The first comments are that 5 stations recorded PGA larger than 20%g (maximum in Cimpina 26%g), and as is shown on the map they are spread in a wide area. A further 6 stations recorded acceleration of 17%g, 4 stations recorded 14%g, and 7 stations between 10 and 12%g. The rest of 7 stations recorded smaller values of 4 to 9%g. *It must be mentioned here that many of these instruments were* located in the basements of multi-storey RC buildings. In Bucharest a variety of PGA values between 7 and 14%g were reported during the main shock and insignificant (2 to 4%g) during the aftershock.

During the August 1986 earthquake, which has had its magnitude recently upgraded to $M_s = 6.8$ (m_b = 6.5 to 6.6), 9 records were obtained at exactly the same locations of Bucharest as in May 1990. As additional comparison the maximum acceleration recorded in 1986 was 28%g in Focsani. The PGA's ranged between 6 and 16%g (in the NS component) and between 4 and 11%g (in the EW component) with the predominant periods ranging between 0.7 and 1.1 seconds. Thus the 1990 peak values are largely similar to those recorded in 1986. Furthermore one record was obtained in INCERC at the basement of a single storey RC building in 1977, that had PGA of 21.5%g (NS component) and 16.5% (EW component). The predominant periods of this record were in the range of 1.4 - 1.6 s (NS component) and 0.8 - 1.0 s (EW component). The 1986 INCERC record at the same location had PGA of 10%g (EW component) and 9%g (NS component) with predominant period of 1.1 second. This supports the suggestion that intermediate depth earthquakes tend to produce longer predominant periods when their magnitude is increasing. Also it must be remembered that the 1986 event had a larger focal depth of 133 km, against 93 km of the 1977 earthquake.

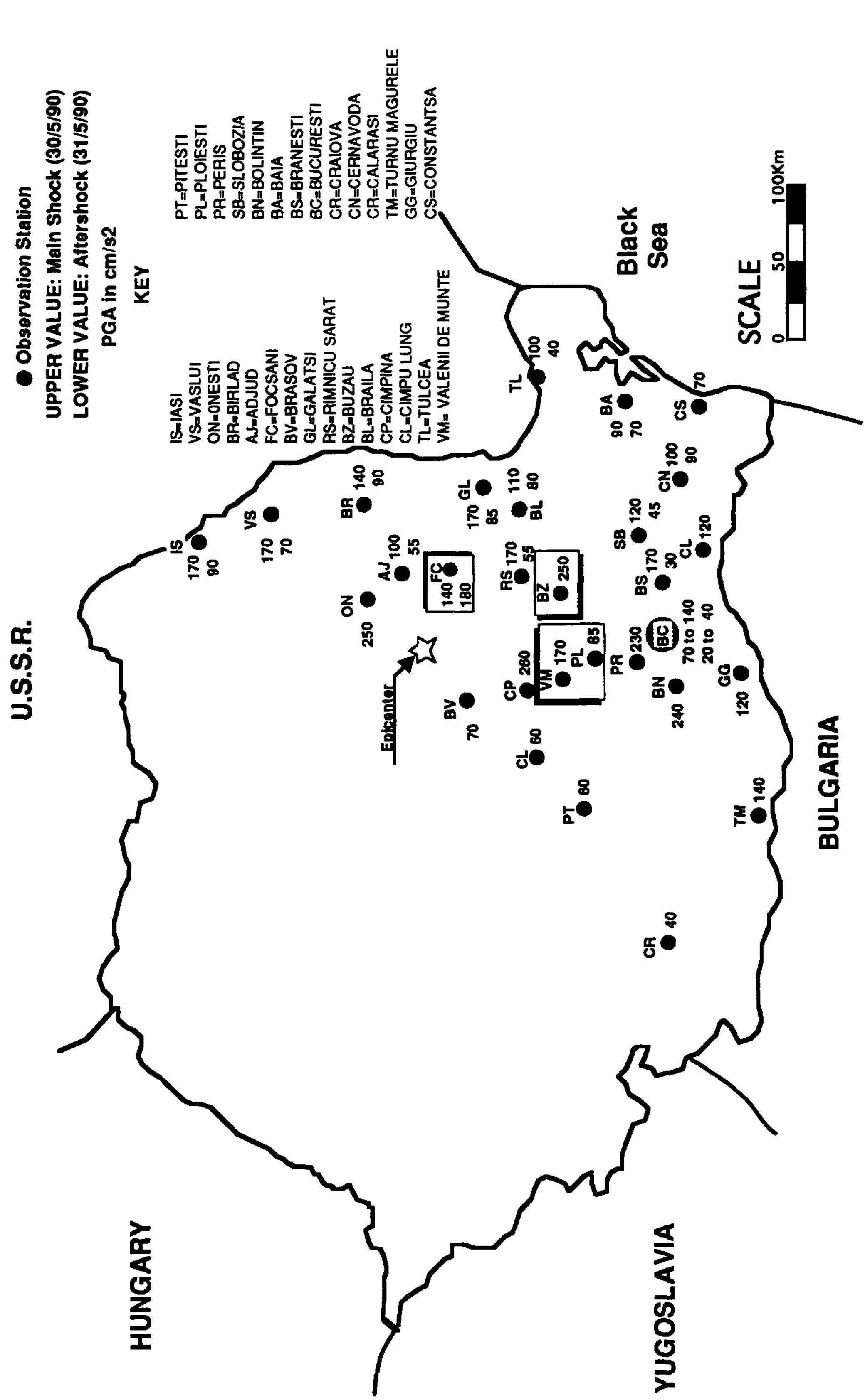

Figure 3: Peak Horizontal Accelerations that were recorded during the main shock and aftershock of May 1990. The squares outline the locations were field surveys around the station were carried out.

Several new lessons seem to emerge with the first information obtained from the 1990 records in Bucharest. First of all it seems that during the main shock in many records the EW component was stronger than the NS component (the opposite to the previous 2 events). Furthermore it seems that the predominant periods were this time much shorter. Thus in Carlton building (Central Bucharest; PGA = 11%g and PGV = 19.2 cm/s) most of the spectral acceleration peaks occurred in a period range of 0.15 - 0.65 seconds, with a 40%g peak (5% damping) at **0.21 s**. In 1986 the predominant period was 1.1 s. The same predominant periods were observed in Panduri Boulevard record (East of city centre; PGA = 14%g; PGV = 7.9 cm/s), with a 50%g peak at **0.23 s**. In 1986 the predominant period was 0.8 s. The same happened in the small town of Peris (24 km North of Bucharest; PGA = 22.5%g and PGV = 14.3 cm/s) where the peak spectral acceleration reached 100%g at **0.28 s**. All these are summarised in Table 5.

Table 5: The characteristics of the strong motion recorded in Bucharest during the 3 large Vrancea earthquakes in the 1977 - 1990 period

Earthq.	m_b	Epic. Dist. (km)	Depth (km)	PGA (cm/s2)	Predom. Period (sec)	Predom. Compon.	Max. MSK Intens.
1977	6.8	98	93	215	0.9 & 1.5	NS	IX
1986	6.6	121	133	60 to 160	0.7 to 1.1	NS	VII+
1990	6.5	164	89	70 to 140	0.25	EW	VII

3. SEISMIC ZONATION AND ASEISMIC CODES

3.1 The distribution of Romania's urban population in seismic zones

Romania has been divided into seismic zones based on the maximum observed or expected intensity (the M.S.K. seismic intensity scale is used in Romania). The seismic zoning map currently in practice is seen in Figure 1 (in this map the seven geographical zones of Romania can also be seen). This map is largely the result of macroseismic investigations after the 1977 earthquake, but it also includes several parts in the North and West of the country where some seismicity, as discussed in the previous chapter, is taking place. It must also be noticed that there are 4 towns in zone VII where actually the design intensity is VII+, these are Iasi (population in 1977 of 265,000), Craiova (221,000), Turnu Magurele (32,000) and Zimnicea (14,000). Furthermore Bucharest is now at intensity zone VIII (instead of VII in the 1963 code). Table 5 shows the surface area of each of these zones and the proportion of the country's area in each seismic zone.

Furthermore for a better understanding of the distribution of Romania's population within each of these zones and the seismic risk involved, the map shown in Figure 4 has been prepared. In this map the urban nucleii of Romania (with population larger than 30,000 people) are located with circles having radii in proportion to their population (in some cases smaller towns have been added to the population of their neighboring urban areas thus considered as a single urban agglomeration). As a first comment on this map we can say that the total population of these urban areas is 7.534 million people which is equivalent to 34.9% of Romania's population in 1977. The official urban population of Romania for 1980 is 49%, mainly because towns of more than 10,000 are considered urban zones. This distinction between urban areas with more than 30,000 population, and the rest of the country, is done here because the distribution of building types in the smaller towns and numerous rural areas of the country is entirely different from that in larger towns (both in terms of the most common structural systems as well as in number of storeys). The seismic zoning has an effect on the design loads for every large residential building constructed in Romania by the state, but smaller mostly privately owned buildings especially in semi-urban and rural areas are not always built according to the code requirements.

Table 6: Surface Area and Urban Population in the seismic zones of Romania

Int. Zone	VI	VII	VII+	VIII	IX
Surface (km^2)	130635	76345	385	29455	680
Romania's surface (%)	55	32.1	0.2	12.4	0.3
Urb.Popul. ($*10^3$)	2068	2124	518	2759	65
% Tot.Urb. Popul.	27.4	28.2	6.9	36.6	0.9

The only large town located in zone IX is Focsani (65,000 people in 1977). The proportion out of the total urban population living in each of the zones is also shown in Table 6. The largest proportion of today's urban population lives in intensity zone VIII, but we note that 55.6% of the country's urban population lives in zones of intensity VI and VII. Nevertheless by overlaying the two maps it may be seen that most of Romania's large towns (80,000 and above) are within the zone that is regularly affected during the occurrence of Vrancea earthquakes. There are only 15 such towns in the west and northwest of the country that are not affected during the occurrence of earthquakes with magnitude larger than 7 in Vrancea.

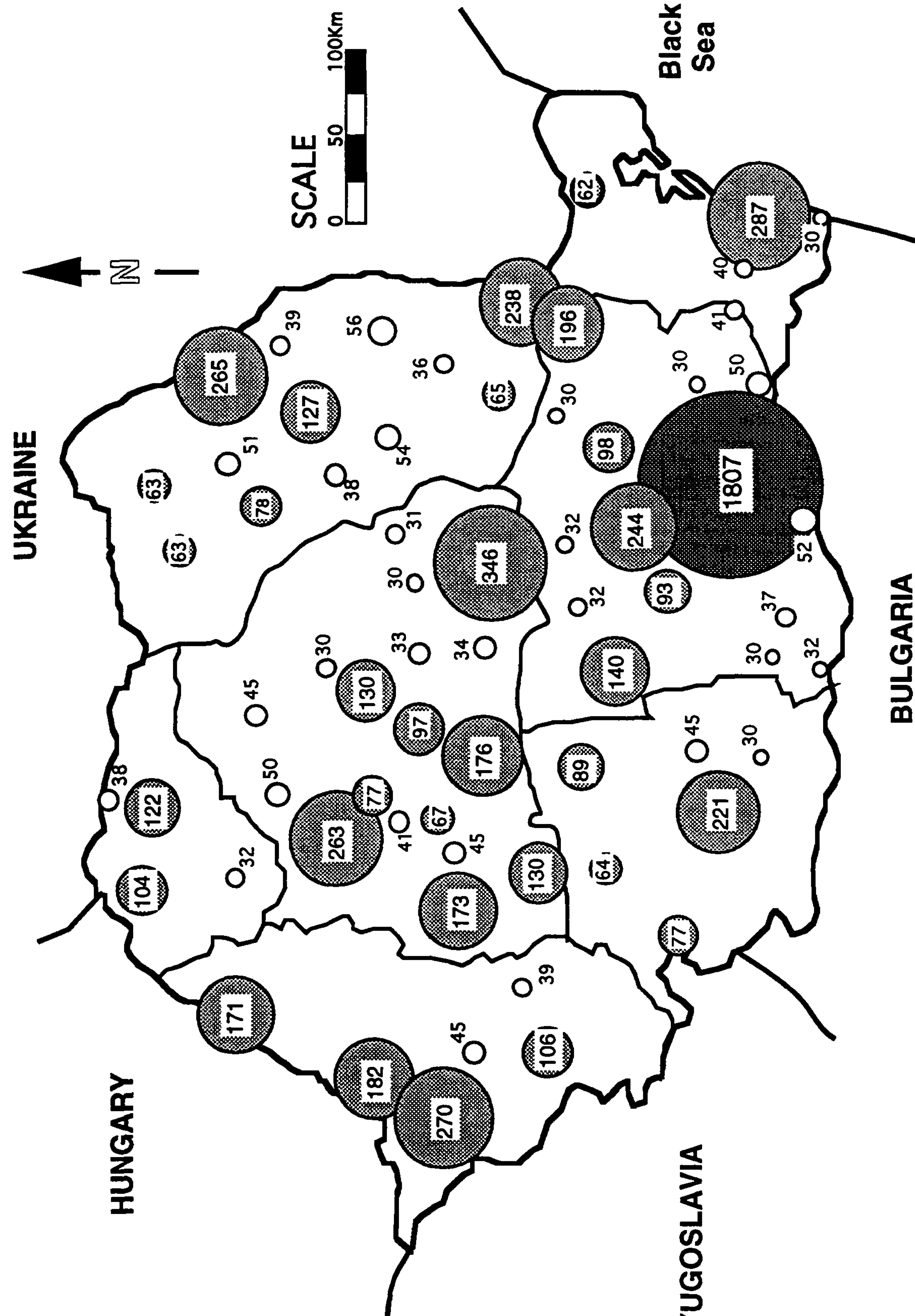

Figure 4: The distribution of urban agglomerations in Romania (population larger than 30,000). Numbers indicate the 1977 population (in thousands).

3.2 The development of seismic construction codes in Romania

The first seismic code of Romania was introduced in 1963. It was based on the USSR code and made seismic design considerations compulsory for state buildings in zones of intensity VII and larger. No base shear force was applied for the design of buildings in zone VI. In 1970 the base shear force was increased by 20% and this remained unchanged until the occurrence of the 1977 earthquake. Soon after this event followed the introduction of a new seismic construction code in 1978 which made compulsory the use of seismic design even in zone VI (practically all over the country). It also significantly increased the seismic coefficient in all zones (see Figure 5). The 1978 code is used at present and requires a seismic base shear coefficient that is equal to:

$$S = \kappa_S \, \beta \, \psi \, \varepsilon \, G$$

where:

κ_S = seismic zone factor (for apartment buildings 0.07 in zone VI; 0.12 in zone VII; 0.16 in zone VII+; 0.20 in zone VIII and 0.32 in zone IX).

β = amplification factor depending on the natural period of the building (Figure 5) and the soil type. It is equal to $3 / T_n$ and ranges between 0.75 and 2 (for normal soils).

ψ = reduction factor accounting for the capability of structure to deform inelastically. It is equal to 0.30 for RC shear wall buildings of less than 5 storeys and 0.25 for more than 5 storeys. ψ equals 0.20 for multispan and multistorey RC framed structures.

ε = coefficient of equivalence between the modal shape of the structure and that of SDOF system (the fundamental mode is considered for buildings less than 40 metres high)

$$\varepsilon \cong \frac{\left(\sum_{\kappa=1}^{n} G_\kappa H_\kappa\right)^2}{G\left(\sum_{\kappa=1}^{n} G_\kappa H_\kappa^2\right)}$$

where:

κ = the no. of storeys; H_κ = height of storey κ;

G_κ = weight of storey κ and G = total gravitational weight of the structure.

This base shear force (S) is then vertically distributed to each floor by the use of the following equation:

$$S_\kappa = S \frac{G_\kappa H_\kappa}{\sum_{\kappa=1}^{n} G_\kappa H_\kappa}$$

In the next chapter the different residential building types that exist in Romania, are discussed. Following the 1986 and 1990 earthquakes a revision of the 1978 seismic code is due to be carried out and is expected to be introduced in 1992. This latest revision is also expected to change the seismic zoning (Figure 1) of the country.

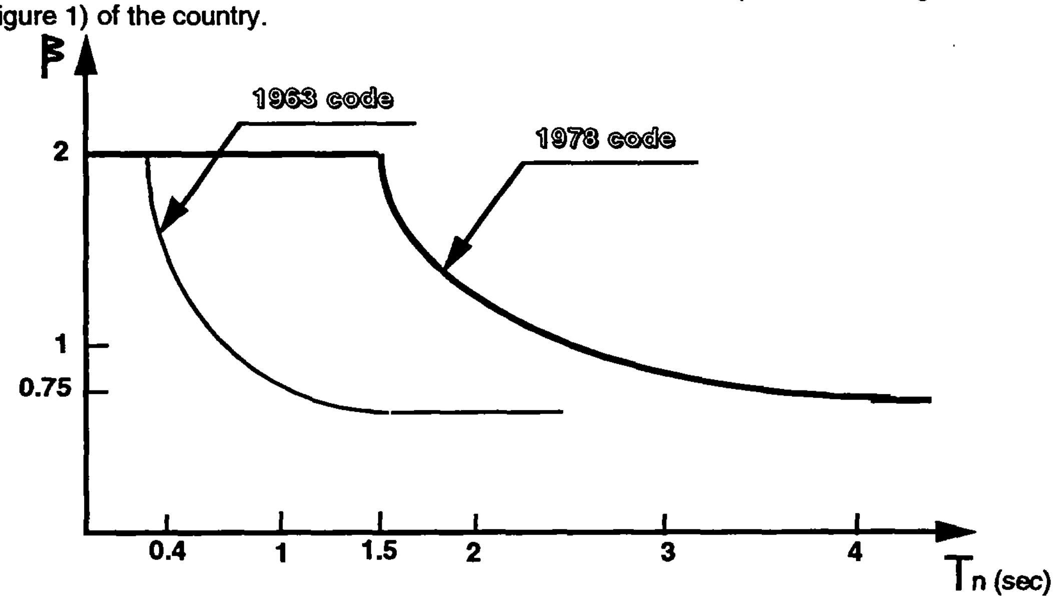

Figure 5: The effect of changes with the introduction of the 1978 code, upon the amplification factor (β).

4. RESIDENTIAL CONSTRUCTION TYPES OF ROMANIA

4.1 Recent tendencies in Romanian residential construction practice

In Romania there is a very large variety of residential building types, depending mainly on the zone and period of construction. The major factors affecting construction practices, during this century, are the introduction of a centralised system of economy in 1948 and the occurrence of two destructive earthquakes in 1940 and 1977. Buildings constructed prior to 1940 did not have any seismic design consideration, while the first seismic code was introduced in 1963 (based on the USSR codes) and was upgraded in 1978. Load-bearing masonry structures are still the main structural type in the rural areas of Romania, where about 45% of the present-day population lives. They are also common in urban areas, but their percentage is rapidly decreasing because of the construction of larger apartment buildings. It is estimated that today only about a quarter of the urban population is living in low-rise masonry buildings.

During the last 30 years a very large number of apartment buildings mainly from reinforced concrete (RC) were built mostly in the urban areas. Figure 6 illustrates this construction boom, as well as the changes in structural systems used (the numbers for the last 5-year period are the planned number of apartments to be constructed). Noteworthy is the tendency to construct more and more RC frames with shear walls or large panel RC buildings (especially after the 1977 earthquake). Thus at present 37% of apartment buildings built since 1956, are of the large RC panel system, with 35% of RC frames with shear walls and the remaining 28% being RC frames without shear walls and reinforced (confined) load-bearing masonry apartment buildings (estimated:13% RC and 15% masonry, unfortunately exact data on construction of load-bearing masonry buildings were not available). Nevertheless in the 1981-1989 period with most of the apartment building construction gradually expanding from the large urban centres to smaller towns, the proportion of masonry has increased. For the 1981-85 period a total of 140,000 apartments in load-bearing masonry system were planned for construction in smaller towns. A further comment on Figure 6 is that the apartment buildings constucted in the post-1965 period are all designed according to the seismic codes applied at the time, while those before, are mostly without seismic design. With an estimated 3.3 people for each apartment it can be said that today about 50% of Romania's polulation lives in mid-rise (3-6 storeys) or multistorey (7-12 storeys) apartment buildings, of which nearly 90% are designed for seismic loads. Finally it is estimated that the proportion of people in urban areas living in buildings constructed after 1955 is 75%, while the proportion of people in rural areas living in houses constructed after 1950 is at present about 50%. In larger cities, there is a significant number of multistorey apartment buildings constructed before 1955, many of which behaved poorly during the 1977 earthquake.

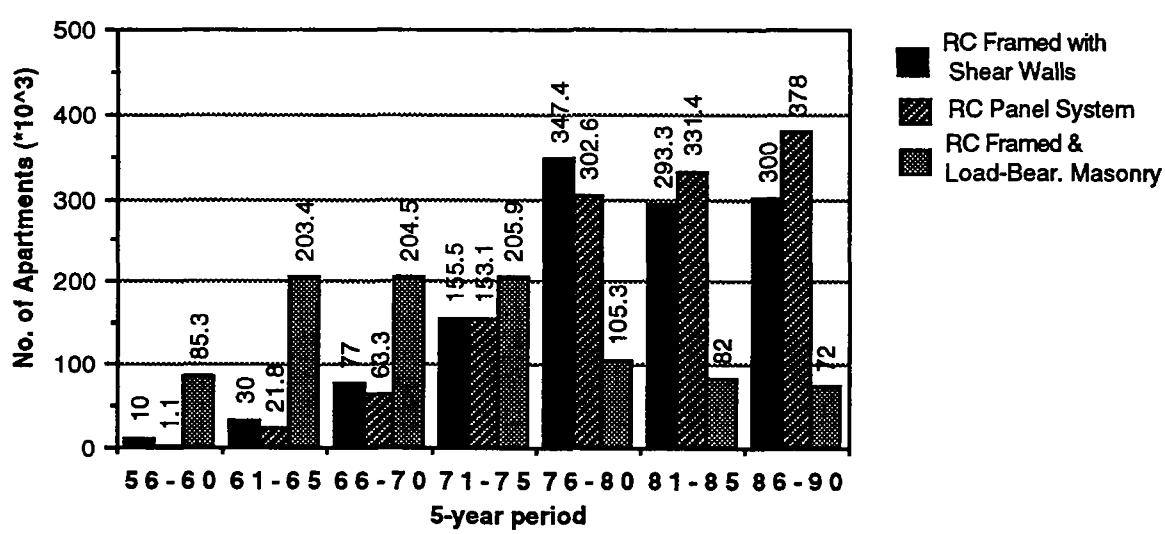

Figure 6: The construction of apartments by structural system during the 1956-90 period in urban areas of Romania.

Both RC framed structures and RC large panel structures now use a high degree of prefabrication. The proportion of prefabrication has been increasing constantly in order to achieve the construction of such a large number of apartment units per year (especially after 1975). Thus the decision to gradually move from the cast-in-situ RC framed structures (without shear walls) to the RC large panel constructions, is not only due to seismic safety, but also due to socio-economic considerations and government policies. Prefabrication of large residential buildings is a tempting idea in regions with serious housing demands, but in the case of earthquake zones, the quality of workmanship and site supervision is of paramount importance. Section 4.3 and chapter 5 further discuss the vulnerability of different building types and corroborate this important point. In Figure 6 it may be seen that in the last 5-year plan the proportion of large panel buildings that have a totally prefabricated structure, for the first time exceeded 50% on the total. At present there are 50 casting factories all over the country producing RC members such as slabs, beams, columns, panels or even complete box units. The construction details have been standardised and an ever increasing number of identical multistorey buildings was constructed until the 1989 revolution. According to data published in 1982, the amount of prefabrication in RC residential buildings was as follows:

– floor members	70%
– shear walls	50%
– beams in multistorey frames	20%
– columns in multistorey frames	5%

There is a significant difference in the prefabrication of framed structures as opposed to the total prefabrication of large panel or box unit buildings. The construction of entirely prefabricated framed structures has been wisely avoided, as opposed to USSR, where more than half of the 25,000 persons killed during the 1988 Armenia earthquake, were occupants of entirely precast RC frame apartment buildings. It must also be mentioned that the proportion of prefabrication in industrial and agro-industrial buildings is much higher, sometimes reaching 90% of the structural system.

In the 1985-1989 period a "**systematisation program**" was adopted, that had as goal the demolition of most of the old load-bearing masonry houses in suburban and semi-urban areas and their replacement with new mid-rise apartment buildings (mostly 3 to 5 storey). This controversial program was implemented in only a limited number of sites and has been abolished by the new government. The changes that occurred recently in the political system of Romania will almost certainly have a significant effect upon the construction sector, with a large part of it being decentralised or even privatised.

4.2 Residential building typology

Figure 7 is a summary chart of the building types that exist in Romania. The building types are divided in three main categories according to their structural system, namely load-bearing masonry systems; framed structures; and panel systems with a view of their seismic vulnerability. These three main structural systems are divided in two different categories according to the consideration or not, of seismic loading in their design. The type of vertical and horizontal structure is also shown. The seismic vulnerability of each structural system is decreasing from top to bottom in the figure. Frame and panel buildings are constructed by state enterprises while the load-bearing masonry buildings and some low-rise RC framed buildings are constructed privately . There is also a significant number of load-bearing masonry apartment buildings (of 3 to 5 storeys) that are constructed by the state especially in the zones of intensity VI and VII.

In order to illustrate the earthquake performance of each of the building types the results of the INCERC1977 post-earthquake damage survey in Bucharest will be presented. These were at first published in the UNESCO, 1982 report on Vulnerability and Seismic Hazard. The book edited by Balan (1982, in Romanian) gives further details of this survey and is one of the best carried out in Europe for the purposes of vulnerability and risk assessment. The results have been converted to a cumulative form for the purposes of this report. In the following sections figures showing the extent of damage sustained by different building types will be presented.

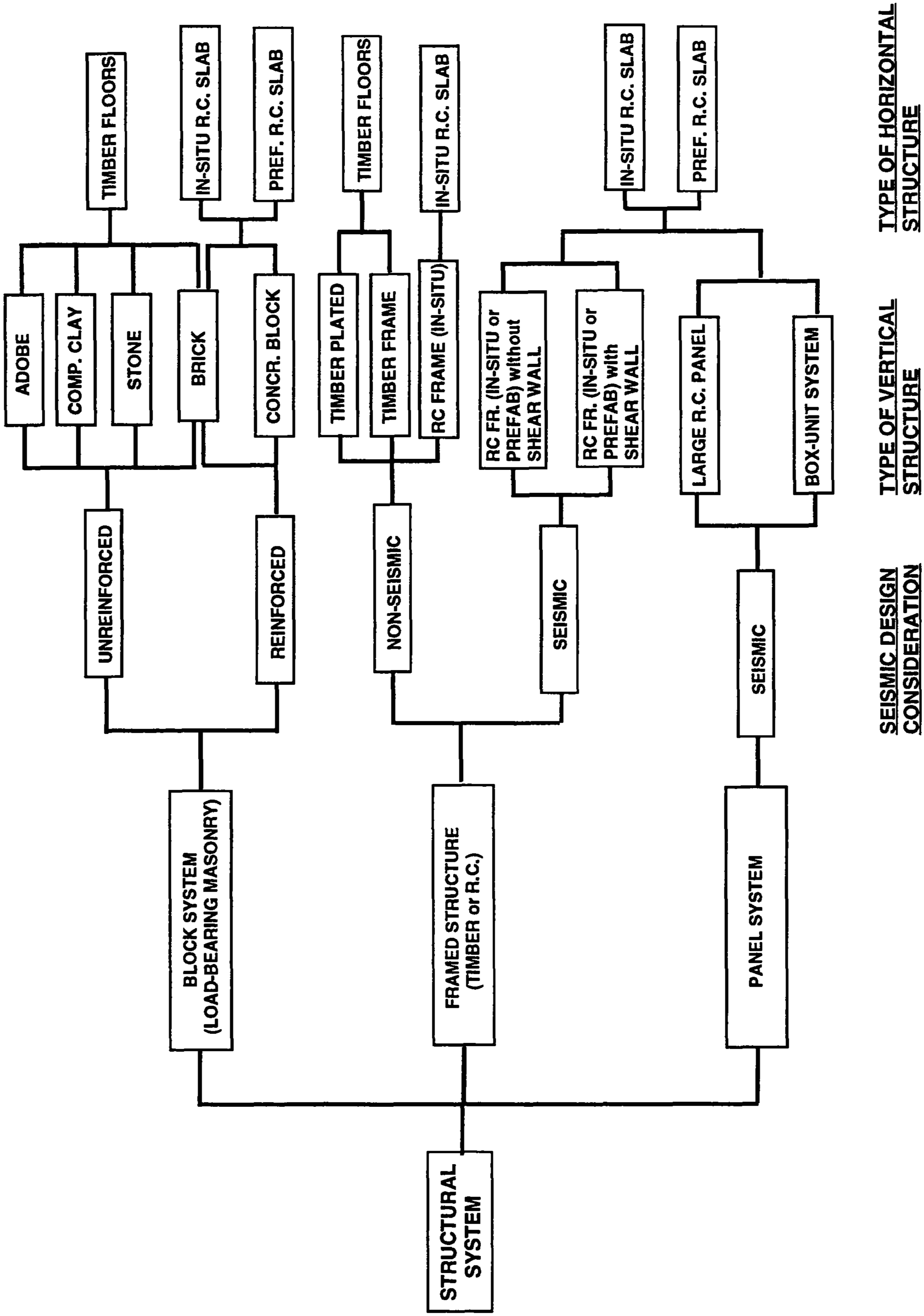

Figure 7: Residential building type distribution in Romania

4.2.1 Load-bearing masonry buildings

As shown in Figure 7, there is a wide variety of masonry buildings. There is a significant difference between such buildings in urban and rural areas. There is also a significant distinction between rural buildings in the large fertile plains and the numerous mountainous and valley zones of the country. Compacted clay and adobe houses are common in the large plain zones of Muntenia, Dobrogea, Moldavia, Oltenia and Banat. Stone masonry houses, although not so common in Romania (unlike Italy, Yugoslavia and Greece) are more numerous in the high hills and mountainous regions of the Carpathians and Transylvania. The proportion of these three building types in relation to the total building stock is decreasing rapidly, because after 1960 most rural houses are built with burned bricks. There are also significant differences in the construction details between new and old buildings, with RC reinforcing elements (ring beam, lintel, slab) becoming increasingly common. The seismic resistance of these buildings is low and due to their relative rigidity they tend to be affected by strong motions with a high frequency content. The Vrancea earthquakes, having intermediate depth, produce seismic motions at the surface with most of their energy concentrated in the lower frequencies. As a result in many cases these buildings survived the 1940 and 1977 earthquakes.

<u>Compacted clay</u>
The walls are 40-60 cm thick made from clay that is reinforced with horizontal tree branches. The walls are plastered with lime-sand mixture. They usually have shallow foundation made from river cobbles. Their height is mostly single storey and the horizontal structure is mostly from timber (Plate 7). In more recent buildings, concrete foundations are also used. Horizontal reinforcing elements in the form of RC ring beams and lintels are also more common in new houses. Their architectural form tends to be quite simple and regular, with reduced storey height (up to 3.5 metres).

<u>Adobe</u>
These are among the most common low-rise old houses in Romania. The walls are thick, made from raw adobe blocks (mixed with straws), and laid with low strength mortar (clay or mud mixed with sand). Foundation and floor elements are similar to those used in compacted clay buildings. In newer buildings of this type, the plaster is reinforced with a mesh that is nailed onto the walls in order to provide a better shielding effect (Georgescu, 1986). RC ring beams and lintels are also more common in recently built houses. Storey height and layout are similar to compacted clay buildings. RC slabs were also introduced (either in-situ or precast hollow planks) in the last twenty years.

Figure 8 shows the results of an extensive damage survey in Bucharest after the 1977 earthquake. The degrees of damage were assigned according to the MSK scale specifications but also a degree of engineering judgement was involved. Appendix B, summarises the definitions for damage degree assignment for masonry and RC buildings. The representation is in cumulative form (left hand axis). ADI is the *average damage index* (degree) at each intensity (right hand axis). The sample comprises 5984 buildings of adobe, compacted clay and timber frame infilled with clay or mud, mostly built before 1900. In the 1990 earthquakes the damage to this type of buildings in Bucharest was much smaller.

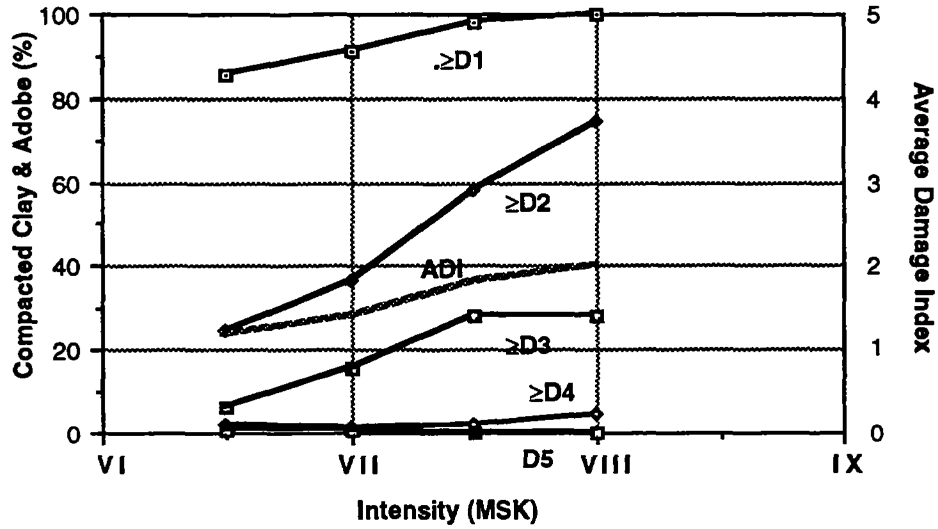

Figure 8: The vulnerability of low-rise compacted clay and adobe buildings (Bucharest, 1977 earthq.).

Stone masonry
These are more common in the mountain zones of the country (Carpathians; Transylvanian Alps). They are very similar to those in Southern Europe, with walls of 40-50 cm thickness, made from cut stones and with larger stones placed at the corners, all laid on low strength mortar. They have wooden lintels above the windows and are somewhat higher than the previous two building types. Vaults are sometimes used as floor elements mainly in older buildings. In more recent years they tend to be two storeyed with the top storey made from timber . RC reinforcing elements and precast RC slabs are also used in recent years. Their plan layout is somewhat more elaborate, with verandas and balconies.

Low-rise brick masonry
This is by far the most common residential load-bearing masonry type. In recent decades more than 75% of all the new low-rise masonry construction is of this type. They are usually one or two storeys high (Plate 8, 9). Similar to the stone masonry buildings, the second floor is often entirely timber framed, especially in mountainous regions. Older urban area buildings in this category have a significantly larger storey height, sometimes exceeding 4 metres. Their architectural layout is more elaborate and less symmetrical, with verandas, porches and various ornamental elements on the façades. The floor element is either wooden or masonry vault. Foundations in older buildings are made from stones laid in lime mortar. These buildings suffered the most damage in 1977. Damage was reportedly more serious in buildings without a basement, because of low quality foundations. Their vulnerability as revealed after the damage surveys of 1977 earthquake in Bucharest is shown in Figure 9. The sample comprises 7483 buildings of brick masonry of 1 or 2 storeys mostly but not exclusively with wooden floors. According to the survey they were all built before the1940 earthquake, and were without any ring beams but possibly had timber lintels. By comparing with Figure 8 we notice that the average damage index is almost equal for both building types. It is useful though to compare the ≥D3 damage line that indicates the amount of buildings that sustained serious and worse damage. Thus we can see that the brick masonry buildings behaved slightly better, and we can also estimate that if adobe and other weaker buildings were located in the central Bucharest zone, that experienced intensity VIII+, their average damage degree should be more than 3.

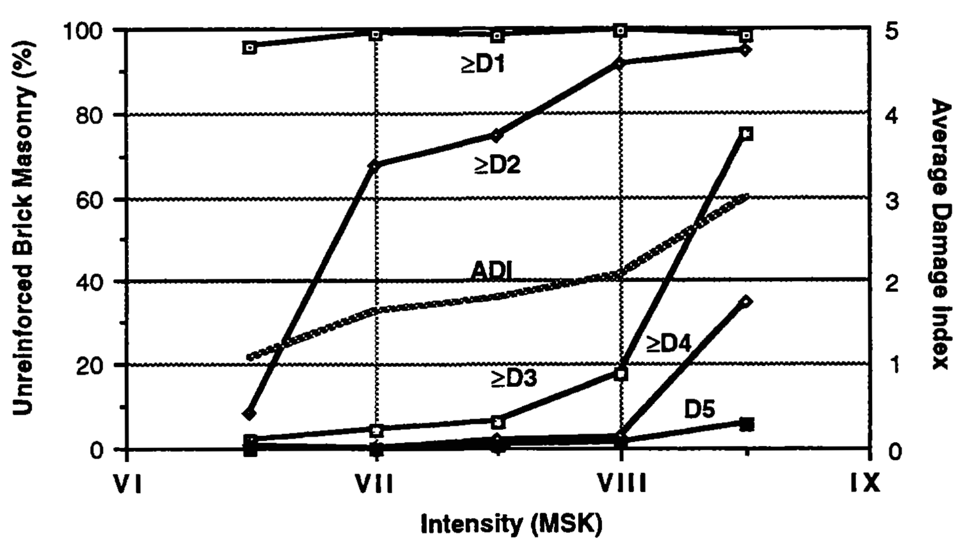

Figure 9: The vulnerability of low-rise unreinforced brick masonry buildings as revealed after a damage survey in Bucharest in 1977.

In newer buildings of this category, RC ring beams and lintels are more common but not yet sufficiently widespead (sometimes the ring beam or the lintel is connected with the floor that is cast-in-situ). RC columns in the corners or wall crossings are also used but more scarcely, even in recent houses. The floors are monolithic or precast RC slabs. Unreinforced concrete or concrete block foundations are more common in recent years. These buildings suffered comparatively less damage than the old ones. Cement mortars are used but information on the common mixtures was not obtained. Balconies and verandas are common in recent buildings. The proportion of openings (windows, doors, verandas) is sometimes too large and in many cases the windows are built too close to the corners or doorways.

Figure 10 shows the vulnerability of pre-1940 reinforced brick masonry houses in a similar manner to Figure 8 and 9. The main difference from Figure 9 is that the buildings of Figure 10 have as horizontal structure cast-in-situ RC slab with embedded ring beams and sometimes RC lintels. The effect of this improvement in construction practice is readily observed, despite the fact that the latter tend to have a more asymetric layout and larger proportion of openings. At intensity VIII the proportion of ≥D4 was only 0.95%, as opposed to 2.5% of the unreinforced brick masonry and 4.4% of the adobe type of buildings.

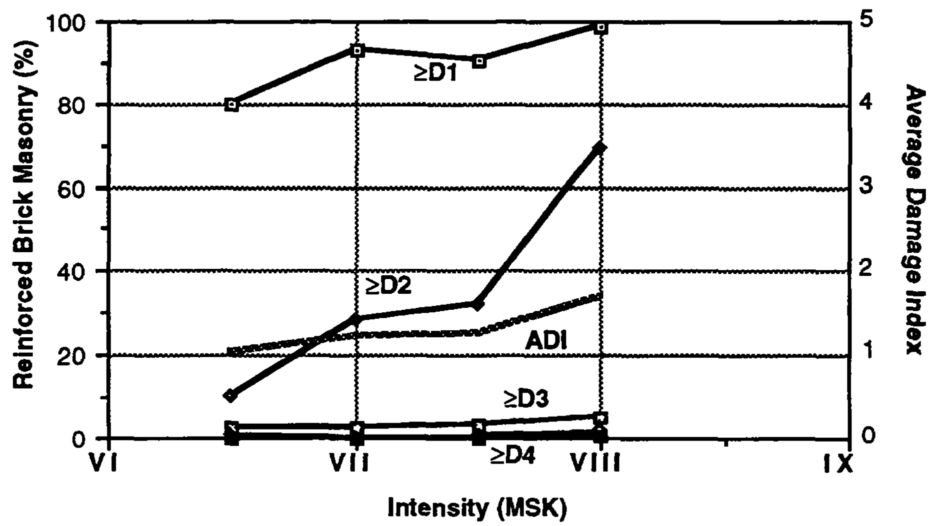

Figure 10: The vulnerability of low-rise reinforced brick masonry buildings, built before 1940 (Bucharest, 1977 earthquake).

Mid-rise masonry apartment buildings

This building type can be divided like the previous one in two categories, according to the time of construction. Older buildings are mostly unreinforced, while newer ones are usually confined (reinforced) load-bearing masonry.

A significantly large number of old apartment buildings of this type exists in Romania, with 3 to 5 storeys. Several such buildings were severely damaged in 1977 in Bucharest and in the small town of Zimnicea (near the Danube, on the border with Bulgaria) . Their number is larger in cities rather than small towns. The floor element is usually wooden, but in some cases (especially in Bucharest) steel or cast-in-situ RC floors were used (Plate 10). The horizontal structure is just bearing on the walls without any other means of tying (similar to the buildings in Spitak, Armenia). Gable walls were made with half brick thickness and were tied only at large distances. Among the three types of horizontal structures (Figure 7) the buildings with RC slabs had a better performance. Solid and perforated bricks were used. The presence of a basement is more common than in the previous category. However their overall behaviour was poorer than that of the good quality low-rise brick masonry houses.

The new confined (reinforced) load-bearing masonry apartment buildings are significantly improved, from the seismic resistance point of view, in comparison to the older ones. Their number is quite large especially in smaller towns located in zones of intensity VI and VII (87% of Romania's territory). Depending on the seismicity of the zone two different systems are used:

(a) in zone VI the longitudinal walls are the load-bearing ones, with the RC slabs bearing on the frontal walls and a middle longitudinal wall. Transversal strengthening walls are placed at 10-12 metres intervals.
(b) in zones VII and VII+ the load-bearing walls are transversal placed at 3-4 metres intervals or they are distributed in both directions encompassing the staircases that are located near the center of gravity.

Perforated bricks, concrete blocks and increasing in recent times lightweight concrete blocks (density 600 kg/m^3) are used in the construction of the walls. The thicknesses are 30 cm for external and 25 cm for internal walls. The mortar strength is also improved, with a mixture of 1:2:6 (equivalent to M5 on EC6 specification). In recent decades this building type has been standardised with the introduction of precast RC elements (slabs and beams) for the optimisation of its construction costs and time. The height of the storeys is usually reduced to 2.7 metres and they are usually 4 to 6 storey high. EEFIT's visit took part almost entirely in seismic zones VIII and IX and therefore this building type was not often encountered. Nevertheless there were a number of smaller privately built houses which were built as confined masonry (Plate 11). With the privatisation of the Romanian economy this building type will become more common. RC columns with four reinforcing bars and stirrups in every 15 to 10 centimetres, are positioned in corners and in some or all the wall crossings (sometimes also on the sides of openings). The columns are connected with the ring beams thus forming framed load-bearing masonry panels. Unreinforced concrete or concrete blocks are used in the foundations that can be up to 3 metres deep. Roofs are usually flat or timber truss of gable or hip and valley form. The amount of openings relative to the total wall area is limited. In 1977 in Bucharest these were among the buildings with the lower average damage. Figure 11, shows the results from the damage survey (sample of 1301 buildings). The buildings in this survey are mostly 4 and 5 storeyed. Their average damage indices are slightly higher than those of Figure 10, possibly because of their longer natural period closer to the predominant periods of the 1977 event. Being high occupancy buildings their importance is increased and the priority should be in preventing their collapse in any future event. For that we should look more carefully at the ≥D4 line that shows the proportion of partially and completely collapsed buildings. It is alarming that at intensity VIII, 7.9% of these buildings were in this category and even at intensity VII the proportion was 5.4%. This is much higher than the damage experienced even by adobe buildings (4.4% and 1.3% respectively). The differences with the buildings in Figure 10 is even larger (0.95% and 0.3% respectively).

Again the average damage index is similar to that of previous figures. However the distribution of the 5 damage degrees differs from the other construction types with significantly worse life thretening behaviour for the mid-rise confined brick masonry construction. The average damage index should thus be used with care in hazard assessment.

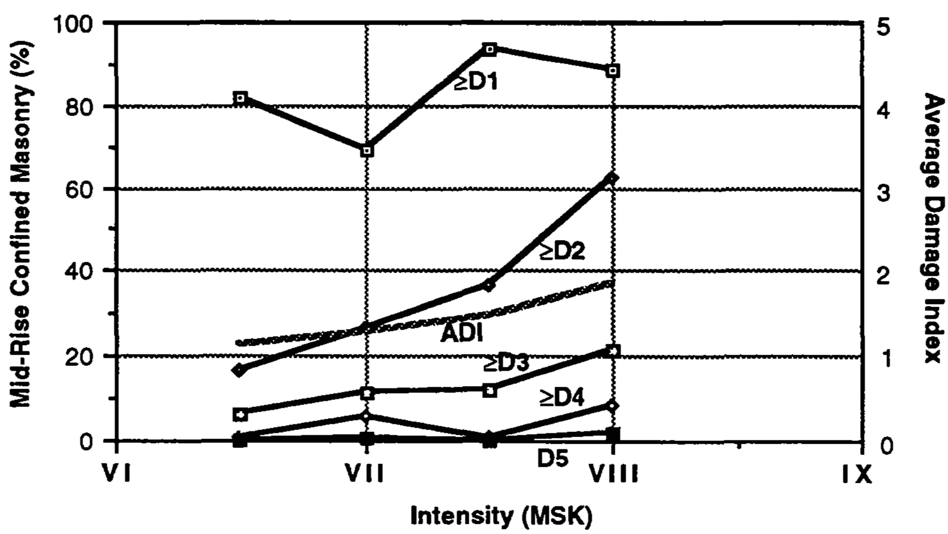

Figure 11: The vulnerability of mid-rise confined brick masonry buildings (Bucharest,1977).

4.2.2 Reinforced concrete buildings

As shown in Figure 7, there are 5 types of RC buildings in Romania. They differ in the degree of their seismic strength and period of construction. The old in-situ frame buildings (built before the 1940 earthquake) are the weakest since they were designed without any seismic consideration and sometimes have discontinuities in the structural frame. After World War II, RC frames of better quality were constructed. Moment-resisting frames started to be built for schools, hospitals and other important structures since the early 50's, but in apartment buildings they are commonly used only since 1970. Shear walls were introduced as early as 1950, but became more common after the introduction of the first seismic construction code in 1963. In Figure 6 we observe that a dramatic change in the structural types used in Romania occurs after 1975. RC framed structures with seismic design but without shear walls and load-bearing masonry buildings were more common until 1975, while the shear wall element is increasing after 1975. Another important attribute distinguishing these buildings is the degree of prefabrication. This has been steadily increased since 1950. In older buildings with precast RC slabs the idea of diaphragm effect (by proper connections with the frame) was not as developed. The shear walls are cast-in-situ or prefabricated, with the latter being increasingly common during the 80's (precast panel construction).

With advances in the prefabrication of structural elements, the RC frame with shear wall system was gradually changed. The new system is a structure of prefabricated shear walls, without the use of frames (UNDP/UNIDO, 1982). This system was used especially for 5 storeyed buildings that do not have shops or open spaces at ground floor. This type of structure is commonly called large RC panel structure, and as Figure 6 reveals it has become the most numerous building type since 1980 (this process was accelerated by the occurrence of the 1977 earthquake).

The large majority of RC buildings were built after the centralisation of the Romanian economy. Consequently their production became more standardised, and a considerable amount of information is available on the construction details of each different RC apartment building type. Figure 12 illustrates in more detail the activity of the state sector in the residential construction. It is estimated that nowadays more than 50% of Romania's population lives in state constructed buildings. Unfortunately exact data on the number of mid-rise and multi-storey buildings are not known, except for Bucharest, where the latter is 67% of the state constructed apartment building stock. Also in Bucharest the distribution between these different building types is as follows:

– RC framed with shear wall	60%
– Large RC panels	23%
– Load-bearing masonry	13%
– RC framed without shear wall	4%

Non-seismic (old) reinforced concrete frames

These were built mostly in large cities since 1930 (in Bucharest they were between 6 and 12 storeys). Their structural layout was inadequate in many cases, and they suffered severe damage during the 1977 earthquake (those built before 1940, were weakened). In many respects they are similar to the confined masonry system, only that they had more storeys. Some of them were underdesigned even for the action of static loads, the dimensions of the frame being small. These buildings are entirely cast-in-situ. A wide range of seismic performance was displayed by this building type, thus e.g. in Bucharest in a small zone that was assigned intensity VIII+ (in the center of the city), the amount of partially and completely destroyed buildings (damage degrees 4 and 5, of the M.S.K. scale) reached **14.9%** while at the same time 33.9% were completely undamaged or with minor cracks in the infill masonry (damage degrees 0 and 1, on the M.S.K. scale) . Their average damage degree, ADI, was 2.14. This clearly indicates a large degree of variety in the strength and performance of this building type. Most of the collapsed structures (60%) were located in street corners and had irregular shapes in plan and in the vertical direction. After 1977, more than 10% of those buildings were demolished. Many of the remaining structures were strengthened, but the exact proportion is not known.

Figure 13 shows the results of the 1977 damage survey . The total number of surveyed buildings was 683 and a total of 30 buildings had damage ≥D4 (4.4%) causing a significant proportion of the 1570 deaths during that event. Most of them were in central Bucharest where the intensity reached VIII+. For comparison we can see in Figure 10 that the proportion of ≥D4 in low-rise unreinforced brick masonry buildings in the same zone reached **34.4%**. At intensity VIII the proportion of ≥D4 drops to **3.5%**, which is better than the post-1950 confined masonry apartment buildings but worse than the low-rise brick masonry buildings (both unreinforced and reinforced).

Seismically designed reinforced concrete frames (with or without shear walls)

These are among the most common multistorey apartment buildings in Romania. Their seismic strength varies considerably depending on their period of construction. Structural discontinuities and inadequate layouts have been avoided, but the joints were not designed to resist moments until about 1970. The dimensions of the frames are adequate, but many of these buildings are designed with soft storeys on the ground floor (Plate12). Their number of storeys is 5 to 12, with common storey height of 2.70 metres (the number of buildings with more than 12 storeys is small). Their degree of prefabrication varies on the period of construction. More recently precast concrete walls are used as non-structural elements instead of infill masonry (Plate 13, 14 and 15). Their size in plan is often very large, reaching even 6 bays (each with its own entrance and staircase) that are separated by expansion joints (Plate 16). The most common incident in these buildings after the May 1990 earthquakes was damage along the expansion joints which are usually filled with pieces of lightweight concrete blocks (due to the lack of other adequate thermoisolation materials, such as glass fibre insulation). These joints in many cases are closed in the façade and back side of buildings with a thick plaster layer. The collapse of large pieces of such plaster in one such building in Bucharest was the cause for the death of two people (see Plates 1 to 6). The width of these joints should be larger in the case of buildings with more than 7 storeys.

The use of shear walls in RC framed buildings started mainly after 1970 (see Figure 6). The technology used for casting the shear walls varies according to the period of construction. The prefabrication of shear walls was introduced after 1975. In the case of long rectangular buildings (the most common type) one or two shear walls are placed in the interior of the building spanning along the longitudinal direction. The floors are two-way slabs totally or partially precast. The frames should have similar stiffness in both longitudinal and transversal directions. The location of staircases is usually close to the center of gravity, otherwise they are built detached from the main structure separated by an expansion joint. The reinforcement of the shear walls was limited until 1977. In some cases reinforcement is only at the top and bottom of the wall and around the openings. The structural layout of the shear walls is similar in concept to that applied to load-bearing masonry buildings with the difference that shear walls are always located in both longitudinal and transverse directions. Their position details are as follows:

(a) The transverse shear walls are located at every 3-4 m centres or at every 6-12 m. The transverse beams bear on the front columns and the internal longitudinal shear wall.
(b) In the case of buildings with shops at the ground floor, all or part of the shear walls at ground floor level are replaced with RC frames. The disadvantage of this solution from the seismic performance point of view is recognized, and after 1977 other solutions are used for such buildings. One solution that is used is to make stronger columns that continue until the 3rd or 4th floor so that the change in building's stiffness is less abrupt.

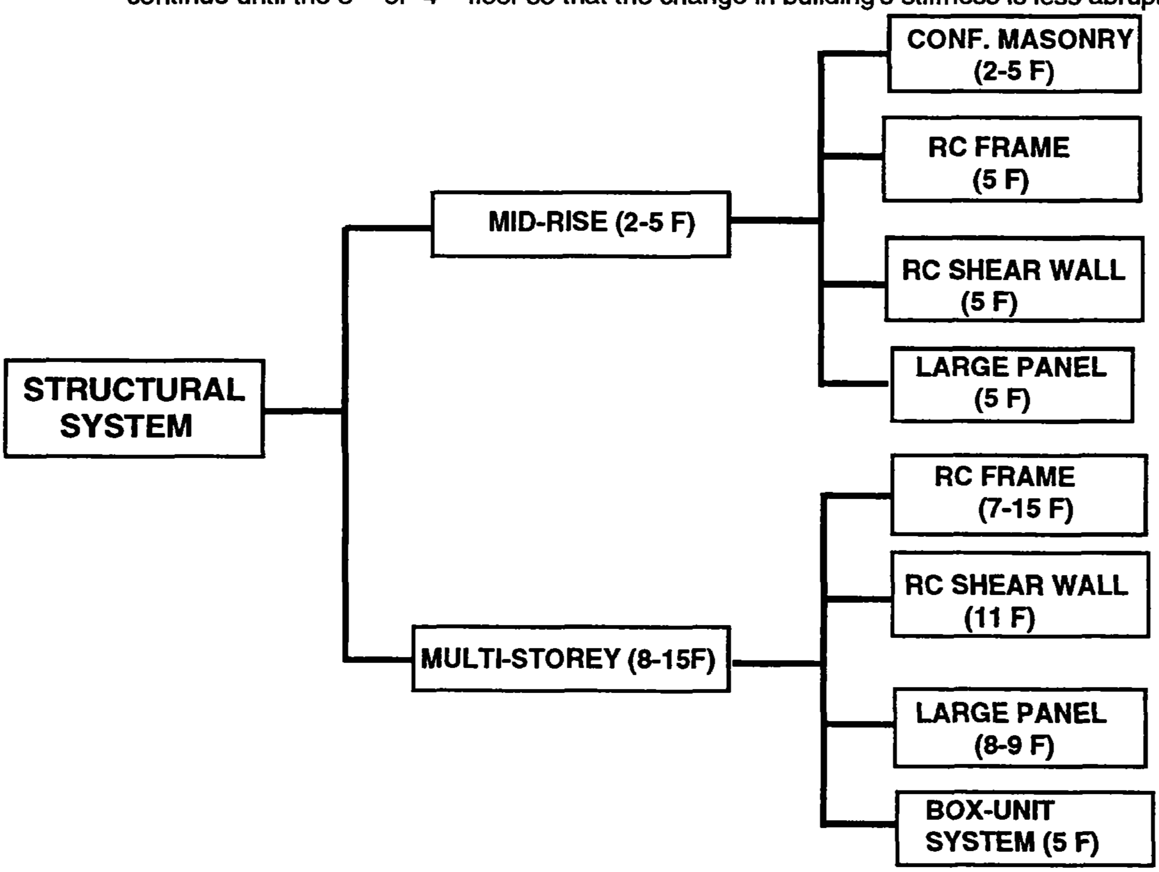

Figure 12: The building type distribution of state constructed large apartment buildings after1950 in Romania.

The density of the transversal shear walls did not affect very much the performance of these buildings as revealed by the surveys carried out after the 1977 earthquake. Nevertheless those with two longitudinal shear walls behaved better than those with a single shear wall. This is because the single longitudinal wall was often sheared by the non-symmetric position of the transverse walls, or it was damaged at the interruption around the staircase. Significant differences in the average damage degree was observed between the 5 and 11 storeyed buildings, with the latter suffering more damage. The cast-in-situ shear walls apparently suffered more than the prefabricated ones. This is because of the inferior quality of concrete used on site, and the less effective reinforcement of the shear walls. Figure 14, shows the result of the 1977 damage survey. The sample size is 613 buildings. Despite the improvement by introducing shear walls, at intensity VIII a proportion of **11%** suffered damage ≥D4 (from which **1.2%** was D5). This is much worse than the non-seismic RC framed buildings built in the 1930-1950 period (≥D4 is only 3.5% at intensity VIII). There were no RC shear wall buildings in the central zone of Bucharest that was assigned intensity VIII+. This unusually high degree of damage can be attributed to a combination of factors such aw: most of the destroyed buildings had a soft ground storey, the location of the 11 storey buildings was in unfavourable ground conditions, many of the older shear walls were not properly reinforced. Nevertheless the proportion of 11% is very high and more attention should be paid in order to improve the performance of these buildings in future earthquakes.

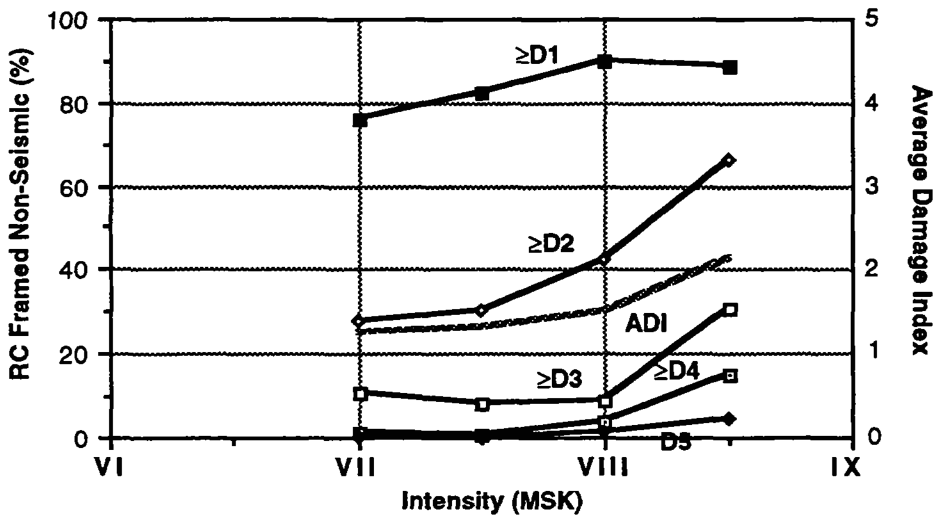

Figure 13: The vulnerability of the old RC framed apartment buildings (Bucharest,1977 earthquake).

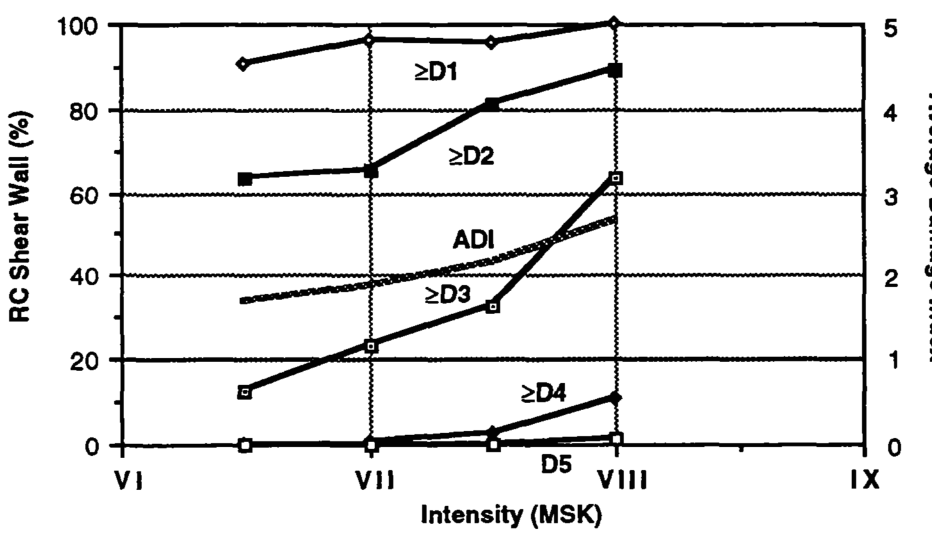

Figure 14: The vulnerability of multi-storey RC shear wall apartment buildings (Bucharest,1977 earthquake).

<u>**Large panel reinforced concrete structures**</u>

This is the most common large residential building type after the 1977 earthquake (see Figure 6). As shown in Figure 12, they are divided in two categories according to the number of storeys (5 or 8-9 floors). These buildings were first introduced in Romania in 1959 with 5 storeys. The connections between the RC panels are of various types depending on the period of construction, with better connections being used as experience with this system increased (EAEE, 1985). All the panels are connected with the horizontal structure through welded and overlapped reinforcements that are subsequently cast with cement mortar or concrete. Thus the structure is an assembly of interconnected shear walls and horizontal RC diaphragms. Not surprisingly the type of connections used in these buildings is the most crucial detail from the seismic safety point of view (Plate 17, 18). The various types of connections are summarised below .

(a) The earliest types of connections were very simple. The panels have smooth lateral sides (with no interlocking) in all their perimeter. At the external end of the panel a 25 mm bar projects from the top and the bottom that is welded to the panels above and below. At the internal end of the panel a 35 mm bar projects at an angle of 45 degrees and is connected to a similar bar projecting from the adjacent panel. Cuts are provided at the connecting corners of the panels so that mortar could be cast subsequently . This system was used until 1964.
(b) The second system was introduced in 1962 and was widely used around the country. The vertical lateral sides of the panel are provided with hollow sockets and corner cut-outs. Vertical bars project from the horizontal lateral side in the case of panels with door openings, but no other vertical projecting bars are used. Instead 6 horizontal projecting bars from the vertical lateral sides are welded with those from the adjacent panel. This connection is subsequently cast with mortar throughout the height of the panel, and the mortar is thus injected into the hollow sockets. This system behaved satisfactorily in many sites during the 1977 earthquake.
(c) The third system was introduced in 1965. The improvement is that the lateral vertical sides instead of having hollow sockets had stagged cut-outs. Horizontal projecting bars were provided in two planes (12 bars in total). Three vertical projecting bars are also used in the horizontal lateral sides of the panel. Otherwise the system is same as (b) and was used until 1979.
(d) This was introduced in 1970. The panels have hollow sockets and nibs in all the four lateral sides, so that a more effective interlocking is achieved. More significantly the connection in this type is cast with in-situ concrete. The six horizontal projecting bars are evenly distributed along the height of the vertical lateral side of the RC panel. This system is widely used especially after 1977.
(e) This system is used in 8-9 storey panel buildings and was introduced in 1973, when the construction of multistorey RC panel buildings started again. The panel has staggered teeth all around its perimeter. Instead of six single horizontal projecting bars, eight evenly distributed projecting hoops are used. A pair of vertical projecting bars is provided at every corner. The hoops were overlapping until 1977, but later they were made to meet head-to-head. Then a spiral bar is placed in-situ and concrete (instead of mortar) is cast. The system behaved well during the 1977 earthquake.

No data on the vulnerability of this building type are available from Bucharest. Nevertheless a similar survey was carried out in the town of Iasi, where 63 large panel buildings of 5 storeys existed. The intensity in this town was not as high as in Bucharest, but **17.6%** suffered failure of the panel (classified by the Romanian engineers as damage degree 4) at intensity VII+. There were no collapses.

<u>Box-unit reinforced concrete structures</u>

This is the building type with the highest degree of prefabrication in Romania. The box-units are complete rooms with openings and panel walls already connected in the factory. Further connections between the boxes are made on site. It was estimated that the construction time of such buildings is only 40% of the RC panel system, with labour reduced by 25%. Box-units are even made in assembly lines. Nevertheless because of the large initial investment (in production factories) and difficulties with transport (traffic and trucks) and lifting on site (cranes), this system has not been used in all the country and is limited to 5 storey buildings. In 1982 there were 2 factories for production of box-units located in Bucharest and Craiova. In 1977, there were few such buildings only in Craiova, where intensity of VII+ was experienced, and their performance was satisfactory. There are mainly two types of box-unit construction technologies:

(a) The four walls are cast horizontally and then lifted and connected. After this the ceiling is connected with the four walls. Subsequently the whole bottomless box is lifted and connected with the floor. The box-units are connected with each other by means of welding projecting bars and casting in-situ concrete around the perimeter, in the construction site. The weight of a typical box-unit is 20 tonnes (17 tonnes in case of lightweight concrete).

(b) The side (separation) walls are assembled first together with the top and bottom slabs. Subsequently the façade and corridor walls are connected with the rest of the assembly to form a complete box. Connection on the construction site is similar to (a).

4.3 Discussion on the vulnerability of the various building types

In Table 7, the proportion of heavily damaged (damage degree D4) and collapsed (damage degree D5) buildings is given. This helps in elucidating the behaviour of the 7 main residential building types during the 1977 earthquake. These results suggest that the vulnerability of new RC buildings, is not significantly reduced. On the contrary we observe that supposedly stronger building types, like RC shear wall or RC large panel buildings (without soft-storey) have a larger proportion of buildings to be demolished. This can be due to the fact that damage degree D4 was assigned to RC buildings as follows:

(a) in case of RC framed buildings: when strong column cracking, or crushing of concrete in columns or buckling of reinforcement in columns or failure of beams, occurred
(b) in case of RC shear wall or RC panels: when a shear wall or panel has failed (failure extent unspecified).

For comparison in masonry buildings this damage degree was assigned in case of leaning of a load-bearing wall. For this reason it is more meaningful to compare the percentages of buildings that have actually collapsed (D5). Up to intensity VII+ the RC buildings behaved better than load-bearing masonry. Nevertheless this is reversed at intensity VIII, where the RC shear wall structures suffered a collapse rate higher than all other building types. With regard to the new RC panel buildings, although none collapsed, the proportion of damage degree D4 is alarmingly high. It is seen that data on intensities higher than VIII are not available for the seismically designed RC buildings (last two types on Table 7). Thus general conclusions cannot be drawn. The behaviour of confined masonry apartment buildings seems more consistent and although they suffered more losses than the low-rise masonry buildings with RC floors, their behaviour up to intensity VIII can be considered as satisfactory (about 8% were demolishable, with only 1.8% being classified as D5).

Table 7: Seismic Vulnerability of the most common building types

Intensity	Io=VI+		Io=VII		Io=VII+		Io=VIII		Io=VIII+	
Dam.Degree	D4	D5	D4	D5	D4	D5	D4	D5	D4	D5
Adobe & Comp. Clay	1.2	0.4	0.9	0.4	0.8	1.0	4.4	0		
Low-rise URBM	0.4	0.1	0.10	0.08	1.0	0.7	1.6	1.0	28.9	5.5
Low-rise RBM	0.6	0.0	0.33	0.0	0.4	0.2	0.71	0.23		
Mid-rise CBM	0.74	0.0	4.6	0.8	0.9	0.0	6.1	1.8		
Non-seismic RC Framed			1.4	0.0	0.9	0.0	2.5	1.0	10.6	4.25
RC Shear Wall	0.0	0.0	1.6	0.0	3.1	0.0	13.5	2.7		
RC Large Panel (Iasi)	17.6	0.0			17.4	0.0				

Notes on Table 7:

- URBM = Unreinforced Brick Masonry with wooden floors
- RBM = Brick Masonry with RC slabs (often with RC ring beam and lintels)
- CBM = Confined Load-Bearing Masonry Apartment Buildings
- The numbers indicate percentage (%) of buildings in each damage level (D4 or D5)

It is understood that the intensities assigned took under consideration the natural period of the buildings. Actually intensities for 3 different period ranges were calculated, namely 0 - 0.15 s (low-rise), 0.15 - 0.25 s (mid-rise) and 0.7 - 1 s (multi-storey). We therefore have to conclude that many of the large apartment blocks constructed after 1950, may perform less satisfactorily in the event of another earthquake having magnitude larger than 7 in Vrancea. This is especially important for those buildings that have been weakened during the 1977, 1986 and 1990 strong earthquakes. In addition it must be remembered that the natural period of the damaged buildings might have been increased by about 25%, which means that if the next large intermediate depth earthquake in Vrancea produces strong motion with predominant long period waves, they will be subjected to an increased loads, in comparison with other undamaged structures with the same number of storeys.

5. SEISMIC VULNERABILITY AND STRONG GROUND MOTION

In recent times with the advance in earthquake related sciences our knowledge on the seismic hazard of earthquake zones around the world has been significantly improved. Furthermore the amount of strong motion observation stations and recordings has increased significantly. This has permitted the improvement in our understanding of attenuation of seismic ground motion with distance and magnitude. However despite these significant advances, the question of quantification and prediction of damage to building types of varying strength, in relation to ground motion severity, remains largely under-researched. This is partly due to the fact that the amount of earthquake records with destructive potential is still quite small and partly because there has rarely ever been a consistent attempt to investigate damage to buildings near to the location of actual recording sites.

Although damage surveys have become more common after the occurrence of major damaging earthquakes, they are not yet standardised and the information is rarely processed to a level higher than simple damage statistics and hardly ever published in a detailed manner. One of the best post-earthquake damage surveys that has ever been carried out was the one carried out by INCERC in Bucharest after the 1997 earthquake. Some elements of this survey were outlined in Chapter 4, Figures 8, 9, 10, 11, 13, and 14 for the six common building types of Romania. Nevertheless even these surveys were done with intensity scales in mind and as a result the information was averaged to each degree of intensity in order to obtain damage probability matrices. Different intensity scales and methods of quantifying the damage (number of damage degrees or mean damage ratios or replacement cost percentages) further enhance the handicap in making this valuable information more useful for future improvement of our knowledge on the seismic vulnerability of buildings and the seismic risk of earthquake prone regions.

5.1. A review on the correlation between intensity and acceleration

Many researchers have obtained relationships between ground motion (mostly peak horizontal acceleration) and various intensity scales (mostly MM, MSK and MCS). Some of the most commonly used conversions between peak horizontal acceleration and intensity are shown in Figure 15 along with some of the most recent studies on the same subject (Gutenberg & Richter, 1956; Ambraseys, 1975; Trifunac & Brady, 1975; Murphy & O' Brien, 1977; Margottini et. al., 1987; Krinitzky & Chang, 1988). Although such relationships are useful for seismic risk assessment, it must be pointed out that most of the intensity values used in these studies portray the average severity of the motion experienced over a given area while the ground motion parameters were recorded in just a point within this area. As a result of more detailed damage surveys we now increasingly know that even within small areas it is possible to have intensities ranging from VI to IX and we cannot certainly correlate any single recording with an average intensity for the location in question. In addition we have seen in recent earthquakes (1986 San Salvador, Shakal et. al. 1987 and Kalamata, Greece, Thompson et. al., 1986) that recorded strong motion might differ significantly even if the distance between the instruments is only 1 km or so. Furthermore in the countries that tend to suffer most of the damage there are only few instruments often installed in basements of large state owned buildings, thus making the scarce recordings less reliable indicators of ground motion.

As it is shown in Figure 15 all the relationships assume a linear correlation between intensity and the logarithm of peak acceleration, but there is a significant difference between the predicted values of acceleration for each intensity level. It is also shown that almost all of the studies predict higher accelerations for each intensity degree, than those values expected in the original definition of the MSK intensity scale.

The amount and type of data as well as the statistical method each study has used, largely influence the expected results. It is thus seen that the Ambraseys relationship predicts by far higher accelerations than any other study mainly due to the fact that his data are from European earthquakes, while most of the other commonly used relationships are based on Western USA recordings. On the other hand the Gutenberg and Richter study predicts much lower accelerations than the Trifunac and Brady relationship, although they use largely the same data set. This is because the former study correlates the peak horizontal accelerations with intensity using the common least squares method, while the latter correlates each intensity degree with the numerical average of the peak horizontal acceleration data available for each intensity level, in order to avoid the large scatter of acceleration values at each intensity level.

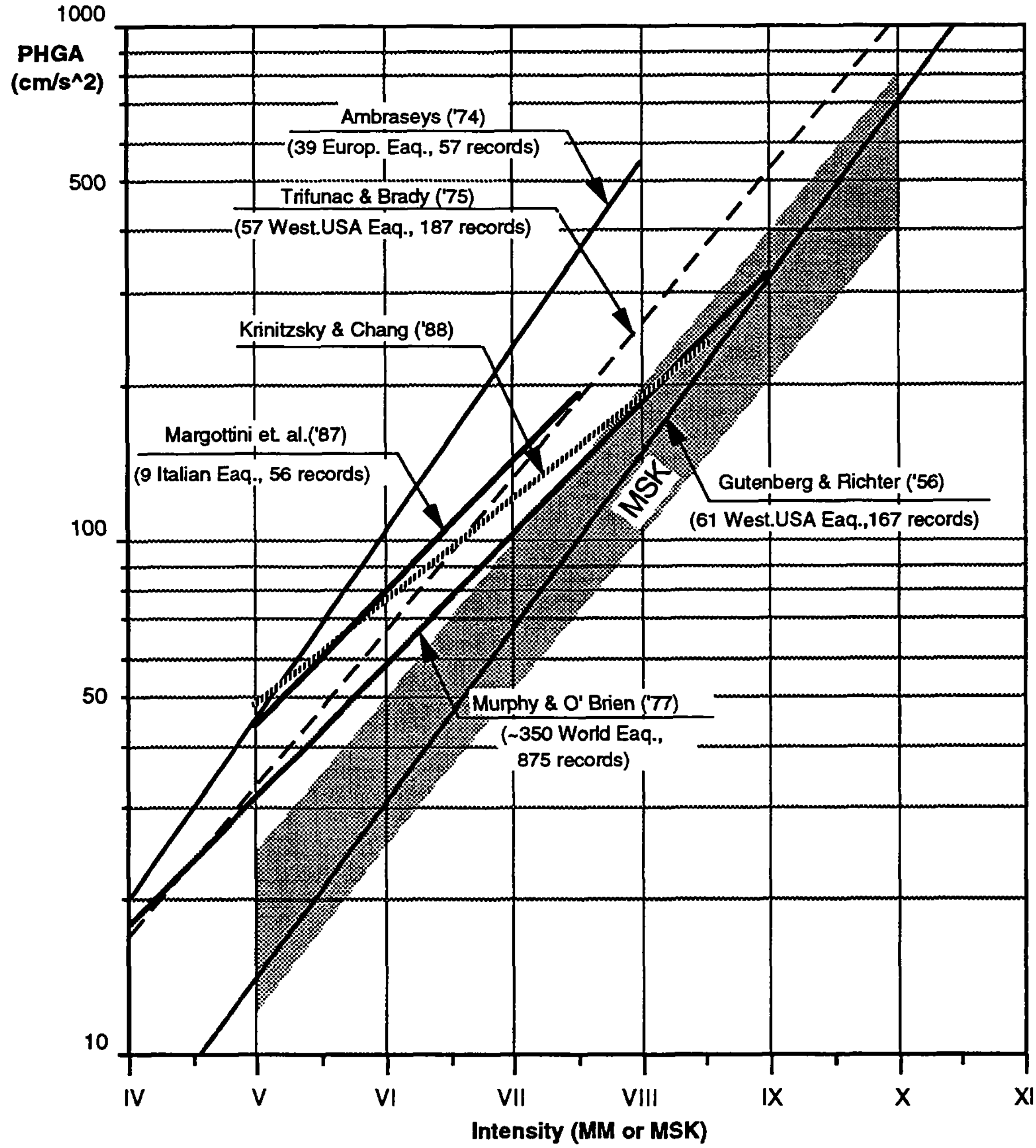

Figure 15: The most important studies on the correlation between Peak Horizontal Acceleration and Seismic Intensity. The shaded area is that of the MSK'64 scale definition. The number of earthquakes, region of the data and the number of records are also shown for each case.

The Murphy and O'Brien relationship is obtained using by far the largest data set composed from worldwide recordings (507 W.USA, 315 Japan and 53 European). All the 875 recordings have peak acceleration ≥1%g and the statistical method used is that of the least squares. This study has also processed the data separately for each of the three aforementioned regions and found similarly to the study of Ambraseys that the peak horizontal accelerations at fixed values of intensity, magnitude and epicentral distance can be up to about a factor of two higher than the corresponding values for Western USA and Japan. Nevertheless it must be pointed that this can be confirmed only for the intensity range of IV-VII. Finally the study by Margottini et. al. is based on a local assessment of the intensity near 56 strong motion instruments in Italy, for the intensity range IV-VII$^{1/2}$, thus making its relationships with actual ground motion more reliable than any other previous study (in the Murphy and O'Brien study, about 60% of the intensity values are believed to have been determined in the vicinity of the recording instrument).

Finally it must be pointed out that all of these studies are largely based on less severe strong motion records and observed intensities. From the 1465 original Murphy and O'Brien records only 462 were in the intensity range VI to VIII, and only about 250 records had a peak horizontal acceleration greater than 50 cm/s^2. Similarly only 22 records in Margottini et. al. study are in the intensity range VI to VII1/2 and only 21 had peak acceleration greater than 50 cm/s^2. The Trifunac and Brady study had only 7 sites with intensity larger than VII, while the Murphy and O' Brien had 19 and the Margottini et. al. only 2. This clustering of data in the lower part is certainly having an effect to the form of these equations. Furthermore if we want an indication of the relationship between peak acceleration and its damage potential it is preferable to use only the data of intensity VI or higher, because all the other data are harmless even to weak masonry buildings. The proportion of data with intensity VI or larger (onset of damage to weak masonry buildings) in the studies discussed above is as follows:

Trifunac and Brady 80.5% (149 records)
Margottini et.al. 39.3% (22 records)
Murphy and O'Brien 33.7% (462 records).

More recently an Italian study published relationships between PGA and MSK intensity, based on damage reports near 58 strong motion instruments, thus giving a more reliable estimation on the effect of ground motion upon damage to buildings. Nevertheless the sample of buildings was very small, thus making the intensity definition less reliable, and most of the intensities are less than VII. It is clear that there is a need for improving our knowledge in this area.

In order to improve the reliability of future vulnerability functions and risk assessment methodologies, the Martin Centre first carried out damage surveys around strong motion instruments after the 1980 Campania earthquake in Southern Italy. Several other surveys have been carried out subsequently in Corinth and Kalamata, Greece (1981 and 1986) and in Gukasyan, Armenia (1988). Correlation between various strong motion parameters (peak acceleration and velocity, response spectral acceleration and velocity, strong motion duration) and damage observed upon masonry buildings have been obtained and the results are encouraging. All of these surveys were carried out in zones where the predominant period of ground motion ranged between 0.2 and 0.45 seconds. The 1990 Vrancea earthquake, has probably produced motions with longer predominant periods and therefore surveys around the recording sites should add more variety and validity to the existing dataset. In section 5.2 the surveys that the EEFIT team carried around 5 strong motion observation sites are discussed.

5.2 Damage surveys around strong motion recording stations

The sites were chosen after inspecting the first information made available to us by the Romanian colleagues on the strong motion recordings obtained after the two earthquakes. It was decided to visit locations of known strong motion recordings and inspect damage distibution around them. Damage surveys were done by the members of the EEFIT team in **Buzau** (recorded PGA = 25%g during the main shock), **Focsani** (PGA = 14%g in the main shock and 18% in the aftershock) and **Valenii de Munte** (PGA = 17% in the main shock). The damage surveys were done within a 400 m radius from the recording instrument, so that the damage distribution of the different building types could be reasonably correlated to the recorded ground motion (see Appendix B for the assignment of damage degrees). Useful conclusions will be drawn when the details of these 5 strong motion records are published. The results of these surveys are discussed in the following sub-sections.

5.2.1 Survey in Buzau

Buzau is situated on the main road linking Bucharest with the northern region of Moldavia (see Figure 1). It is a town of regional importance and capital of the homonymous county, with population of 98,000 (in 1977). Its epicentral distance is 78 km, and in the 1978 Code it is located at the seismic zone of intensity VIII. The observed intensity in the 1977 earthquake was VII+. As shown on the map (Figure 3) the 25%g PGA recorded in Buzau is the second highest during the main shock (and the third highest ever recorded in Romania). The instrument is located at the basement of a 6 storey RC office building, near the centre of the town and it was ideally situated between two zones of the town that had completely different building stock. On one side there were only old low-rise masonry buildings, while on the other there are only multi-storey RC shear wall or large panel apartment blocks. In total 145 buildings were surveyed, from which 105 (72.5%) were low-rise masonry buildings.

Among the old masonry buildings there was a considerable variation in building types: low-rise brick masonry (Plate 19) being the most numerous (78 buildings), followed by adobe and compact clay buildings (27). Among the brick masonry buildings there were only 8 buildings with two storeys, while all of the adobe-clay type of buildings were single storeyed (Plate 20). In addition 10 (12.8%) of the brick masonry buildings were reinforced either with RC lintels or with ring beams (RC slabs were in only 2 buildings). The usual storey height of the brick buildings was 3.3 metres, while the adobe houses are lower (2.7 metres). The condition of this housing stock was also quite varied in level of maintenance. Almost all of these houses were built long before 1977, thus surviving all the three earthquake sequences. Evidence of repaired cracks after the 1986 earthquake was widespread (Plate 21). Figure 16 shows the damage degree histograms for the three main building types. For masonry buildings D1 was assigned in the case of small and isolated cracks, D2 in case of wider and more numerous cracks, D3 in case of dislodged masonry, separation of walls in corners (Plate 22), complete chimney collapse or strong diagonal cracks, D4 was given to a few buildings that had lost the upper half of their gable walls. This is somewhat different to the damage degree classification used by the Romanian engineers in 1977 when partial gable wall collapse was assigned only D2 and total gable wall collapse D3.

The average damage of brick masonry buildings was similar to that of the adobe buildings. This is mainly due to the non-symmetrical layout effects on the brick buildings. The brick buildings were of much better quality from architectural and structural points of view (Plate 23). However their complex layout (large entrance porches, large openings, higher storey height, verandas etc.) had a negative effect in comparison to the humble adobe-clay type of buildings that have a very simple layout in plan with small openings and lower storey height. Nevertheless, if the motion was more severe the average damage degree of the latter would surely be higher (as seems to be suggested by Figures 8 and 9). This damage distribution is equivalent to MSK intensity VII or slightly smaller.

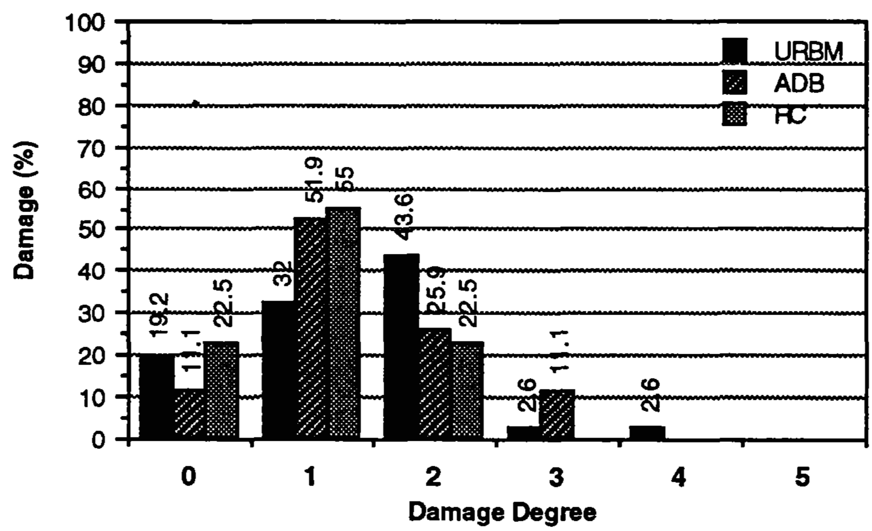

Figure16: Damage degree histograms of the three main building types in the survey of Buzau (URBM = Unreinforced Brick Masonry; ADB = Adobe-Clay; RC = RC framed and RC shear wall).

If PGA - I_{MSK} relationships were to be used, the expected intensity would be VIII. This is a strong indication that in the case of intermediate depth earthquakes, PGA is not a good ground motion parameter for vulnerability analysis. In Table 8, the survey results around strong ground motion instruments that recorded peak acceleration around 25%g in four different earthquakes are summarised. We notice that Buzau has the lowest damage of all. **MRSA** is the **mean spectral acceleration at 5% damping between the two horizontal components and at the period range of 0.1 to 0.3 seconds**, typical for low-rise masonry buildings.

The product between acceleration and velocity has been suggested by Sandi (1986 and 1990) as a good means of measuring the intensity of ground motion. In this report we first introduce the idea of correlating damage with the product parameter **MRSA*MRSV** which is the product between the aforementioned mean response spectral acceleration and the respective response spectral pseudovelocity (at the same period range). The complete analysis of the Buzau record will shed more light onto the effect of these and other strong motion parameters (Arias intensity, strong motion duration, velocity, displacement etc.). It seems that the MRSA in Buzau was less than 450 cm/s^2 and the product MRSA*MRSV should be less than 7000 cm^2/s^3. In the North of the instrument there is another area with a concentration of masonry buildings (of newer construction) that was also surveyed (about 25 buildings). In this zone that is just outside the 300 metre radius, the damage was even smaller (just a few cases of D1). These buildings were not included in the damage distribution of Figure 16.

Table 8: Damage to low-rise masonry buildings for PGA around 25%g

SITE	PGA (cm/s2)	MRSA (cm/s2)	MRSA*MRSV (cm2/s3)	≥D1 (%)	≥D2 (%)	≥D3 (%)	≥D4 (%)	D5 (%)
Brienza ('80)	224	540	8262	74	60	30	0	0
Corinth ('81)	281	625	10500		57	27	6	0
Kalamata ('86)	270	620	10478	100	85	65	32	5
Buzau ('90)	250	< 450	< 7000	83	46	7	2	0

The reinforced concrete buildings suffered little damage (ADI = 1.00) and none had damage degree 3 or larger. D1 was assigned in case of non-structural cracks and small damage along the expansion joint or cracks at the cast-in-situ joints between the RC panels. D2 was assigned in case of small column or beam cracks, or wide cracks along the expansion joint (Plate 24), cracks between the panels (Plate 25) or diagonal cracks to the infill masonry (Plate 26).

5.2.2 Survey in Focsani

Focsani is the capital town of Vrancea county (see Figure 1) with a population of 65,000 (in 1977). Although it has a smaller epicentral distance (40 km) the PGA there reached only 14%, while during the aftershock it was the only station that recorded a higher peak (18%). In 1977 the town experienced an intensity near to VIII. In 1986, the town was affected the most among other towns in the region and PGA had reached 26% and the observed intensity was VII+.

The location of the instrument in Focsani was in the basement of a 5 storey RC hotel (Hotel Uca), situated in the centre of the town. The buildings surveyed in Focsani were quite different from those in Buzau with a variety of building types and uses (Hotel, Post-Office, Bank, County Library, Telecommunications building, Department store and residential apartment blocks and small load-bearing masonry houses of high architectural value). In total, 88 buildings were surveyed with 48% being of RC frame or shear wall construction. In the Code of 1978 the risk of Focsani was upgraded from intensity VIII to IX. Many of the masonry buildings were already strengthened from repair works carried out after the 1977 and 1986 earthquakes. From this point of view Focsani is a unique town around the world, since it has suffered three earthquakes causing intensity VII-VIII in a short interval of 13 years. The RC buildings surveyed were mostly 4-5 storeyed (93%), while 60% of the masonry buildings were single storey with the rest two storeyed. About 40% of the masonry buildings were strengthened and no adobe-clay type of building existed in the surveyed area. The RC buildings were almost equally divided between framed and shear wall structures (20 framed and 22 with shear walls). From most points of view the sample of buildings in Buzau is of a smaller strength than that in Focsani. In Figure 17 the damage degree histograms in Focsani are shown, the same principles were followed in assigning a damage degree as for Buzau (section 5.2.1).

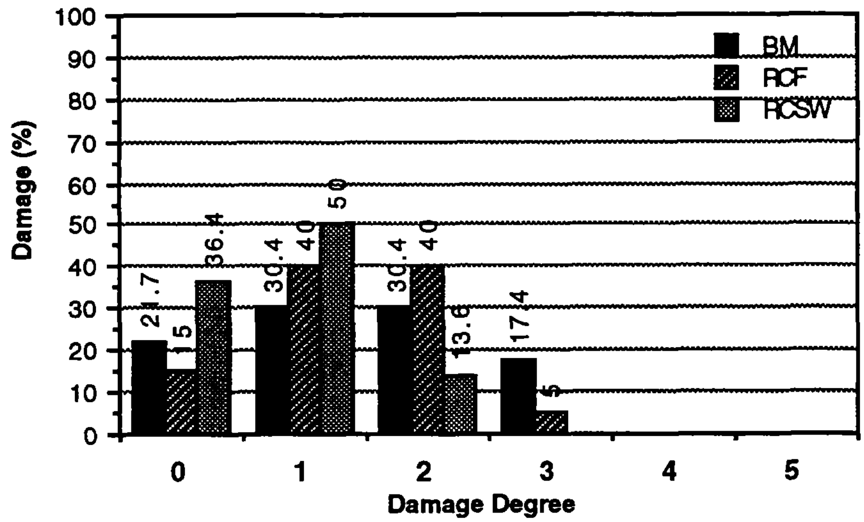

Figure 17: Damage degree histograms of the three main building types in the survey of Focsani (BM = Brick Masonry; ; RCF = RC framed; RCSW = RC shear wall).

It is also evident that the damage in Focsani appears to be somewhat greater than in Buzau, this being quite incompatible with the lower recorded peak acceleration. It is expected that the greater damage in the surveyed area was actually due to the combined effect of the 1986, 1990 earthquakes. Therefore we have to conclude that these damage distributions are more compatible with the 1986 acceleration time history, rather than the 1990 recording. The mean response spectral acceleration of the 1986 recording was 400 cm/sec^2 with the predominant period occurring at 0.48 seconds.

5.2.3 Survey in Valenii de Munte

Valenii de Munte is a small small town (10,500 people in 1977) at the foot of the Carpathian mountains (see Figure 1). It is situated at an epicentral distance of 90 km. The recorded PGA during the main shock reached 17%g. 48 buildings were surveyed there, 65% being masonry and the rest RC. Most of the masonry buildings (77%) were unreinforced brick and single storeyed (80%). Most of the RC buildings were 5 storeyed apartment blocks (76%) of RC frame (58%), or RC shear wall construction. There were also 7 reinforced brick masonry buildings of which only one had some fine cracks (D1). Due to the small number of RC buildings (17) we will not try to estimate the damage degree distribution for each of the two types. We will point out though that only 2 buildings were assigned damage D2, and these were 5 storeyed RC frame structures having a long rectangular shape and shops at the ground floor (soft storey). Their damage was strong evidence of pounding all along the height of the expansion joints between the bays. In Figure 18 the damage degree histograms in Valenii de Munte are shown.

Compared with the other two surveys in Buzau and Focsani we notice a significant difference. Although unfortunately the 1986 acceleration for Valenii de Munte is not known at the moment, the impression from our survey is that it must have been lower or nearly equal to that recorded this year, because there was no evidence of previous damage and repair works. For further comparison in Table 9 the results of three other damage surveys where recorded PGA was similar to Valenii de Munte are given. We notice a wider variation than in Table 8, with Gukasyan (Armenia, 1988) suffering more damage than the two Italian and the Romanian town. We must notice though that the MRSA in Gukasyan is noticeably higher than in the two towns of Italy and almost double that of Valenii de Munte. This emphasises the lower reliability of predicting damage based only on PGA values. Another reason for the lower damage to the masonry buildings in Valenii de Munte is that they were good quality brick masonry mostly one storeyed, while those in the Italian towns were heavier stone masonry many of them two storeyed. In Gukasyan they were built from lightweight tuff volcanic stone and they were mostly single storeyed (EERI - NRC, 1989).

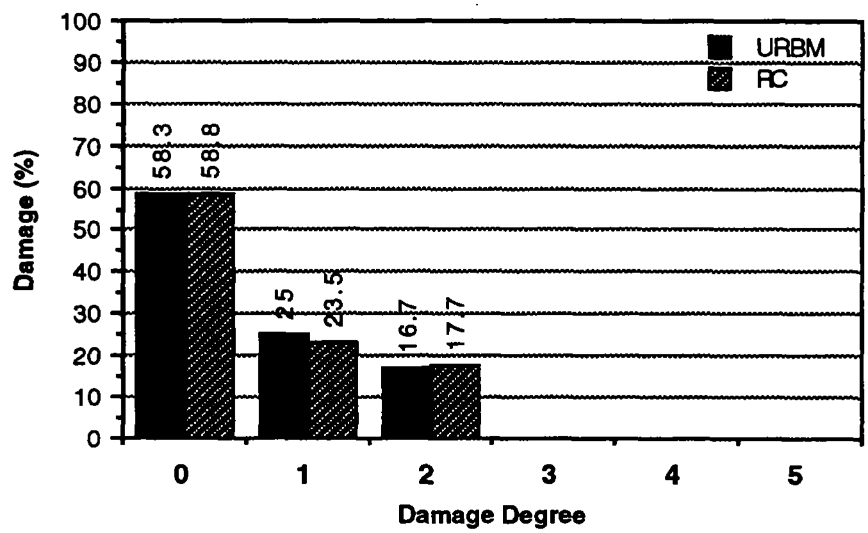

Figure 18: Damage degree histograms of the two main building types in the survey of Valenii de Munte (URBM = Unreinforced Brick Masonry; RC = Reinforced Concrete).

Table 9: Damage to low-rise masonry buildings for PGA around 17%g

SITE	PGA (cm/s2)	MRSA (cm/s2)	MRSA*MRSV (cm2/s3)	≥D1 (%)	≥D2 (%)	≥D3 (%)	≥D4 (%)	D5 (%)
Calitri ('80)	177	340	3366	59	37	0	0	0
Bagnoli ('80)	188	305	2654	76	36	4	2	0
Gukasyan('88)	181	490	4165		75	45	0	0
Valenii ('90)	170	260		36	13	0	0	0

5.2.4 Brief surveys in Ploiesti and Mirinescu street, Bucharest

In addition two more sites were visited in the town of **Ploiesti** (recorded PGA 8.5% in the main shock) and **Mirinescu Street in Bucharest.**

In the case of Ploiesti (see Figure 1), the instrument was located in the basement of a 10 storey RC shear wall apartment block, that was severely damaged along its expansion joint (Plate 27, 28) and also had considerable beam cracks at ground and second floor level. This building has also been damaged during the 1986 earthquake and many of the occupants expressed fears for its structural safety and disatisfaction for the quality of the repair work. The building was equipped with an additional accelerometer at the roof level, that recorded a peak of 30%g. All the other buildings around this site, where large apartment blocks of similar construction and similarly they were cracked along their expansion joints (like in Colentina, Bucharest). The total number of buildings though is too small to make any reliable statistical analysis.

In Mirinescu Street (Bucharest) there was no evidence of damage. The area has mainly two storeyed load-bearing masonry houses of good structural and architectural quality, similar to the building type that suffered the smallest damage in 1977 (Plate 29). The details of the motion recorded in an instrument located at the basement of a two storeyed brick masonry building are not yet known.

6. CONCLUSIONS

The main conclusions and lessons to be learned from this earthquake are summarised below.

1. The earthquakes of 30-31 May 1990, were quite strong but did not cause severe damage to buildings or other structures. The magnitude of the main shock was 6.7 and had it been a shallow event the amount of damage would have been much greater. Nevertheless its focal depth of about 80 kilometers is somewhat shallow for the Vrancea region, and that is why the effects of the shaking were felt strongly and there were 8 fatalities.

2. It was pointed out that the seismicity in the Vrancea region is of paramount importance to Romanian engineers. According to our analysis the next earthquake of magnitude 7 or greater has more than 80% probability to occur during the following 40 years with the most likely period being after 25 years.

3. The standards of seismic design in Romania are quite advanced, but there is a problem with the implementation and policing of the construction activity. This is expected to be more acute now that the country is changing towards a market economy. The amount of prefabrication was found to be very high but the quality of connections although improved over the years still needs to be improved further.

4. Due to the lack of significant damage and the lack of information on the building practices in Romania in the West, an overview of the building technology of this country was prepared for the purposes of this report. The seismic vulnerability of the six most common structural types of buildings was illustrated with the results of the 1977 INCERC damage survey in Bucharest.

5. A review of correlations between macroseismic intensity and peak recorded acceleration was presented as an introduction for the justification of the usefulness of damage surveys in the vicinity of recording instruments.

6. The amount of strong earthquake recordings obtained after this earthquake sequence is very large. This is a success of the Romanian Building Research Institute and it is hoped that their analysis will contribute to a better understanding of issues related with the damage potential and nature of ground shaking.

7. The damage surveys carried out by EEFIT around three recording stations, have shed new light regarding the damage potential of ground shaking. The surveys proved that peak ground acceleration is not adequate parameter for estimating the strength of shaking, especially in case of intermediate depth earthquakes. On the other hand response spectral acceleration seems a promising means to describe the strength of motion much more accurately. More surveys are needed in order to built-up a statistically significant sample which will allow reliable conclusions to be drawn.

ACKNOWLEDGEMENTS

EEFIT would like to acknowledge the following:

- The Science and Engineering Research Council (SERC) for providing the financial support for all thee mebers of the mission team to Romania.

- The Building Resaerch Institute of Romania (INCERC) for accepting the EEFIT team and providing priceless support without which this report would not have been possible.

- Eng. Emil-Sever Georgescu, for his support throughout the field trip in every respect and for providing important reference material and background information.

- Dr. Horea Sandi, Head of the Seismology and Earthquake Engineering Division of INCERC, for his kind advise and scientific lectures during our meetings in INCERC.

- Dr. Olga Stancu for her support during the field trip in Vrancea and her priceless help in helping us during the surveys.

- The British Council Staff at the British Embassy of Romania in Bucharest.

REFERENCES

Ambraseys, N. (1975). The Correlation of Intensity with Ground Motion. *Proceedings of the 14th Assembly ESC.*

Balan, S., Cristescu, V. and Comea, I. (1982). *Cutremurul de pamint din Romania de la 4 Martie 1977.* Romanian Academy Publication, Bucharest (in Romanian).

Coburn, A.W., Hughes, R.E., Nash, D.F.T. and Spence, R.J.S. (1982). Damage Assessment and Ground Motion in the Italian Earthquake of 23.11.80. *Proceedings of the 7th E.C.E.E.* Vol. 1, Athens, Greece.

EAEE (1985). *Prefabricated Large Panel Buildings in Romania.* European Association of Earthquake Engineering, Working Group on Prefabricated Panel Buildings. Sofia, November 1985.

EERI - NRC (1989). *Armenia Reconnaissance Report.* Earthquake Spectra Special Supplement, August 1989.

Georgescu, E-S. (1986). The earthquake behaviour of earthen and low-strength masonry rural buildings in Romania. *Proceed. of the Middle East and Mediterranean Regional Conference on Earthen and Low-Strength Masonry Buildings in Seismic Areas.* Middle East Technical University in Ankara, September 1986, Turkey.

Gutenberg, B. and Richter, C.F. (1956). Earthquake Magnitude, Intensity, Energy and Acceleration. *Bulletin of the Seismological Society of America,* Vol. 46.

Krinitzsky, E.L. and Chang, F.K. (1988). Intensity - Related Earthquake Ground Motions. *Bulletin of the Association of Engineering Geology* Vol. 4.

Margottini, C., Molin, D., Narcisi, B. and Serva, L. (1987). Intensity vs. Acceleration: Italian Data. In the *Proceedings of the Workshop on Historical Seismicity of Central-Eastern Mediterranean Region.* Published jointly by ENEA and the International Atomic Energy Agency. ENEA CRE Cassacia, Roma - Italy.

Murphy, J.R. and O' Brien, L.J. (1977). The Correlation of Peak Ground Acceleration Amplitude with Seismic Intensities and other Physical Parameters. *Bulletin of the Seismological Society of America,* Vol. 67, No., 1977.

Radu, C., Radulescu, D., Sandi, H. (1991). Some data and considerations on recent strong earthquakes of Romania. *French Association of Earthquake Engineering (AFPS).* Domaine de Saint-Paul, B.P. 1, 78470 Saint-Remy-les Chevreuse, Paris, France.

Sandi, H. (1986). An Engineer's Approach to the Scaling of Ground Motion Intensities. *Proceedings of the 8th E.C.E.E.,* Vol. 1, Lisbon, Portugal.

Sandi, H. (1990). An Approach to the Development of an Up-to-Date Intensity Scale. *Proceedings of the 9th E.C.E.E.,* Moscow, U.S.S.R.

Shakal, A. F., Huang, M., Linares, R. (1987). The San Salvador Earthquake of October 10, 1986 - Processed Strong Motion Data. *Earthquake Spectra* Vol. 3, No. 3.

Thompson, P., Cuny, C., Coburn, A.W., Cheretis, J. and Georgoussis, G. (1986). *Earthquake Damage Assessment and Recovery Issues in Kalamata Region, Peloponnese, Greece.* Joint Report by INTERTECT (Dallas, U.S.A.) and The Earthquake Planning and Protection Organisation OASP (Athens, Greece).

Trifunac, M.D. and Brady, A.G. (1975). On the Correlation of Seismic Intensity Scales with the Peaks of Recorded Strong Ground Motion. *Bulletin of the Seismological Society of America,* Vol. 65, No.1, February 1975.

UNDP/UNIDO (1982). State of the Art Report on Prefabricated Concrete Buildings in Romania. UNDP Project RER/79/015, *Building Construction under Seismic Condition in the Balkan Region.* Bucharest, April 1982.

UNESCO (1982). National Report of Romania on Observed Vulnerability of Buildings. Working Group B. UNDP Project RER/79/014, *Earthquake Risk Reduction in the Balkan Region.* Skopje, November 1982.

APPENDIX A:

CATALOGUE OF SIGNIFICANT EARTHQUAKES IN ROMANIA (1091-1990)

Date (Y/M/D)	GMT	Epicentre (^{0}N-^{0}E)	I_o	Depth (km)	M_S (M_l)
1091		45.7 - 26.6	VII	i	(6.1)
1107/8/8	3:00	45.7 - 26.6	VII	i	(6.1)
1126/8/8	0:00	45.7 - 26.6	VII	i	(6.1)
1170/4/1		45.7 - 26.6	VIII	i	(6.7)
1196/2/13	7:00	45.7 - 26.6	VIII	i	(6.7)
1230/4/10	7:00	45.7 - 26.6	VIII+	i	(6.9)
1327		45.7 - 26.6	VIII	i	(6.7)
1446/10/10	4:00	45.7 - 26.6	VIII	i	(6.7)
1471/8/29	10:	45.7 - 26.6	VIII	i	(6.7)
1516/11/24	12:	45.7 - 26.6	IX	i	(7.2)
1523/6/9		45.7 - 26.6	VII	i	(6.1)
1543		45.7 - 26.6	VII	i	(6.1)
1545/7/19	8:	45.7 - 26.6	VIII	i	(6.7)
1569/8/17	5:	45.7 - 26.6	VIII	i	(6.7)
1590/8/10	20:	45.7 - 26.6	VIII+	i	(6.9)
1598/11/2	2:	45.7 - 26.6	VII	i	(6.1)
1599/8/4		45.7 - 26.6	VII	i	(6.1)
1604/5/3	3:	45.7 - 26.6	VIII	i	(6.7)
1605/12/24	15:	45.7 - 26.6	VIII	i	(6.7)
1606/1/13	1:	45.7 - 26.6	VII+	i	(6.4)
1620/11/8	13:	45.7 - 26.6	VIII+	i	(6.9)
1620/12		45.7 - 26.6	VII+	i	(6.4)
1637/2/1	1:30	45.7 - 26.6	VII+	i	(6.4)
1650/4/19		45.7 - 26.6	VII	i	(6.1)
1679/8/9	0:	45.7 - 26.6	VIII	i	(6.7)
1681/8/18	0:	45.7 - 26.6	VIII	i	(6.7)
1701/6/12	0:	45.7 - 26.6	VII+	i	(6.4)
1711/10/11	0:	45.7 - 26.6	VII	i	(6.1)
1738/6/11	10:	45.7 - 26.6	VIII+	i	(6.9)
1778/1/18	5:45	45.7 - 26.6	VII	i	(6.1)
1790/4/6	19:29	45.7 - 26.6	VIII	i	(6.7)
1793/12/8	6:10	45.7 - 26.6	VII	i	(6.1)
1802/10/26	10:55	45.7 - 26.6	IX	i	(7.5)
1829/11/26	1:40	45.7 - 26.6	VII+	i	(6.4)
1838/1/23	18:45	45.7 - 26.6	VIII	i	(6.7)
1868/11/13	7:45	45.7 - 26.6	VII+	i	(6.4)
1868/11/27	20:39	45.7 - 26.6	VII	i	(6.1)
1880/12/25	14:30	45.7 - 26.6	VII	i	(6.1)
1894/3/4	6:35	45.7 - 26.6	VII	i	(6.1)
1894/8/31	12:20	45.7 - 26.6	VII	i	(6.1)
1908/10/6	21:39	45.5 - 26.5	VII+	150	(6.4)
1912/5/25	18:01	45.7 - 27.2	VII	80	(6.1)
1934/3/29	20:06	45.8 - 26.5	VII	90	(6.1)
1940/10/22	6:37	45.8 - 26.4	VII	122	(6.1)
1940/11/10	1:39	45.8 - 26.7	IX	133	7.4 (m_b6.9)
1945/9/7	15:48	45.9 - 26.5	VII+	75	6.5
1945/12/9	6:08	45.7 - 26.8	VII	80	6
1977/3/4	19:22	45.3 - 26.3	VIII	109	7.2 (m_b6.8)
1986/8/30	21:28	45.6 - 26.3	VII+	143	6.9 (m_b6.6)
1990/5/30	10:40	45.8 - 26.5	VII+	89	6.7 (m_b6.5)
1990/5/31	2:15	45.8 - 26.5	VII	79	6.1

APPENDIX B:

ASSIGNMENT OF DAMAGE DEGREES TO MASONRY AND RC BUILDINGS

	Masonry	**Buildings**
	Load bearing walls:	*Non-load bearing walls:*
D 1	Fine cracks (< 3mm), plaster dislodged	Cracks < 10mm
D 2	Cracks between 3 and 10mm	Wider cracks spreading diagonally or dislodging of wall
D 3	Wider cracks spreading diagonally, dislodging of wall or partial corner failure	Partial collapse, usually at top of gable wall
D 4	Partial collapse (significant leaning of structural wall or loass of one or more corners)	Total collapse of one of more walls
D 5	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:*
D 1	No damage	Small boundary cracks
D 2	Cracks < 10mm, usually near joints	Severe cracking, usually diagonal
D 3	Severe cracking,some loss of concrete	Collapse or severe crashing of the infill panels
D 4	Buckling of column reinforcement, large loss of concrete	N/A
D 5	Complete or partial collapse of building	N/A

Plate 1: General view of the apartment building Colentina no. 81, Block 84/I. This is where the two Bucharest fatalities occurred due to the collapse of the plasterboard along the seismic gap, as a result of pounding between the two structural parts of the large complex.

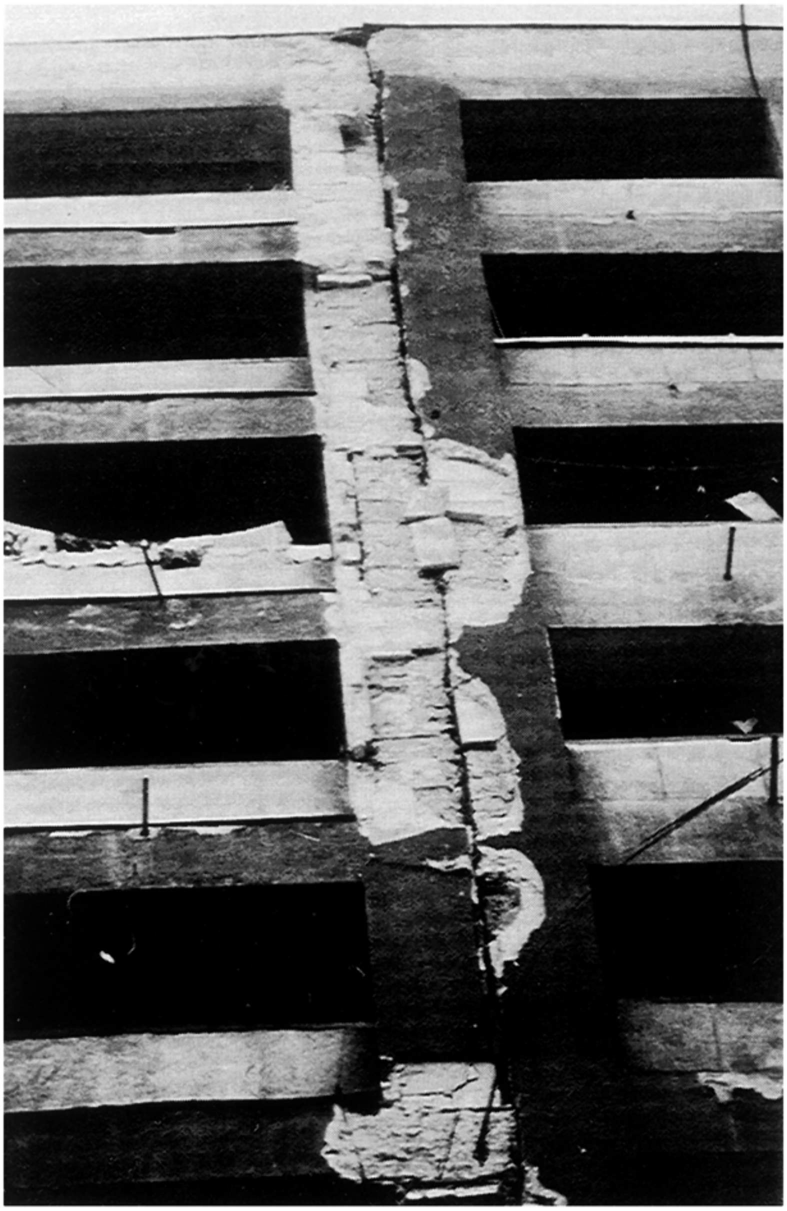

Plate 2: Close view of the collapsed plasterboard along the seismic gap line.

Plate 3: The unfortunate location of the entrance of the store, exactly along the seismic gap line contributed to the death of the two people, who were trying to run out of the building.

Plate 4: Colentina district, damage down the seismic gap line, due to pounding.

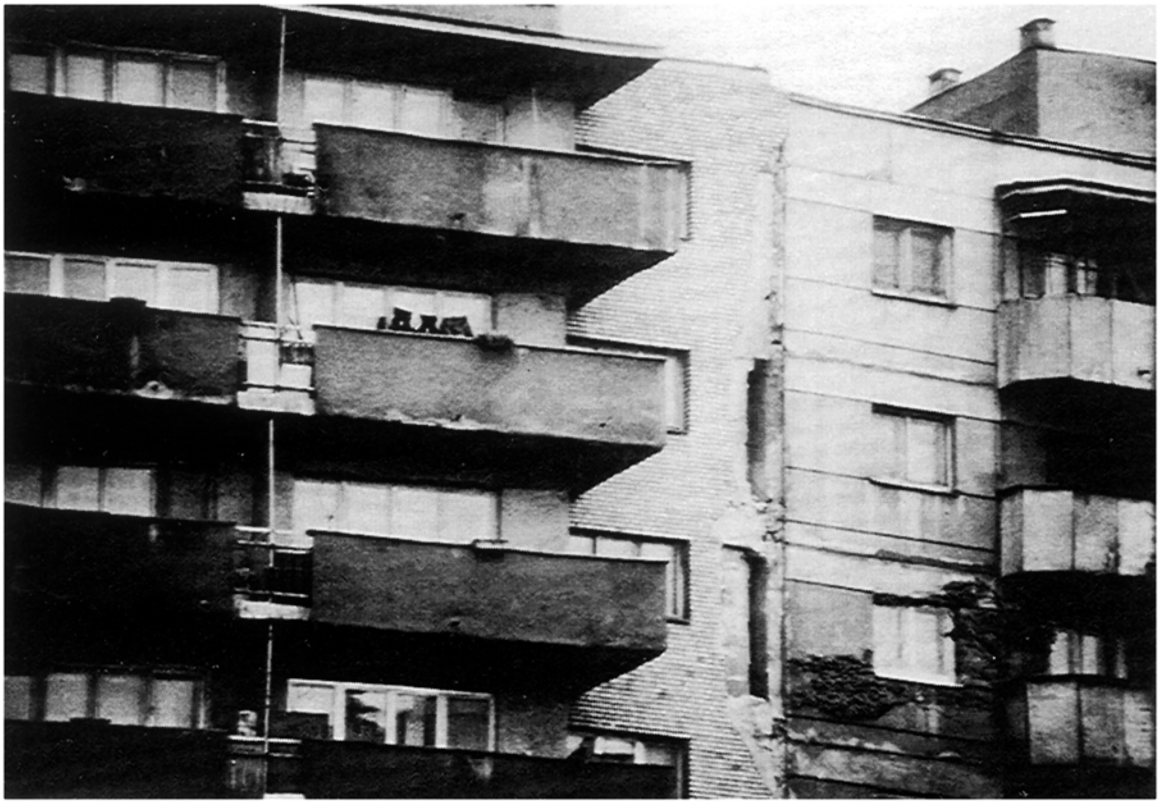

Plate 5: Damage to the expansion joint in another part of Bucharest.

Plate 6: Similar damage down the seismic gap line, in the town of Buzau, 78 km from the epicentre. Office of the Local Planning Authority, opposite to the strong motion observation station of Buzau.

Plate 7: Old compact clay house (Location: Focsani survey, 400 metres from instrument).

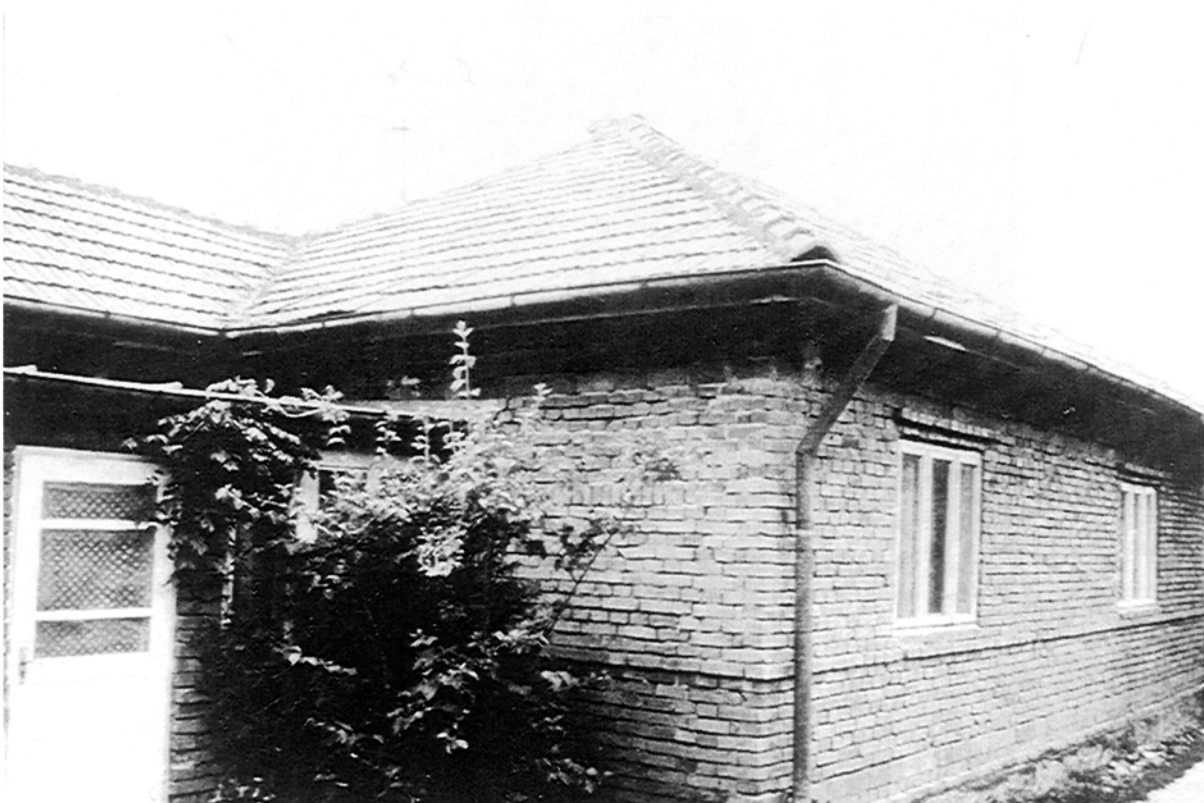

Plate 8: Typical single storey unreinforced brick masonry in cement-lime mortar.

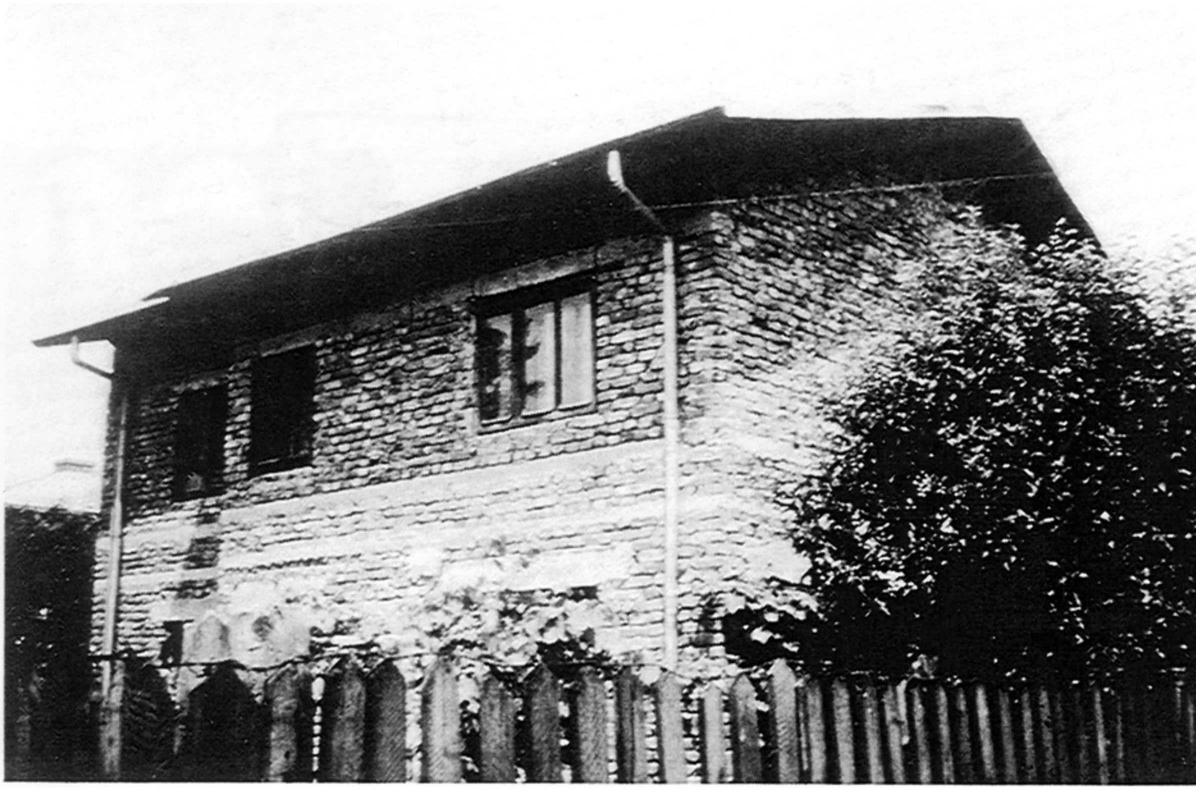

Plate 9: Two storey brick masonry house with RC slab, RC lintels and ring beam in the ground floor.

Plate 10: Typical 5 storey confined masonry apartment building, more commonly found in zones of intensity VI and VII.

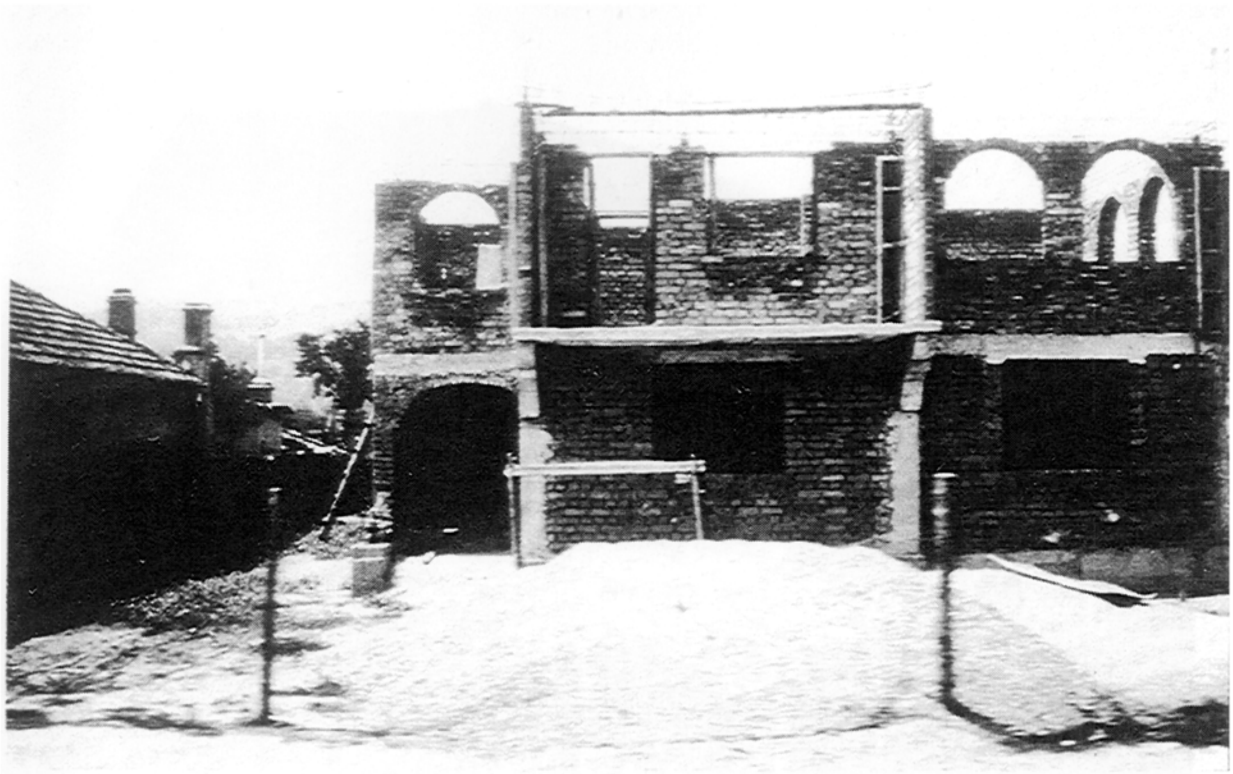

Plate 11: Two storey reinforced brick masonry house. Notice the reinforcing bars in the top floor and columns in the ground floor.

Plate 12: Monolithic RC frame building with RC wall panels instead of infill masonry (Location: Focsani).

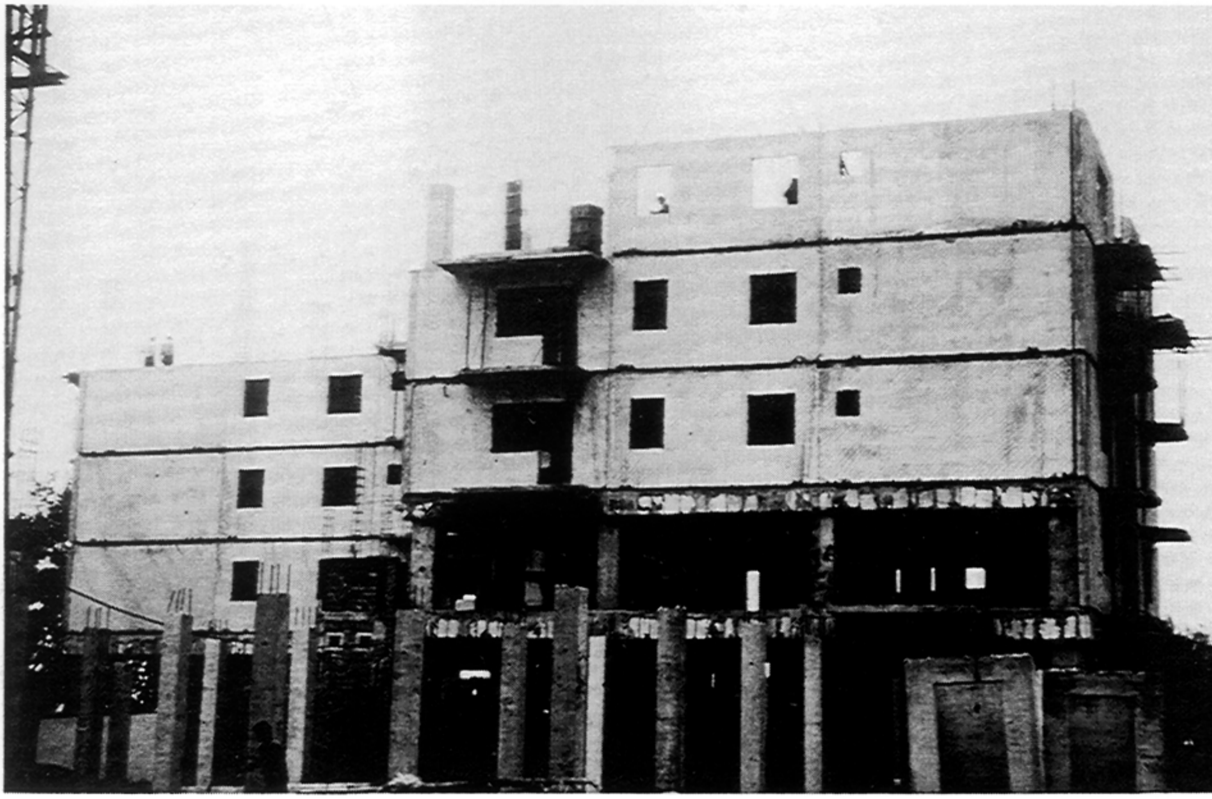

Plate 13: Similar RC frame building under construction (Location: Focsani)

Plate 14: Similar RC frame building in final stage, before plaster is applied (Location: Focsani)

Plate 15: Three types of frame infill in a single building due to material supply shortage (Location: Focsani).

Plate 16: Typical RC with shear walls large apartment block with several expansion joints and entrances (Location: Bucharest).

Plate 17: Detail of connections in an RC large panel building under construction.

Plate 18: RC large panel construction before plastering. Notice the poor details in the connections.

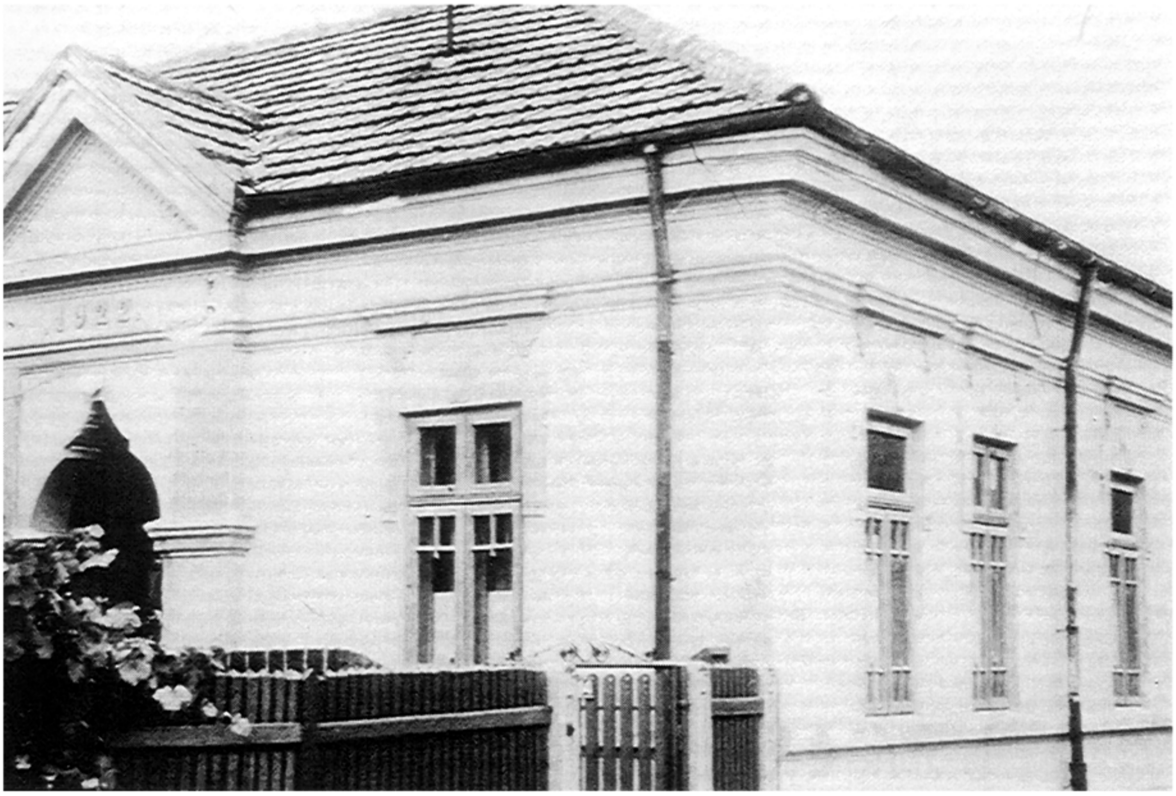

Plate 19: Typical single storey brick masonry building in the Buzau survey.

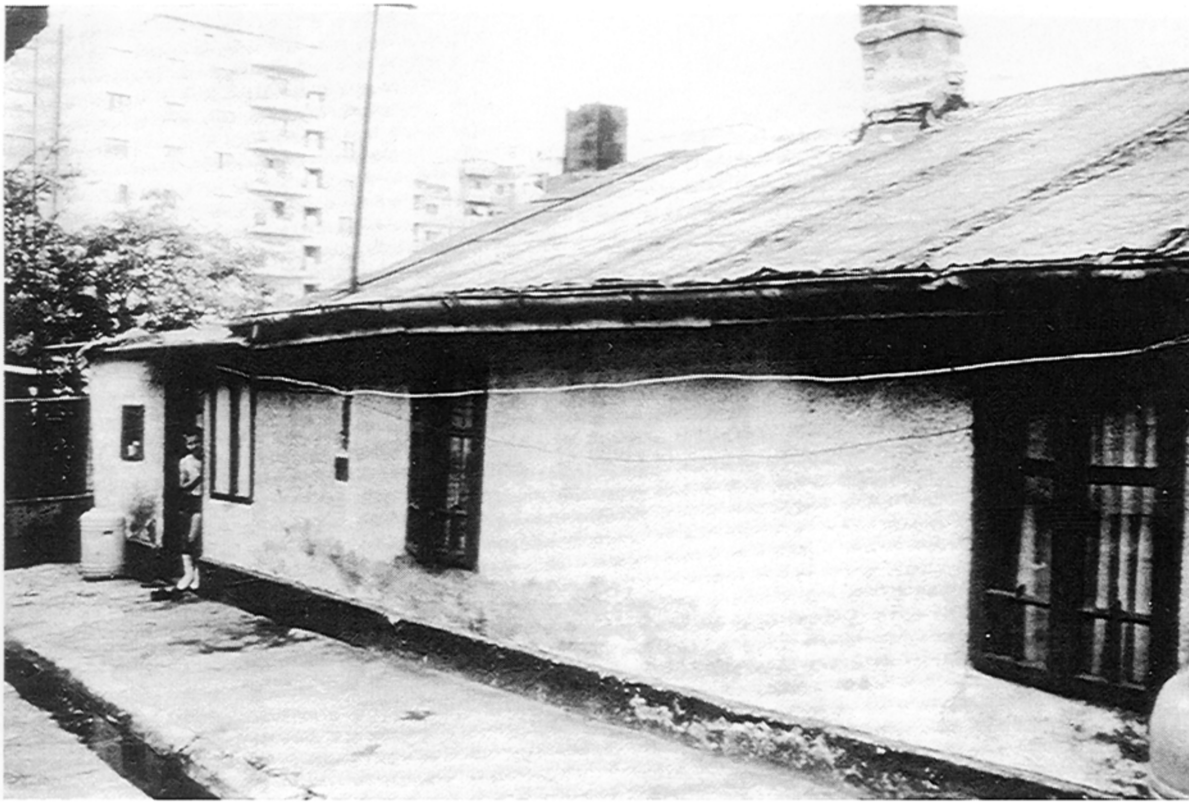

Plate 20: Typical adobe house in the Buzau survey

Plate 21: Evidence of repairs after the 1986 earthquake was widespread (Location: Buzau survey).

Plate 22: Old adobe house with wall separation (Location: Buzau survey).

Plate 23: The architectural layout of many of brick masonry buildings was elaborate (Location: Buzau survey).

Plate 24: Damage to the expansion joint (Location: Buzau survey).

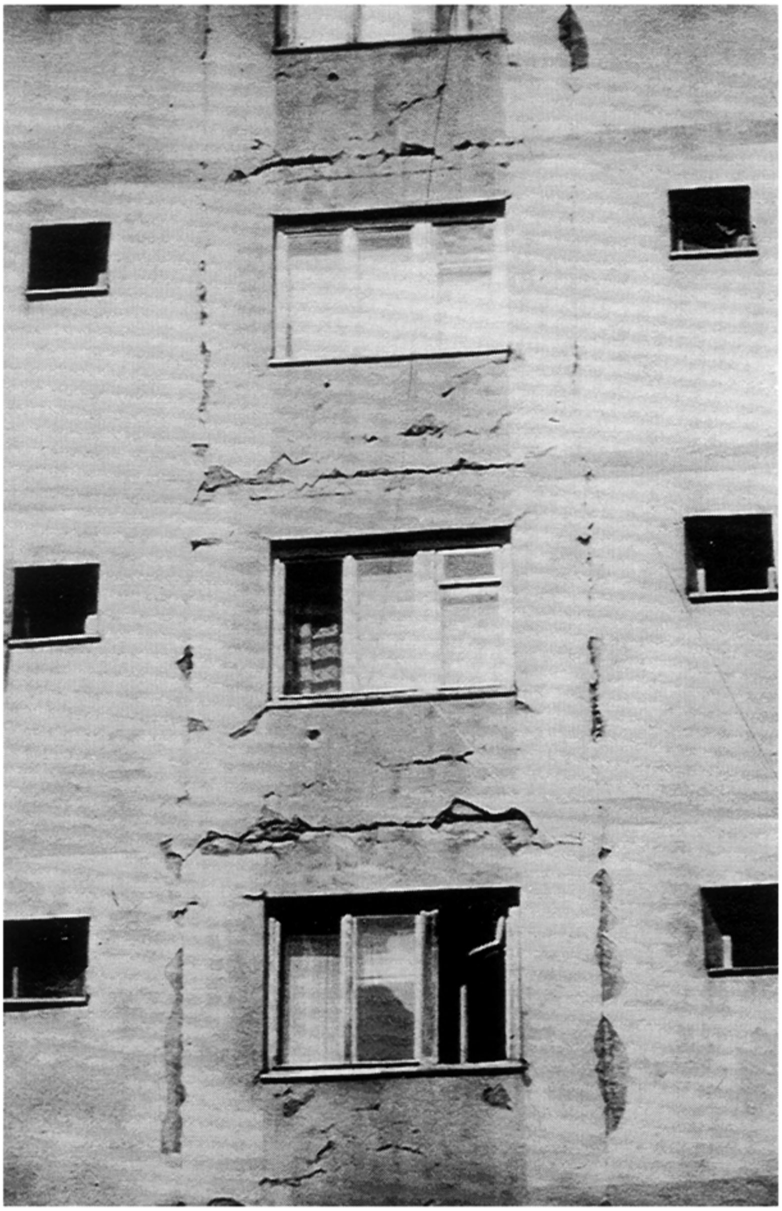

Plate 25: Widespread cracks around the panel perimeter (Location: Ploiesti survey)

Plate 26: Diagonal cracks in the infill masonry (Location: Buzau survey).

Plate 27: Ploiesti survey: the building where the strong motion instruments were installed. For a close view of the damage see Plate 28.

Plate 28: Close view of the damage due to pounding between the two structural parts of different shape and height.

Plate 29: Typical building in Mirinescu street survey (Bucharest).