OBSERVATIONS ON THE BEHAVIOR OF BUILDINGS IN THE ROMANIA EARTHQUAKE OF MARCH 4, 1977

NBS SPECIAL PUBLICATION 490

U.S. DEPARTMENT OF COMMERCE / National Bureau of Standards

NATIONAL BUREAU OF STANDARDS

The National Bureau of Standards[1] was established by an act of Congress March 3, 1901. The Bureau's overall goal is to strengthen and advance the Nation's science and technology and facilitate their effective application for public benefit. To this end, the Bureau conducts research and provides: (1) a basis for the Nation's physical measurement system, (2) scientific and technological services for industry and government, (3) a technical basis for equity in trade, and (4) technical services to promote public safety. The Bureau consists of the Institute for Basic Standards, the Institute for Materials Research, the Institute for Applied Technology, the Institute for Computer Sciences and Technology, the Office for Information Programs, and the Office of Experimental Technology Incentives Program.

THE INSTITUTE FOR BASIC STANDARDS provides the central basis within the United States of a complete and consistent system of physical measurement; coordinates that system with measurement systems of other nations; and furnishes essential services leading to accurate and uniform physical measurements throughout the Nation's scientific community, industry, and commerce. The Institute consists of the Office of Measurement Services, and the following center and divisions:

Applied Mathematics — Electricity — Mechanics — Heat — Optical Physics — Center for Radiation Research — Laboratory Astrophysics[2] — Cryogenics[2] — Electromagnetics[2] — Time and Frequency[2].

THE INSTITUTE FOR MATERIALS RESEARCH conducts materials research leading to improved methods of measurement, standards, and data on the properties of well-characterized materials needed by industry, commerce, educational institutions, and Government; provides advisory and research services to other Government agencies; and develops, produces, and distributes standard reference materials. The Institute consists of the Office of Standard Reference Materials, the Office of Air and Water Measurement, and the following divisions:

Analytical Chemistry — Polymers — Metallurgy — Inorganic Materials — Reactor Radiation — Physical Chemistry.

THE INSTITUTE FOR APPLIED TECHNOLOGY provides technical services developing and promoting the use of available technology; cooperates with public and private organizations in developing technological standards, codes, and test methods; and provides technical advice services, and information to Government agencies and the public. The Institute consists of the following divisions and centers:

Standards Application and Analysis — Electronic Technology — Center for Consumer Product Technology: Product Systems Analysis; Product Engineering — Center for Building Technology: Structures, Materials, and Safety; Building Environment; Technical Evaluation and Application — Center for Fire Research: Fire Science; Fire Safety Engineering.

THE INSTITUTE FOR COMPUTER SCIENCES AND TECHNOLOGY conducts research and provides technical services designed to aid Government agencies in improving cost effectiveness in the conduct of their programs through the selection, acquisition, and effective utilization of automatic data processing equipment; and serves as the principal focus wthin the executive branch for the development of Federal standards for automatic data processing equipment, techniques, and computer languages. The Institute consist of the following divisions:

Computer Services — Systems and Software — Computer Systems Engineering — Information Technology.

THE OFFICE OF EXPERIMENTAL TECHNOLOGY INCENTIVES PROGRAM seeks to affect public policy and process to facilitate technological change in the private sector by examining and experimenting with Government policies and practices in order to identify and remove Government-related barriers and to correct inherent market imperfections that impede the innovation process.

THE OFFICE FOR INFORMATION PROGRAMS promotes optimum dissemination and accessibility of scientific information generated within NBS; promotes the development of the National Standard Reference Data System and a system of information analysis centers dealing with the broader aspects of the National Measurement System; provides appropriate services to ensure that the NBS staff has optimum accessibility to the scientific information of the world. The Office consists of the following organizational units:

Office of Standard Reference Data — Office of Information Activities — Office of Technical Publications — Library — Office of International Standards — Office of International Relations.

---

[1] Headquarters and Laboratories at Gaithersburg, Maryland, unless otherwise noted; mailing address Washington, D.C. 20234.
[2] Located at Boulder, Colorado 80302.

Observations on the Behavior of Buildings in the Romania Earthquake of March 4, 1977

George Fattal, Emil Simiu, and
Charles Culver

Center for Building Technology
Institute for Applied Technology
National Bureau of Standards
Washington, D.C. 20234

Sponsored by:

Agency for International Development
Department of State
Washington, D.C. 20523

U.S. DEPARTMENT OF COMMERCE, Juanita M. Kreps, Secretary

Dr. Sidney Harman, Under Secretary
Jordan J. Baruch, Assistant Secretary for Science and Technology

NATIONAL BUREAU OF STANDARDS, Ernest Ambler, Acting Director

Issued September 1977

Library of Congress Catalog Card Number: 77-600035

National Bureau of Standards Special Publication 490
Nat. Bur. Stand. (U.S.), Spec. Publ. 490, 168 pages (Sept. 1977)
CODEN: XNBSAV

U.S. GOVERNMENT PRINTING OFFICE
WASHINGTON: 1977

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402
(Order by SD Catalog No. C13.10:490). Stock No. 003-003-01841-4. Price $3.
(Add 25 percent additional for other than U.S. mailing.)

PREFACE

Following the destructive earthquake of March 4, 1977, the Government of Romania requested U.S. technical assistance in connection with determining the safety of damaged buildings, major dam sites and hydroelectric stations. The Office of Foreign Disaster Assistance, Agency for International Development (AID) dispatched a team of engineers, geologists and seismologists to Romania on March 12, 1977 in response to this request.

The team included representatives from the National Bureau of Standards, (NBS), U.S. Geological Survey (USGS), Bureau of Reclamation and Corps of Engineers. They arrived in Romania on March 14, 1977. The NBS and USGS members spent six days in-country and the Bureau of Reclamation and Corps members, eleven days.

During their visit, team members worked directly with Romanian scientists and engineers in assessing the damage caused by the earthquake. They also developed recommendations for technical assistance related to repair and strengthening of damaged structures, strong motion instrumentation, and exchange of scientific and technical data relating to earthquakes for inclusion as part of a U.S. government assistance program for Romania.

This report presents observations on the behavior of buildings by team members from the National Bureau of Standards. The purpose is to document the performance of buildings designed to resist earthquakes as well as those which had not been so designed. This information should be of interest to practicing professionals and researchers. Since this is the first time AID has sent a team of this particular type into a disaster area to provide on-site technical assistance, the report can also serve to illustrate the useful technical data which can be obtained from similar missions in the future.

TABLE OF CONTENTS

ABSTRACT

Observations are presented of the damage to buildings resulting from the earthquake of March 4, 1977 in Romania. The report was prepared by engineers from the National Bureau of Standards who participated as members of the U.S. government team dispatched to Romania under the auspices of the Office of Foreign Disaster Assistance, Agency for International Development. A summary of the team's activities is included. Background data on the seismic history of Romania, the characteristics of the earthquake and descriptions of damage to specific buildings are also included. The types of building construction and the history of the development of seismic design requirements for buildings in Romania are discussed. Recommendations are presented for needed building research based on the observations.

Key words: Buildings; building codes; earthquakes; natural disasters; Romania; structural engineering.

1. INTRODUCTION

On Friday, March 4, 1977 at 9:21 P.M. local time a destructive earthquake of magnitude 7.2 (modified Richter scale) occurred in the Vrancea region west of Focsani in the southeast corner of the Romanian Carpathian Mountains approximately 150km northeast of the capital city of Bucharest (Figure 1.1). The earthquake occurred at a depth of approximately 110km and was felt over a large area. There was considerable damage in Bucharest (pop. approximately 2 million) and the town of Craiova (pop. 230,000), Zimnicea and Alexandria. Over 30 buildings, among which were many five and ten story apartment structures, collapsed in Bucharest. Statistics on casualties and property damage compiled as of the end of April [12][1] indicate that the earthquake:

- killed 1,578 people including 1,424 in Bucharest and 154 in the rest of the country
- injured 11,221 including 7,598 in Bucharest and 3,723 in the rest of the country
- destroyed or seriously damaged 33,000 housing units in high-rise apartment flats and conventional type dwellings (35,000 families, more than 200,000 persons homeless)
- caused lesser damage to 182,000 other dwellings
- destroyed 374 kindergartens, nurseries, primary and secondary schools and badly damaged 1,992 others
- destroyed six university buildings and damaged 60 others
- destroyed one orphanage and damaged 15 others

---

[1]Figures in brackets indicate literature references at the end of the report

- destroyed 11 hospitals and damaged 228 other hospitals and 220 polyclinics (health care centers)
- destroyed or damaged almost 400 cultural institutions such as theaters and museums
- damaged 763 factories
- directly affected over 200,000 people

This report summarizes the field investigations conducted by the National Bureau of Standards members of the U.S. technical team dispatched to Romania by the Agency for International Development. A summary of activities for the entire team and a list of individuals contacted while in Romania are included in the appendices.

The NBS investigations were confined to Bucharest and the city of Craiova. Their purpose was to assess the performance of buildings and the types of damage which occurred. Background data on the seismic history of Romania, the types of building construction and comparisons between U.S. and Romanian seismic design practice are also included in this report.

1.1 Earthquake Characteristics

The earthquake occurred on Friday evening March 4, 1977 at 9:21 P.M. local time (12:21:54.3 GMT). The location of the epicenter was 45.84N, 26.73E about 150km northeast of Bucharest (Figure 1.1). The focal depth was about 110km. About 250 aftershocks had been recorded in a week to ten days following the main shock. Most had a magnitude below 4; the largest was magnitude 4.5. Information from Dr. Karl Fuchs, a German seismologist from Karlsruhe, indicated the epicenters for these aftershocks were moving southward and occurring at depths of about 50km based on data obtained from his instruments installed in the epicentral region.

One strong motion record was obtained from an instrument installed at the Building Research Institute (INCERC) in Bucharest. The Building Research Institute is located in the eastern part of the city (Boulevard Pantelimon - Figure 2.2). The Japanese SMAC-B accelerograph (natural frequency-10 Hz, damping-100% critical) was located in the basement of a one-story reinforced concrete building. The instrument and installation were inspected by Chris Rojahn from the U.S. Geological Survey who indicated it had been properly installed and maintained. A copy of the record is shown in Figure 1.2. A peak acceleration of 0.2g in the north-south direction occurred about 18 seconds into the record. The peak accelerations in the east-west and vertical directions were 0.16g and 0.12g, respectively. The record, unlike most obtained from other destructive earthquakes, is characterized by a single strong pulse with a period of about 1.4 seconds. The pulse was recorded in the north-south and east-west components but not the vertical component of motion. In view of the relatively long period of the pulse, one would expect that this earthquake motion would be more severe for flexible, low frequency structures than for stiffer, high frequency ones.

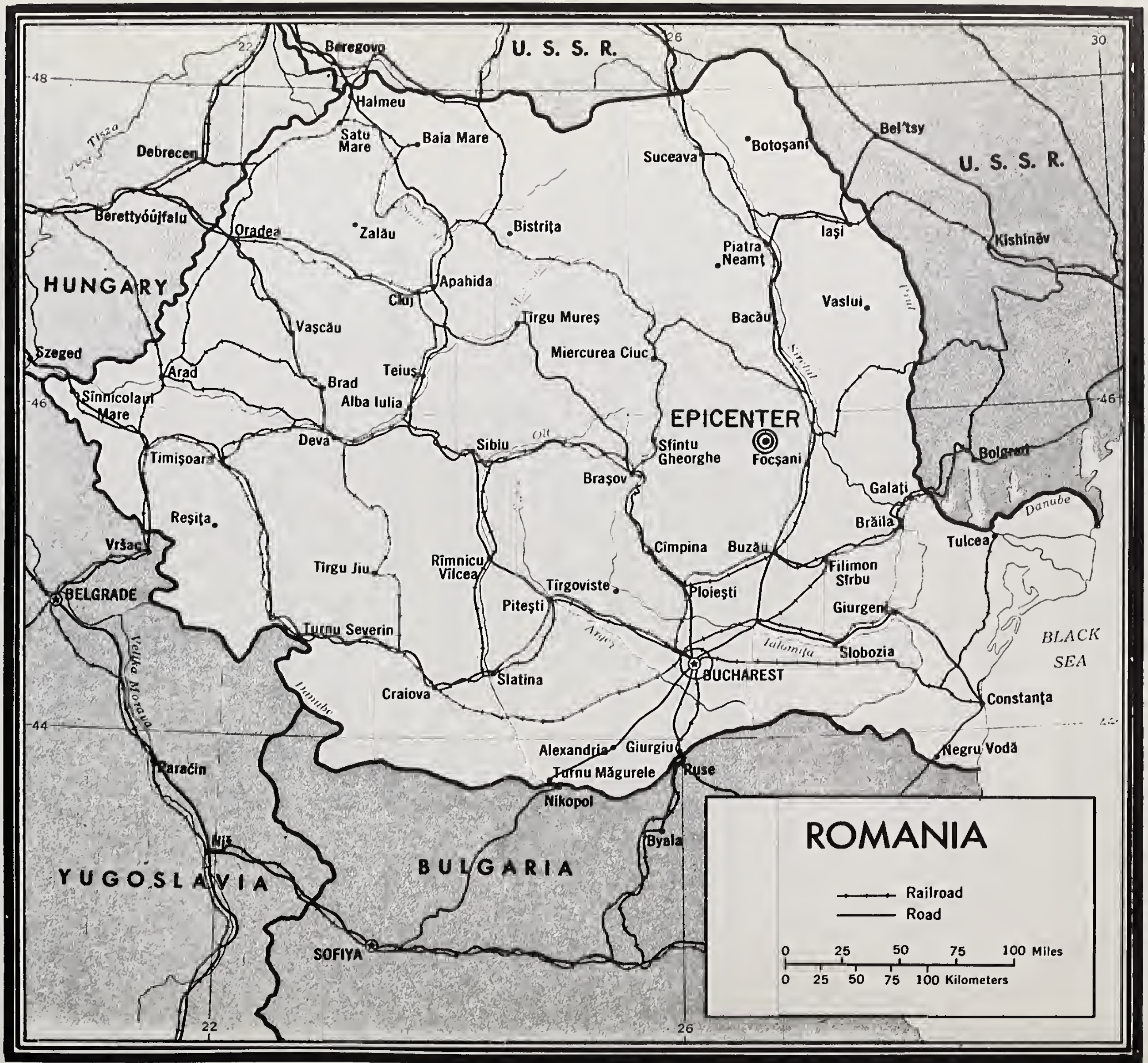


Figure 1.1. Location of Earthquake of March 4, 1977

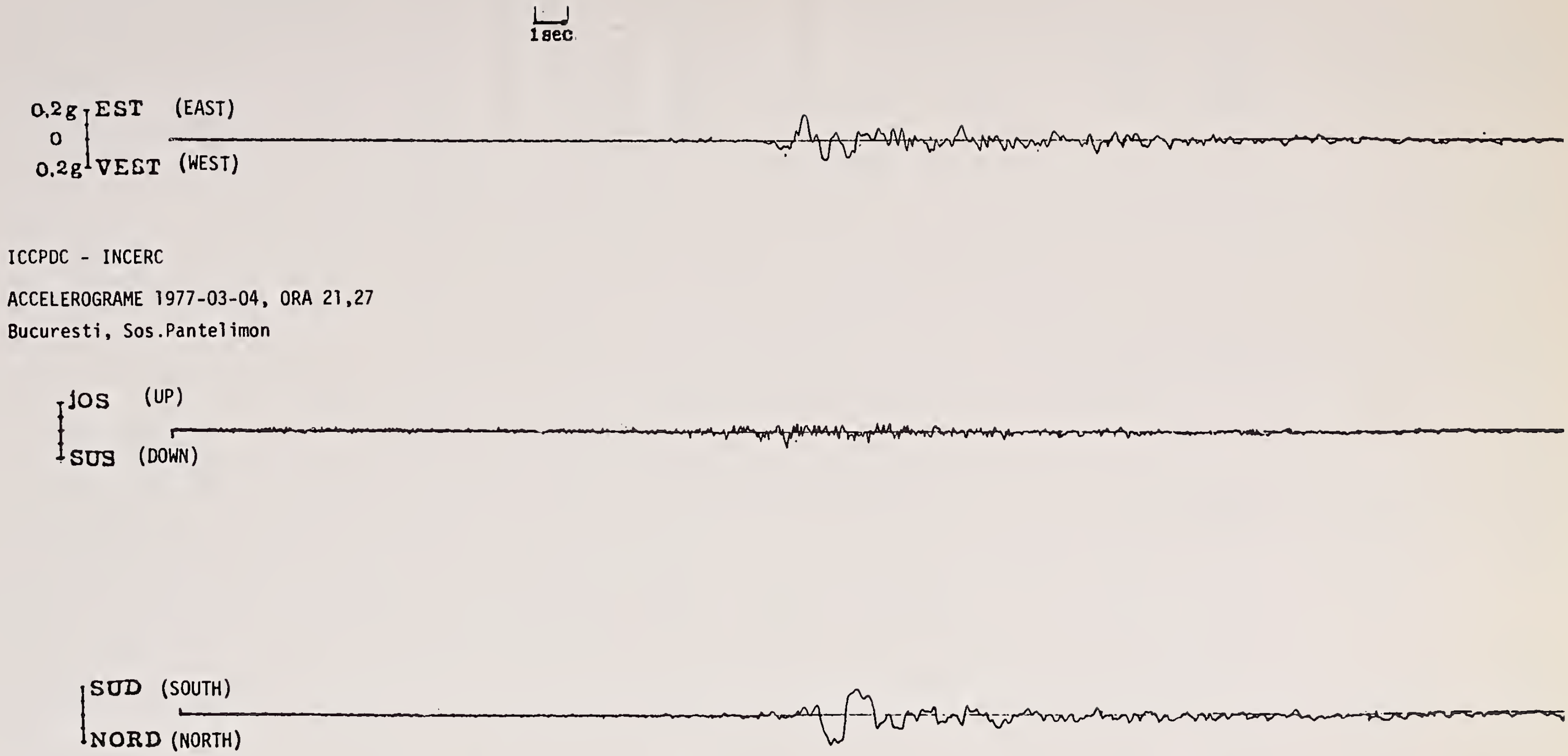


Figure 1.2. Copy of Portion of Strong Accelerograph Record - Building Research Institute, Bucharest

2. BACKGROUND

2.1 Seismic History

The earthquake of March 4 occurred in a region characterized by a long history of damaging earthquakes dating from Roman times. Among the strongest earthquakes mentioned in historical records, Ref. 1 lists the 1445 earthquake, the shock of which was felt as far as Moscow; the 1472 earthquake which destroyed the Neamtu (Moldavia) Monastery Church; the 1683 earthquake which caused the collapse of the Suceava (Moldavia) fortress towers; the 1738 earthquake which damaged the Palace in Bucharest; the 1802 earthquake which caused the collapse of the Coltea tower in Bucharest (located in the present center of the town). Ref. 3 mentions the occurrence in 1832 of what has been referred to as "the great earthquake". Data obtained from the Environmental Data Service of the National Oceanic and Atmospheric Administration indicate that 172 recorded earthquakes occurred in Romania between 1908 and February 1976 (Appendix C). Over 70 percent of these occurred in the area between 45-46 degrees North latitude and 26-27 degrees East longitude in the Vrancea mountain region where the March 4 earthquake occurred. In contrast to most damaging earthquakes in the United States which occur at shallow depths, i.e., less than about 30km (18 miles), most damaging Romanian earthquakes occur at much greater depths, i.e., 100-150km (60-90 miles). The data in Appendix C [2] illustrate the latter trend.

Observations of the historical record by the U.S. Geological Survey suggested the possibility that large deep Romanian earthquakes may occur in pairs. This tendency was displayed in 1912, 1940 and 1945 when large earthquakes occurred with few if any aftershocks as indicated in the following table:

YEAR	DATE	MAG	DEPTH	COMMENTS
1908	Oct 6	6.8	150	No significant 1/ aftershocks
1912	May 25	6.0	100	One or two significant aftershocks
1912	May 25	6.3	100	One or two significant aftershocks
1934	Mar 29	6.3	150	No significant aftershocks
1940	Oct 22	6.5	150	One significant aftershock (Nov 8)
1940	Nov 10	7.4	150	Many significant aftershocks
1945	Sept 7	6.5	100	One significant aftershock (Sept 14)
1945	Dec 9	6.0	100	No significant aftershocks
1977	Mar 4	7.2	100	No significant aftershocks

1/ Magnitude greater than 4.5

An earthquake of magnitude 6.5 occurred on October 22, 1940. No significant aftershocks followed until one of magnitude 5.5 on November 8, followed by the severe earthquake of magnitude 7.4 on November 10. Similarly in 1945, the magnitude 6.5 earthquake of September 7 was followed by only one significant aftershock until the magnitude 6.0 earthquake on December 9. Earthquake activity prior to the shock of March 4 included a magnitude 5.5 earthquake on October 1, 1976. No earthquakes were recorded in February 1977. Based on only three cases, however, it is difficult to conclude there is a trend for large Romanian earthquakes to occur in pairs.

The November 10, 1940 earthquake occurred at a depth of about 130km in the same region as the March 4 shock. The felt area for the two shocks was comparable although in 1940 there was less damage in Bucharest and no damage in Craiova, a city approximately 175km west of Bucharest. However, in 1940, there was considerable damage in the epicentral area north of Bucharest. There were 30 recorded aftershocks with the largest having a magnitude of 5.5.

Damage to buildings in the 1940 earthquake is summarized in a report by Niculescu [3] from which the following information is excerpted. The earthquake of November 10 was reported as intensity 9 (Mercalli-Cancani-Sieber scale) in Bucharest and strong ground shaking lasted 42 seconds. No instrumental readings were obtained. The shock of October 22 was of shorter duration and of intensity 7 in Bucharest. The primary direction of motion in Bucharest was east-west.

Many roofs, chimneys and walls of buildings were damaged. Apparently very few roofs collapsed but many were displaced laterally as much as 20cm. No observations of torsional motions of buildings were reported. Gable walls supported on columns displaced up to 18cm. Vertical and horizontal cracks and evidence of torsion were observed in chimneys. More chimney damage was observed on buildings with high pitched roofs. This may have been due to the fact that chimneys on buildings of this type were higher and consequently more flexible (with longer natural periods). There was some damage to buildings as a result of foundation settlement. Differences in damage between adjacent buildings was attributed to the variation of soil conditions throughout the city.

The most significant building collapse in the 1940 earthquake was the Carlton building, a 47m high, 11 story reinforced concrete frame structure, in which 130 lives were lost. A fire in the building followed the collapse. A commission of experts which investigated the failure found design and construction errors in that many of the beams required for wind bracing were missing and less concrete and steel were used than required by the design specifications. The commission felt that similar design and construction errors were present in almost all apartment buildings built in the same manner in Bucharest.

2.2 Soil Conditions

Bucharest is located on a plane inclined slightly in a north-south direction as shown in Figure 2.1b. Two small rivers, the Dimbovita in the southern portion and the Colentina in the northern, traverse the city from northwest to southeast, the latter interconnecting a series of lakes within the city as shown in Figure 2.1a. The Dimbovita river bed consists of alluvial deposits of fine sands with pockets of organic matter. Soil immediately adjacent to the river is loess and soft clay with an average depth of 5m with local deposits as deep as 9m. The soil profile in Figure 2.1b shows the general area is underlain by layers of gravel, clay, and sand. The average thickness of the sand layer underlying the whole area in Figure 2.1b is about 15m and the silty clayey sand layer about 60m. Allowable bearing pressures for design vary from 100 - 150 kN/m^2 in the vicinity of the Dimbovita River to 200 - 300 kN/m^2 in higher regions of the plane. The level of the ground water table varies from about 2m in the vicinity of the Dimbovita River to 10 - 15m (Figure 2.1a).

2.3 Building Systems and Construction Practices

2.3.1 Building Profile of Bucharest

Both in population and area, Bucharest is the largest city in Romania, with a metropolitan area of about 400km^2. The Dimbovita and Colentina rivers which run through the city on parallel courses are shown in Figure 2.2. The principal thoroughfares converge toward the center of the city, where they frequently merge and intersect other streets in oblique patterns, creating a gridwork of polygonal blocks having the irregular configurations illustrated in Figures 2.2 and 2.3.

Most of the structures within a 3km radius of the center of Bucharest were built before 1940. Among these are a number of historic monuments and churches (Figures 2.4 and 2.5), massive educational, administrative and cultural facilities with ornate features reminiscent of earlier architectural styles (Figures 2.6 and 2.7), and small residential buildings (5 stories or less) consisting of unreinforced masonry load-bearing walls with, in most cases, concrete or wood floors. In general, however, the building profile of central Bucharest is dominated by multistory concrete frame structures built before 1940 (Figures 2.8 and 2.9), none of which were designed for earthquake loads. In the aftermath of the destructive earthquake in November 10, 1940, rudimentary earthquake design provisions, patterned after the Italian code, were introduced for use on a voluntary basis. Mandatory seismic design regulations went into effect in 1957, followed by later versions in 1963 and 1970.

It has been noted that during World War II (1939-1945), there was virtually no construction activity in Bucharest. When the city resumed its outward growth after the war, fundamental transformations began to occur in building construction. In multistory residential buildings, most notable was the trend towards greater utilization of precast structural and non-structural elements, and increased use of framing plans having rectangular, symmetric configurations (Figures 2.10 and 2.11). Currently, precast panel systems together with cast-in-place reinforced concrete shear wall systems constitute about 80 percent of new building construction in Bucharest. The other systems are mainly buildings of load-bearing masonry construction, complete moment-resisting concrete frames and buildings that fall under the category of special structures.

2.3.2 Reinforced Concrete Frame Buildings

Within a 30-year span prior to 1940, a large number of concrete frame structures were erected in Bucharest*. They are typically 8 to 12 stories high. The first story is generally higher than the rest and is almost void of walls to accommodate stores and other non-residential facilities. In the upper stories, masonry infill walls and partitions are used liberally to provide enclosure for apartment or office space, and to function as lateral bracing against wind action. As a result, the structure is characterized by laterally stiff upper stories resting on relatively flexible columns at the

---

*According to Beles and Ifrim [1], reinforced concrete as a load-resistant material for tall buildings was introduced in Bucharest after 1910.

ground level. Because they are commonly built adjacent to each other and more or less conform to the irregular patterns of the city blocks, the end and corner units tend to be particularly non-uniform in layout. A good illustration is provided by the framing plan of the eight-story building shown in Figure 2.12, which is reproduced from the paper by Niculescu [3]. The paper presents the author's observations of building damage caused by the earthquake in November 1940. It cites major shortcomings in building design and construction practices in the years prior to 1940 among factors tha contributed to the inadequate seismic performance of the reinforced concrete frames in general. In view of the similarities in the response of these buildings in both the November 1940 and March 1977 earthquakes, it is of interest to cite the following observations made in that article with regard to building practices prior to 1940:

> *There existed no building inspection at the time. Each owner was responsible for his building and could be prosecuted only in case of catastrophe or damage. Only certain prescriptions specified by the authorities had to be satisfied. The plans had to be drawn by an architect with a degree. Even though, in general, reinforced concrete design was based on German prescriptions, they could be altered according to the wishes of the architect or the user or even the contractor eager to build as inexpensively as possible.*
>
> *According to German prescriptions, sufficiently stiff buildings did not need to be analyzed for wind loads. Some stiffening guidelines were provided together with recommendations to the effect that in cases of doubt, the extent of stiffening should be agreed upon with the inspection authorities. In Bucharest, a reinforced concrete frame*

was assumed to be sufficiently stiff even in the absence of stiffening walls so that the structures were not analyzed for horizontal loads. In addition, the German specifications did not require analysis of concrete frames in accordance with established frame theory, even when the building was seven stories or higher. Consequently, not even the bending moment formulas given therein were used, nor was it considered necessary to vertically align the columns in different floors. Columns of height-to-thickness ratio greater than 15 were designed for axial compression only. Because it was common practice to divide the residences of different floors in various ways, the columns had to be so arranged that they could not be connected directly to the beams. In fact, there were columns that were aligned through all upper floors and rested only on a girder at the lower floor. On the other hand, the width of the beams was kept within 14cm so that they would be concealed in the rooms. It is easy to visualize what the stiffness of such a building will be with respect to horizontal loads.

The earthquake revealed many erroneous practices. For example, a column in one building had been destroyed at the base. It was found that ties were missing over a height of 60cm. Furthermore, the vertical bars were connected by simply overlapping the hooks with no additional overlap. It was noted here that structural drawings provided no information on the ties so that their relative positioning was left to the whim of the construction worker.

It also appeared that in certain cases vibrations had caused a pulverizing (grinding) effect on the concrete. In the case of a column that had been perforated by a layman in order to accommodate a heating pipe, the concrete was found to have only the resistance of lime mortar. The destruction of the concrete was probably caused by repetitive stress. In one case, concrete strength was found to be 65 kg/cm2 (6.4 MN/m2). In the case of the Carlton building, a court-appointed commission cited design and construction errors of the type found in almost all Bucharest apartment buildings, notably, inadequate wind bracing and deficiencies in the quality of materials used. As to the causes that may have contributed to the collapse, the commission noted the unfavorable position of the 47m high corner tower with respect to the direction of the earthquake motion, possible amplification of earthquake effects due to the proximity of the foundation to the water table, the location of the building at the end of a row of tall buildings, and the cantelevering of the first floor theater seats from the columns. According to the author, resonance may also have occurred. *

---

*In a 1965 paper [1] Beles and Ifrim attributed the collapse to the failure of a first floor concrete wall column and the imperfect transmission of the loads to the foundation because of a lack of continuity of columns.

2.3.3 Reinforced Concrete Shear Wall Buildings

Cast-in-place concrete shear wall systems commonly used in Bucharest conform to two basic configurations. The cellular layout shown in Figure 2.10 is normally used for buildings of up to 12 stories high (although occasionally it has been used for buildings of up to 18 stories high). It is, in fact, a mixed system combining cast-in-place transverse and longitudinal shear walls with cast-in-place transverse concrete frames in the intermediate bays at the ground floor level to provide relatively large uninterrupted spaces needed for store use. In the upper stories, the transverse beams, exterior walls and floor slabs are frequently precast. Symmetry is maintained in both transverse and longitudinal directions.

The box layout shown in Figure 2.11 also has double symmetry. It consists of an orthogonal array of intersecting shear walls dividing the space into square and rectangular modules for apartment use. The system has been used for buildings up to five stories high. The nonstructural fascia walls (not shown) and the floor slabs are precast. This is a total shear wall system in the sense that no use is made of either cast-in-place or precast concrete framing elements (beams, columns, frames) in the construction. Slip forming as well as conventional formwork are utilized in the erection of both systems.

Design specifications for concrete strength are based on Romanian standard 7cm cube tests. The concrete strength commonly used in shear wall buildings is about 20 MN/m^2 (grade B200) for cast-in-place concrete and 25 MN/m^2 for the precast slabs which are usually 130mm thick. The steel used for main reinforcement is usually grade PC52 having an ultimate strength of 510 MN/m^2.

2.3.4 Precast Reinforced Concrete Panel Structures

Since they were first introduced in 1962, industrialized building systems have outpaced all other forms of construction in Bucharest. Two well-refined principal schemes (with 3 or 4 variations each) have emerged for use in high-rise (9 stories or more) and mid-rise (5 stories) building construction. Buildings above 5 stories are required to have elevators. Consequently, construction of 6 to 8 story buildings is uncommon because of the economic factor.

The plan of a high-rise panel building with a doubly symmetric configuration is shown in Figure 2.13. Selected details of vertical and horizontal joints are displayed in Figures 2.14 and 2.15, respectively. Structurally, the system appears to be fairly substantial as evidenced by the following. The use of panels with castellated edges provides for longitudinal and transverse interlock between joint concrete and prefabricate. Mechanical coupling at the joints is effected by means of lapped reinforcement in the form of loops, and two longitudinal steel bars within the loops provide added bearing area against the concrete core. Additional longitudinal bars outside the core are used in vertical joints to supplement flexural tension reinforcement needed for flange action under lateral loads. The longitudinal bars in the vertical joints are lapped and welded together at horizontal junctions to achieve vertical continuity as indicated by the section details in Figure 2.14. It appears that these are the only tension ties in the "flanges" of the system since, according to the available details, the vertical panels are not mechanically coupled to the horizontal joints at their base. Note that all exterior panels are double walls with insulation in between. Materials specifications for the system call for B300 grade joint concrete (30 MN/m^2 standard cube strength) and PC52 grade steel for main reinforcement.

Figures 2.16 to 2.18 show specific details of a panel system used in the construction of midrise buildings. In this case, mechanical union in the joints is achieved by means of splice bars welded to the transverse reinforcement of adjacent panels. Longitudinal bars, used singly in vertical joints and in pairs in horizontal joints, provide added bearing area for the transfer of tension across the connections. The coupling of the floor panels is somewhat different (Figure 2.17a). The top bars are splice welded while the bottom bars are bent up 90 degrees and lapped. This particular scheme gives greater continuity to the floors at the supports than the lapped loop arrangement used in the high-rise building system. Another basic difference between the two systems is that the wall panels in the mid-rise building are mechanically coupled at their base (Figure 2.17b), so that all vertical bars are rendered continuous across the horizontal joints, as opposed to the case of the high-rise building in which only the longitudinal bars of vertical joints are coupled. On the basis of these differences alone, it appears that the lateral response of the low-rise building should be closer to that of a monolithic shear wall structure. The materials specifications for this system are the same as before (PC52 grade steel and B300 concrete). Concrete strength in the panels is about 25 MN/m^2. In Figure 2.16, alpha-numeric notation is used to designate special reinforcement. Prefix P stands for 3-mm wire mesh. The special lintel reinforcement designated by prefix C consists of two longitudinal bars (the size of which depends on the seismic zone) connected by 4-mm cross bars.

Prefix C is also used to designate the 4-mm peripheral bars (shown dotted). The amount of the main reinforcement, designated by numbers, are specified according to the seismic zone. For instance, the total weight of the steel used in the double-wall exterior panel shown in Figure 2.16a is 47.2kg and 57.4kg for seismic zones 6 and 7, respectively.

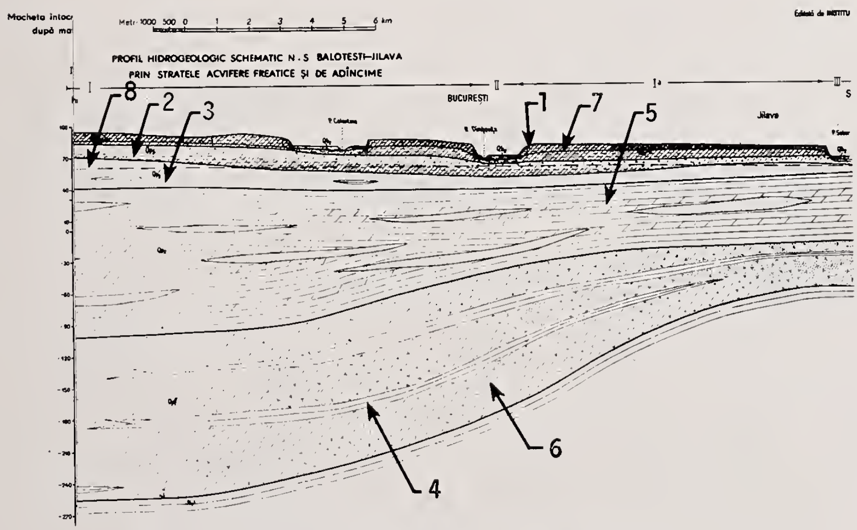
2.3.5 Other Building Systems

Brick or concrete block masonry bearing wall construction is used for buildings 5 stories or less in height. The lower stories have reinforced concrete columns (essentially serving the function of pilasters) at corners and intermediate locations along the walls, and are cast after the walls are laid. Reinforcement within the walls and positive connections between the walls and the columns provide the capacity for integral action in out-of-plane flexure (normal loads) and under lateral loads (membrane forces). This practice is also followed in the upper stories where reinforcement and ties become less substantial.

Moment-resistant reinforced concrete multi-story frames with non-structural masonry walls laid after the frame is cast are rarely used at present. This type of construction had been used mostly in the mid-to-late fifties for isolated instances (i.e., one of a kind) in downtown Bucharest, e.g., the structure on the site of the Carlton building destroyed by the 1940 earthquake. These buildings are usually 8 to 10 stories high.

A well known example of special buildings is the 25-story Intercontinental Hotel in downtown Bucharest (Figure 2.19). It is a modern reinforced concrete frame structure. In plan it looks like an equilateral triangle with truncated vertices and concave circular sides. Another example is the computer center for the Ministry of Transportations and Telecommunications which collapsed in the March 4, 1977 earthquake (Figure 3.46). This was a 3-story reinforced concrete building of flat slab construction in which the floors were supported by 9 columns arranged in a square pattern (this building is described in greater detail in Section 3.3).

(a) Phreatic contour map of Bucharest



1-Clayey sandy silt
2-Gravel
3-Sand
4-Clay
5-Silty clayey sand
6-Marl
7- Phreatic water level
8- Subterranean piezometric head

(b) Soil profile of Bucharest (Sect. I-I)

Figure 2.1. Hydrogeological map of Bucharest

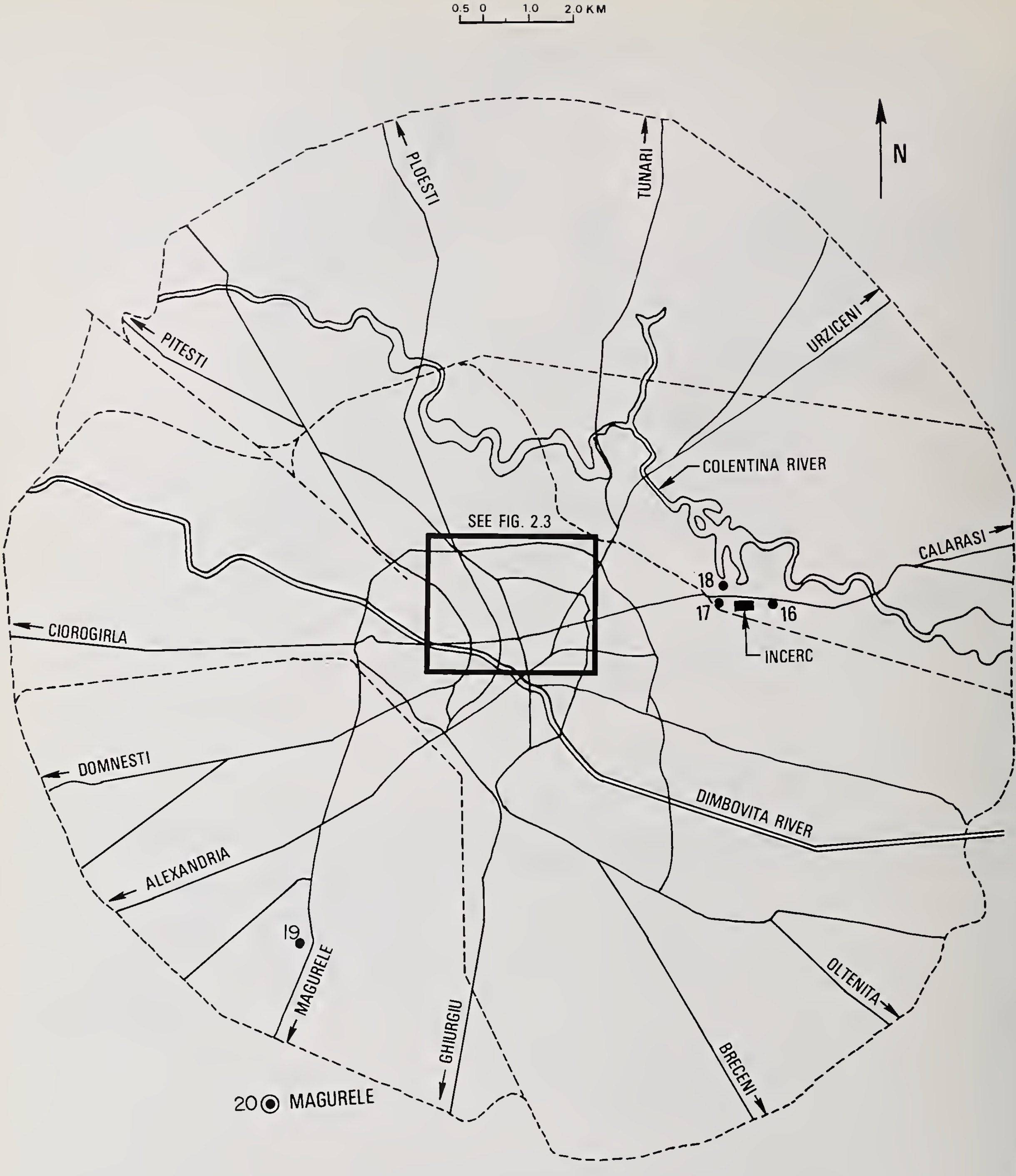


Figure 2.2. Map of metropolitan area of Bucharest.

Figure 2.3. Street map of downtown Bucharest. Arrow indicates location of Intercontinental Hotel.

Figure 2.4. Russian church with characteristic pear-shaped domes on Ghica Street.

Figure 2.5. Historic Stavropoleos Church near central post office.

Figure 2.6. Romanian Atheneum on Victoriei Avenue.

Figure 2.7. Central House of the Army on Victoriei Avenue.

Figure 2.8. View of downtown Buchar

Figure 2.9. General view of Bucharest
looking towards northwest.

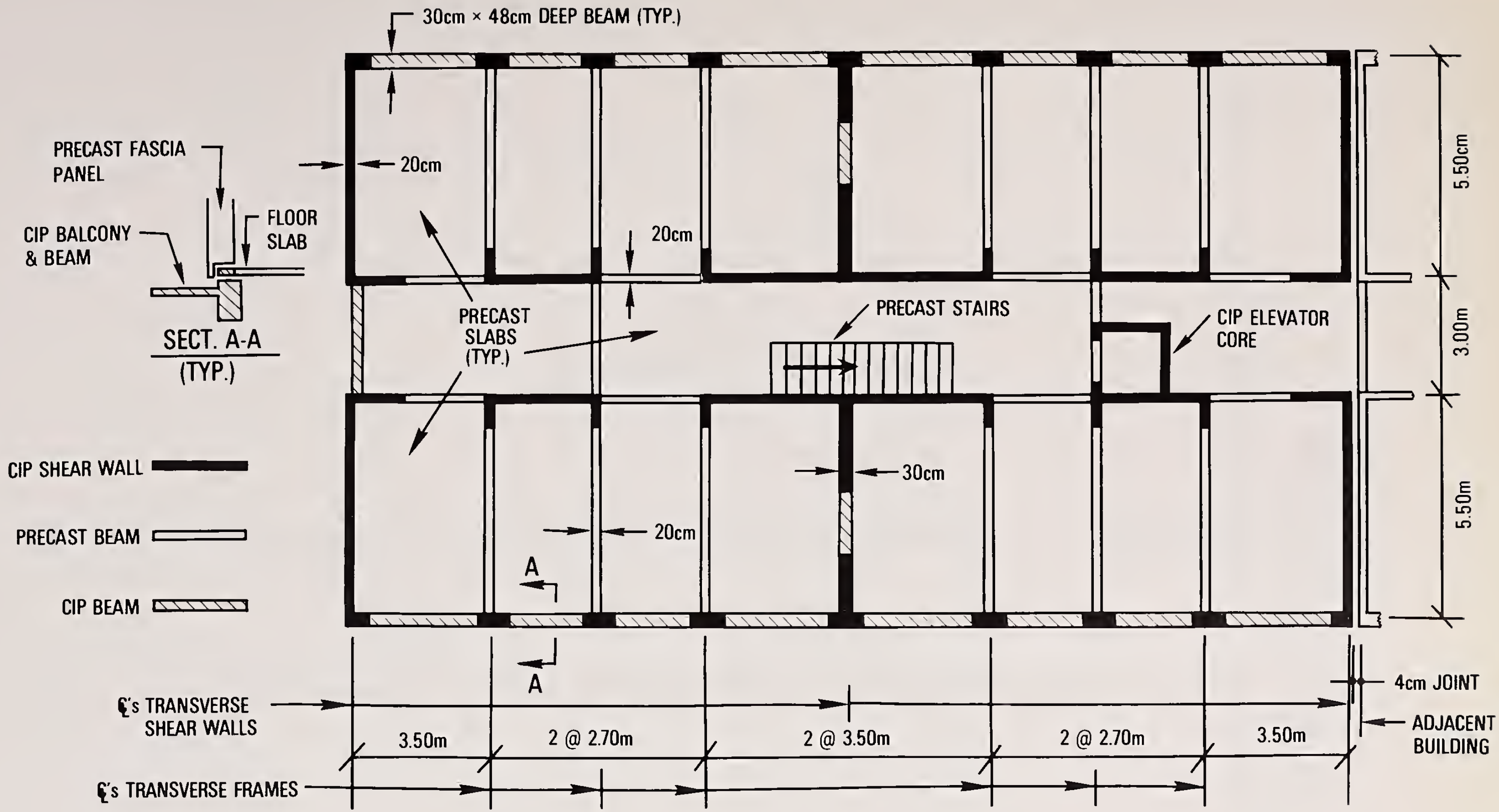


Figure 2.10. Typical plan of ten-story concrete shear wall and frame building (end unit).

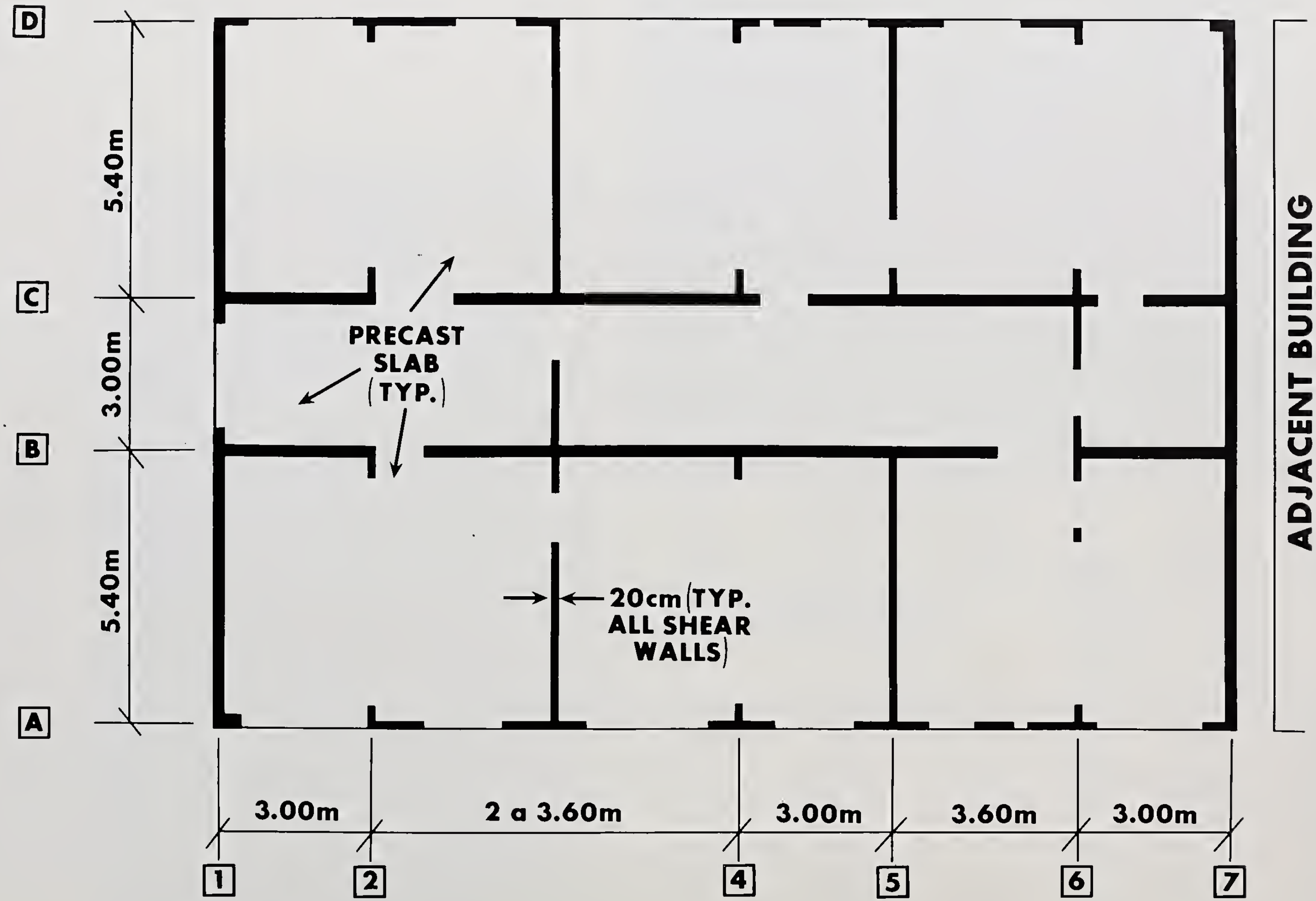


Figure 2.11. Typical plan of five-story cast-in-place concrete building with precast slabs (end unit).

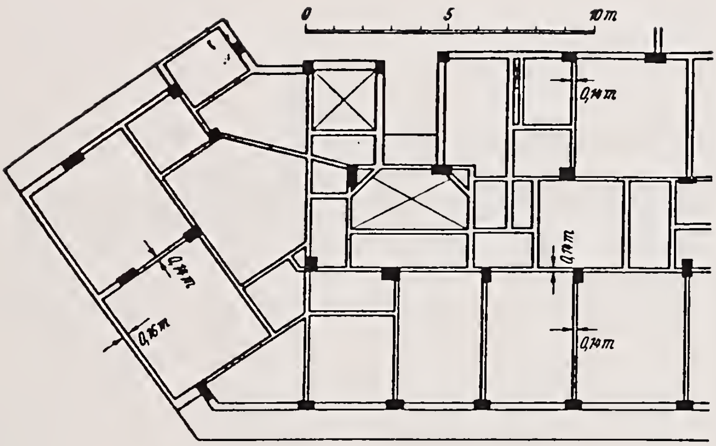


Figure 2.12. Plan of 8-story concrete frame structure in Bucharest built before 1940.

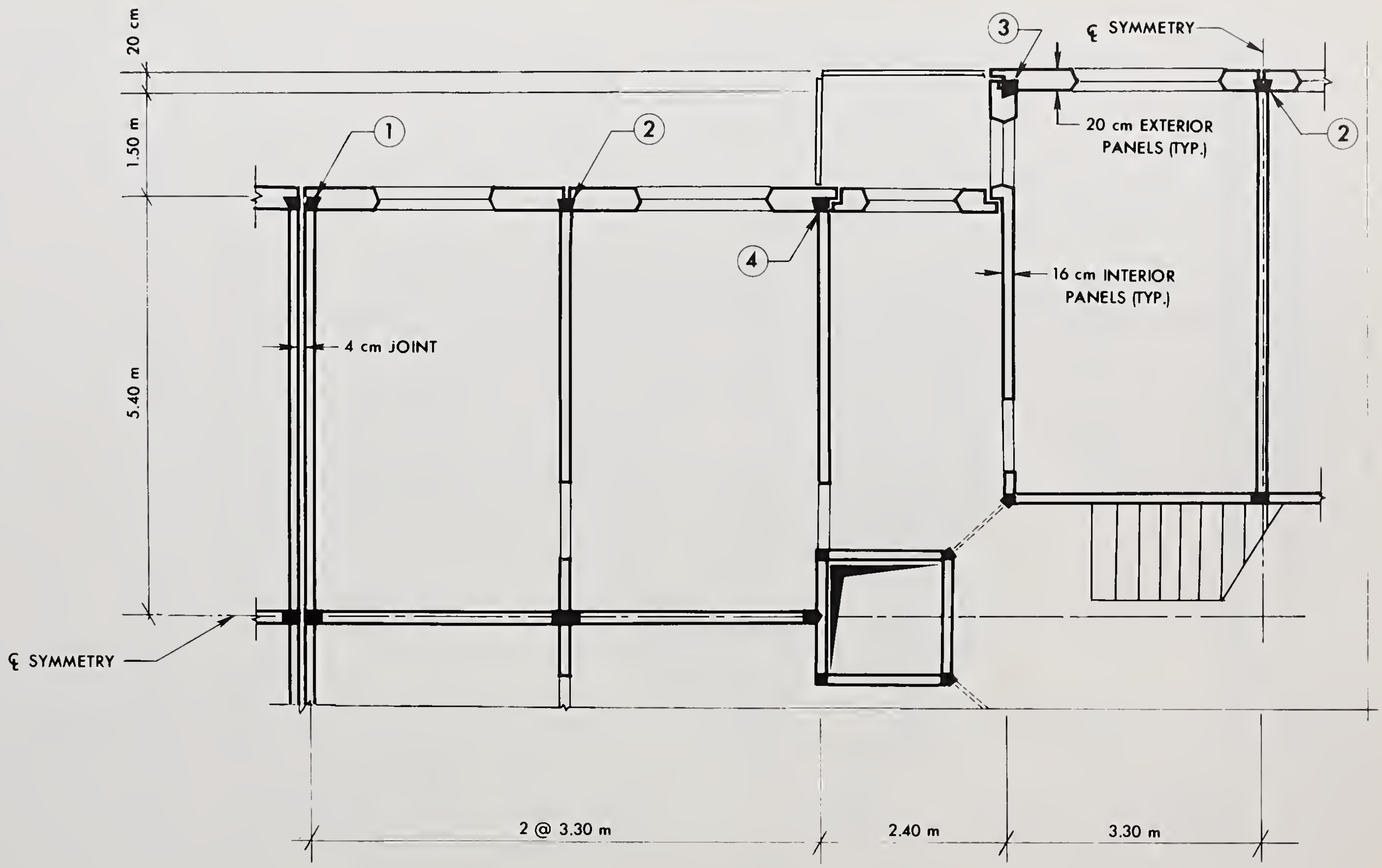


Figure 2.13. Plan of high-rise precast concrete panel building (interior unit).

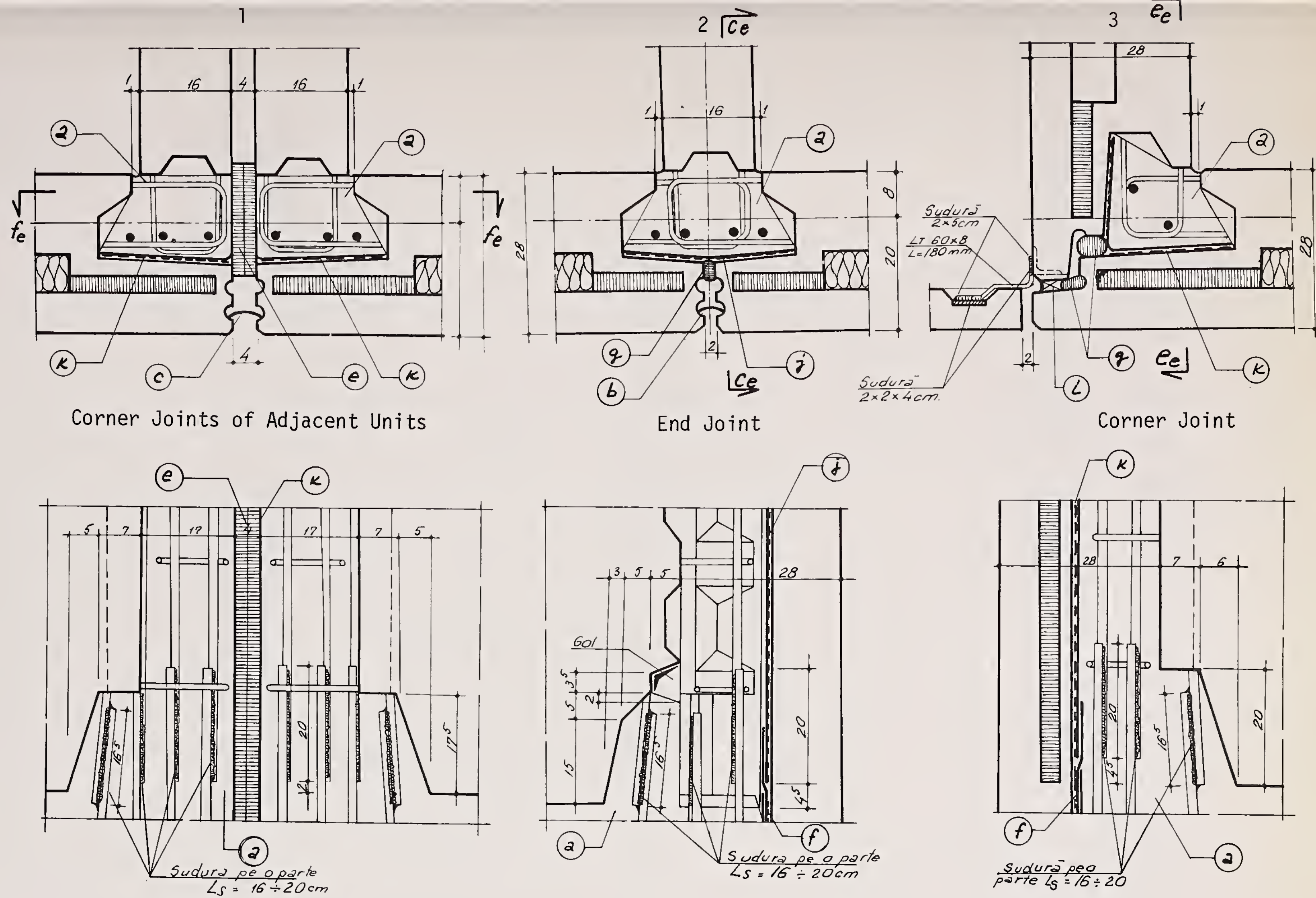


Figure 2.14. Vertical joint details of high-rise panel building system.

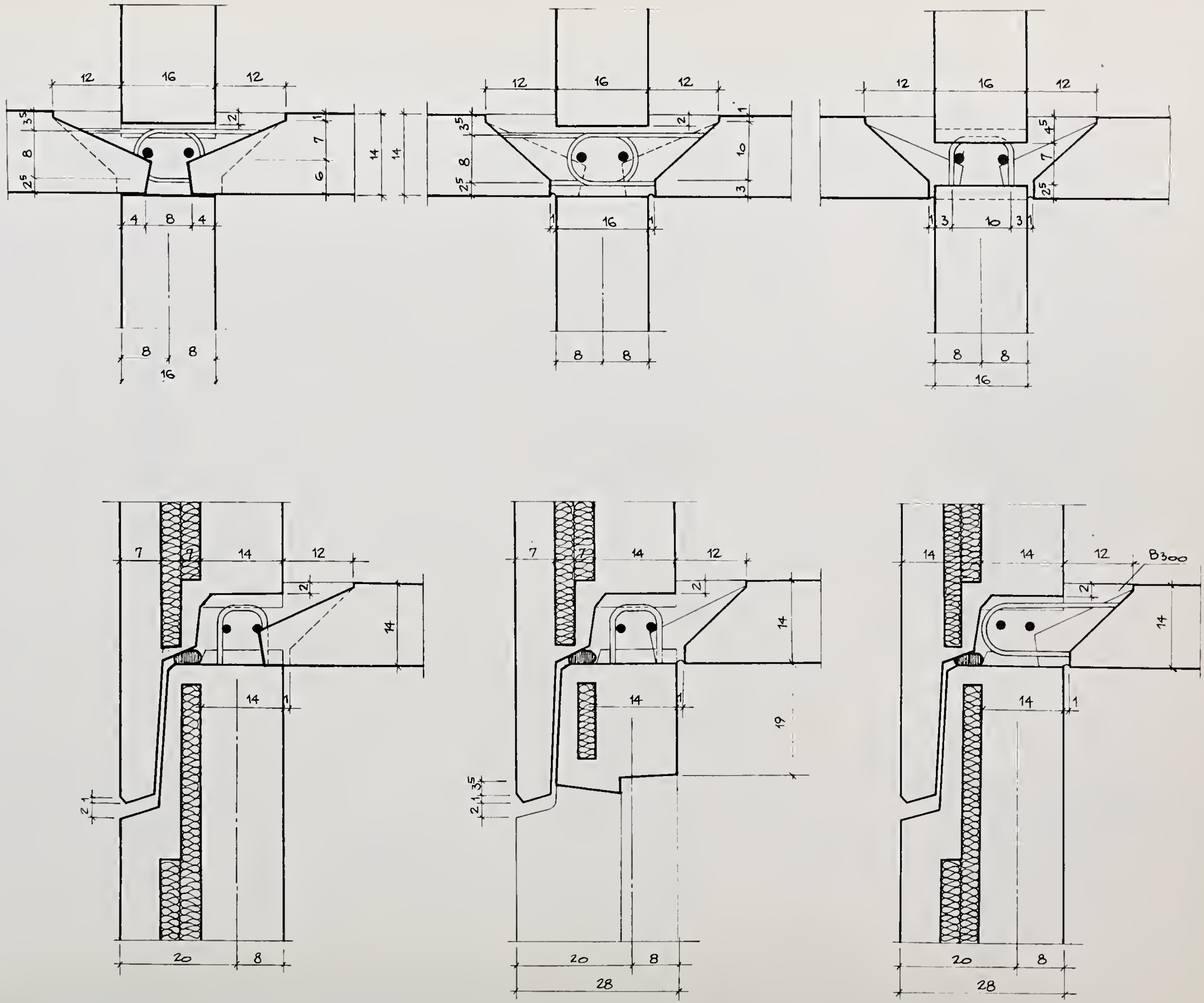
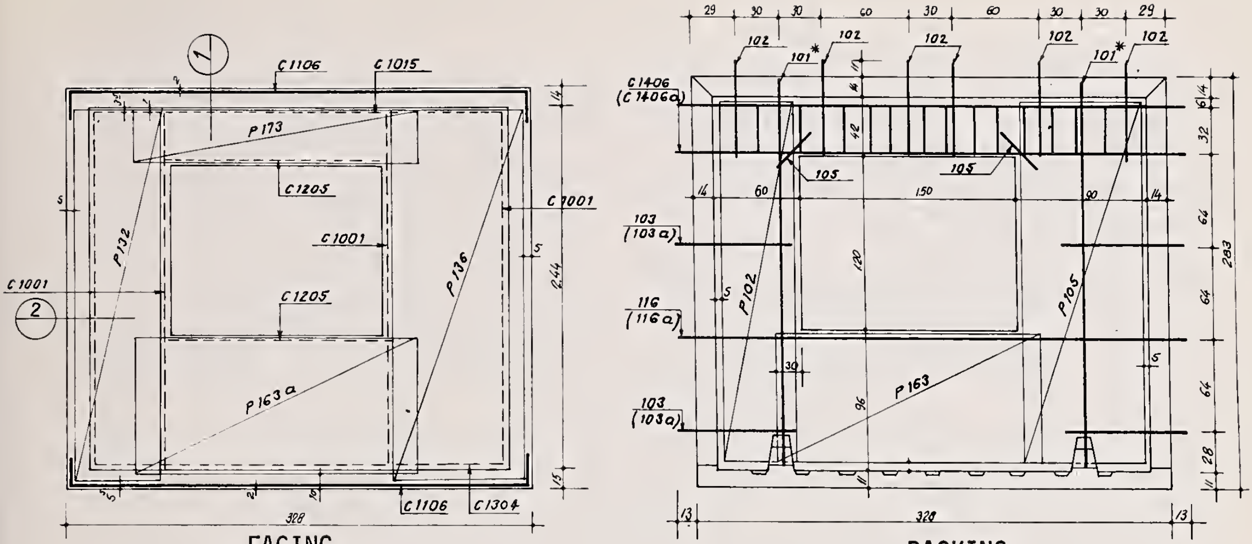
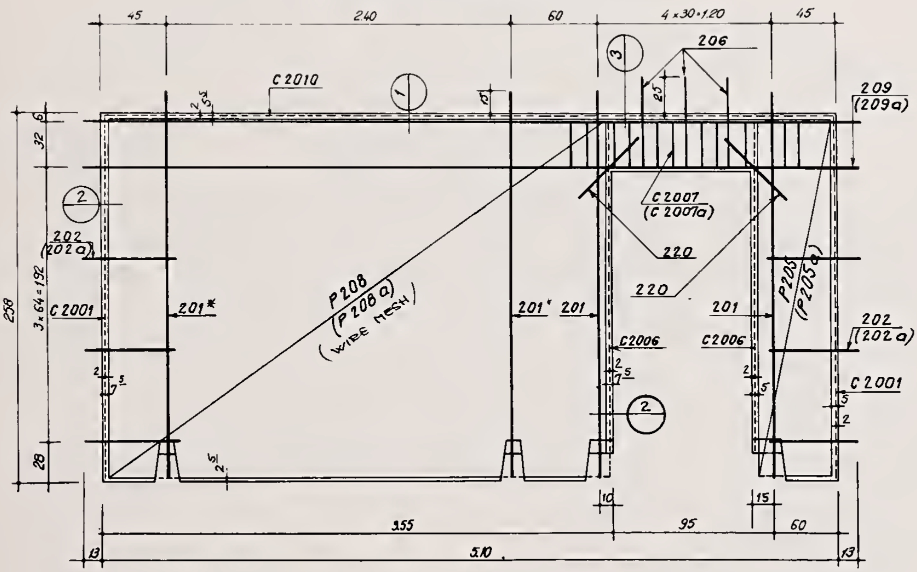


Figure 2.15. Horizontal joint details of high-rise panel building system.

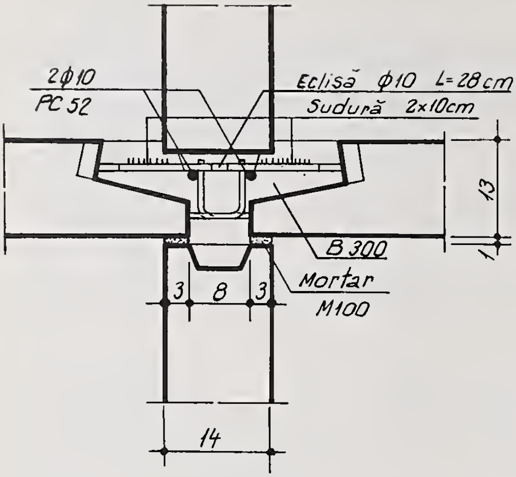


(a) Interior Wall Panel

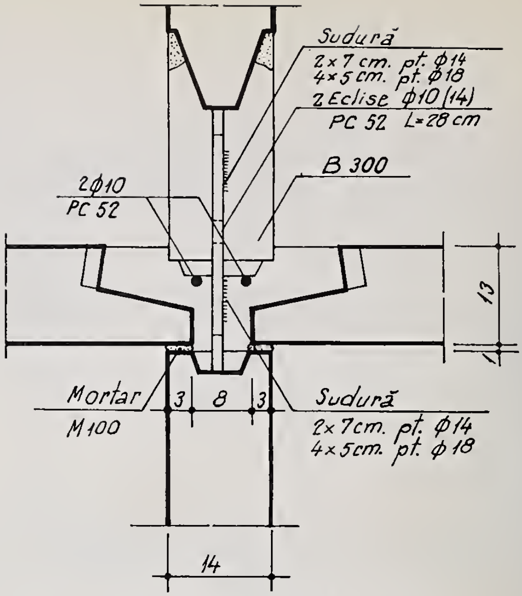


(b) Interior Wall Panel

Figure 2.16. Typical panel details for mid-rise precast building.

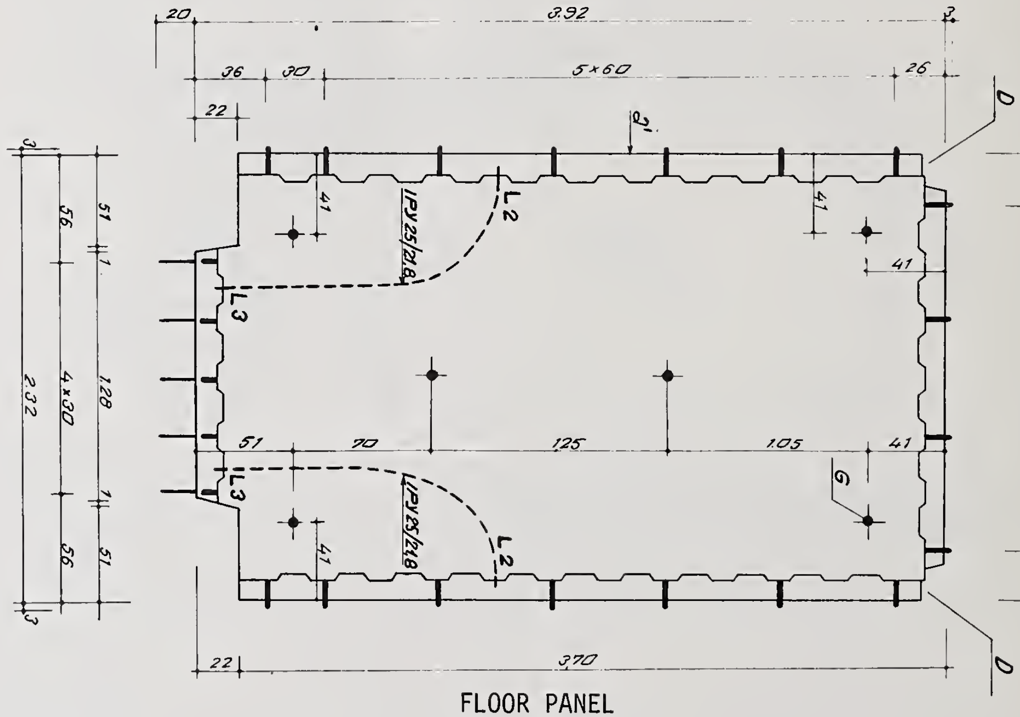


(a) connection of floor panels



(b) connection of wall panels

HORIZONTAL JOINT CONNECTION DETAILS



FLOOR PANEL

Figure 2.17. Floor panel and horizontal joint details of mid-rise precast building system.

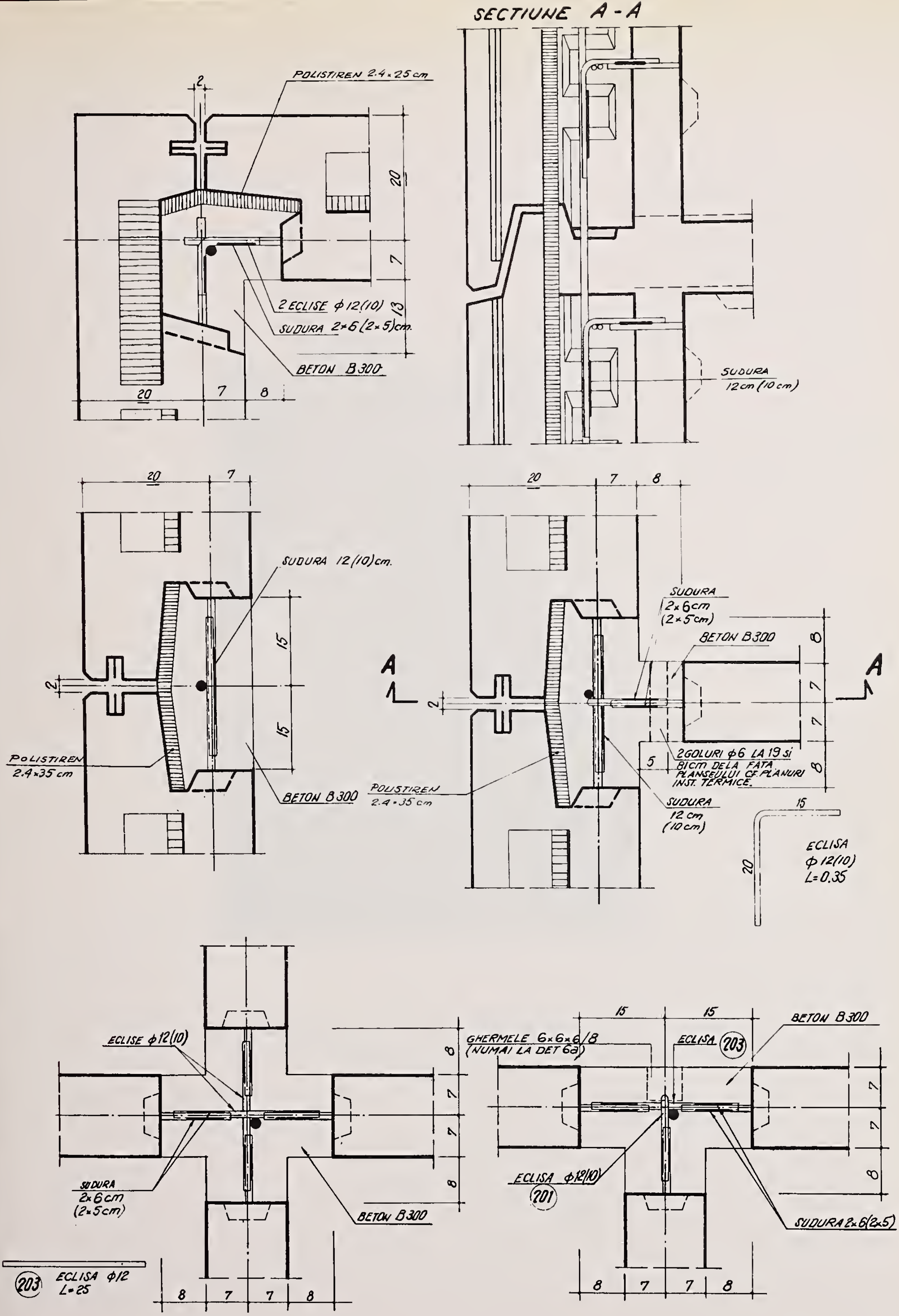


Figure 2.18. Vertical joint details of mid-rie panel building system.

Figure 2.19. The Intercontinental Hotel in downtown Bucharest.

3. SURVEY AND ANALYSIS OF EARTHQUAKE DAMAGE

3.1 General

This section presents a survey of earthquake damage in the cities of Bucharest and Craiova, based on field observations by the NBS members of the U.S. team, supplemented by information acquired from the Romanian engineers and scientists who worked with the team. The survey focuses on damage to buildings since there was no evidence of significant overall damage to the country's infrastructure. Power, water, rail and air service were restored very quickly after the earthquake. Ports were unaffected.

Out of an estimated total of 33 buildings destroyed in Bucharest, all but three were reinforced concrete masonry infilled frames that were built before 1940 and were of the type described in Section 2.3.2. Two post-WW II structures that collapsed were sections of multi-story apartments built after 1960. The third building was a modern concrete structure which housed the computer facilities of the Ministry of Transportation.

In the city of Craiova, although many structures sustained moderate to heavy damage, it was reported that no collapses had occurred. Among heavily damaged structures examined were the art museum and the city administration headquarters, both of which had historic significance, and load-bearing masonry apartment buildings. A number of multi-story apartment buildings examined exhibited minor facade damage mostly confined to the lower stories.

3.2 Pre-1940 Structures

It was learned from INCERC that most of the destroyed or heavily damaged buildings in Bucharest were located within a three-pronged downtown area roughly identified by the broken lines shown in Figure 3.1. The left branch of this area extends towards the west along the Dimbovita River, the central portion runs along Magheru Boulevard north, and the right branch extends towards the northeast more or less parallel to the Boulevard of the Republic. The large solid circles within that area designate the approximate locations of 26 damaged building sites, half of which are numbered for referencing purposes in the text. The rest of the circles designate sites about which no specific information was acquired.

Unless otherwise noted, all of the buildings discussed below are multistory concrete frames reinforced with plain steel bars and infilled with unreinforced masonry walls. Buildings examined but not identified on the map are not assigned numbers in the text. When used, the names of the buildings are for identifying purposes only and sometimes refer to nearby landmarks.

Buildings 1 through 4 - Group of Devastated Buildings

Figures 3.2 to 3.6 exhibit the remains of four apartment buildings that were totally destroyed during the earthquake. These photographs were taken by an INCERC survey team before removal of the debris. It should be noted that by March 14, the day the U.S. technical team arrived in Bucharest, all of the rubble from these and most other destroyed building sites had been completely cleared.

Figure 3.5 exhibits a rare instance of a collapsed building which was an interior unit. In general, the collapsed structures were either isolated buildings or were the end or corner units of a block of adjacent buildings. A plausible explanation of this trend is provided by the fact that the corner and end units tend to be less symmetric and more irregular in plan and structural configuration than those at the interior. Consequently, torsional effects, including the likelihood of coupling of the torsional and translational modes, and unfavorable stress distribution under lateral loads, would tend to be relatively greater in the end and corner units.

There may be still another factor involved. Numerous instances of pounding damage were witnessed along the joints of adjacent buildings. It is possible that under certain conditions the two joints flanking an interior building unit could constrain and damp the response of the system in such a manner as to create less unfavorable stress conditions at the lower levels. The single joint next to an end unit is less effective in providing such confinement nor will it necessarily help restore symmetry or reduce torsional effects. This difference provides a possible explanation of collapses of end units that were triggered by failures at the lower story levels (Figures 3.7 and 3.8). In addition, the pounding could have been aggravated by the fact that the end building was higher in certain cases, than the adjacent building.

A peculiar phenomenon associated with the collapse of the older structures is evidenced by the extensive pulverization of the concrete as indicated in Figures 3.2 and 3.4. A member of the INCERC team that examined wrecked structures immediately after the earthquake noted that in many instance the concrete had disintegrated completely and a haze of dust particles hung in the air surrounding the sites. This pulverization effect, also noted during the November 1940 earthquake (see Ref. 3 and Section 2.3.2), may possibly be attributed to the cumulative degradation of concrete through exposure to cyclic loading during past earthquakes. Well-rounded ends of story-high concrete columns that had pried loose and fallen to the ground (see, for instance, Figures 3.3 and 3.6), as opposed to jagged ends normally found in concrete failures, may be evidence of internal pulverization of concrete in the joints attributable to past extreme events. It should also be noted that the use of relatively low-strength concrete (as low as grade B100 with approximately 10 MN/m^2 cube strength) was prevalent in the construction of pre-1940 buildings.

Building 5 - Wilson Apartments

A typical example of partial building collapse precipitated by failure in the lower stories is provided by the photographs in Figures 3.7 and 3.8 taken soon after the earthquake. Referred to as the Wilson Building, this was a 12-story apartment structure in which failure of the first story columns caused the corner bays to shear off from adjacent bays and vertically impact the ground. The collapse of the top story roofs shown in Figure 3.7 was possibly a follow-up action triggered by this impact.

The framing configuration of this building is not apparent from the photograph. In Figure 3.8, the two large pillars in the foreground appear to be reinforced concrete columns, judging from a joint failure at the (present) fifth floor level. Otherwise, there appears to be no reinforced concrete exterior columns in the collapsed portion of the building. The use of two wythes of clay tile units in the construction of exterior walls was common practice for this type of structure.

Building 6 - Snagov Building

Figures 3.9 and 3.10 show the partial collapse of a three-story residential building located on Snagov Street. The building appears to be a massive, stiff structure with thick exterior walls, partitions, and floor slabs. This is one of the rare instances where a stiff low-rise structure had collapsed. The large story height, large perforations in the exterior walls and slender piers between openings were features that may have contributed to its demise.

Building 7 - Continental Building

Figures 3.11 to 3.13 show different views of a collapsed multi-story apartment building named after the Continental Bar in the first story. The building was located on Ghica Street, west of the Boulevard of the Year 1848. By reference to Figure 3.11, which was taken before removal of the debris, it appears that the structure was U-shaped in plan, with the end wings joining other buildings to form a ring around a central courtyard.

Except for a portion of the left wing, this structure was leveled by the earthquake. It was learned that a 19-year old boy trapped within that section was saved 11 days after the earthquake. Figure 3.12, taken the same day, shows the structural configuration of the left wing which was three bays deep. Note the common use of double and single wythe brick infill walls at the exterior and interior of the building, respectively. Although not designed to function structurally, these walls invariably took the brunt of the earthquake-induced lateral forces before failure.

The neighboring structures of this site, including the church with the pear-shaped domes (Figure 3.13), showed no indication of visible damage. In Figure 3.11, the tall structure in the foreground and to the right, is an 11-story building with its center bay braced over the full height by concrete diagonals. It also showed no damage other than the loss of lower story filler walls that may have been caused by earthquake action or by the force of falling debris; in the figure, the first story is concealed behind the rubble.

Building 8 - Dunres Building

Figures 3.14 to 3.17 are different views of the partially collapsed Dunres building located on Boulevard Balcescu across from the Intercontinental Hotel. The name refers to the restaurant on the first floor. Figure 3.14, taken before removal of the debris, shows daylight coming through the upper story windows that might indicate an open yard in the rear. It is therefore presumed that the building has a V-shaped configuration.

Structurally, the building is characterized by flexible first and second stories, where stores and offices are located (Figures 3.14 and 3.15) and stiff upper stories with masonry infill walls and partitions to provide for apartment space. The upper three stories are progressively setback and therefore, do not conform to the typical floor plan layout of the stories below (Figures 3.15-3.17). None of these features are particularly desirable from the standpoint of earthquake-resistant construction.

Building 9 - Lido Building

Figure 3.18 shows an 8-story apartment building with stores in the ground floor, located at the end of a block behind the Lido gas station. From Figure 3.19, the building appears to be U-shaped in plan, overlooking an interior court, and ajoining shorter structures on both sides. Approximately the left half of the building collapsed to the ground. The rest of the building was severely damaged, probably beyond repair. In the standing portion, the first story exterior columns had failed at the top (Figure 3.18), while those at the interior exhibited shear failure at the base (Figures 3.20 and 3.21).

This building and the one on the left may have caused mutual damage. In Figure 3.19, it appears that the top story corbel of the building on the left had pounded against the 7th story column of the Lido building, causing it to fail at the base. In way of retribution, the collapsing roof of the Lido building had taken along portions of the roof and masonry wall of the adjacent building (see Figure 3.18 which was taken before clearing of the wreckage).

The failure of an interior column (Figure 3.20) exposed four vertical bars, approximately 20mm in diameter, and a single tie, approximately 6mm in diameter, spaced from the base a distance equal to the depth of the column. Four hooked footing dowels projected into the base of the column. The column that failed in shear (Figure 3.21), did not appear to have any ties within the height of the spalled concrete which was twice the depth of the column. All bars were plain.

Building 10 - Casata Building

Figures 3.22 to 3.26 show the partial collapse of a 10-story building which is the end unit of a block of buildings along the east side of Boulevard Magheru. General views of the building before and after removal of the debris are shown in Figures 3.22 and 3.23, respectively. The wreckage in Figure 3.22 indicates extensive pulverization of the concrete, which was observed at other sites as well.

In Figure 3.23 it is noted that the 9th and 10th stories of this building, both of which are set back, have much longer transverse spans than the stories below. It appears that the setback wall of the 10th story is supported only by the 9th story roof slab. Figure 3.24 shows pounding damage observed between this and the next buildings. Note the slight tilt of this building to the right. Pounding damage had also occurred between other buildings in this row (Figure 3.25).

Figure 3.26 shows shear failure observed in an interior first story column. A total of six vertical bars were used in the column, three in each face, and approximately 16mm in diameter. The single ties were approximately 6mm in diameter and were bent only around the corner bars. In general, concrete cover was insufficient and at certain locations non-existent. Note, for instance, that on the unspalled top surface of the column, ties as well as vertical bars are visible with one of the ties inclined 15 degrees with respect to the horizontal.

<u>Building 11 - Turist Building</u>

Figure 3.27 is a scene of downtown Bucharest viewed from Piata Romana towards Boulevard Magheru south. Taken in April 1974, this photograph shows on the right the 9-story Turist building which was heavily damaged during the March 4, 1977 earthquake. The Casata building, which can also be seen, is the last unit of the left block in the background. This photograph shows a restaurant on the first floor of the Turist building which also uses the sidewalk as part of its dining area.

A partial view of this building taken after the earthquake (Figure 3.28) exhibits substantial structural and non-structural damage in the lower stories. Fallen plaster on the sidewalk, the detached soffit of the canopy above it (Figure 3.29), and the ruptured facing of the second story wall on the east side, which had to be propped to prevent collapse, are instances of potential hazards to pedestrians (sidewalk users, shoppers, cafe patrons, etc.) associated with the collapse of non-structural exterior elements that can occur even when the building itself survives the earthquake. Falling masonry units resulting from premature failures of inadequately bonded exterior walls and piers are another potential source of hazard to the pedestrians. A case in point is the type of masonry construction illustrated in Figure 3.31 which shows a damaged pier between the second story balconies. The exposed masonry beneath the fallen plaster is characterized by poorly bonded units (no head joint mortar used) having different sizes and shapes.

The worst apparent structural damage occurred at the relatively flexible first story level within the end bay area as evidenced by the extensive amount of timber shoring and bracing placed around the periphery of the building both inside and outside (Figure 3.30). Loss of vertical support within this area had occured as a result of column failures such as shown in Figure 3.32. Although the spacing of the ties (approximately 6mm in diameter) appeared adequate, they did not prevent buckling of the face bars, probably because they were bent around the corner bars only. The proximity of the ties to the column surface was noted here as in other places. Several heavy timber logs were placed next to the column to shore up the floor above this location. Metal bars attached to the logs, timber cross-braces and horizontal back-to-back steel channels are probably used to brace the auxiliary support system against forces of possible aftershocks. Elsewhere (Figure 3.33), timber posts were used to shore the damaged floor slab in the story above. The brick filler wall at this location appeared to have stopped short of the ceiling.

Metalimport Building

Figures 3.34 and 3.35 show another partially collapsed concrete structure characterized by setback stories at the top. Approximately a 2-bay by 3-bay corner section was completely demolished while the rest of the structure was probably damaged beyond repair. Daylight showing through the back windows of the upper stories suggests the possiblity of an open courtyard in the rear. If this were the case, the building would have an L-shaped configuration which is not particularly conducive to earthquake-resistant construction.

It appears that some of the masonry walls in this building served a load-bearing function. For instace, Figure 3.35 indicates that the collapsed stairs and landing were entirely supported by multiple wythe masonry walls outside and inside the building since concrete framing elements are conspicuously absent in this region. Elsewhere, the interior masonry walls in the upper stories (the plastered walls in the plane of Figure 3.34) are at least partially supporting the bearing loads transmitted by the setback walls above. Also note that in the first interior bay successive stories are partitioned differently.

Mercerie Building

The heavily damaged exterior of this building (Figures 3.36 and 3.37) made it appear as though it was on the verge of collapse. The characteristic diagonal X cracks in the lower story piers indicated the structure had experienced cyclic load reversals. As was usually the case in other buildings, the most severe structural damage had occured within the relatively flexible first story where substantial shoring had to be installed to prevent further distress.

Building 12 - The Ambassador Hotel

The rear view of the Ambassador Hotel shown in Figure 3.38 provides a typical demonstration of the type of relatively moderate but widespread damage found in pre-1940 infilled structures in downtown Bucharest. The stiff masonry walls, initially attracting most of the lateral forces, were among the first victims of the shock. Particularly susceptible were the lower story walls. This stems from the fact that masonry filler walls were not specifically designed to provide either gravity or lateral load resistance and therefore had the same thickness throughout the building.

Building 13 - The Post Office Building

The Post Office Building shown in Figure 3.39 is one of the very rare instances where steel frame structures are used in building construction (reportedly only two or three steel structures are found in Bucharest). The architecture of this building is reminiscent of steel structures built in the U.S. in the 1930's. It was indicated that there was no visible damage to the main structure. Non-structural damage was relatively minor and was mostly confined to cracking and spalling of brittle facade elements adjacent to lower story openings.

3.3 Post WWII Structures

As noted earlier, only three post-WWII structures collapsed in Bucharest. One of these structures was a multistory residential building located in the western part of the city. It was reported that a section of this building had collapsed to the ground and that wreckage had already been removed. The two remaining structures that were surveyed are buildings 14 and 15 indicated on the map shown in Figure 3.1.

Building 14 - Lizeanu Building

This row of buildings stretches along the north side of Stefan Cel Mare Avenue directly east of Lizeanu Street. Approximately the three end bays nearest Lizeanu Street fell to the ground and assumed a slightly tilted posture after the collapse of the first story columns. Reportedly, the building was further tilted in an initial attempt by the demolition crew to overturn it. Subsequently, wrecking operations were carried out from the top down so that at the time it was inspected, all the stories above the fourth (counting the collapsed first story) had been already removed (see Figure 3.40).

It was learned that before the earthquake, this section of the building was in the process of being underpinned to rectify a foundation problem encountered earlier. Therefore, it is conceivable that one of the main reasons, if not the only reason, for the collapse may be attributed to the presence of this temporary remedial condition at the time the earthquake struck.

Figures 3.40 through 3.45 present several photographs of the building taken from different angles at the time of the visit, except for the rear corner view shown in Figure 3.41, which was taken by an INCERC crew soon after the earthquake, before demolition work had begun. It was reported that the building is a concrete frame structure with shear walls in the upper stories. The presence of a concrete frame may be ascertained from Figures 3.42 and 3.43 and also from the detached reinforced concrete column on the ground shown in Figure 3.41. Because of the finish, it is not possible to establish the composition of the transverse walls shown in Figure 3.40. However, Figures 3.41 to 3.43 indicate that the exterior longitudinal walls in the back of the building are unreinforced masonry filler walls.

Figure 3.44 shows structural damage in the first story columns in the back of the standing portion of the building and the timber shoring used to prop other damaged sections. There was no other major visible damage to the exterior of the rest of the building. Figure 3.45 shows the front view of the building facing Stefan Cel Mare Avenue. Note the extent of delamination of concrete cover underneath the upper story floor slabs and the concrete stairs shown in Figures 3.43 and 3.45.

<u>Building 15 - Computer Center</u>

The collapse of the building that housed the computer facilities of the Ministry of Transportation and Telecommunications was the most severe single incident in terms of its impact on the Romanian economy. As a result, 400 people were put out of work and millions of dollars worth of materials were destroyed, including computer hardware and software which are not readily replaceable.

A northeast view of the center before the earthquake is shown in Figure 3.46. The three-story building in the foreground and the access tower in the background (better seen in Figure 3.49) were structurally disjoint from the computer building in the center which collapsed.

Figure 3.47 shows the plan of the building together with a sectional elevation of an end bay and the configuration of first story columns. The building was a three-story concrete flat slab structure in which 30m square floor and roof slabs were supported by nine columns spaced 12m on centers in both directions, and were cantilevered 3m beyond the exterior columns. All second and third story columns were square and prismatic. The first story columns varied from a square cross-section at the top to the flared configuration shown in Figure 3.47. The columns were approximately 0.5m x 0.5m in cross section, except the flared sections of the first story columns were larger. All columns were capped with shallow wide capitals at the top.

The exterior walls consisted of precast concrete sections hung from the peripheral edges of, and anchored to, the slabs at each story level. They stopped short of the floor below, as indicated in the same Figure. The floor slabs were approximately 0.5m thick and had square cellular hollow cores so that a longitudinal repetitive strip in either orthogonal direction had the sectional configuration of a wide flange beam with equally spaced web stiffeners on both sides. The ruptured slab shown in Figure 3.52 further illustrates the type of construction used. The specified material strengths were 30 MN/m^2 (grade B300) for the concrete and 510 MN/m^2 for the reinforcement bars. There were no shear walls in this structure.

Figures 3.48 to 3.55 provide a description of the situation after the earthquake. Figures 3.48 and 3.49 are views of the north side of the computer building after the collapse. Note the tilted and detached walkway slabs between this building and the access tower. The latter did not exhibit any visible signs of damage. Except for broken window panes and some damaged interior partitions, the 3-story structure west of the computer building was likewise undamaged. Figure 3.50 is a partial view of the south side of the building after the collapse. The hung exterior walls which did not originally extend to the floor below, now do. These walls may have been instrumental in keeping the structure from being totally levelled to the ground.

To the extent that could be established, nearly all the columns in the building had failed. In most cases the failures had occurred at the tops of the column just below the capital. Figures 3.55 and 3.51 exhibit the failures of first and second story columns, respectively, in this manner. Within the main computer room at the second story level, one column had ruptured at midheight, as indicated in Figure 3.54. The exterior second story column shown in Figure 3.53 appears to have failed in flexure after loss of support elsewhere (note the rotation at the top of this column conforming to the tilt of the sagging floor slab above).

At the time of the visit to the site, it was learned that investigations into the collapse of the computer building were underway but that as yet no definite conclusions had been reached. With regard to the failure sequence, it was mentioned that according to one lay person who happened to witness the incident, the building collapsed progressively from the roof downwards. The reliability of such secondary sources of information, is, of course, unknown.

Based on field inspection and supplementary information acquired at the site, certain conditions that may have been factors contributing to the collapse are cited as suggestions for further verification by the Romanian investigators. It is noted, for instance, that the computer equipment was installed on raised floors, as is commonly done to simplify wiring. This brings out the possibility that a premature failure of the raised floor may introduce a live load dynamic amplification factor (as a result of the impact of the computer equipment falling on the concrete floor) not normally considered in design. Figure 3.53 provides evidence that the raised floor had failed (although not necessarily prematurely). In Figure 3.52, it appears that the central processor had punctured the second and first story floor slabs and ended up on the basement floor.

Another deficiency noted by reference to the failures shown in Figures 3.54 and 3.55 is the absence of substantial column ties needed to resist the shear forces due to transverse loads and torsional effects. The fact that most columns in the building failed at the top is significant in that respect. It may be that the building was simply not designed for the seismic intensity it actually experienced or that the design provisions for ties were inadequate. In either case, the problem is probably related to deficiencies in the existing codes.

A third possibility is that the fate of the entire building appeared to be dependent upon the structural integrity of a single column. Stated differently, there seemed to be no alternate paths of load transfer to the foundation, if, for some reason, one of the columns were to be removed. Mechanisms for the containment of localized failures, or built-in redundances to provide for alternate means of load transfer, if duly recognized by the codes, are design concepts that would reduce the likelihood of progressive collapse under the action of extreme events.

Building 16 - Concrete Shear Wall Building

Among relatively recent structures examined were a number of buildings located close to INCERC (Figure 2.2) on Boulevard Pantelimon in the eastern section of the city. The three structures that were surveyed in greater detail than others were built well after 1970 and therefore were designed according to the latest seismic provisions issued in 1970 (see condensed version presented in section 4.3). These buildings are identified by numbers 16 to 18 on the general map of Bucharest shown in Figure 2.2.

The first building inspected was an 11-story concrete shear wall structure having the cellular layout described in section 2.3.3. It was located at the end of a row of multistory apartment houses and had a layout almost identical to that shown in Figure 2.13 (the plans were examined prior to the visit). The front of the building, a partial view of which is shown in Figure 3.56, gave no visible indication of damage. The end view of the building likewise revealed no damage except for some minor cracking in the lower story shear wall couplers (Figure 3.58). It should be noted that in buildings which were not seriously affected by the earthquake, and this was one of them, a visual assessment of external structural damage was often rendered difficult by the presence of facing walls used for insulation purposes. A case in point is the delamination of the first story facing wall shown in Figure 3.59 which, without close inspection, could be mistakenly associated with structural damage.

At the first story level, the structural system in the transverse direction consisted of three shear walls and six intermediate frames as indicated in Figure 2.13. Within the first story restaurant, the only structural damage noted was the cracking of one of the frame columns shown in Figure 3.60. Within the second story apartments inspected, considerable damage was noted in the concrete shear wall lintels above door openings, particularly those in the transverse direction. Elsewhere, a good deal of cross-cracking in the plaster of transverse walls (Figure 3.61) had occurred, an indication of reversed cyclic response. The direction of the transverse walls approximately coincided with the N-S component of the ground motion which attained a maximum acceleration of 0.2g according to the strong motion records obtained from the accelerograph located on INCERC grounds nearby (see Figure 2.2).

On the whole, the extent of structural damage was moderate and was mostly confined to the first three stories. Apparently, the building occupants were not evacuated. One of the proposed repair schemes discussed with INCERC engineers involved casting of additional concrete shear walls within the transverse frames of the first three stories.

Figure 3.57 shows a 12-story building under construction located on Boulevard Pantelimon right across from the shear wall structure. This photograph was taken mainly to illustrate the generally poor quality masonry construction frequently observed, even in the most recent structures. The reason for using brick in the lower stories and concrete block in the upper stories was not established although it might have been influenced by availability of materials at the time of construction. Note the lintel frames over the eighth story window openings. Subsequently, these are filled with concrete as in the lower stories.

Building 17 - Concrete Frame Structure

One of the buildings examined was an 11-story concrete frame structure that was under construction and had not yet been fully occupied. Figure 3.62 shows the building viewed from the side. It was noted that during the earthquake, the frame had pounded against the box-type shear wall structure next to it and had broken one of its diaphragms.

This was another instance where low quality workmanship and materials were found in the construction of masonry filler walls, as well as walls built exterior to the frame (Figures 3.62 and 3.68). Appearance is not a factor here because the walls are plastered before the structure is put in service. However, it seems that the structural integrity of the walls is not a principal concern either. It almost appears as if the only purpose is to create enclosure by using whatever materials happen to be at hand and to cut down on the consumption of mortar to the bare minimum needed to keep the units from falling off during construction.

The ground floor of this building was actually two stories in one, with a built-in mezzanine accessed from within (Figures 3.63 and 3.69). The longitudinal bay nearest the street was two stories (about 8m) high as seen in Figure 3.63 and, therefore, the unsupported length of front exterior columns in the transverse (approximately N-S) direction was twice that of columns at other locations. Without shear walls in this area, the lateral resistance of the first two stories was entirely dependent on continuous frame action.

Structural damage within this area was generally moderate but quite widespread. Several of the transverse beams below the third story floor slab exhibited failures near the supports and had to be shored by metal pipes at intermediate locations as indicated in Figures 3.69 and 3.70. The latter shows the single stirrup noted below a section of spalled concrete approximately equal in length to the height of the beam. The most extensive damage occurred in one of the long exterior columns. Figures 3.64 and 3.65 show the elaborate steel system used to keep it under lateral confinement. The concrete had spalled at about 1.5m above the floor and extended another meter beyond. The column was about 850mm square. The main reinforcement was about 25mm in diameter. The ties were spaced about 300mm and were approximately 6mm in diameter. The ends of some of the ties that had opened up appeared to have been bent 90 degress around the corner bars. Figures 3.66 and 3.67 show two additional columns that were not damaged as severely. The ties at both locations were about 6mm in diameter spaced at 100mm on centers. The upper stories of this building were not examined.

Building 18 - Concrete Frame Structure

The next building examined was similarly a reinforced concrete frame structure in which the first story elements exhibited a similar but somewhat less extensive damage pattern. Figures 3.71 and 3.72 show two transverse beam failures characterized by diagonal cracking at the column supports. A close-up view of the beam shown in Figure 3.72 revealed two positive bars of unequal size and a single stirrup in the region of the spalled concrete. The smaller bar (in the background) appeared to have buckled. At another location, the spalled concrete at the top of a column (Figure 3.73) exposed a bent corner bar but no ties.

Within the second story, considerable cracking and spalling of plaster had occurred, but this was generally not extensive enough to permit assessment of structural damage, if any. One exception was the cracking of the stairway at the second story landing shown in Figure 3.76. In connection with the partition wall within a second story apartment unit (Figures 3.74 and 3.75), two peculiarities were noted. First, the cracked plaster was exceedingly thick (about 100mm). Second, the partitions did not conform with the framing arrangement. The wall in this case was located behind a transverse structural beam and the space in between had been filled with masonry units overlain with plaster (note its thickness in Figure 3.75). At the time of inspection, most of the brick units had pried loose and fallen to the floor.

Figure 3.77 provides a further illustration of the general lack of emphasis placed on the quality of masonry construction when used in filler walls, parapets and other non-load bearing applications. Exposed concrete columns at several locations suggests that there may have been considerable shedding of masonry units from the walls of this building during the earthquake. Figure 3.78, which is the rear view of the same building, shows substantial pounding damage to this structure and the one adjacent to it, including the partial collapse of several masonry filler walls and possibly structural damage to a fifth story corner column in the adjacent building. In the building further to the left (Figure 3.79), many of the piers that had been already repaired but not plastered over, showed classical X cracking patterns.

In this general area, several other instances of pounding damage were witnessed, such as shown in Figures 3.80 and 3.82 to 3.84. The former was a frame building located next to a shear wall structure which showed no visible signs of exterior damage (Figure 3.81). The most severe pounding damage observed had occurred as a result of the collision between a taller frame building and a shorter structure having adjacent corners.

Figures 3.83 and 3.84 are two opposite views of the damage which seemed to have occurred mostly in the frame structure including the rupture of a corner column in the story opposite the top of the next building.

Other Recent Structures

A number of apartment buildings of relatively recent construction were examined in the southern part of the city near Boulevard Magurele. These buildings are collectively identified by the number 19 on the map shown in Figure 2.2. The first building was a 5-story precast panel structure shown in Figures 3.85 and 3.86. No evidence of damage, not even signs of hairline cracks, was found inside or outside the building.

The second building, shown in Figure 3.87, was a 10-year old cast-in-place concrete shear wall structure in which breakage of window glass and appearance of hairline cracks near the first story stairway landing were the only visible signs of damage. This observation was also corroborated by the superintendent of the building. Because the first story was used for residential purposes, its stiffness was more compatible with the rest of the structure than would have been the case had that area been designed for store usage. Note the superior quality of the exposed masonry used in the construction of the end wall of the building.

The third building, shown in Figure 3.89, was a 4-year old slip-formed shear wall structure which was likewise used exclusively for residential purposes. There was no visible damage on the outside of the building. Inside, there were minor cracks in the lintels above some of the doorways to the first story apartments. According to one of the tenants, there was no other damage elsewhere in the building.

The fourth building, shown in Figure 3.90, consisted of two 11-story rectangular structures interconnected at their corners. It was an 11-year old slip-formed shear wall building in which the only visible damage was confined to the first story hallway area where insignificant superficial cracks were observed in the walls and at the stairway junctions. Inspection of exposed structural elements within the basement showed them to be free of cracks.

Figure 3.88 is a view showing some of the other typical layouts used in multistory apartment construction in this section of the town. Insofar as could be discerned by cursory visual examination, there were no conspicuous signs of major exterior damage to these structures.

There was also some evidence of physical damage to buildings in the suburban town of Magurele, 12km due south of the center of Bucharest (see Figure 2.2). The modern looking 5-story hotel where the members of the U.S. technical team were stationed was located on the campus of the National Center for Physics(Figure 3.92). Although this building suffered no exterior damage, several interior walls at the first-story level exhibited substantial damage, through-cracks and detachment along horizontal and vertical joints. The upper stories were relatively free of major damage. Next to the hotel, the 11-story institutional building shown in Figure 3.93 had suffered substantial superficial damage including stripping of plaster from stiff precast masonry facade panels in the fourth story and from masonry coupler walls (on the right), and through-cracks in the first story slab and masonry side wall (not shown). The interior of this building was not examined.

Among modern buildings in Bucharest, observed mostly in passing, the National Theater next to the Intercontinental Hotel (Figure 3.91) exhibited cracking in one of the exterior columns supporting the overhanging roof and minor spalling in the tower facade. The Intercontinental Hotel in the same block (see Figures 2.3 and 2.19) did not appear to have suffered significant structural damage. However, substantial and costly damage of a non-structural nature, such as detachment, cracking and relative displacement of interior partitions, had occurred within the building. In general, flexible buildings or flexible parts of buildings (i.e., soft first stories) appeared to have been subjected to motions large enough to cause the type of nonstructural damage associated with partial loss of functionality or serviceability. Relatively speaking, such losses were considerably less severe in the case of stiffer structures.

3.4 Structures in Craiova

The city of Craiova is located 185km west of Bucharest on the bank of the river Jiu, a tributary to the Danube 60km to the south. According to a 1972 estimate, Craiova is the seventh largest city in Romania. It is located in seismic zone 6 as indicated in Figure 4.1. Consequently, its buildings were not designed for seismic loads, except those post-1970 structures which were of exceptional importance (class I category in Table 4.1) and for which the P. 13-70 provisions were applicable (section 4.2), were designed using a seismic acceleration coefficient of $k_s = 0.03$ (or 3 percent of gravitational acceleration) as indicated in Table 4.1.

During the brief visit to Craiova, only a few buildings could be examined. Upon arrival at the city government headquarters (Figure 3.94), the members of the visiting team were met by the vice mayor of Craiova who supplied general information about the city. It is 17 centuries old, has a current population of 230,000, a university with 7 departments and 7,000 students, and manufacturing facilities for chemicals, electrical equipment and machinery. Total casualty figures were not supplied but it was indicated that a substantial number of the human casualities had occurred in the streets as a result of falling debris.

The city is built on four terraces, rising successively from the river in the southwest. The second terrace from the river had felt the worst shock, except in the Craiovitsa section, where the structure of the soil is different, there was less damage. In general, the buildings in the rest of the city including industrial plants suffered only minor damage.

No major building collapses had occurred in the city. A number of brick houses and a block of new brick apartment buildings had been damaged. The most serious damage had occurred in old and historic buildings such as the city administration headquarters building (Figure 3.95) and the art museum (Figure 3.99). The former had been damaged in the November 10, 1940 earthquake, but the effect of that earthquake had been much less severe. In the present earthquake, some of the 1940 cracks that were repaired had reappeared.

The city government office building (Figure 3.94) where the meeting was held showed no damage. After the meeting, the first building toured was the city administration headquarters (Figures 3.95-3.98). Major vertical cracks had occurred at the top of the brick wall of the central tower as seen in Figure 3.95. The cracking had developed just below the pyramidal high pitched roof and had propagated downward on both sides, putting the structural integrity of the steeple in jeopardy (Figure 3.96). To maintain stability, two Vierendeel type steel joists were installed horizontally on the near face and were anchored by cables to the rear portion of the tower as indicated in Figures 3.95 and 3.96. There was also evidence of roof damage due to collapse of the cupolas, as shown in Figure 3.95.

Within the building, the structure of the ceiling consisted of a series of spherical and cylindrical brick vaults supported by massive brick columns and bearing walls. At one location a portion of the ceiling had collapsed (Figure 3.97). At other locations the ceiling and the archways were severely cracked and displaced relative to the supports (Figures 3.97 and 3.98). The repair of this building will require the expertise of professionals specialized in the restoration and preservation of historical monuments.

The second historical structure examined was the art museum (Figures 3.99-3.103). It was a massive masonry bearing wall building with a heavy timber floor system. Most of the damage was attributed to the collapse of the chimneys that had fallen inside and outside of the building. Figure 3.99 shows the damage caused to the exterior facade and balcony by a fallen chimney. In one instance, a cupola had fallen into the building and had damaged the flooring below. Figure 3.100 exhibits the exposed timbers of a floor damaged by falling debris. The floor joists were approximately 0.3m square, spaced at about 0.6m on centers. Falling debris had also caused breakage of a decorative skylight above one of the first story rooms (Figure 3.101). The brick walls in the first story, which were about 0.5m to 0.7m thick, were not seriously cracked, although a good deal of decorative plaster had spalled off both walls and the ceiling.

The attic of the building was accessed by narrow stairs that had been seriously damaged. Within the attic, the high-pitched timber roof was supported by a timber framing system laterally stiffened by knee braces as shown in Figure 3.102. At a few locations splitting of knee braces and frame members were noted. Figure 3.103 shows the splitting of one of the laterally unbraced brick chimneys projecting beyond the roof. In general, the extent of damage was such that the building will be relatively easy to repair without compromising its historical appearance. On the other hand, retrofitting it for resistance against future earthquakes will not be as simple. One possibility discussed with the Romanian engineers involved stiffening of the attic floor to give it increased diaphragm capability to improve the distribution of lateral forces to the shear walls below.

The third building examined was a 5-story unreinforced masonry load-bearing wall system. Two of the wings of the building shown in Figures 3.104 and 3.105 exhibited cracking of first story piers. The latter also shows the development of a major vertical crack between two first story abutting walls at one corner of the building. It appeared that most of the damage had occurred in the first story where many of the exterior piers exhibited severe diagonal cross cracking as shown in Figure 3.106. By reference to this figure, two distinct modes of failure can be identified. In each of the four different instances shown, the wide pier had attained its collapse limit state, characterized by through tension cracking along the diagonal(s), while the slender pier located next to it had developed horizontal flexure cracks at the ends (a first limit state), without appreciably losing its vertical load-bearing capacity.

Inside the first story, the solid interior walls were not as severely damaged (Figure 3.107) but most lintels above openings had received a good pounding (Figure 3.108). One of the repair schemes that was being considered for this building was the application of gunite over wire mesh on both sides of the first and second story walls. The Romanian engineers also expressed interest in learning more about the structural rehabilitation methods used in the United States, particularly tuckpointing of masonry and use of epoxy resins in the repair of concrete and masonry cracks.

Elsewhere in the city, two or three instances of pounding damage between buildings were noted during the survey such as shown in Figure 3.109. There were also a number of buildings with partly missing roofs and fallen chimneys. Inspection of 10-story reinforced concrete shear wall buildings, some of which were reportedly erected quite recently using the slip-form method, revealed minor non-structural damage, mainly in the first story exterior insulation walls as seen in Figure 3.110.

3.5 Summary of Observations

The devastating effect of the March 4 Romanian earthquake is evidenced by the casualty figures and property losses that have been reported in Section 1. The full impact of the earthquake in terms of human suffering, disruption of productive activities and expenditures in relief and rehabilitation is not readily quantifiable but probably will magnify the cited disaster statistics manyfold.

The fact that most of the casualties occurred in Bucharest and as a result of the collapse of pre-1940 apartment buildings underscores the immediate necessity to address the problem existing in similar structures which, even though survived this earthquake, may well have been rendered structurally inadequate to resist forces of future earthquakes of even lesser intensity than experienced on March 4. Considered collectively, these buildings shared many of the deficiencies noted below:

1. Low concrete strength, viz., $10 MN/m^2$ or less.
2. Cumulative damage resulting from past extreme events such as the 1940 and 1977 earthquakes and World War II.
3. Inadequacies in ties and stirrups including excessive spacing and improper anchorage.
4. Insufficient reinforcement cover.
5. Flexible first story supporting stiff upper stories.
6. No lateral bracing other than non-structural masonry walls having low membrane capacity and not specifically designed to resist lateral forces.

7. Irregular layout and structural discontinuities.
8. Progressive setbacks of upper stories often without auxiliary supports or bracing below.
9. Structural and non-structural modifications during service often carried out without particular attention being given to their effect on the overall integrity of the system.
10. Low bond between concrete and steel reinforcement attributable to use of plain bars and insufficient splices and anchorages.
11. Proximity of buildings giving rise to distinct possibility of pounding damage in an earthquake.
12. Improperly bonded exterior masonry walls and the potential hazard to pedestrians arising from their premature failure.

Particularly disturbing was the extensive disintegration of concrete that had occurred at the destroyed pre-1940 building sites as noted by independent observations following both the present and November 10, 1940 earthquakes. This phenomenon may be associated with low specified concrete strength or possibly with cumulative damage attributable to past extreme events as noted in the first two items above. A third possibility is that less cement may have been used in the concrete than called for by design. In this respect, it is interesting to note that the commission investigating the collapse of the Carlton building after the 1940 earthquake had found the concrete and steel used in its construction were less than required by the design specifications (see Section 2.1 and Ref. 3).

It is hoped that investigations will be undertaken to determine the extent of degradation of in-situ concrete in the pre-1940 buildings, and whether or not it will be economically feasible to retrofit (rather than replace) any of these structures in a manner that would reduce the seismic risk involved to a level that would be locally acceptable. This would require sampling and testing of concrete specimens extracted from the sites to determine their composition and mechanical properties, as well as definition of a minimum level of acceptable seismic risk as part of the decision making process.

From the standpoint of structural safety, the performance of post-WWII buildings in Bucharest was remarkably good. This is a positive reflection on the seismic design and construction practices introduced in the 1950's and progressively refined into the comprehensive provisions currently in force (see Section 4.3). As noted in Section 3.4, of the three post-WWII building collapses in Bucharest on record, the two that were examined by the NBS team presented certain unusual circumstances which may have adversely affected their seismic response.

It appears, however, that the current seismic provisions were somewhat less effective in mitigating substantial economic losses associated with structural and non-structural damage of a less serious nature, judging from the damage statistics compiled in Section 1. Although these figures provide no specifics as to extent and nature of damage in relation to type, age and location of buildings affected, they include institutional and public facilities such as hospitals, health care centers, nurseries, kindergartens, university buildings and manufacturing plants, many of which were probably erected during the post-WWII period when seismic requirements had gone into effect.

Specifically, the numerous instances of pounding damage of buildings witnessed both in Bucharest and Craiova suggest the need to reexamine the rationale behind the building separation requirements currently in force. In this respect it would be useful to attempt to reconcile the respective drift and building separation provisions of the P.13-70 document [6] and the Uniform Building Code [8] which are derived from fundamentally different principles.

Secondly, potential hazards to pedestrians and dwellers from falling debris in an earthquake can be reduced by improving the quality of materials and workmanship used in the construction of masonry walls for non-structural applications. Examples of exposed masonry construction of superior quality (see Figures 3.90 and 3.93, for instance), suggest that the skills required to bring about such improvements are at hand.

A third area deserving attention is the practice with regard to the use of ties and stirrups in concrete columns and beams, respectively. There were cases of failures where ties and stirrups appeared to be minimal and improperly anchored. In other instances they seemed to be quite adequate. The differences could not be altogether explained in terms of apparent differences in the design loads. The question thus arises whether there could be discrepancies between the structure as built and the code recommendations and also whether these recommendations are adequate, particularly in situations where the lateral resistance of a building is entirely dependent on continuous frame action.

Fourth, the prospects of survival of buildings in an earthquake could be improved by incorporating, at the design stage, alternative mechanisms of load transfer aimed at the containment of local failures, should a relatively small portion of a building become structurally impaired. This is particularly important in the design of buildings in situations where a collapse would be most catastrophic in terms of human casualties, economic loss, and disruption of services deemed essential to the post-disaster recovery effort.

It is finally noted that the damage resulting from this earthquake could provide a basis for assessing the relative merits of various building systems in resisting earthquake loads. An important question to be resolved is to determine the engineering implications of the fact that stiff structures, both precast and cast in place, generally performed better than more flexible types of buildings. It is possible that resonance may have adversely influenced the response of flexible structures with fundamental periods in the order of 1 to 1.5s, as suggested by the characteristics of the only strong motion record available (Figure 1.2). On the other hand, the magnitude and frequency content of this earthquake may have been such that stiffer structures were not put to test in a manner that would provide sufficient indication about whether they are capable of post-cracking ductile behavior. It is felt that the opportunity exists for these and other critical questions to be resolved through a coordinated research effort involving field investigations and selective testing of scaled buildings and prototype components, coupled with analytical studies of seismic response using mathematical models of buildings and digitized time-histories of the available earthquake records.

Figure 3.1.. Map of downtown Bucharest showing area heavily affected by the earthquake and location of damaged structures.

Figure 3.2. Cafe Scala apartment building totally destroyed by the earthquake.

Figure 3.3. Opposite view of the Cafe Scala building ruins.

Figure 3.4. Avintul building near Piata Rosetti destroyed by the earthquake.

Figure 3.5. Multistory apartment on Sahia Street near corner of Piata Rosetti totally destroyed by the earthquake.

Figure 3.6. View of wreckage of the multistory apartment building at Boulevard Hristo Botev No. 10.

Figure 3.7. General view of failure of Wilson apartment building.

Figure 3.8. Close-up of damaged Wilson building.

Figure 3.9. Partial collapse of three-story residential building on Snagov Street.

Figure 3.10. Close-up view of the collapsed bay of the building on Snagov Street.

Figure 3.11. General view of the collapse of the multistory Continental building before removal of the wreckage.

Figure 3.12. View of Continental building after partial removal of debris.

Figure 3.13. Standing left wing portion of Continental building with Russian church in the background.

Figure 3.14. View of partially collapsed Dunres building before removal of debris.

Figure 3.15. Southeast view of Dunres building after clearing of debris.

Figure 3.16. View of Dunres building looking north.

Figure 3.17. Close-up view of Dunres building.

Figure 3.18. Partial collapse of Lido building.

Figure 3.19. Close-up view of Lido building after removal of suspended wreckage.

Figure 3.20. Failure of interior first story column at the base indicating lateral displacement.

Figure 3.21. Shear failure at the base of first story interior column.

Figure 3.22. General view of the collapse of the end bays of Casata building.

Figure 3.23. Casata building after removal of debris.

Figure 3.24. Damage due to pounding of the Casata building with the next structure.

Figure 3.25. Pounding damage to other structures in the Casata building block.

Figure 3.26. Damage to interior column in the first floor.

Figure 3.27. Scene of downtown Bucharest showing the Turist building before the earthquake.

Figure 3.28. General view of the Turist building after the earthquake.

Figure 3.29. Close-up view of detached soffit of the canopy over the sidewalk.

Figure 3.30. West view of Turist buildin
showing substantial facade
and masonry damage.

Figure 3.31. Close-up of damaged masonry
pier beneath spalled plaster.

Figure 3.32. Interior view showing failure of first story column at the base.

Figure 3.33. Timber logs shoring partially damaged floor slab of second story.

Figure 3.34. Partial collapse of 10-story Metalimport building.

Figure 3.35. Left view of the collapse Metalimport building.

Figure 3.36. Damage of eight-story Mercerie building.

Figure 3.37. Another view of Mercerie building showing configuration of upper story setbacks.

Figure 3.38. Damage of multistory Ambassador Hotel, rear view.

Figure 3.39. Post Office and Telecommunications building showing damage to exterior walls.

Figure 3.40. End view of partially collapsed Lizeanu building taken from Lizeanu Street looking towards east.

Figure 3.41. Rear view of collapsed portion of Lizeanu building before removal of debris.

Figure 3.42. Rear corner view of collapsed end bays after tearing down of walls by wrecking crew.

Figure 3.43. Close-up view of region collapsed stairs.

Figure 3.44. Rear view of collapsed end bays looking west.

gure 3.45. Front view of the collapsed bays taken from Stefan Cel Mare Avenue looking north.

Figure 3.46. Computer center of the Ministry of Transportation and Telecommunications before the earthquake.

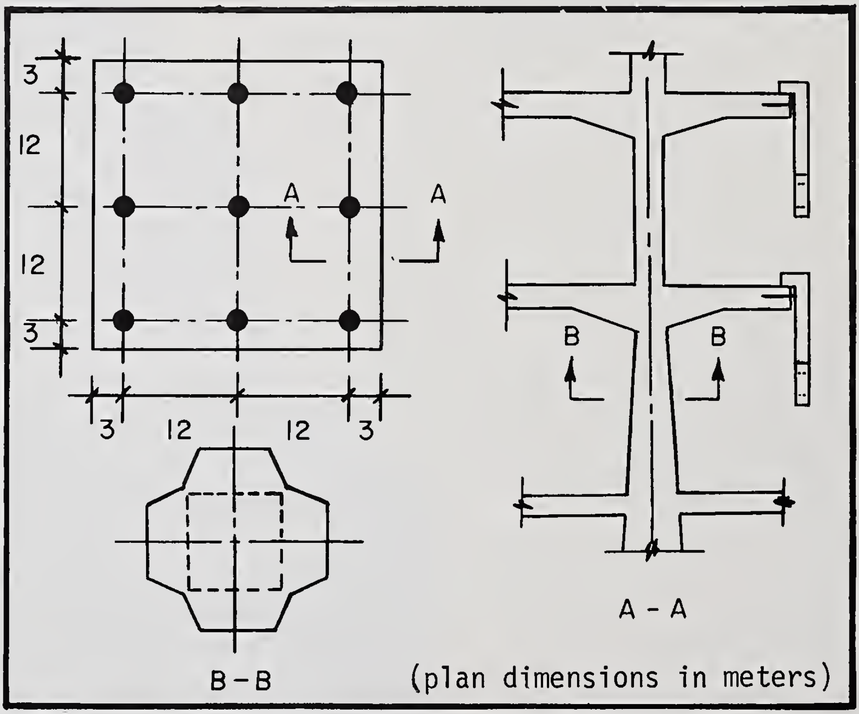


Figure 3.47. Structural details of the computer building.

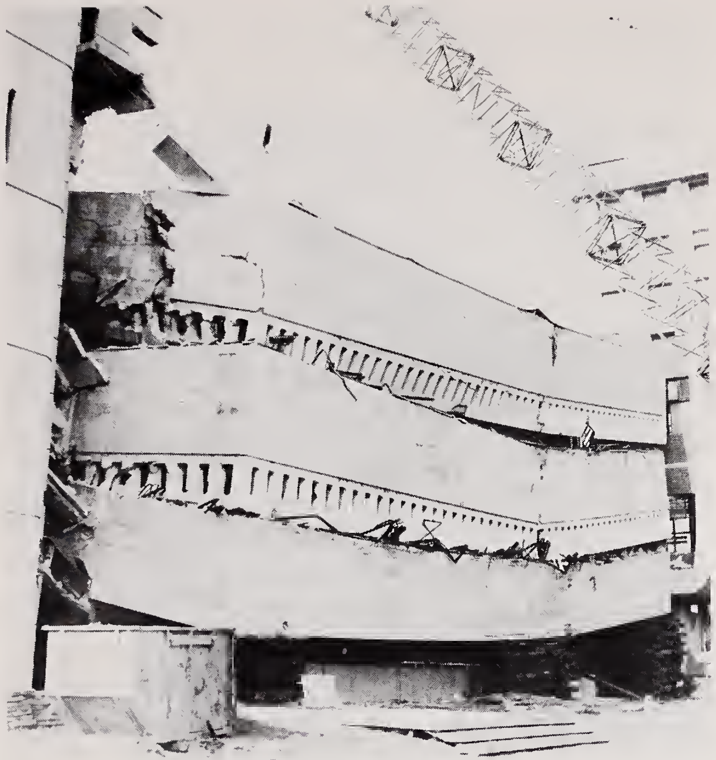


Figure 3.48. Northeast view of the computer center building after the earthquake.

Figure 3.49. Northwest view of the computer center building after the earthquake.

Figure 3.50. South side of computer building after the earthquake-

Figure 3.51. Interior view of second story wreckage.

Figure 3.52. Damaged central computer unit.

Figure 3.53. Inside view of the main computer room in the second story.

Figure 3.54. Failure of second story column located in the air conditioning room.

Figure 3.55. Failure at the top of the northwest column in the first story.

Figure 3.56. Partial front view of 11-story apartment building on Boulevard Pantelimon (Building No. 16).

Figure 3.57. Multistory apartment building under construction across from Building 16.

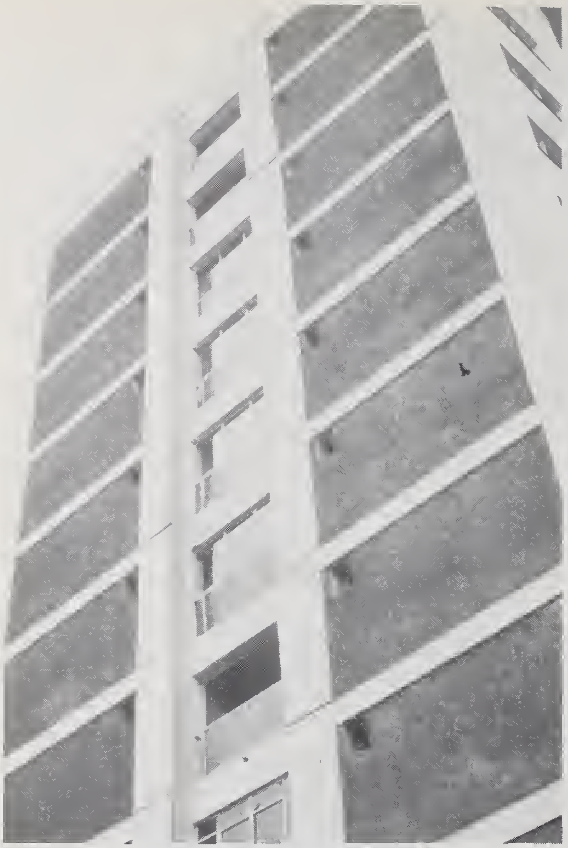


Figure 3.58. End view of upper storie
Building 16.

Figure 3.59. View of rear corner of Building 16 showing detached insulation wall.

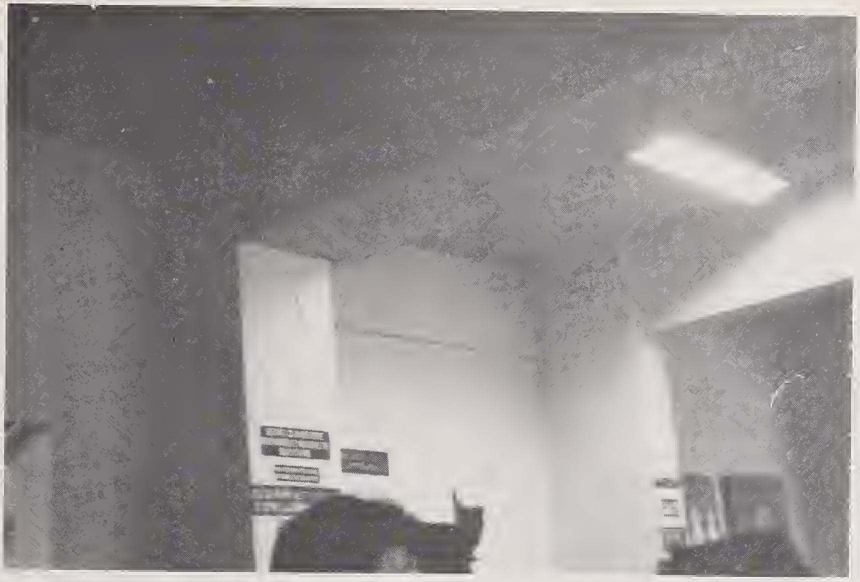


Figure 3.60. Interior view of first story restaurant in Building 16.

Figure 3.61. Cracking of wall plaster inside second story apartment Building 16.

Figure 3.62. Side view of 11-story concrete frame building on Pantelimon Avenue. (Building No. 17)

Figure 3.63. General view of longitudinal bay within the first story of the building.

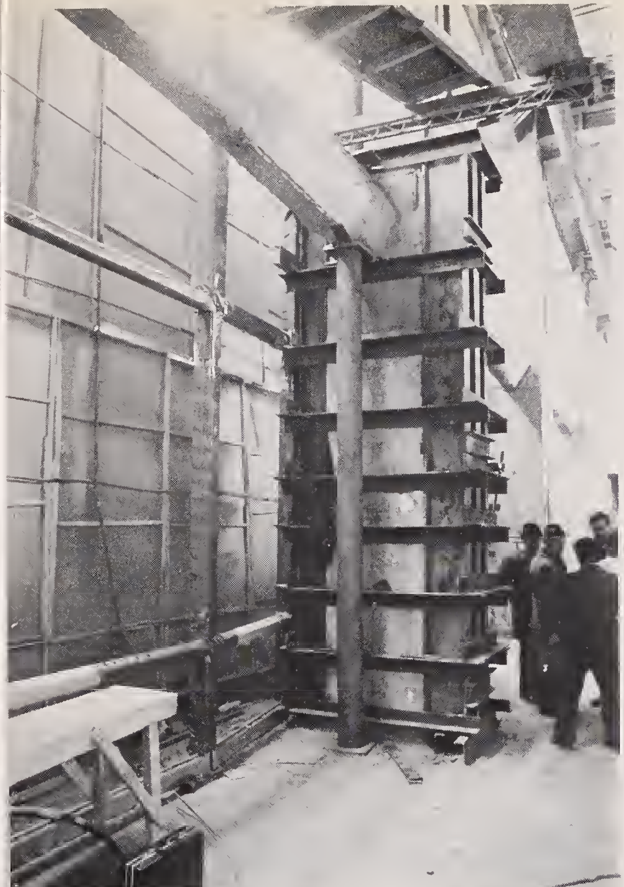


Figure 3.64. Circumferential confinement of a ruptured exterior column in the first story.

Figure 3.65. Close-up view of steel framing used for confinement of ruptured column.

Figure 3.66. Damage at the base of fir
story exterior column.

Figure 3.67. Damage at the base of first story interior column.

Figure 3.68. Exterior masonry walls at the first story level.

Figure 3.69. Metal pipes used in shoring damaged transverse beams.

Figure 3.70. Failure of transverse beam at the support.

Figure 3.71. Shoring of damaged first story beam in multistory concrete frame building. (Building No. 18)

Figure 3.72. Close-up and opposite view of damaged first story beam.

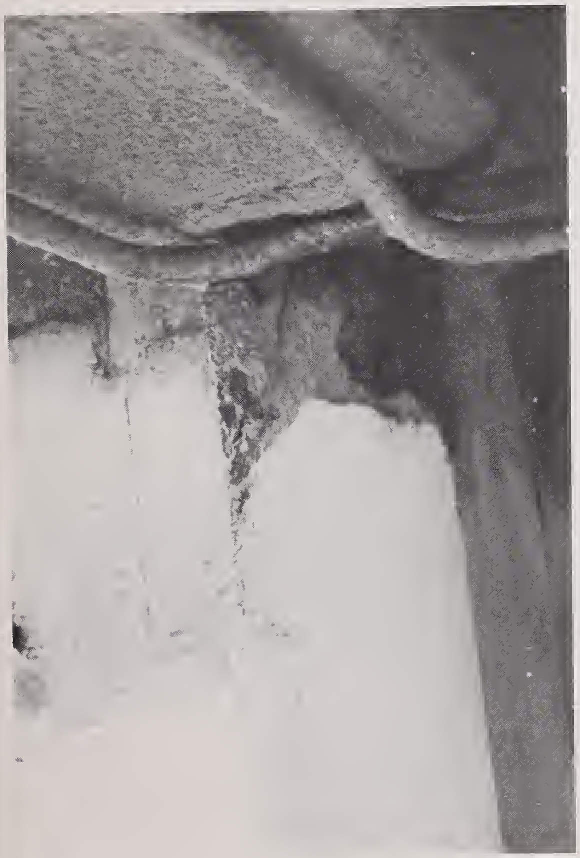


Figure 3.73. Damage at the top of first story column.

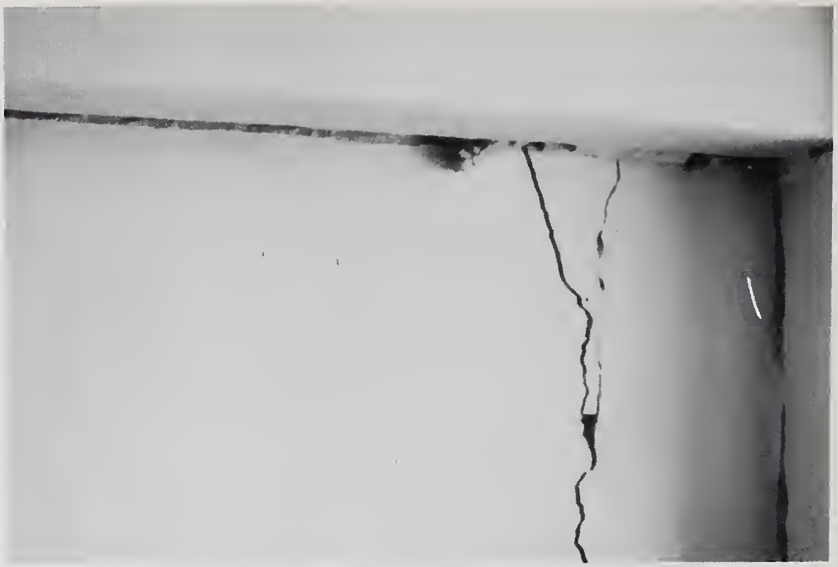


Figure 3.74. Cracking of wall within second story apartment unit.

Fiqure 3.75. Detached masonry units between ceiling and wall.

Figure 3.76. Damage within second story hallway.

Figure 3.77. Exterior view of multistory apartment building.

Figure 3.78. Rear view of building sh
in Fig. 3.77 showing
pounding damage.

Figure 3.79. Cracking of exterior piers
in multistory building.

Figure 3.80. General view of superficial damage in concrete frame building next to shear wall building on the left.

Figure 3.81. Front view of 10-story shear wall building.

Figure 3.82. Pounding damage to adjacent buildings.

Figure 3.83. Heavy pounding damage to buildings with adjacent corners.

Figure 3.84. Same pounding damage viewed from the opposite side.

Figure 3.85. Front view of 5-story large panel building.

Figure 3.86. End view of 5-story panel building.

Figure 3.87. Eleven-story shear wall building.

Figure 3.88. Row of 11-story apartments.

Figure 3.89. Eleven-story slip-formed shear wall building.

Figure 3.90. Another 11-story slip-forme
shear wall building.

Figure 3.91. National Theater next to Intercontinental Hotel.

Figure 3.92. Five-story hotel on campus of National Center for Physics in Magurele.

Figure 3.93. Superficial damage of multistory building on campus of National Center for Physics.

Figure 3.94. City government office building in Craiova.

Figure 3.95. View of city administration headquarters building in Craiova showing damaged tower and roof.

Figure 3.96. Close-up view of damaged tower of city administration building.

Figure 3.97. Partial collapse of vaulted ceiling within administration building.

Figure 3.98. Other damage to the ceiling within the administration building.

Figure 3.99. Exterior damage to the Art Museum in Craiova.

Figure 3.100. Partial collapse of the second story floor of the museum.

Figure 3.101. Damage of decorative skylight within the first story of the museum.

Figure 3.102. Timber framing of the roof within the museum attic.

Figure 3.103. Ruptured masonry chimney in the museum attic.

Figure 3.104. General view of load-bearing masonry apartment building.

Figure 3.105. Damage of first-story exterior bearing walls.

Figure 3.106. Failure of first-story piers.

igure 3.107. Damage of interior masonry wall.

Figure 3.108. Damage of lintel wall in first story.

Figure 3.109. Pounding damage between adjacent buildings.

Figure 3.110. Cracking of facing wall in multistory apartment build

4. BUILDING PRACTICES

4.1 General

Many buildings in Bucharest subjected to the earthquake of March 4 were designed to resist earthquakes. It is therefore of interest to examine both the behavior of these buildings and the seismic design requirements with a view to assess the adequacy of the latter.

The material in this chapter is presented for this purpose. A summary of Romanian seismic design requirements based on an NBS translation of the original Romanian text, P. 13-70, issued in 1970, is presented in Section 4.3. The principal differences between these and the preceding requirements P. 13-63, issued in 1963 [4] are discussed in Section 4.4. Section 4.5 gives a description of the microzoning of Bucharest which went into effect in 1973. For purposes of comparison, a brief summary of the main earthquake design provisions of the 1976 Uniform Building Code is compiled in Section 4.6.

4.2 Introduction

As indicated in Ref. 3, before the November 10, 1940 Romanian earthquake, structures were generally designed for vertical loads only, and were seldom checked for wind forces. No earthquake loads were considered in design. The seismic design recommendations introduced in the aftermath of the 1940 earthquake were based on the provisions of the Italian building code but were not made mandatory.

Following a heavy increase in construction activity in the late 1940's and especially in the 1950's, a research effort was undertaken and used as the basis to develop a draft seismic design standard in 1957. This is described in some detail in Ref. 5. Building code requirements representing a modified version of this draft were issued in 1963 as a legally binding document under the designation P. 13-63 [4]. This document was superseded by the latest edition P. 13-70 issued in 1970 [6]. The fundamental principles of the 1957, 1963 and 1970 documents are essentially similar, each successive document incorporating a number of refinements based on the experience gained from practical application of the provisions, observations reported following major earthquakes, and research results obtained in Romania and internationally.

4.3 Summary of P. 13-70 Seismic Design Requirements [6].

The magnitude of the lateral seismic design loads is given by:

$$S_{kr} = k_s \beta_r \psi \eta_{kr} Q_k \tag{4.1}$$

where S_{kr} designates the force acting at level k corresponding to the r-th vibration mode, k_s is a coefficient dependent on the seismic zone and importance of the structure (Table 4.1), β_r is a coefficient dependent on foundation soil conditions and natural period T_r (in seconds) of the structure in the r-th vibration mode, ψ is a coefficient dependent on the damping and ductility characteristic of the structure, η_{kr} is a coefficient dependent on the modal participation factor, and Q_k is the sum of all gravity (dead and live) loads above level k. Live loads associated with the weight of residential building occupants in buildings where large concentrations of people occur infrequently, or with the weight of moveable installations, equipment, etc., are multiplied by a reduction factor of 0.8.

Table 4.1 - Values of coefficient k_s

Class	Description[a]	Seismic Zone[b]			
		6	7	8	9
I	Structures of exceptional importance[c]	0.03	0.05	0.08	0.12
II	All structures, except those of classes I, III & IV	-	0.03	0.05	0.08
III	One-story structures and other structures for which losses in case of failure would be moderate	-	0.02	0.03	0.05
IV	Structures of secondary importance for which losses in case of failure would be small	-	-	-	0.03

[a]More precise definitions of classes I, III, IV are given in Reference 6.

[b]Seismic Zone Map for Romania is shown in Figure 4.1.

[c]For certain types of structures (e.g., nuclear installations) special criteria may be used as required by the appropriate regulatory agencies.

The coefficient β_r is specified as $0.6 \leq \beta_r = 0.8/T_r \leq 2.0$ except that (a) for structures other than masonry or reinforced concrete, if the foundatio soil consists of rock or consolidated gravel, $0.48 \leq \beta_r = 0.64/T_r \leq 1.6$, and (b) if the soil conditions are unfavorable*, $0.9 \leq \beta_r = 1.2/T_r \leq 2.5$. These exceptions are not applicable if the local soil conditions are taken into account in microzoning maps. In calculating the period T_r, the elasticity of foundation soil is taken into account. As a result, T_r will be greater and β_r will be smaller (subject to the lower limitation specified above) than if T_r were calculated assuming fixity at the foundation level.

*Detailed descriptions of soils for which the specified relationship applies are given in Reference 6.

The specified values of ψ are 1.0 for frame structures, 1.2 for structures with shear walls, 1.3 for structures with bearing walls, 1.8 for tall, very flexible structures (e.g., antennas, etc.), 2.0 for water tanks and 1.2 for all other structures.

The coefficient η_{kr} is determined as follows:

$$\eta_{kr} = u_{kr} \frac{\sum_{i=1}^{n} Q_j u_{jr}}{\sum_{j=1}^{n} Q_j u_{jr}^2} \tag{4.2}$$

where u_{kr} is the coordinate at level k of the r-th modal shape.
In addition, it is required that the following relation be satisfied:

$$\sum_{k=1}^{n} S_{k1} \geq 0.02 \sum_{k=1}^{n} Q_k \tag{4.3}$$

In designing structures for which $T_1 \leq 1s$ and the height $H \leq 40m$, only the fundamental vibration mode need to be taken into account. If $T_1 > 1s$ or $H > 40m$, the first three vibration modes must be taken into account. In this latter case

$$N = \left[\sum_{r=1}^{3} N_r^2 \right]^{\frac{1}{2}} \tag{4.4}$$

where N_r is the force in any given member induced by the loads S_{kr} (k=1, 2, ...n), and N is the total force in that member due to seismic loads.

Earthquake induced vertical loads equal to the product $ck_s\ Q_k$ are taken into consideration in the design of structural and non-structural components and connections. Values for c are as follows:

$c = \pm 4$ for axial force when predominent in a member, for shear force in girders and for gravity loads on large-span cantilever beams.

$c = \pm 6$ for concentrated vertical force on girders, punching shear force on large-span flat slabs, and gravity load on heavily-loaded cantilever beams.

$c = \pm 5$ for connections of post-tensioned structural members.

$c = \pm 3$ for connections at supports and of elements with heavy load concentrations due to seismic effects and for seismic forces normal to wall surface of non-load bearing walls.

$c = \pm 10$ for seismic forces in any direction acting on balconies, cornices, water tanks within buildings and industrial installations supported by braced frames.

$c = \pm 15$ for seismic forces in any direction acting on parapets, ornamental elements, statues, towers and stacks of low height.

Where the vertical resisting elements depend on diaphragm action for distributing the shear at any level, the structure is designed for torsional moments equal to the shear force times a distance e. This distance is defined as follows: $e = e \pm e_2$ where e_1 = distance between mass and stiffness centers and $e_2 = c_eB$. (c_e = 0.05 for ordinary structures, c_e = 0.075 for structures that have an unfavorable distribution in plan of the members resisting the earthquake-induced torsion, and B = maximum building dimension in plan normal to the seismic force).

Special provisions for the design of foundation and basements specify that buildings over 25m in height be provided with a rigid box construction for the floor over the basement, basement walls and foundations. For seismic zones 8 and 9, it is recommended that the floor over the basement be cast in place in buildings with several stories.

Special provisions for seismic-resistant elements in cast-in-place reinforced concrete structures require (a) a minimum concrete strength equivalent to grade B200, having a corresponding Romanian standard cube strength of approximately 20 MN/m^2 (3,000 psi), (b) a minimum strength and ductility of steel reinforcement conforming to grades OB38 (380 MN/m^2 or 55,000 psi) or PC52 (520 MN/m^2 or 75,000 psi), (c) minimum negative reinforcement in beams at column supports varying between 0.5% and 2% for OB38 steel and between 0.4% and 1.5% for PC52 steels, and corresponding minimum positive reinforcement of 30% of the negative steel at the same location, (d) use of stirrups at beam supports over a minimum distance equal to 25% of the span length and (e) ultimate flexural strength in beams in excess of the ultimate shear strength. For columns the minimum reinforcement requirements are (a) 1% and 0.8% of OB38 and PC52 steels, respectively, for corner columns, and 0.8% and 0.6% of OB38 and PC52 steels, respectively, for all other columns and (b) use of ties spaced at 100mm or less for a height of 600mm or 1/6 the floor height, whichever is less, at both column ends and extending over the beam depth at supports. In addition, a recommendation is made against designs of rigid (shear wall) structures resting on flexible columns at the ground floor level. In the event that such structures are designed, the code requires that the forces corresponding to Eq. 4.1 be increased by

25% in calculating the earthquake loads on columns supporting shear walls.

Special provisions for frame and shear wall structures require that shear walls be designed for shear forces equal to 1.5 times those obtained from Eq. 4.1. Where possible, it is recommended that frame structures be so designed as to ensure development of plastic hinges in beams rather than in columns under the action of earthquake loads. Additional special provisions for prestressed concrete, steel, masonry and wood structures and non-structural elements are found in the P. 13-70 document.

4.4 Summary of Differences Between the P. 13-70 and the P. 13-63 Seismic Design Requirements [4].

In P. 13-70, the P. 13-63 requirements have been reformulated to provide a more comprehensive view of the factors that influence the satisfactory performance of structures subjected to seismic action. General provisions have been included aimed at achieving structures with high ductility under repetitive loading. A provision has been included that allows the use, in special cases, of more advanced calculation procedures than those presented in the requirements.

The provisions included in the P. 13-63 document pertaining to the degree of seismicity assumed in calculations have been deleted, and the notion of degree of seismicity used therein has been discarded. Instead, the values of the seismic acceleration used in calculations (the coefficient k_s) have been specified as a function of the importance of the structure and the degree of seismic intensity of the zone in which the structure is located.

In the P.13-70 requirements, the values of k_s have been so chosen that the level of seismic protection is better balanced between the various seismic zones and the classes of importance of the structures. For zone 8, the value of $k_s = 0.05$ has been maintained. However, k_s has been changed from 0.025 to 0.03 in zone 7, and from 0.10 to 0.08 in zone 9.

In view of the sensitivity of precast structures to seismic loads, the P.13-70 requirements stipulate that, in zones believed not to be seismic, such structures shall be checked for a seismic load equal to at least the minimum value specified in the requirements.

The spectrum curve representing the coefficient β has been modified, the ratio between its maximum and its minimum value being reduced from 5 to 3.3.

On the basis of observations of the Mexico (1957), Niigata (1964) and Anchorage (1964) earthquakes, β has been increased for structures on poor foundation soils and decreased for structures on very stiff ground.

Provisions on limiting the effect of foundation rotation upon the natural periods of vibration have been modified on the basis of recent research.

Values of ψ have been specified for a larger number of structural types as functions of damping and ductility in accordance with results of U.S. research.

The limits within which it is necessary to consider higher vibration modes, and the criteria for the superposition of the latter, have been modified.

Additional criteria have been introduced pertaining to the vertical seismic loads. Vertical loads in zone 8 have been reduced by 20% to 60%. Seismic loads at supports and connections have been specified as functions of k_s. Additional seismic design provisions have been made for special industrial installations.

Provisions have been introduced for checking lateral displacements and other phenomena (cracking, accelerations, velocities).

Load combinations have been specified in a separate section (Appendix I of the P.13-70 requirements).

Additional provisions have been included in the P.13-70 requirements on the design for torsional effects induced by earthquake loads and on the superposition of horizontal and vertical seismic loads.

Provisions have been added that take into account post-elastic structural behavior and the favorable effect of ductility on structural performance under seismic loads. These provisions include the following: (1) At any given frame joint, the sum of moment capacities of columns shall exceed the sum of moment capacities of beams; (2) An increase in the horizontal shear force assumed to act on shear walls, to take into account the absence of ductility; (3) To ensure a ductile behavior of reinforced concrete members, higher grade concrete than was specified by the P. 13-63 requirements shall be used in all structures designed to withstand seismic loads. Also, reinforcing steel is specified that exhibits acceptable ductility. The reinforcement ratios are limited to moderate values. Reinforcement shall be provided in compression zones undergoing earthquake-induced stresses. It is also noted that the P. 13-70 provisions on spacing and sizes of ties are more severe than those of the P. 13-63 requirements.

Restrictions on the height of large panel prefabricated structures have been deleted. Provisions have been added aimed at improving the construction and detailing of prefabricated structures, of masonry structures, and on non-structural elements.

Provisions have been added pertaining to the design of tall structures, viz. (a) the first floor of tall structures in zone 8 shall be cast-in-place; (b) the design of structures with shear walls and a flexible ground floor shall be avoided.

4.5 Microzoning of Bucharest

Figure 4.2 shows the microzoning map of Bucharest which was issued in 1973 by the Romanian Institute of Standardization for use as a supplement to the existing mandatory seismic design regulations (see Section 4.2). The microzones in this map have been determined in accordance with the approach established by Medvedev [7], taking into consideration the seismo-geological features of the locality.

According to the seismic risk map of Romania shown in Figure 4.1, Bucharest falls within seismic zone 7. In microzoning the city, seismic zone increments are calculated on the basis of the depth of the hydrostatic level below the ground and seismic rigidity $V\rho$ of the terrain using the relation

$$C_s = 1.67 \log \frac{V_o \rho_o}{V_n \rho_n} + e^{-0.04h} \tag{4.5}$$

where C_s designates the seismic zone increment, V_o and V_n designate the speed of propagation of longitudinal waves in standard rock and soil under consideration, respectively, ρ_o and ρ_n are the respective densities of the corresponding materials, and h is the depth of the hydrostatic level below the ground surface. The standard material that is used as reference datum for soils is usually granite.

The microzoning supplement contains a clause permitting correction of the specifications of the standard, the designer assuming responsibility for such correction with the advice and consent of the client, in the event enough data is available that might lead to a different assessment of the local geological factors involved. In addition, it calls for seismological studies to be carried out at the site when designing structures of special importance in accordance with the technical prescriptive documents currently in force.

4.6 Summary of Seismic Provisions of the 1976 Uniform Building Code [8]

The total lateral seismic force assumed to act non-concurrently in the direction of each of the principal axes of a structure is:

$$V = ZIKCSW \tag{4.6}$$

where Z is equal to 3/16, 3/8, 3/4 and 1 in seismic zones 1, 2, 3 and 4, respectively; I, the occupancy importance factor is equal to 1.5, 1.25 and 1 for essential facitities, buildings where the primary occupancy is for assembly use for more than 300 persons in one room, and all other buildings, respectively, K is a coefficient depending upon type or arrangement of resisting elements (e.g., K = 1.33 for buildings with a box system; K=0.67 for buildings with a ductile moment resisting space frame with the capacity to resist the total required lateral force; K=2.5 for elevated tanks on four or more cross-braced legs and not supported by a building); C is a coefficient determined in accordance with the formula

$$C = \frac{1}{15\sqrt{T}} \leq 0.12 \tag{4.7}$$

T being the fundamental period of vibration of the building or structure in seconds in the direction under consideration; S is a coefficient determined in accordance with the formulas:

$$S = 1.0 + \frac{T}{T_s} - 0.5\left[\frac{T}{T_s}\right]^2 \quad \text{for } \frac{T}{T_s} \leq 1.0 \tag{4.8}$$

$$S = 1.2 + 0.6\frac{T}{T_s} - 0.3\left[\frac{T}{T_s}\right]^2 \quad \text{for } \frac{T}{T_s} > 1 \tag{4.9}$$

T_s being the characteristic site period ($0.5s \leq T \leq 2.5s$), and W is the total dead load plus the snow load if it exceeds 30 psf, plus 25 percent of the floor live load in case of storage and warehouse occupancies. Where

T_s is not properly established (see UBC Standard 23-1), S is taken as 1.5 except that if T > 2.5s, S may be determined by assuming T_s = 2.5s. The product CS need not exceed 0.14. The total lateral force V is distributed over the height of the structure as follows:

$$V = F_t + \sum_{i=1}^{n} F_i \tag{4.10}$$

Where F_t, the concentrated lateral force acting at the top structure is taken as 0 where $T > 0.7s$ and as $F_t = 0.07\ TV \leq 0.25V$ when $T \leq 0.7s$.

The remaining portion of the total base shear V shall be distributed over the height of the structure including level n according to

$$F_x = \frac{(V-F_t)\ w_x\ h_x}{\sum_{i=1}^{n} w_i\ h_i} \tag{4.11}$$

where w_i is the portion of w located at or is assigned to level i, and h_i is the height above the base to level i. Parts or portions of structures and their anchorages are designed for lateral forces in accordance with

$$F_p = ZIC_pSW_p \tag{4.12}$$

where W_p is the weight of the part or portion, and C_p is a coefficient depending upon the part or portion being considered (e.g., C_p = 0.20 for walls and partitions, C_p = 1.0 for cantilever parapet and ornamentation). If C_p = 1.0, the values of I and S need not exceed 1.0.

Figure 4.1. Seismic zone map of Romania.

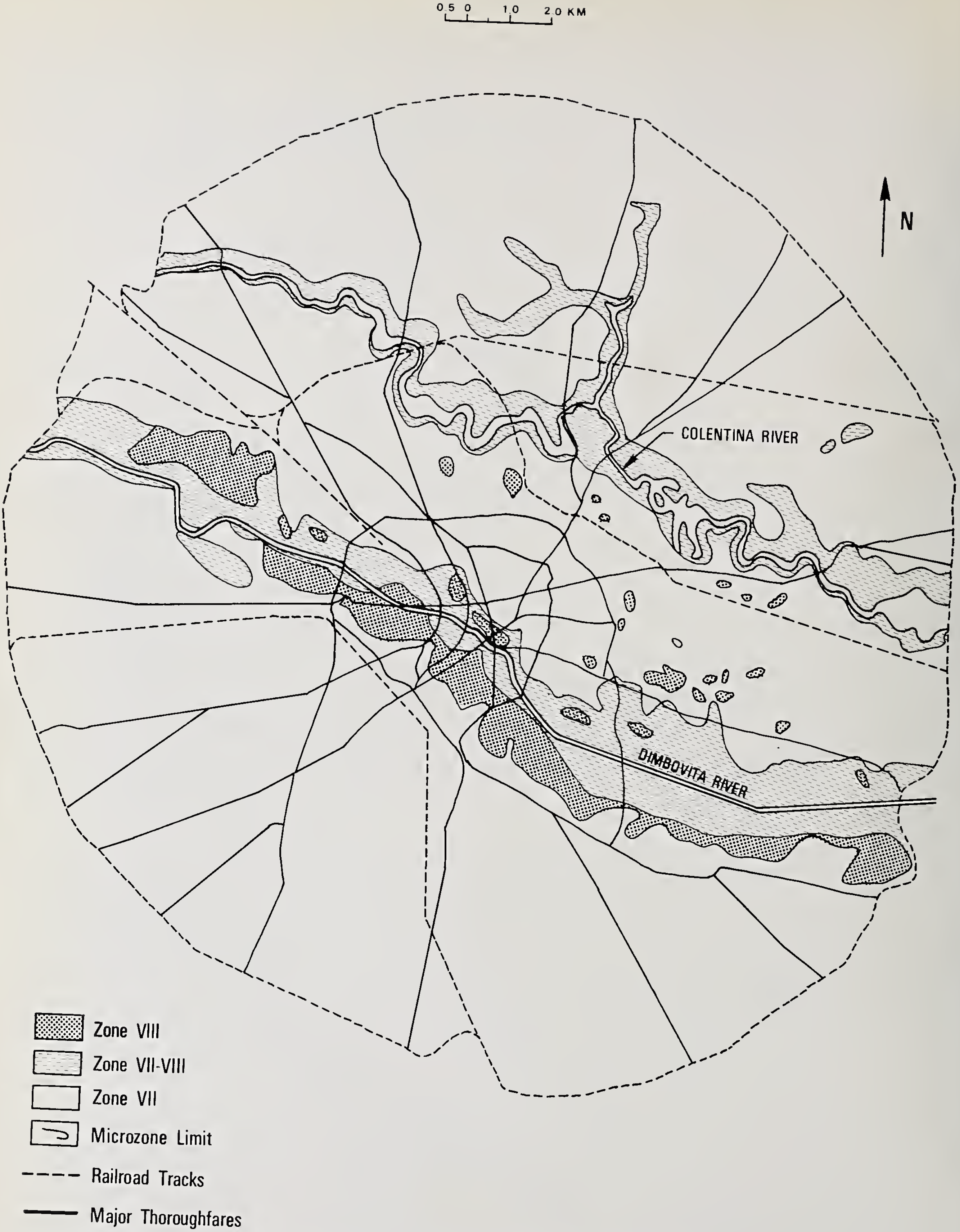


Figure 4.2. Microzone Map of Bucharest.

5. RECOMMENDATIONS

The authors developed a number of recommendations as a result of their observations of the building performance described in this report. Recommendations related to reconstruction of earthquake damaged buildings and the development of improved building practices for future construction are included.

RECOMMENDATION 1:

A SYSTEMATIC EVALUATION SHOULD BE MADE OF ALL BUILDINGS IN BUCHAREST ERECTED PRIOR TO THE ADOPTION OF EARTHQUAKE DESIGN REQUIREMENTS AND A HAZARD ABATEMENT PLAN SHOULD BE DEVELOPED.

Inspections of damaged buildings are already being carried out by teams of Romanian engineers. Quite naturally, these are concentrating on buildings in which the visible damage was sufficient to raise doubts about their safety. This program should be extended to include buildings which did not suffer apparent extensive damage. Particular attention should be paid to the possibility of cumulative damage particularly progressive deterioration of concrete with time. Such a program could mitigate the damages produced by future earthquakes.

RECOMMENDATION 2:

CONSIDERATION SHOULD BE GIVEN TO THE INCLUSION IN THE ROMANIAN DESIGN REQUIREMENTS OF MODERN PROVISIONS FOR THE PREVENTION OF PROGRESSIVE COLLAPSE.

The design of the computer building discussed in Chapter 3 illustrates the need for such requirements. The building collapsed as a result of column failures. However, failure of any one of the columns, particularly the central column, was probably sufficient to incapacitate the structure beyond repair. Had the design of the building incorporated, for instance, concepts for the containment of local failures or for the provision of alternate load paths to the foundation, it is possible that the extent of the failure would have been significantly less.

RECOMMENDATION 3:

DESIGN REQUIREMENTS FOR BUILDING SEPARATIONS SHOULD BE REVIEWED AND POSSIBLY IMPROVED.

The frequency of occurrence and extent of pounding damage resulting from out-of-phase vibrations of adjacent buildings raise questions about the adequacy of existing requirements for building separation. The damage was observed in older buildings as well as those constructed under current design requirements. Construction practices should also be reviewed to ensure field compliance of building separation requirements specified by design.

RECOMMENDATION 4:

ANALYTICAL AND EXPERIMENTAL STUDIES SHOULD BE CARRIED OUT TO ASCERTAIN THE BEHAVIOR OF THE PRECAST PANEL BUILDINGS SUBJECTED TO THE EARTHQUAKE OF MARCH 4 IN BUCHAREST.

One of the advantages of precast panel building systems is the speed of erection in the field [9]. Despite the fact that large panel systems have been extensively utilized in Europe in the construction of apartment buildings, a recent review indicates a general paucity of experimental information on the subject [10]. The performance of the panel buildings in Bucharest should therefore be studied in detail. Although the strong motion record for the earthquake would seem to indicate that these buildings were not subjected to significant forces, this should be verified by digitizing the record and carrying out analytical time history studies. This is particularly necessary before any conclusions are drawn regarding the possible reassessment of the design requirements for panel builings. These buildings should also be carefully inspected. Vibration tests could be carried out to determine the natural periods for the buildings and comparisons made with pre-earthquake measurements or calculations. These comparisons could reveal the extent of any internal damage not immediately apparent through visual inspection.

RECOMMENDATION 5:

A COOPERATIVE RESEARCH PROGRAM SHOULD BE INITIATED BETWEEN THE UNITED STATES AND ROMANIA IN THE FIELD OF LARGE PANEL STRUCTURES.

In view of the potential for increased use of precast panel construction, more data is needed on the performance of these systems under earthquake loading. The behavior of the joints between the panels, their ductile capacity and energy dissipative characteristics should be studied. A cooperative research program could take advantage of the Romanian construction experience with panel buildings, laboratory data on their dynamic performance available at INCERC and the analytical and experimental research facilities available in the U.S.

RECOMMENDATION 6:

INCREASED ATTENTION SHOULD BE PAID TO CONSTRUCTION QUALITY CONTROL IN ROMANIA.

The damage produced by the earthquake clearly indicated the need for improved construction practices. Inadequacies in the amount of reinforcement and cover, and in the spacing and anchorage of stirrups and ties provided in concrete structures, and shortcomings in workmanship and the quality and quantity of materials used in masonry construction are a few examples.

RECOMMENDATION 7:

THE RESPONSE SPECTRUM USED FOR DESIGN PURPOSES IN ROMANIA SHOULD BE REEXAMINED.

The accelerograph record from the earthquake of March 4 would seem to indicate a concentration of energy in the frequency range between 1 and 1.5 sec. Romanian engineers indicated that this also appeared to be the case for previous earthquakes. Available strong motion records could be used to generate response spectra and statistical techniques employed to select an appropriate design spectrum from the results [11].

RECOMMENDATION 8:

THE STRENGTHENING AND REPAIR PROCEDURES ADOPTED FOR DAMAGED BUILDINGS SHOULD BE AIMED AT REDUCING LARGE DIFFERENCES AND DISCONTINUITIES IN STRENGTH AND STIFFNESS BETWEEN ADJACENT FLOORS.

In many cases, building damage occurred as a result of large differences in stiffness between the first and upper floors. The soft story behavior observed in previous earthquakes was apparent. Repair techniques involving only strengthening of the damaged elements in the lower stories will not eliminate potential recurrence of a similar situation in future earthquakes. The repair procedure should provide for increasing the stiffness of the lower stories to make it more consistent with that of the stories above.

6. ACKNOWLEDGMENTS

The information presented in this report was obtained by the National Bureau of Standards while participating as part of a technical assistance team sent to Romania by the Agency for International Development (AID). The authors appreciated the opportunity to be a part of the team. Special recognition is due AID for the initiative and foresight demonstrated in assembling this team to provide assistance to Romania and to gather scientific and technical data on damages caused by the earthquake of March 4, 1977. Staff from the Office of Foreign Disaster Assistance, AID, including E.E. Anderson, Christian Holmes, William Dalton and Weston Emery were very helpful in making necessary arrangements for the team.

Staff from the Building Research Institute (INCERC) in Bucharest, including R.T. Constantinescu, Scientific Director of the Institute and G. Serbanescu, Head of the Structural Dynamics and Earthquake Engineering Laboratory, were especially helpful in making arrangements for visiting damaged buildings, setting up meetings with design engineers responsible for reconstruction and providing background material on building design and construction. Adriana Mihalus provided translation assistance and served as a guide for the team.

Sidney Smith, Science Liaison Attache for the U.S. Embassy in Bucharest provided assistance in obtaining technical information from the Government of Romania on the earthquake damage to buildings. The authors appreciate the assistance and hospitality of the United States Ambassador to Romania, Harry Barnes.

Many of the photographs and all the structural drawings used in this report were supplied by Romanian research, design and construction organizations.

Ann Heffernan and Linda Sacchet of the Office of Housing and Building Technology typed the several manuscripts of the report.

7. REFERENCES

1. Beles, Aurel A. and Ifrim, Mihail, "Engineering Aspects of Earthquakes in Romania in the Light of Modern Investigations," Proceedings of the Second World Conference on Earthquake Engineering, Volume III, Gakujutsu Bunken FukyuKai, Tokyo, Japan, Second Edition, 1965.

2. Von Hake, Carl A., Meyers, Herbert, "Earthquake Data File Summary," Environmental Data Service, National Oceanic and Atmospheric Administration, Boulder, Colorado, May 1976.

3. Niculescu, Christea, "Die Gebaudeschaden des Bukarester Erdbebens vom 10 November 1940." Die Bautechnik, Heft 30/31, pp. 321-326, July 18, 1941, Berlin.

4. Earthquake Resistant Regulations A World List 1973, International Association for Earthquake Engineering, c/o International Institute of Seismology and Earthquake Engineering, Building Research Institute, Tokyo, Japan, April 1973.

5. Titaru E., and Cismigiu A., "On the Romanian General Design Specifications for Civil and Industrial Buildings in Seismic Areas. Examples," Proceedings of the Second World Conference on Earthquake Engineering, Volume III, Gakajutsu Bunken Fukyu-Kai, Tokyo, Japan, Second Edition, 1965.

6. Provisions For The Design of Civil and Industrial Constructions in Seismic Regions, P. 13-70, Colectia de normative si instructiuni, No. 44, Redactia Publicatiilor pentru Constructii, Bucharest, 1970 (in Romanian).

7. Medvedev, S.V., Engineering Seismology, Gosudarstvennoe Izdatel'stvo Literatury po Stroitel'stvu, Arckhitekture i Stroitel'nym Materialam, Moscow, U.S.S.R., 1962 (in Russian). (English translation published by Israel Program for Scientific Translations, Jerusalem, 1965, available through U.S. Department of Commerce Clearinghouse for Federal and Technical Information, Springfield, Virginia 22151).

8. Uniform Building Code, 1976 Edition, International Conference of Building Officials, Whittier, California, June 1976.

9. "Ten Story Building Goes Up in 27 Days," Engineering News Record, April 14, 1977, pp. 99.

10. Zeck, Una Irelan, "Joints in Large Panel Precast Concrete Structures," Publication No. R76-16, Order No. 535, Massachusetts Institute of Technology, Cambridge, Massachusetts, January 1976.

11. Newmark, N.M., Blume, J.A., and Kapur, K.K., "Seismic Design Criteria for Nuclear Power Plants," Journal of the Power Division, ASCE, Volume 99, 1973, pp. 287-303.

12. Romanian Earthquake Disaster and Proposed A.I.D. Rehabilitation Program, A.I.D. Assessment Team Report, May 9, 1977.

APPENDIX A

COMPOSITION OF SURVEY TEAM AND SUMMARY OF ACTIVITIES

Members of the U.S. technical team which visited Romania following the earthquake of March 4, 1977 included:

National Bureau of Standards

Charles Culver (Team Leader)
Disaster Research Coordinator
Center for Building Technology

George Fattal
Research Structural Engineer
Center for Building Technology

U.S. Geological Survey

S.T. Algermissen
Branch of Earthquake Hazards

T. Leslie Youd
Research Civil Engineer
Branch of Engineering Geology

Christopher Rojahn (joined team on 3/16/77)
Research Civil Engineer
Seismic Engineering Branch

Karl Steinbrugge
Consultant to U.S. Geological Survey

Bureau of Reclamation

Gerald F. Bowles
Concrete Dams Section

Frederick O. Ruud
Head, Special Studies and Testing Section
Mechanical Branch, Design Division

Corps of Engineers

Ernest L. Dodson
Chief, Soil Mechanics Branch
Office, Chief of Engineers, U.S. Army

The following account summarizes the activities of the U.S. team during their visit to Romania from March 14, 1977 through March 22, 1977:

March 14, 1977 - Team arrived in Bucharest. Met by Ambassador H.G. Barnes and Science Attache Sidney Smith. Established residence at Nuclear Physics Institute at Magurele, 15 km south of Bucharest. Briefed by Chris Rojahn, USGS, on activities of other U.S. investigators from National Academy of Sciences and Earthquake Engineering Research Institute.

March 15, 1977 - Briefed by Romanian seismologists on the details of the March 4 earthquake. Met with Head of National Council of Science and Technology and representatives from Building Research Institute and the Design Institute (ICPT). Visited damaged and collapsed buildings in central portion of Bucharest.

March 16, 1977 - Team again met with National Council of Science and Technology to discuss our work plans. NBS team members later met with representatives from ICPT and discussed the types of building construction in Bucharest. Also visited damaged apartment building. USGS team members (Algermissen, Steinbrugge) began collecting data for an isoseismal map by surveying damage pattern throughout the city. Youd visited INCERC and discussed the geotechnical program underway there. BuRec and Corps of Engineers team members met with Romanian counterparts concerned with dam safety and planned site visits to dams and hydroelectric plants. (Description of groups itinerary is not included in this report).

March 17, 1977 - NBS team members and Youd visited Craiova and surveyed damage to municipal building, museum and apartment complex. Met with local engineers and discussed potential schemes for damage repair and rehabilitating structures which had not been designed for earthquakes. USGS team members continued damage pattern survey in Bucharest.

March 18, 1977 - NBS team members visited modern buildings in Bucharest near INCERC which had been damaged by the earthquake. Also visited several large panel buildings in western portion of city which had not been damaged. Youd met with representatives from ICPT and discussed behavior of building foundations during the earthquake. USGS team members continued damage pattern survey in Bucharest.

March 19, 1977 - NBS team visited INCERC and discussed earthquake related building research being carried out in Romania. NBS and USGS team members met with U.S. Embassy personnel to discuss the earthquake and answer questions of concern to them. Youd visited epicentral area and Focsani checking rivers for evidence of liquefication and observing building damage in villages.

March 20, 1977 - NBS and USGS team members departed Romania.

Youd observed building performance and geotechnical effects of the earthquake in the following cities: Galati, Braila, Chiscani, Slobozia.

March 21, 1977 - Youd met with representatives from INCERC and discussed elements for inclusion in a possible Romanian-American cooperative geotechnical program.

March 22, 1977 - Youd departed Romania.

APPENDIX B

TEAM CONTACTS IN ROMANIA

During their visit, the NBS team members made contact with the following individuals in Romania:

U.S. Embassy

Harry G. Barnes - U.S. Ambassador to Romania
Richard N. Viets - Deputy Chief of Mission
Richard Scissors - Economic Affairs Officer
Sidney G. Smith - Science Liaison Attache

Building Research Institute (INCERC)

Romulus T. Constantinescu
Horea Sandi
G. Serbanescu
T. Zorapapel
A. Mihalus (Interpreter)

Consiliul National pentru Stiinta Technologie

Ioan Ursu, Head, National Council of Science and Technology
George Filipas

ICPT

Anatolie Cazacliu
Petru Vernescu

Center for Earth Physics and Seismology

Ion Cornia

United Nations Development Programme

B.R. Devarajan

APPENDIX C

SUMMARY OF ROMANIAN EARTHQUAKES

Obtained from Earthquake Data File
Environmental Data Service
National Oceanic and Atmospheric Administration

Date	Time GMT	Latitude (North)	Longitude (East)	Depth (km)	Magnitude			Max. Modified Mercalli Intensity*
					Body	Surface	Other	
Oct. 6, 1908	21:39:48.0	45.50	26.50	150			6.75	VIII
May 25, 1912	18:01:42.0	45.75	27.25	100			6.00	VIII
Jun 14, 1913	09:33:12.0	43.50	25.50				6.75	X
Jan 26, 1916	07:38:00.0	46.00	24.00				6.50	IX
Nov 23, 1928	04:23:30.0	47.50	30.00				5.60	VI
Nov 1, 1929	06:57:21.0	45.90	26.50	160			5.75	VII
Mar 29, 1934	20:06:51.0	45.75	26.50	150			6.25	VIII
Jul 13, 1935	00:03:46.0	46.00	26.25	150			5.25	VI
Jul 13, 1938	20:15:17.0	45.75	26.75	150			5.25	VI
Sep 5, 1939	06:02:02.0	45.75	26.50	150			5.25	VI
Jun 24, 1940	09:57:24.0	45.75	26.75	150			5.50	VI
Oct 22, 1940	06:37:00.0	45.75	26.50	150			6.50	VIII
Nov 8, 1940	12:00:44.0	45.50	26.00	150			5.50	VI
Nov 10, 1940	01:39:09.0	45.75	26.50				7.40	X
Nov 11, 1940	06:34:16.0	46.00	26.75	150			5.50	VI
Nov 19, 1940	20:27:12.0	46.00	26.50	150			5.25	VI
Sep 7, 1945	15:48:22.0	46.00	26.75	100			6.50	VIII
Dec 9, 1945	06:08:45.0	45.00	26.50	100			6.00	VII
Nov 3, 1946	18:47:01.0	45.75	26.50	150			5.50	VI
May 29, 1948	04:48:55.0	46.00	26.75	150			5.75	VI
Jul 14, 1949	11:09:52.0	43.50	20.50					VII
Jan 16, 1950	04:25:01.0	45.70	26.80	128				IV
May 10, 1950	02:08:45.0	47.50	26.70					VI
Jun 20, 1950	01:18:36.0	46.00	27.00					V
Jul 14, 1950	06:29:51.0	45.70	26.80					V
Mar 18, 1951	11:32:27.0	46.20	26.60					V
Jan 16, 1952	23:54:31.0	45.70	26.80					
Jun 3, 1952	05:53:22.0	45.70	26.80					
Aug 3, 1952	16:36:15.0	45.70	26.80	160				
May 17, 1953	02:33:52.0	46.00	26.50					
Nov 16, 1953	15:37:54.0	45.25	20.00					VI

Date	Time GMT	Latitude (North)	Longitude (East)	Depth (km)	Magnitude			Max. Modified Mercalli Intensity*
					Body	Surface	Other	
Oct 21, 1954	12:03:18.0	45.00	26.50					
May 1, 1955	21:22:53.0	45.50	26.50	128				V
Jun 28, 1955	07:14:07.0	44.00	20.50					V
Jul 10, 1955	11:37:54.0	44.00	20.50					V
Mar 11, 1956	15:05:54.0	45.60	26.70	130				
Apr 18, 1956	12:52:26.0	46.23	27.66				4.75	
May 7, 1956	03:54:12.0	45.90	26.96	130				
Jun 30, 1956	01:50:20.0	44.00	29.00					
Nov 4, 1956	01:23:55.0	45.67	26.85					
Nov 18, 1956	16:02:30.0	45.72	26.76	160				
Dec 14, 1956	00:11:17.0	47.60	20.60					
Jun 29, 1957	23:32:00.0	45.68	26.75	130				
Dec 2, 1957	04:21:57.0	45.75	26.30	150				
Dec 16, 1957	04:49:59.0	43.25	20.50					
Dec 16, 1957	05:06:28.0	43.25	20.50					
Dec 23, 1957	23:38:36.0	45.39	26.88					
Feb 14, 1958	22:28:58.0	44.25	20.75					V
Mar 27, 1958	17:20:18.0	45.75	26.75					
Jun 9, 1958	18:47:12.0	45.75	26.75					
Jun 25, 1958	07:22:06.0	45.75	27.00					
Jul 24, 1958	23:03:15.0	45.90	26.60	150				
Aug 1, 1958	02:11:25.0	45.75	26.50	150				
Aug 9, 1958	09:34:24.0	43.10	20.80				4.00	VI
Aug 10, 1958	12:37:43.0	43.10	20.80					
Nov 11, 1958	23:07:10.0	45.50	27.10					
Feb 16, 1959	14:13:52.0	45.75	26.75					III
Apr 16, 1959	11:01:36.0	45.90	26.60	150				
Apr 29, 1959	01:35:29.0	46.13	26.22					V
May 27, 1959	20:38:25.0	45.72	20.96					VIII
May 27, 1959	21:46:16.0	45.75	21.25					
May 28, 1959	02:01:42.0	45.75	21.25					
May 28, 1959	06:10:30.0	45.75	21.25					
May 31, 1959	12:15:43.0	45.89	27.39					VI
Jun 26, 1959	13:44:40.0	45.86	26.53					
Jun 30, 1959	07:26:20.0	45.41	27.29					
Jul 12, 1959	05:18:06.0	43.00	21.50					
Jul 22, 1959	03:01:30.0	45.96	25.92	150			4.00	
Aug 2, 1959	03:33:06.0	45.90	26.90	150			4.28	
Aug 19, 1959	15:32:07.0	45.86	24.52	196				
Aug 29, 1959	19:34:10.0	43.80	20.50					

Date	Time GMT	Latitude (North)	Longitude (East)	Depth (km)	Magnitude			Max. Modified Mercalli Intensity*
					Body	Surface	Other	
Oct 1, 1959	16:04:52.0	46.00	26.90	160				
Nov 10, 1959	18:02:36.0	46.00	26.90	160			4.82	
Jan 4, 1960	12:51:52.0	45.00	27.00				5.90	
Jan 5, 1960	06:07:33.0	46.00	26.90	160				
Jan 26, 1960	20:27:05.0	46.00	26.50	150			5.60	
Feb 21, 1960	11:47:17.0	45.75	21.00					
Feb 26, 1960	13:33:42.0	45.40	26.10	110				
Apr 28, 1960	19:47:18.0	45.50	25.25					
Apr 30, 1960	01:54:12.0	45.75	26.75					
Jun 18, 1960	23:16:18.0	45.75	26.25					
Oct 13, 1960	02:21:12.7	45.40	25.80	063				
Oct 22, 1960	19:17:47.9	45.90	21.20	025				
Oct 24, 1960	15:46:00.0	45.75	20.00					
Jun 11, 1961	17:06:15.0	46.00	27.00	150			3.50	
Jun 29, 1961	18:08:48.0	45.50	26.60	100			4.70	
Nov 18, 1961	03:18:44.2	45.50	26.70	100				
Feb 27, 1962	21:34:10.8	45.70	26.40	146				
Aug 30, 1962	07:46:27.1	45.50	26.00	108				
Nov 9, 1962	02:14:47.4	45.70	26.70	129				
Jan 14, 1963	18:33:24.2	45.90	26.70	117				
Jun 17, 1964	13:38:15.9	45.70	26.50	145				
Jan 10, 1965	02:52:23.9	45.80	26.60	128	5.30			
Apr 12, 1965	19:14:28.1	45.10	25.70	060	4.10			
May 11, 1965	22:35:59.3	45.90	26.90	084	4.40			
Sep 16, 1965	00:40:11.6	46.10	27.10	045	4.20			
Dec 25, 1965	10:17:55.3	43.00	20.80					
Jan 18, 1966	20:20:25.0	46.00	26.80	069	4.40			
Jun 28, 1966	00:01:32.9	45.60	26.40	158	4.20			
Sep 4, 1966	01:29:29.6	45.70	26.60	136	4.10			
Oct 2, 1966	11:21:44.8	45.70	26.50	140	5.20			
Oct 15, 1966	06:59:18.9	45.60	26.40	140	4.90			
Oct 16, 1966	02:39:50.8	45.80	26.50	129				
Dec 14, 1966	14:49:59.4	45.70	26.40	150	4.90			
Dec 29, 1966	06:30:02.1	45.50	26.40	129	4.30			
Feb 27, 1967	21:00:42.6	44.84	26.60	046	4.90			
Mar 5, 1967	17:22:54.5	45.81	26.81	131	4.40			
Mar 5, 1967	18:54:20.5	45.28	25.95	064	4.10			
Apr 4, 1967	18:06:06.5	45.73	26.36	161	4.60			
May 13, 1967	20:04:29.6	45.67	26.81	144				
May 26, 1967	17:33:00.3	45.39	26.13	162	4.20			
Jul 25, 1967	12:33:23.5	45.80	26.50	146	3.90			

Date	Time GMT	Latitude (North)	Longitude (East)	Depth (km)	Magnitude Body	Magnitude Surface	Magnitude Other	Max. Modified Mercalli Intensity*
Jan 6, 1968	10:23:49.1	45.80	26.60	163	4.60			
Feb 9, 1968	13:22:53.9	45.60	26.40	122	4.60			
Feb 24, 1968	13:23:53.4	45.80	26.60	134	4.40			
Mar 15, 1968	22:56:34.6	43.80	20.50	033	4.20			
Aug 14, 1968	15:47:01.3	45.70	26.50	128	4.30			
Sep 21, 1968	11:05:52.9	45.70	26.59	128	4.30			
Oct 20, 1968	23:15:04.0	45.72	26.57	123	4.60			
Nov 20, 1968	01:51:13.9	45.72	26.80	110	4.00			
Nov 26, 1968	09:53:49.4	45.70	28.10	028	4.40			
Jan 15, 1969	08:46:29.4	45.56	26.41	135	4.50			
Apr 12, 1969	20:38:39.6	45.25	25.02	008	5.20			IV
Jul 27, 1969	09:01:28.1	45.65	26.43	163	4.20			
Dec 21, 1969	19:06:22.2	45.56	26.93	034	4.60			
Jan 2, 1970	07:31:37.9	45.46	26.30	134	4.40			
May 30, 1970	14:22:54.5	46.13	27.11	059	4.00			
Jun 5, 1970	12:00:32.9	45.65	26.61	129	4.40			
Jul 9, 1970	21:08:18.5	45.70	26.47	143	4.60			
Jul 10, 1970	14:18:58.8	47.72	25.63	033	4.70			
Jul 18, 1971	16:18:22.8	45.71	26.31	137	4.60			
Jul 22, 1971	10:37:15.8	45.57	26.13	128	4.20			
Sep 8, 1971	04:10:18.2	45.78	27.02	140	3.40			
Apr 16, 1972	00:03:31.7	45.52	26.43	136	4.60			
Jun 28, 1972	01:43:56.5	43.04	20.48	033	4.90			
Aug 23, 1972	18:00:31.3	45.84	26.77	082	4.70			
Oct 1, 1972	00:56:25.5	45.79	26.17	155	4.60			
Oct 1, 1972	04:32:00.2	43.48	21.46	003	4.90	4.3		
Dec 25, 1972	12:53:16.4	45.80	26.73	132	4.30			
Jan 5, 1973	12:37:47.2	45.55	26.58	131	4.40			
Jan 25, 1973	05:29:10.2	45.69	26.27	156	3.80			
Mar 12, 1973	09:55:46.7	45.57	26.23	138	3.70			
Mar 31, 1973	23:34:07.9	45.72	26.65	159	4.10			
Jul 19, 1973	10:51:19.0	43.21	22.25	010				
Aug 20, 1973	15:18:28.3	45.73	26.47	073	5.60			
Aug 23, 1973	14:52:42.3	45.66	21.11	039				
Aug 23, 1973	16:56:26.9	45.69	21.11	051				
Aug 23, 1973	18:23:43.8	46.30	21.51	086				
Sep 7, 1973	19:37:51.8	45.79	26.48	140	4.60			
Oct 23, 1973	10:50:58.6	45.68	26.49	174	4.90			
Oct 25, 1973	01:19:16.6	45.54	26.52	137	3.80			
Dec 21, 1973	02:46:36.8	45.55	26.47	172	4.00			

Date	Time GMT	Latitude (North)	Longitude (East)	Depth (km)	Magnitude			Max. Modified Mercalli Intensity*
					Body	Surface	Other	
Jan 5, 1974	04:28:03.9	45.43	25.67	090	3.90			
Feb 22, 1974	13:40:48.9	45.64	26.30	149	4.20			
Apr 4, 1974	10:26:07.8	46.08	27.44	033				
Apr 17, 1974	01:31:33.9	45.97	21.14	033	5.60			
Jun 10, 1974	05:11:00.3	45.64	26.46	167	4.20			
Jul 17, 1974	05:09:23.0	45.75	26.53	145	5.10			
Jul 26, 1974	11:29:14.6	43.62	22.01	033	3.80			VI
Aug 30, 1974	17:42:59.4	44.17	20.37	033				
Feb 8, 1975	08:21:18.0	45.31	25.99	023	4.60			
Feb 25, 1975	02:42:47.4	45.69	26.45	147	4.10			
Mar 2, 1975	13:21:13.5	45.81	27.14	033	3.80			
Mar 7, 1975	04:13:05.1	45.85	26.62	021	4.90	4.5		
Mar 8, 1975	16:39:24.8	45.77	26.58	146	4.20			
Mar 31, 1975	08:28:46.2	45.63	26.35	140	4.70			
May 13, 1975	12:18:55.1	45.60	26.73	144	3.0			
May 26, 1975	18:14:22.3	45.60	26.99	062	3.0			
May 26, 1975	22:01:42.6	45.52	26.96	050	3.8			
Sep 5, 1975	00:39:22.9	45.68	26.63	155	4.0			
Oct 6, 1975	21:16:54.3	45.61	26.59	088				
Feb 3, 1976	13:29:16.4	45.31	25.99	184	3.7			

*Intensity Data, 1901-1955 from "Seismicity of the European Area" by Vit Karnik, D. Reidel Publishing Co., Dordrecht, Holland 1969.

☆ U. S. GOVERNMENT PRINTING OFFICE : 1977--240-848/314

NBS-114A (REV. 7-73)

U.S. DEPT. OF COMM. BIBLIOGRAPHIC DATA SHEET	1. PUBLICATION OR REPORT NO. NBS SP 490	2. Gov't Accession No.	3. Recipient's Accession No.

4. TITLE AND SUBTITLE	5. Publication Date
Observations on the Behavior of Buildings in the Romania Earthquake of March 4, 1977	September 1977
	6. Performing Organization Code: Center for Build. Tech.
7. AUTHOR(S) George Fattal, Emil Simiu and Charles Culver	8. Performing Organ. Report No.
9. PERFORMING ORGANIZATION NAME AND ADDRESS NATIONAL BUREAU OF STANDARDS DEPARTMENT OF COMMERCE WASHINGTON, D.C. 20234	10. Project/Task/Work Unit No. 11. Contract/Grant No.
12. Sponsoring Organization Name and Complete Address (Street, City, State, ZIP) Office of Foreign Disaster Assistance, AID Department of State Washington, DC 20523	13. Type of Report & Period Covered: Final 14. Sponsoring Agency Code: AID

15. SUPPLEMENTARY NOTES Library of Congress Catalog Card Number: 77-600035
Special report of observations by technical team sent to Romania by the Department of State at the request of the Government of Romania following the earthquake of March 4, 1977.

16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

Observations are presented of the damage to buildings resulting from the earthquake of March 4, 1977 in Romania. The report was prepared by engineers from the National Bureau of Standards who participated as members of the U.S. government team dispatched to Romania under the auspices of the Office of Foreign Disaster Assistance, Agency for International Development. A summary of the team's activities is included. Background data on the seismic history of Romania, the characteristics of the earthquake and descriptions of damage to specific buildings are also included. The types of building construction and the history of the development of seismic design requirements for buildings in Romania are discussed. Recommendations are presented for needed building research based on the observations.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)*

Buildings; building codes; earthquakes; natural disasters; structural engineering.

18. AVAILABILITY [X] Unlimited	19. SECURITY CLASS (THIS REPORT)	21. NO. OF PAGES
[ ] For Official Distribution. Do Not Release to NTIS	UNCLASSIFIED	168
[ ] Order From Sup. of Doc., U.S. Government Printing Office Washington, D.C. 20402, SD Cat. No. C13.10:490	20. SECURITY CLASS (THIS PAGE)	22. Price
[ ] Order From National Technical Information Service (NTIS) Springfield, Virginia 22151	UNCLASSIFIED	$3.00

USCOMM-DC 29042-P74

NBS TECHNICAL PUBLICATIONS

PERIODICALS

JOURNAL OF RESEARCH reports National Bureau of Standards research and development in physics, mathematics, and chemistry. It is published in two sections, available separately:

• **Physics and Chemistry (Section A)**
Papers of interest primarily to scien orking in these fields. This section covers a br ge of physical and chemical research, with r emphasis on standards of physical measur , fundamental constants, and properties of m ssued six times a year. Annual subscription: D c, $17.00; Foreign, $21.25.

• **Mathematical Sci Section B)**
Studies and com s designed mainly for the mathematician and etical physicist. Topics in mathematical statis eory of experiment design, numerical analysi retical physics and chemistry, logical design rogramming of computers and computer syst nort numerical tables. Issued quarterly. Annual su scription: Domestic, $9.00; Foreign, $11.25.

To begin publication as a single journal, July-August, 1977

DIMENSIONS/NBS (formerly Technical News Bulletin)—This monthly magazine is published to inform scientists, engineers, businessmen, industry, teachers, students, and consumers of the latest advances in science and technology, with primary emphasis on the work at NBS. The magazine highlights and reviews such issues as energy research, fire protection, building technology, metric conversion, pollution abatement, health and safety, and consumer product performance. In addition, it reports the results of Bureau programs in measurement standards and techniques, properties of matter and materials, engineering standards and services, instrumentation, and automatic data processing.

Annual subscription: Domestic, $12.50; Foreign, $15.65.

NONPERIODICALS

Monographs—Major contributions to the technical literature on various subjects related to the Bureau's scientific and technical activities.

Handbooks—Recommended codes of engineering and industrial practice (including safety codes) developed in cooperation with interested industries, professional organizations, and regulatory bodies.

Special Publications—Include proceedings of conferences sponsored by NBS, NBS annual reports, and other special publications appropriate to this grouping such as wall charts, pocket cards, and bibliographies.

Applied Mathematics Series—Mathematical tables, manuals, and studies of special interest to physicists, engineers, chemists, biologists, mathematicians, computer programmers, and others engaged in scientific and technical work.

National Standard Reference Data Series—Provides quantitative data on the physical and chemical properties of materials, compiled from the world's literature and critically evaluated. Developed under a world-wide program coordinated by NBS. Program under authority of National Standard Data Act (Public Law 90-396).

NOTE: At present the principal publication outlet for these data is the Journal of Physical and Chemical Reference Data (JPCRD) published quarterly for NBS by the American Chemical Society (ACS) and the American Institute of Physics (AIP). Subscriptions, reprints, and supplements available from ACS, 1155 Sixteenth St. N.W., Wash. D. C. 20056.

Building Science Series—Disseminates technical information developed at the Bureau on building materials, components, systems, and whole structures. The series presents research results, test methods, and performance criteria related to the structural and environmental functions and the durability and safety characteristics of building elements and systems.

Technical Notes—Studies or reports which are complete in themselves but restrictive in their treatment of a subject. Analogous to monographs but not so comprehensive in scope or definitive in treatment of the subject area. Often serve as a vehicle for final reports of work performed at NBS under the sponsorship of other government agencies.

Voluntary Product Standards—Developed under procedures published by the Department of Commerce in Part 10, Title 15, of the Code of Federal Regulations. The purpose of the standards is to establish nationally recognized requirements for products, and to provide all concerned interests with a basis for common understanding of the characteristics of the products. NBS administers this program as a supplement to the activities of the private sector standardizing organizations.

Consumer Information Series—Practical information, based on NBS research and experience, covering areas of interest to the consumer. Easily understandable language and illustrations provide useful background knowledge for shopping in today's technological marketplace.

Order **above** *NBS publications from: Superintendent of Documents, Government Printing Office, Washington, D.C. 20402.*

Order **following** *NBS publications—NBSIR's and FIPS from the National Technical Information Services, Springfield, Va. 22161.*

Federal Information Processing Standards Publications (FIPS PUBS)—Publications in this series collectively constitute the Federal Information Processing Standards Register. Register serves as the official source of information in the Federal Government regarding standards issued by NBS pursuant to the Federal Property and Administrative Services Act of 1949 as amended, Public Law 89-306 (79 Stat. 1127), and as implemented by Executive Order 11717 (38 FR 12315, dated May 11, 1973) and Part 6 of Title 15 CFR (Code of Federal Regulations).

NBS Interagency Reports (NBSIR)—A special series of interim or final reports on work performed by NBS for outside sponsors (both government and non-government). In general, initial distribution is handled by the sponsor; public distribution is by the National Technical Information Services (Springfield, Va. 22161) in paper copy or microfiche form.

BIBLIOGRAPHIC SUBSCRIPTION SERVICES

The following current-awareness and literature-survey bibliographies are issued periodically by the Bureau:

Cryogenic Data Center Current Awareness Service. A literature survey issued biweekly. Annual subscription: Domestic, $25.00; Foreign, $30.00.

Liquified Natural Gas. A literature survey issued quarterly. Annual subscription: $20.00.

Superconducting Devices and Materials. A literature survey issued quarterly. Annual subscription: $30.00.

Send subscription orders and remittances for the preceding bibliographic services to National Bureau of Standards, Cryogenic Data Center (275.02) Boulder, Colorado 80302.