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The Joint U.S.-Romanian Seminar on Earthquakes and Energy was held September 2-9,1985, at the Romanian Building Research Institute (INCERC) in Bucharest. During that week over seventy researchers from Romania and the United States met in intense and vivid discussions on topics of building research, with the focus on earthquakes and energy conservation. This volume contains the forty-one papers on the subject of earthquakes. They were presented in the following four sessions:

- Session I: Experience of Past Earthquakes. Performance of Buildings and Behavior of Occupants. Summary of Lessons. (eleven papers)

- Session II: Evaluation of the Existing Building Stock. Vulnerability and Risk Analysis. Repair and Strengthening of Structures. (twelve papers)

- Session III: Earthquakes Preparedness. Critical Facilities. Urban and Sociological Aspects. (six papers)

- Session IV: Structural Performance Under Earthquake Loadings. Structural Design. (twelve papers)

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SPONSORED BY THE NATIONAL SCIENCE FOUNDATION (U.S.) AND THE NATIONAL COUNCIL FOR SCIENCE AND TECHNOLOGY (ROMANIA)

BUCHAREST 2-9 SEPTEMBER 1985

VOLUME 2: EARTHQUAKES

ORGANIZING GROUPS:
ARCC
UCLA
INCERC

EDITORS:
SAMUEL ARONI
ROMULUS CONSTANTINESCU

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Joint U.S.-Romanian Seminar on Earthquakes
and Energy. Bucharest, 2-9 September, 1985.

Volume 1: Introduction and Summary
Volume 2: Earthquakes
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1.2

VOLUME 2: TABLE OF CONTENTS

INTRODUCTION

PAPERS:

I.1 S. Aroni, and M. E. Durkin :
Injuries and Occupant Behavior in Earthquakes. 3

I.2 H. Sandi :
The Romania Earthquake of March 4, 1977: Notes on the Effects, the Post-Earthquake Reaction, and the Future Action Needs. 41

I.3 P. Mazilu, M. Ieremia, M. Neicu, and L. Rosca :
Seismic Behavior of Some Industrial and Telecommunication Structures: Solutions for Seismic Risk Reduction. 67

I.4 C. Dalban, and E. Dragomirescu :
Some Conclusions on the Behavior of the Steel and Composite Structures Subjected to Seismic Action. 77

I.5 M. Mihailescu :
Dynamic Behavior of Spatial Structures and Shells During the March 4, 1977, Earthquake in Romania. 85

I.6 G. Sandulescu :
Considerations of the Causes of Collapse or Damage of Some Dwellings in Bucharest During the March 4, 1977, Earthquake. 92

I.7 D. Anastasescu, and R. Marinov :
Earthquake Protection of Existing Low-Rise Residential and Mixed-Occupancy Buildings. 99

I.8 A. Nestor, E. Iliescu, and G. Pitis :
Remarks and Conclusions Concerning the Effects of the 1977 Earthquake in the Prahova County: The Behavior of People and Buildings, Social and Technical Actions for Recovery After the Earthquake. 109

I.9 C. Mihai, S. Covali, and G. Palamaru :
Behavior of Residential Buildings in Jassy During the 1977 Vrancea Earthquake. 117

I.10 A. Mihalache, A. Liulica, and V. Mihailovici :
Design and Performance of Prefabricated Buildings Located at Sites with Difficult Ground Conditions During the March 4, 1977, Earthquake. 122

I.11 A. Mihalache :
Seismic Behavior of Dwellings with Coupled Shear Wall Structure Located at Sites with Difficult Foundation Conditions During the Earthquake of March 4, 1977. ...129

II.1 B.G.Jones, D.M.Manson, C.M.Hotchkiss, and M.J.Savonis :
Determination of Building Stock for Urban Vulnerability Analysis and Earthquake Loss Estimation. 143

II.2 R. Constantinescu :
Romanian Technology for Strengthening Reinforced Concrete Structures by Sheathing with Fiberglass Tissues Glued with Epoxy Resins. 168

II.3 H. Sandi :
Current Possibilities for Analysis and Mitigation of the Seismic Risk Affecting the Existing Building Stock. 178

II.4 H. Sandi, D. Cazacu, C. Constantinescu and M. Stancu :
A Summary of Studies on the Seismic Vulnerability of Buildings, Carried out in Bucharest Subsequent to the March 4, 1977, Earthquake. 194

II.5 C. Constantinescu, and M. Stancu :
Statistical Studies for the Prediction of Seismic Damage of Reinforced Concrete Structures. 231

II.6 M. Simonici :
Research on Strengthening Methods for Earthquake Damaged Masonry. 242

II.7 D. Vasilescu, and D. Diaconu :
Possibilities of Assessing Seismic Resistance of Old Buildings. 252

II.8 C. Mihai :
Observations Concerning Vulnerability and Seismic Risk of Residential Buildings of Medium Height. 265

II.9 V. Fierbinteanu, M. Balcu, D. Petrovici, and M. Dima :
Choice of Strengthening Solution by the Evaluation of Post-Shaking Strength Capacity of a Multistory Building Having a Steel Skeleton Structure and Brick Masonry Infilling. 272

II.10 M. Neicu, and Z. Apostol :
The Strengthening Strategy of Existing Buildings Damaged by Earthquakes. 285

II.11 M. Mironescu, A.Bortnowschi, T.Brotea, and A.Stanescu : Simplified and Medium Complexity Methods to Estimate the Resistance and Deformation Capacity of Existing Buildings. Case and Statistical Studies Using These Methods. 297

II.12 L. Neagoe, L. Baltateanu, and St. Mihailescu : Strengthening Solutions for Some Large Span Framed Structures. 312

III.1 H. J. Lagorio : Seismic Performance of Critical, Emergency Service Facilities. 323

III.2 S. Georgescu : Premises of a Romanian Earthquake Preparedness Program For Bucharest. 340

III.3 S. Georgescu, and D. Mandruta : Architectural, Engineering and Individual Reaction Elements Concerning the Opportunity to Evacuate Apartments During the Earthquake. 351

III.4 A. Cristescu : Town Planning Contribution for the Urban Vulnerability Mitigation. 364

III.5 D. Abraham, G. Gheorghe, N. Gheorghe, and M. Kivu : The Social Response to Earthquakes. 381

III.6 D. Abraham, A. Cristescu, G. Gheorghe, and M. Kivu : Social and Urban Aspects of Seismic Protection of Towns 388

IV.1 D. P. Abrams : Computation and Testing of Dynamic Response for Reinforced Concrete Structures. 4o1

IV.2 H. Sandi, M. Stancu, O. Stancu, and C. Constantinescu : A Biography of a Large-Span Structure, Pre- and Post-Earthquake, After the Provisional and Final Strengthening. 428

IV.3 S. Georgescu, and D. Radulescu : Architectural Demands and Limits Imposed by the Aseismic Structural Concept: Case Study Starting from the Damage Caused by the March 4, 1977, Vrancea Earthquake on a Building Type with Peculiar Architecture and Structure. 440

IV.4 D. Diaconu, D. Vasilescu, and M. Manolovici :
Important Aspects Concerning Seismic Response of Entirely Precast Structures for Single-Story and Multi-Story Industrial Halls. 458

IV.5 D. Diaconu, and P. Cosmulescu :
Aspects of Dynamic Interaction Between Structural and Nonstructural Members. 475

IV.6 D. Diaconu, D. Vasilescu, and S. Marinescu :
Peculiarities in the Seismic Behavior of Various Structural Systems Made of Entirely Precast Large Panels....488

IV.7 G. Sandulescu, and M. Decusara :
Seismic Risk Reduction Using Prestressing in Reinforced Concrete Structures. 500

IV.8 F. Dabija, D. Capatina, and S. Dabija :
Dynamic Analysis of Multistory Building Structures with Cores of Precast Components Assembled by Prestressing. 509

IV.9 D. Constantinescu :
Analysis of the Seismic Response of a 9-Story Large Panel Building 522

IV.10 D. Georgescu :
Spatial Interaction Effect on Behavior of Single-Story Mill Buildings Subjected to Seismic Action. 545

IV.11 D. M. Ghiocel, A. Popovici, and D. Ghiocel :
Seismic Risk Evaluation for Buildings Including Soil-Structure Interaction Effects....554

IV.12 G. Ciuhandu, and A. Mihaescu :
Studies Concerning the Behavior of Reinforced Concrete Shear Walls with Openings Under Alternating Horizontal Loading....562

UNITS OF MEASURE 573

INTRODUCTION

The Joint U.S.-Romanian Seminar on Earthquakes and Energy was held September 2-9,1985, at the Romanian Building Research Institute (INCERC) in Bucharest. The seminar was supported by the National Science Foundation (NSF) Division of International Programs (Grant No. INT 85-03889) and the Romanian National Council for Science and Technology (CNST), under the U.S.-Romanian Cooperative Science Program. During that week over seventy researchers from Romania and the United States met in intense and vivid discussions about topics of building research, with the focus on earthquakes and energy conservation. The seminar provided an opportunity for exchanging scientific research information, and encouraging the establishment and planning of future joint research cooperation.

The groundwork for the seminar began in the spring of 1984, when the Architectural Research Centers Consortium (ARCC) was searching for ways to further international cooperation. ARCC is an organization of over forty academic institutional members devoted to supporting and encouraging research in the fields of architecture and building. The membership of ARCC includes leading university-based research centers and has a broad geographical distribution within the United States. ARCC has also conducted seminars with other overseas researchers, including those in Sweden and the United Kingdom. The initiative for this seminar was taken by Professor Samuel Aroni of UCLA. With the encouragement and help of NSF, he traveled to Romania in September 1984 and met with Dr.Eng. Romulus Constantinescu of INCERC, the Romanian Building Research Institute. INCERC is a large and impressive research organization in Bucharest under the direction of the Central Institute for Research, Design, and Guidance in Civil Engineering (ICCPDC), established in 1950 and consisting of four sections and six laboratories. Earthquake research and energy conservation are prominent in its activities, representing two of its four sections. During the September 1984 visit, an agreement was reached to organize a seminar, to be held at INCERC in Bucharest as a first step of collaboration between Romanian and American institutions and researchers in areas of mutual interest. The two subjects selected were earthquake issues and energy conservation. In addition to their intrinsic importance, the decision to focus on both of them stemmed from the belief in the advantage of synergism. There is some interaction between them, and having two subjects for the seminar enhanced the possibilities of finding areas of future collaboration, and encouraged cross-fertilization of research ideas.

We are well aware of the earthquake dangers facing the Pacific coastline of the United States, as well as many other locations within the national boundaries. Significant research has been conducted and much more is needed. Among the important topics of recent research interest have been the problems of old buildings, their repair, strengthening and reconstruction, issues of vulnerability and risk of both buildings and lifelines, non-structural elements, performance of emergency facilities during earthquakes, issues of human behavior and injuries, and planning for earthquake preparedness and disaster mitigation. Romania is also located in a seismic region and suffered greatly from the earthquake of March 4, 1977, in which some 1,600 persons were killed, over 11,000 were injured, 33,000 buildings collapsed or were severely damaged, industrial facilities were seriously damaged, and damage totaled over $2 billion. There is much to be learned from this major earthquake, which has been studied in great detail by Romanian researchers and is the subject of a recent comprehensive Romanian book. The third most important event in the modern seismic history of Romania was the recent earthquake near Tulcea in the eastern part of the country, on November 13, 1981. The epicenter was near settlements which have developed rapidly in recent years, and the behavior of modern high-rise construction as well as the non-structural damage are of particular interest. The Romanian earthquakes are of special international importance because of the proximity of a large number of prefabricated industrialized buildings. This is probably the first time that such newer buildings have been subjected to major earthquakes on such a large scale, and their seismic behavior is of great interest. Serious seismic research in Romania has gone on for a long time at their Building Research Institute (INCERC), both in Bucharest and at the Jassy branch of the ICCPDC, where some of the earliest earthquake testing facilities, including shaking tables, were developed.

During the last ten years, energy conservation in buildings has been the subject of research interest in the United States. The use of solar energy, active and passive systems, utilization for hot water and space heating, and the upgrading of existing buildings have all been topics of both field work and research activity in both countries. Romania has also put an emphasis on energy conservation at a larger urban scale. Romanian solar installations during the last five years have included some 600 projects for hot water or space heating and some 14,000 apartments. The solar hot-water installation in Baneasa (Bucharest), consisting of 2239 apartments, is the largest in Europe and possibly in the world. Industrial applications include an interesting ice manufacturing plant using solar energy, and large projects for heat recovery from industry for storage and use by some 20,000 apartments for both hot water and space heating. A two-story experimental solar house, which includes four apartments, has been erected at INCERC for comparative research of active and passive systems for both space heating and hot water.

The seminar consisted of an opening session, followed by four working sessions, with the participants divided into two groups discussing earthquakes and energy respectively. The presentations and discussions on the subject of earthquakes covered the spectrum of seismic vulnerability and behavior of buildings, urban systems and critical facilities, as well as human behavior and injuries during earthquakes. Those dealing with energy conservation, included solar passive and active systems, retrofitting, daylight applications, total building performance, and problems of energy conservation at an urban scale. Each of the four working sessions, concentrating on a specific group of related topics, consisted of one or two presentations of American papers, a summary of the relevant Romanian papers presented by a rapporteur, and an open discussion. The seminar was enriched by field visits to a large scale solar installation and to the Jassy Seismic Testing Station and laboratory. The final day, devoted to research needs and areas of future cooperation, proved to be very fruitful and productive.

Romanian participants included engineers, architects, planners, and sociologists from INCERC and over a dozen other institutes, centers, laboratories, and universities throughout Romania. They prepared sixty-one papers, thirty-seven on the subject of earthquakes and twenty-four on energy topics. The American team consisted of nine academics, from seven different universities, each with a paper on earthquakes (four papers) or on energy (five). A bilingual program and abstracts of all the papers was prepared by INCERC and distributed at the seminar. The seminar was co-chaired by Dr. Constantinescu and Professor Aroni.

We would like to thank all those who in various ways contributed to the seminar and made it possible, including all the seminar participants. Mr. George Matache of CNST, and Eng. Valeriu Cristescu, the General Director of ICCPDC and INCERC, provided significant help and guidance. Eng. Emil Sever Georgescu of INCERC was of invaluable help in the seminar organization. The excellent work of a number of staff, and scientific translators at INCERC is gratefully acknowledged. Our gratitude is expressed to Dr. Gerson Sher, Ms. Bonnie H. Thompson, and Ms. Deborah L. Wince of the NSF Division of International Programs and to Dr.William Anderson and Mr. Gifford Albright of the NSF Directorate of Engineering for their support and assistance. In the United States , the seminar participants were selected with the help of an Advisory Committee consisting of Professor David S. Haviland (Dean, School of Architecture, Rensselaer Polytechnic Institute), Dr. Frederick Kringold

(Associate Dean for Research and Extension, College of Architecture and Urban Studies, Virginia Polytechnic Institute and State University, and President of ARC), and Professor Samuel Aroni.

The work of the joint seminar is presented in three volumes. Volume 1 contains an introduction and summary of all papers, sessions and discussions. Significant contributions were made in the writing of this volume by Professor Daniel Abrams, Professor Volker Hartkopf, Professor Henry Lagorio, Dr. Horea Sandi, Professor Robert Shibley, and Eng. Teodor Teretean. Volume 2 contains the forty-one papers on the subject of earthquakes, and volume 3 the twenty-nine papers on topics of energy. The editorial help of Mr. William Fulton, in the United States, is much appreciated. The reproduction of these volumes was performed at INCERC.

The American participants express sincere thanks and gratitude to the Romanian hosts for their outstanding hospitality, both scientifically and socially. Finally, we hope that this publication will prove to be useful and will further contribute to the achievements of the goals of the seminar.

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SESION I : EXPERIENCE OF PAST EARTHQUAKES. PERFORMANCE OF BUILDINGS AND BEHAVIOR OF OCCUPANTS. SUMMARY OF LESSONS.

I.1 INJURIES AND OCCUPANT BEHAVIOR IN EARTHQUAKES

Samuel Aroni *
Michael E. Durkin **

ABSTRACT

Life safety is the foundation of all earthquake hazard reduction measures. Therefore, we should base these measures on empirical evidence rather than speculation. Yet, until very recently, we had little hard information about the etiology of earthquake injuries in different U.S. building types.

This paper proposes a comprehensive conceptual framework for earthquake injuries. It presents preliminary results of an epidemiological study of the role of physical environment and occupant behavior in earthquake injuries, involving the injuries during the 1978 Santa Barbara, 1979 Imperial County, and 1983 Coalinga, California, earthquakes. We also describe our work in progress concerning the injuries in the 1985 Chile earthquake. For the past earthquakes we document, not only the type of injury, but also the physical agent responsible. We analyze the relationship of the injury to factors such as building type, damage level and personal characteristics. Of major importance was the behavior of the building occupants during the earthquake. We describe how specific actions either contributed to or helped to prevent earthquake injuries.

We review several previous studies of occupant behavior in past U.S. earthquakes. These involve the behavior of patients and staff in five hospitals heavily damaged in the 1971 San Fernando earthquake, and the response of office workers in the five story Imperial County Services Building in the 1979 Imperial County earthquake.

Finally, some suggestions are made for future research on the subject.

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INTRODUCTION

During the past two decades, over half a million people have been killed or injured in earthquakes. On July 28, 1976, the Tangshan, China earthquake killed 240,000 people and injured 164,000 more.(1) This year, the Chile earthquake of March 3, 1985, claimed 180 lives and at least 2,572 injured.(2)

The United States, due primarily to the absence of a large seismic event occurring in a heavily populated area and improved seismic engineering practices, so far has been spared the high mortality of other countries. According to the U.S. Geological Survey(3), as shown in Table 1, some 1,029 were killed by earthquakes in California during a period of 168 years. More specifically, in the 1906 San Francisco earthquake, 315 were killed outright, 352 were reported missing and about 400 were injured.(4) The 1964 Alaska earthquake was responsible for 130 deaths and 50 injuries.(5) By far, the largest number of casualties in a recent U.S. earthquake occurred in the San Fernando earthquake of 1971 when 58 died and 2,543 injured required hospital treatment and 3,000 more were treated at Red Cross first aid stations.(6) No fatalities occurred in the three California earthquakes discussed at length in this paper, 1978 Santa Barbara, 1979 Imperial County, and 1983 Coalinga, but there were at least 85,78, and 211 injuries, respectively.

What happens when our luck runs out? Today, nearly 20 million Californians live in zones of major earthquake risk. A 1980 FEMA report estimates that if a great earthquake hit the Newport-Inglewood fault in Southern California at 4:30 p.m. on a weekday, when many people are either inside or just leaving their office buildings, 23,000 people would be killed and 91,000 others would require hospitalization. If, on the other hand, this earthquake occurred in the early morning (i.e. at 2:30 a.m.) when most people are at home, the number of estimated casualties would drop to 4,000 dead and 18,000 hospitalized. In addition, the FEMA report estimates the number of injuries not requiring hospitalization to be from 15 to 30 times the number of death.(7) The estimated dollar loss in buildings and contents is $62 billion.

Even if the preceding estimates are high, it is clear that large numbers of injuries and fatalities can overwhelm available medical resources, as well as create long-term social and economic disruption unless we find a way to reduce them. It is also apparent that smaller versions of these catastrophic losses can result from moderate earthquakes which are certain to strike California and other seismically active areas in the United States during the forseeable future.

Significantly, the overwhelming majority of earthquake-induced injuries and fatalities occur in buildings. Hazardous earthquake-induced building performance can range from complete building collapse, with its obvious implications for life safety, to the injury-producing behavior of structural and non-structural elements and building contents.

Ensuring life safety has inspired the evolution of earthquake hazard reduction measures in the U.S. Codes and other regulations are designed to ensure life safety by preventing building collapse. For example, the Field Act, requiring the seismic

TABLE I: Destructive California Earthquakes, 1812-1980 (according to the U.S. Geological Survey)(3)

Date	Location	Lives Lost	Dollar Loss at the Time of the Quake (million)
1812	San Juan Capistrano	40	--
1857	Fort Tejon	--	--
1865	San Francisco	--	0.5
1868	Hayward	30	0.35
1872	Owens Valley	27	0.25
1892	Vacaville	--	0.23
1898	Mare Island	--	1.4
1899	San Jacinto	6	--
1906	San Francisco	700	500.0
1915	Imperial Valley	6	0.9
1918	San Jacinto and Hemet	--	0.2
1925	Santa Barbara	13	8.0
1926	Santa Barbara	1	--
1932	Humboldt County	1	--
1933	Long Beach	115	40.0
1940	Imperial Valley	9	6.0
1941	Santa Barbara	--	0.1
1941	Torrance-Gardena	--	1.1
1949	Terminal Island	--	9.0
1951	Terminal Island	--	3.0
1952	Kern County	14	60.0
1954	Eureka-Arcata	1	2.0
1955	Terminal Island	--	3.0
1955	Oakland-Walnut Creek	1	1.0
1957	San Francisco	--	1.0
1961	Terminal Island	--	4.5
1969	Santa Rosa	--	8.35
1971	San Fernando	65	504.95
1975	Oroville	--	2.5
1978	Santa Barbara	--	12.0
1979	Imperial County	--	30.0
1980	Mammoth Lakes	--	1.5
	Totals	1,029	$1,201.83

retrofit of unreinforced masonry schoolhouses, was enacted following the 1933 Long Beach earthquake because it was felt that many children could have been injured if classes had been in session. Similarly, parapet ordinances were passed after the Bakersfield earthquake, during the 1950's, because it was thought that falling parapets had been responsible for the numerous deaths and injuries following the Long Beach earthquake. More recently, observations of non-structural damage following the Alaskan and San Fernando earthquakes, and speculation about its potential contribution to earthquake injuries prompted regulations requiring the fastening of hazardous non-structural elements.

In spite of the potential of buildings for injury and disruption, surprisingly little is known about 1) how people are actually injured, 2) what elements or building types are particularly hazardous, 3) how people behave during and immediately after an earthquake to avoid or induce injury, 4) what effects such as health status, age and prior training have on injury, and 5) what can be done to mitigate particular dangers. Most of the research on earthquake hazard reduction has focused on overall building structure. While traditional engineering studies certainly result in stronger and safer buildings, more research is needed on the particular aspects of buildings that have actually caused injury in past earthquakes.

There is virtually no empirical data on the contribution of the physical setting to injury.

There have been a number of reports of injuries in past earthquakes and similar disasters. This information ranges from anecdotal stories in the news media and journals to formal epidemiological studies that attempt to describe the incidence of mortality and injuries resulting from these disasters.(8,9,10,11,12) Few of these published studies, with the exception of one study by Glass and his associates(13), have systematically attempted to investigate possible associations between injury and selected aspects of the physical setting. Nor have associations between injury and such factors as time of day, age and behavior been suggested in any but a few of the studies.

Several studies of occupant behavior during earthquakes in hospitals(14), office buildings(15,16), and homes(17,18), have been conducted in recent years. However, because of the dearth of empirical data, potentially misleading "conventional wisdom" about how to avoid injury in earthquakes has accumulated. This "conventional wisdom", based on overly general assumptions of building performance in earthquakes and on the capability of occupants to perform recommended actions, needs urgent reappraisal. For example, although doorways occasionally survived the collapse of unreinforced masonry buildings, the recommendation to stand in a doorway is not sufficiently specific for type of building or type of doorway to be particularly useful to occupants. A recent study of occupant response in the Imperial County Services Building (a reinforced concrete building with a metal floor to ceiling interior partition system) during the 1979 Imperial County earthquake noted that persons who took refuge in doorways often experienced them buckling around them, and several occcupants were struck by the door as it swung shut.(15,16) Observations described in this paper indicate that many people injured while leaving their dwellings would have escaped injury had they stayed inside. Clearly, what is needed are more specific recommendations derived from a basis of empirical data from recent and future earthquakes, all over the world, leading to comprehensive hazard mitigation planning and education.

This paper proceeds to discuss a proposed conceptual framework for a comprehensive approach to earthquake injuries and their prevention and mitigation, a framework which involves the interaction between four groups of factors and four time phases related to the earthquake. Next, we describe in detail preliminary results of an injury and occupant behavior study of four recent earthquakes. They are the 1978 Santa Barbara, the 1979 Imperial County, and the 1983 Coalinga earthquakes, in California, and the earthquake in Chile of March 3, 1985. The observations are discussed and some conclusions are drawn.

This work is based on an epidemiological study of the role of physical environment and occupant behavior in earthquake injuries, conducted together with our collegues Professor Jess F. Kraus and Research Epidemiologist Ann H. Coulson, of the School of Public Health at UCLA, and supported by the National Science Foundation. The study has the aims of increasing available knowledge of how people are injured, and escape injury, in earthquakes; identifying the contribution of aspects of the physical setting such as structure, layout, and building contents, as well as identifying the contribution of occupant behavior and personal characteristics to injury in earthquakes. We are using the approach of epidemiology, which is the study of the distribution and determinants of disease (in this case trauma) in human populations[19,20], for effectively determining the susceptibility of different types of persons to specific types of injury in specific types of physical settings.

To evaluate the importance of personal characteristics and individual actions, it is necessary to identify the distribution of these aspects both among persons suffering injury and among uninjured persons under similar circumstances. In this study, for the four earthquakes mentioned above, we are using a **case-control approach**. The **case group**, i.e. those suffering clearly earthquake related injuries, is mainly identified from the reviews of hospital, Red Cross, and physicians' records. We are interviewing as many of these injured persons as possible, using a questionnaire developed in the study. This comprehensive questionnaire covers injuries or medical problems associated with the earthquake; during the main event, an aftershock, or in the recovery period while searching through rubble, cleaning up debris, inspecting damage, or working as a volunteer, in the National Guard, or in the police force. It deals with all the activities of the person from the beginning of shaking through injury and treatment. It documents the details of the physical environment and the injury, as well as the level of damage sustained by any buildings involved in the injury. It asks about prior training or experience with other earthquakes, and any conclusions drawn by the injured for their behavior in future earthquakes under similar circumstances. Finally, it determines any chronic medical problems that the person had, their influence on the injury and the aggravation that the injury may have produced. The injured person is also asked about other people present in the same place when the injury occurred, and the names of uninjured friends, assumed by us to have similar personal characteristics as the injured. The **control group**, selected from both "same place" and "friend" categories, is being interviewed using a special but similar questionnaire. Both field interviews and the analysis of results are presently being conducted for all four earthquakes involved in the study.

CONCEPTUAL FRAMEWORK

The comprehensive problem of earthquake injuries, understanding the details of the etiology of these injuries, and the various actions needed for their prevention and mitigation, is a very complex one. It involves many aspects and a number of disciplines. The need exists for some model, or conceptual framework, which is at the same time comprehensive, simple, and flexible. Such a conceptual framework is shown in Figure 1, inspired by a similar matrix proposed by Haddon(21,22) for the problem of prevention of motor vehicle crash injuries.

FIGURE 1: Conceptual Framework for Earthquake Injuries

FACTORS

PHASES	Human	Physical	Socio-Economic	Circumstantial
Pre-Earthquake				
Earthquake				
Recovery				
Long Range				

The model uses four fundamental phases of general applicability to earthquakes: the pre-earthquake, earthquake, recovery, and long range phases. Within each phase, in focusing on injuries, we can consider four groups of factors which influence injuries in various ways; namely human, physical, socio-economic, and circumstantial factors. Each of the 16 phase-factor interactions represents an area of specific sets of concerns within the comprehensive picture of the matrix as a whole. The phases also remind us of the "continuing" aspect of earthquake injuries. Beyond the obvious injuries during the earthquake, injuries can and do occur during the recovery period that are earthquake-related, and there is evidence of long range psychological and emotional injury, as well as some physical ones. The long range phase merges into the pre-earthquake phase for the next event, with particular significance when a seismic gap is identified, or another reason arises for an earthquake warning or prediction.

Hopefully, the conceptual framework will serve some useful purposes in that it should facilitate identifying the various roles the interactions play in mitigating and preventing injury. Let us exemplify some interactions by discussing a number of issues.

The human factors include personal characteristics such as age, sex, state of health, etc. At the pre-earthquake phase they affect personal preparedness planning and receptivity for training towards greater protection from injury. During the earthquake they influence behavior and the probability of either greater safety or increased likelihood for injury. For example the starting and propagation of fires, with subsequent injuries, during both the earthquake and early recovery phases, are due to a combination of both human and other factors. Human factors during the recovery phase would impact the medical problems of the homeless, another aspect of earthquake related injuries. Also, human factors, including curiosity and lack of discipline, contribute to tsunami losses, sometimes at great distances from the epicenter. One example is the 11 deaths at Crescent City, California, in 1964, due to the tsunami generated by the Alaskan earthquake hours earlier. A long range-human factors interaction, influenced by physical and circumstantial histories, is, for example, the prevailing attitude that exists about specific earthquake dangers. While fires have occurred during U.S. earthquakes,(23) with 1906 San Francisco being a major example, the predominant fear is of building and other collapses. This is in contrast to Japan, where the fear of earthquakes is dominated by the fear of fire.(24) An obvious explanation for the Japanese reaction is their past repeated experience, for example the Kanto earthquake of 1923, when the majority of the 90,000 deaths were caused by fire.(24)

Physical factors include all the characteristics and variability of the built environment, as well as those of local and regional seismicity. These factors obviously have a major impact on injuries and for the considerations appropriate for each of the four phases. Included in the physical factors are non-structural elements, as well as building contents.

The socio-economic factors are a large group, including institutional factors, cultural aspects, and the variability of circumstances of families, communities and regions, all of which affect issues of injuries at the different phases. During the pre-earthquake period, they are relevant for considerations of planning, preparedness and education. The performance of social organizations, for example hospitals, and the various industrial and work environments, during all earthquake phases, can have a major impact on injuries. Social roles and relations, as well as human characteristics, among those in the same location during the earthquake phase, may account for one person being injured and another not. For instance, in the Coalinga earthquake, a husband who left his living room to exit the house directly through the front door was uninjured, while his wife who left the same living room to exit circuitously through the kitchen was injured by broken glass. The wife was seeing to her children who were in the backyard playing. In the recovery phase, socio-economic factors are also important in providing the needed organization and resources, including those for the homeless, and in making a difference to the health of individuals.

Finally, the group of circumstantial factors, for example the time and season of the earthquake, can have a profound influence. In the United States, so far, we have been very lucky with respect to the timing of damaging earthquakes. The 1933 Long Beach earthquake, which caused extensive damage to pre-Field Act school buildings, occurred at 5:45 in the afternoon when the schools were empty.(25) The 1964 Alaska earthquake, although 8.4 in magnitude, struck a sparsely populated area in the late afternoon of Good Friday, when offices and commercial establishments were closed in the heavily damaged downtown Anchorage. The death toll of the 1971 San Fernando earthquake certainly would have been considerably higher if the event had occurred three hours later when between 100 and 300 staff would have occupied the first-story area of the Olive View Hospital Psychiatric building, which was crushed during the earthquake, and in the areas of the main hospital building, which were destroyed by the collapsed stairway towers.(26) As far as the season of the earthquake is concerned, major secondary sources of injury and damage associated with California earthquakes are landslides and dam failures (primarily in winter or spring) and uncontrolled fires (primarily in summer and fall).(3)

Our own study is mainly concerned with the earthquake and part of the recovery phases, and deals primarily with the human and physical factors. Much work needs to be done, in many earthquake prone countries, to provide the understanding and empirical data required to answer the many questions raised by the conceptual framework.

EARTHQUAKE INVESTIGATIONS

We shall present and discuss next results of our current study of injury and behavior of four earthquakes, namely the 1978 Santa Barbara, 1979 Imperial County, and 1983 Coalinga, all in California, and the Chile earthquake of March 1985. Some relevant details of these earthquakes are given in Table 2. Our results are preliminary since not all interviewing and analysis have been completed.

TABLE 2: Details of the Earthquakes Investigated

Earthquake	Magnitude	Maximum Intensity (Modified Mercalli)	Date	Time	Dead	Injured
Santa Barbara	5.7	VIII	August 13, 1978 (Sunday)	3:55 P.M.	---	85
Imperial County	6.6	VII	October 15, 1979 (Monday)	4:16 P.M.	---	78
Coalinga	6.7	VIII	May 2, 1983 (Monday)	4:42 P.M.	---	211
Chile	7.8	VIII	March 3, 1985 (Sunday)	7:47 P.M.	180	2,572+

Santa Barbara, California, 1978

Santa Barbara is a coastal community in southern California, some 160 kilometers northwest of Los Angeles. In 1978, together with neighboring Goleta, (where the University of California, Santa Barbara (UCSB) campus is located), it had a population of about 143,000. The August 13, 1978 earthquake struck on

Sunday at 3:55 P.M., and had a magnitude of about 5.7. The resulting ground motion had a marked directional asymmetry, which caused greater intensity west of Santa Barbara near Goleta. Eleven aftershocks occurred within the first 20 minutes after the main event.(27)

Santa Barbara does have some old buildings, but most residential buildings are single family homes. and low rise apartments of wood construction. There are only four buildings in Santa Barbara which are 5 or more stories high. In Goleta, except for a few old wood frame and adobe structures, the buildings are relatively modern (20-30 years old). On the UCSB campus, there are some multistory reinforced concrete structures. No serious injuries occurred in the earthquake as a result of structural failures. No major damage was reported to conventional residential houses. However, in Goleta, 324 mobile homes were damaged, 68% in a major way. Four such homes were rendered uninhabitable, and one was destroyed by fire. Overall, minor damage occurred to 68 apartment buildings, with an average of 6 units each, and to 80 businesses, many of them stores. The total financial loss caused by the earthquake was initially estimated to be $7.31 million, some 47% of which was sustained by facilities at the UCSB campus. This included damage to buildings, elevators, utilities, light fixtures,, ceilings, and laboratory supplies and equipment. West of Goleta, a soil fill failure caused the derailment of a freight train some 10 minutes after the earthquake, without any injuries.(27)

A total of 85 injured people were treated at two local hospitals, including 1 by paramedics and 2 by private doctors. This number also includes 9 people who were injured during the recovery phase. The incidence of injury was thus 0.6 per 1,000 population. The injuries were generally minor, with only 3 persons hospitalized. This included a hip injury, to a very old lady who fell during the earthquake and the short hospitalization of two people with cardiac problems.

We started the investigation of the Santa Barbara injuries over six years after the event, and, in this mobile community, the delay caused difficulties in locating the injured persons. Our preliminary results are based mainly on hospital records. At present, interviewing continues, with about 14% of the injured having been interviewed as of August 1985.

1. Who were injured?

Table 3 presents the distribution of the injured by age and sex. It can be seen that the majority were young (55.3% below 31 years of age) and male (67 %).

2. What were the injuries?

The types of injuries are shown in Table 4. The majority of the injuries were lacerations, abrasions, and contusions (56%). The majority of the fractures and sprains were twisted ankles and minor fractures to fingers, wrists, and toes. Some unusual injuries included cat scratches during the the earthquake, and a dog bite later, which we assume to be earthquake related. One of the burns was caused during the earthquake by exposure to chemicals, at home, and the second by hot soup, at work in a restaurant. Finally, there was an accidental exposure to fertilizer chemicals during the earthquake, and exposure to toxic fumes during cleanup operations. As can be seen from Table 5, the most vulnerable parts of the body to injury were arms, hands, and feet.

TABLE 3: Santa Barbara Injuries: Age and Sex Distribution

Age	Number	Percent[a]	Sex	Number	Percent
0-10	5	5.9			
11-20	12	14.1			
21-30	30	35.3			
31-40	17	20.0			
41-50	10	11.8	Male	57	67.0
51-60	4	4.7	Female	28	33.0
61-70	3	3.5	Total	85	100.0
71-80	2	2.4			
80+	2	2.4			
Total	85	100.1			

[a]Because of rounding, the total does not equal 100%

TABLE 4: Santa Barbara Injuries: Type of Injury

Type of injury	Number[a]	Percent
Lacerations, Abrasions	33	33.0
Contusions	23	23.0
Head Injuries	5	5.0
Fractures, Sprains	19	19.0
Muscle Strain	3	3.0
Back Injuries	4	4.0
Anxiety Reactions	2	2.0
Eye Injuries	3	3.0
Cardiac	2	2.0
Animal Bites	2	2.0
Burns	2	2.0
Other	2	2.0
Total	100	100.0

[a]Numbers include several cases of multiple injuries to the same person

TABLE 5: Santa Barbara Injuries: Part of Body

Part of Body	Number[a]	Percent[b]
Head, Face	7	7.4
Eyes	3	3.2
Neck	3	3.2
Shoulders	4	4.2
Arms, Hands	33	34.7
Chest	2	2.1
Back	5	5.3
Hip	1	1.1
Legs	4	4.2
Knees	9	9.5
Ankles	9	9.5
Feet	15	15.8
Total	95	100.2

[a] Numbers include several cases of multiple injuries to the same person

[b] Because of rounding, the total does not equal 100%

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3. Where did the injuries occur?

As shown in Table 6, most of the injuries (57.6%) happened in or near the home. The 31.8% in "business" locations were mostly in grocery or liquor shops, but also at work. The 7 "outside" injuries included 3 falls (off bicycle, thrown from truck, and a fall in the garden), 3 injuries while driving, without the involvement of another car (including an anxiety attack) and an injury to a toe while jogging. Two (2) of the "home" injuries happened in mobile homes. Both cases involved relatively minor head injuries from fallen objects. The very small number of injuries found in mobile homes in very surprising, in view of the large number of such structures damaged. One plausible explanation is the timing of the earthquake, on a Sunday afternoon during the summer, when people were ouside their mobile homes, and before dinner preparations. Also, additional minor injuries probably did occur and did not require hospital treatment. Our 85 injuries represent a lower bound.

TABLE 6: Santa Barbara Injuries: Location

Location	Number	Percent
Home	49	57.6
Business	27	31.8
Outside	7	8.2
Unknown	2	2.4
Total	85	100.0

4. **When did the injuries occur?**

It has already been mentioned that out of the total of 85 injuries, 9 (10.5%) occurred after the main shock, during the immediate recovery phase. Seven (7) were cleanup and rescue operations, 1 was the dog bite, and 1 was an anxiety attack.

5. **How did the injuries occur?**

Table 7 presents a general classification of the manner of injury. Overall, a total of 19 injuries (22.4%) were related in one form or another to broken glass.

The objects involved included a bookcase falling in an office and causing back lacerations and a compression fracture, and a broken sliding glass door.

The falls listed are considered to be "primary," i.e. happening very quickly after the earthquake and apparently before much other movement. They include a fall off a stool, while taking a shower, and an old person just standing in the kitchen. The average age was 40 years.

Those injured leaving buildings were not hit by any object. They represent anxious people trying to escape environments they considered very dangerous. Actually, they would have been safer staying put. The exiting involved jumping through a window, through doorways, running out through a glass door, jumping over a railing, etc., and all except one of these persons were male. The average age was 30 years.

The large group who "bumped into something" represents people who undertook a variety of actions inside the built environment. They collided with a chair, a sofa, a fireplace, and walls and doors.

TABLE 7: Santa Barbara Injuries: Manner of Injury

Manner of Injury	Number	Sex: Male	Sex: Female	Percent
Hit by Objects	15	7	8	17.6
Falls	15	6	9	17.6
Leaving Buildings	9	8	1	10.6
Bumped into Something	19	14	5	22.4
Other	27	22	5	31.8
Total	85	57	28	100.0

We would like to point out again the "lucky" timing of the earthquake, on a Sunday afternoon during the summer. The UCSB campus was almost empty of people. Had the campus been occupied, the estimate has been made of "at least 25 persons killed, and several hundred seriously injured"(28).

Imperial County, California, 1979

On Monday, October 15, 1979, at 4:16 P.M., a destructive earthquake, of magnitude 6.6, shook the Imperial Valley, California, just north of the Mexican border. The population of the neighboring areas of El Centro, Imperial, Brawley, and Calexico, was about 50,000. Imperial County is for the most part a flourishing, extensively irrigated, agricultural area.[29]

Buildings in the area are of varying ages and construction. Most of the commercial buildings are one and two-story structures, many relatively old and built of unreinforced masonry, but there is also a representative group of recent relatively modern structures. In general, there was only minor structural damage to the typical commercial construction, although a few buildings were subsequently condemned. There was some parapet damage, some architectural damage, but also widespread spilling of library racks, store merchandise, and other shelved items. The most significant structural damage was to the six-story Imperial County Services Building, which had to be demolished later. It was also the subject of a detailed earlier study of earthquake behavior.[15, 16] The total estimated damage of the earthquake was $30 million.[29]

A total of 78 people were found to have been treated at the local hospitals, at Brawley, El Centro, and Calexico. This number also included 6 people who were injured during the recovery phase. The incidence of injury was thus 1.5 per 1,000 population. Four (4) people were hospitalized, for a back injury, two leg and foot fractures, and a child for severe burns by hot water from a kettle. The burns required one month of hospitalization, while the others were less than one week.

Our preliminary results are based only on hospital records, and the numbers are probably low. We have started interviewing and, because of a relatively stable population, hope to find a large fraction of the injured even after six years have elapsed.

1. Who were injured?

Table 8 presents the distribution of the injured by age and sex. Almost half (46.2%) are below the age of 31 years, and the majority (61.5%) are women.

2. What were the injuries?

The types of injuries are shown in Table 9. Of the people suffering anxiety reactions, including hysteria, there were ten women and one man. The average age of the 11 was 32.2 years, with 8 of the 11 below 40 years. Table 10 shows the most vulnerable parts of the body to have been arms and hands, followed by back and head.

3. Where did the injuries occur?

As shown in Table 11, we know only the location of 35 injuries, most of which (57.1%) were in or near the home. The rest (42.9%) were in business, including a variety of stores and shops. None happened outside.

TABLE 8: Imperial County Injuries: Age and Sex Distribution

Age	Number	Percent[a]
0-10	7	9.0
11-20	16	20.5
21-30	12	16.7
31-40	18	23.1
41-50	12	15.4
51-60	5	6.4
61-70	4	5.1
71-80	2	2.6
80+	1	1.3
Total	78	100.1

Sex	Number	Percent
Male	30	38.5
Female	48	61.5
Total	78	100.0

[a]Because of rounding, the total does not equal 100%

TABLE 9: Imperial County Injuries: Type of Injury

Type of Injury	Number[a]	Percent[b]
Lacerations, Abrasions	22	25.3
Contusions	16	18.4
Head Injuries	6	6.9
Fractures, Sprains	12	13.8
Muscle Strain	1	1.2
Back Injuries	11	12.6
Anxiety Reactions	11	12.6
Eye Injuries	1	1.2
Cardiac	0	0.0
Animal Bites	4	4.6
Burns	2	2.3
Other	1	1.2
Total	87	100.1

[a] Numbers include several cases of multiple injuries to the same person

[b] Because of rounding, the total does not equal 100%

TABLE 10: Imperial County Injuries: Part of Body

Part of Body	Number[a]	Percent[b]
Head, Face	10	12.5
Eyes	1	1.3
Neck	3	3.8
Shoulders	4	5.0
Arms, Hands	22	27.5
Chest	2	2.5
Back	15	18.8
Hip	1	1.3
Legs	7	8.7
Knees	7	8.7
Ankles	4	5.0
Feet	4	5.0
Total	80	100.1

[a] Numbers include several cases of multiple injuries to the same person

[b] Because of rounding, the total does not equal 100%

TABLE 11: Imperial County Injuries: Location

Location	Number	Percent[a]
Home	20	57.1
Business	15	42.9
Outside	0	0.0
Unknown	52	---
Total	87	100.0

[a] Percentages calculated of the 35 known locations

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4. When did the injuries occur?

Out of the 78 injuries, 6 (7.7%) occurred during the recovery phase. Two (2) were cuts on glass in cleanup operations, and 4 were dog bites, 2 of children.

5. How did the injuries occur?

Table 12 presents a general classification of the manner of injury. Overall, a total of 9 injuries (11.5%) were related in one form or another to broken glass.

The objects involved in the injuries include a broken plate glass window, hitting a woman while exiting a business, hot water causing burns, a water bottle, a cotton bale, and a car falling off a jack and catching a worker's arm between the tire and the fender.

The falls listed, based on hospital records and, so far, without confirmation by interview, are considered to have been the primary manner of injury. They include a fall off a step ladder, standing in the kitchen, in a market over spilled items, etc. The average age was 39 years.

As mentioned above only 1 of those injured while leaving a building was hit by an object. The exiting includes 2 jumps through windows, 2 cases of group panic, cuts and twisted knees. The average age was 37.8 years.

As before, the group listed to have "bumped into something" includes people hitting doors or doorways, chairs and tables, and being thrown against walls. The manner of injury of about one third of the "other" group is unknown.

TABLE 12: Imperial County Injuries: Manner of Injury

Manner of Injury	Number	Sex		Percent
		Male	Female	
Hit by Objects	13[a]	5	8[a]	16.5
Falls	14	4	10	17.7
Leaving Buildings	8[a]	6	2[a]	10.1
Bumped into Something	13	4	9	16.5
Other	31	11	20	39.2
Total	79	30	49	100.0

[a]Numbers include a female who was hit while leaving a building

Coalinga, California, 1983

Coalinga, with a population of about 7,250, is located in the western part of the San Joaquin Valley approximately 60 miles southwest of Fresno, and midway between San Francisco and Los Angeles. It served initially as a coaling station for the railroad, and later as a center for oil exploration and commercial activities for the surrounding agricultural area. On Monday, May 2, 1983, at 4:42 P.M., an earthquake of magnitude 6.7 occurred. A large aftershock followed slightly more than 3 minutes later.(30)

The earthquake devastated Coalinga's central business district. Most of the structures in this area were old, over sixty years old, and built mainly of unreinforced brick masonry with timber roofs. There were some 35 of them, and they suffered major destruction. Coalinga had 2041 one- to four-family dwellings and over 100 mobile homes. A few of the dwellings were old and built of adobe, tile, unreinforced hollow concrete block, and unreinforced brick. Their overall performance was very poor. Most of the dwellings were of wood-frame construction. Their performance ranged from minor damage to subsequent demolition. Those designed to be earthquake-resistant performed very well. Virtually all mobile homes suffered some damage, and many fell from their supports. There was one major fire, in the central business district, and three minor residential fires.(30) The overall damage was over $35 million.

So far we have identified 211 people who have been injured. This represents an overall incidence of injury of 29 per 1,000 population. There were no fatalities. Most injured people were treated at seven area hospitals and by the American Red Cross (ARC). The above number includes 10 injuries in subsequent aftershocks, mainly that of July 25, 1985, and 49 injuries during the recovery phase. The number excludes injuries listed by ARC but judged by the local hospital staff not to be earthquake-related, 17 injuries to ARC staff members during recovery, and 137 illnesses (such as upper respiratory infections) that were treated at the Red Cross shelter.(31) The 211 injured included 17 people hospitalized (8%). They represent an incidence of hospitalization of 2.3 per 1,000 population. The late afternoon earthquake found many residents at home making dinner. Luckily, no fatalities occurred during this earthquake.

The interim results, presented in this paper, are based on both hospital records and extensive interviewing. We approached 141 people, were refused by 8 (5.7%), and have used our questionnaire so far in 133 interviews. These plus 56 hospital and ARC records, for a total of 189, are analyzed and presented here. An additional 22 injured were identified recently in a random short questionnaire used in conjunction with another study. They will be discussed separately below. Interviewing is now proceeding of both injured and controls.

Before discussing the 189 injuries in greater detail, consider Table 13 which shows the distribution of places where treatment was provided. The vast majority (83.1%) were at hospitals and ARC. Table 14 presents data on the degree of damage to buildings associated with injury. This is information obtained during interviews with our questionnaire, and is available for 100 injuries. The 89 cases listed under "unknown" also contain injuries outside of the built environment.

TABLE 13: Coalinga Injuries: Places of Treatment

Place	Numbers[a]	Percent
Hospitals	116	51.8
American Red Cross	70	31.3
Doctors' Offices	16	7.1
Other[b]	15	6.7
Unknown	7	3.1
Total	224	100.0

[a]Numbers include cases treated in multiple places

[b]Mainly at home

TABLE 14: Coalinga Injuries: Degree of Damage to Buildings Associated with Injury

Degree of Damage	Number	Percent	Percent[a]
None	3	1.6	3.0
Minor	45	23.8	45.0
Major	24	12.7	24.0
Demolished	28	14.8	28.0
Unknown(or not applicable)	89	47.1	--
Total	189	100.0	100.0

[a]Percentage calculated excluding unknown or not applicable cases

1. Who were injured?

Table 15 presents the distribution of the injured by age and sex. We have indicated both total and hospitalized injuries, and also the incidence of injury per 1,000 population. (The 1980 census provides the most recent breakdown by age of the population of Coalinga. Since 1980, the recorded population of Coalinga was increased by about 500). It is apparent that the injuries in both categories are highest among the elderly. They seem to be more susceptible to injury, and may also reside in older houses that are more prone to damage. The majority of the injured are women, the percentage increasing to 60.5% if we exclude 12 National Guardsmen, all male, who were injured during the recovery phase. However, the ratios of men to women reverse if we consider the sexes of the hospitalized injuries.

TABLE 15: Coalinga Injuries: Age and Sex Distribution

Age	Total Injuries Number	Total Injuries Percent[a]	Total Injuries Rate[b]	Hospitalized Injuries Number	Hospitalized Injuries Percent	Hospitalized Injuries Rate
0-10	16	8.5	13.9	0	0.0	0
11-20	26	13.8	19.5	4	23.5	3.0
21-30	37	19.6	30.4	2	11.8	1.6
31-40	23	12.2	24.7	4	23.5	4.3
41-50	22	11.6	30.0	0	0.0	0
51-60	11	5.8	15.0	2	11.8	2.7
61-70	25	13.2	44.6	1	5.9	1.8
71-80	10	5.3	26.5	1	5.9	2.7
80+	9	4.8	42.7	3	17.6	14.2
Unknown	10	5.3		—	—	
Total	189	100.1		17	100.0	

Sex	Number	Percent	Percent[c]	Hospitalized Number	Hospitalized Percent
Male	82	43.4	39.5	11	64.7
Female	107	56.6	60.5	6	35.3
Total	189	100.0	100.0	17	100.0

[a] Because of rounding, the total does not equal 100%

[b] Rate of injury per 1,000 population, of the particular age group.

[c] Percentages calculated excluding 12 National Guardsmen, all male, injured during the recovery phase.

2. What were the injuries?

The types of injuries are shown in Table 16. The majority were lacerations, abrasions and contusions (54.4%). Some of the head injuries and fractures led to hospitalization. Of the people suffering anxiety reactions, there were 10 women and 2 men. The average age of the 12 was 48 years, ranging from 24 to 71 years of age. Only 1 of the burns occurred in the kitchen, which is surprising in view of the hour and the fact that 9 other injuries occurred in the kitchen to people being hit by various objects, as well as some identified falls in the kitchen. The other burn happened to a volunteer during recovery. "Other" injuries included stress induced psoriasis, swallowed poison by a child from a spill, and a few earthquake-associated illnesses.

Table 17 presents the distribution of injured parts of body. The most vulnerable are arms and hands, head and face, and feet. We have also indicated the parts involved in 15 of the 17 hospitalizations. The other 2 included an anxiety attack and stress, and a sunstroke suffered by a National Guardsman.

TABLE 16: Coalinga Injuries: Type of Injury

Type of Injury	Number[a]	Percent[b]
Lacerations, Abrasions	75	31.4
Contusions	55	23.0
Head Injuries	21	8.8
Fractures, Sprains	34	14.2
Muscle Strain	7	2.9
Back Injuries	14	5.9
Anxiety Reactions	12	5.4
Eye Injuries	5	2.1
Cardiac	2	1.0
Animal Bites	2	1.0
Burns	2	1.0
Other	9	3.8
Total	238	

[a] Numbers include several cases of multiple injuries to the same person

[b] Because of rounding, the total does not equal 100%

TABLE 17: Coalinga Injuries: Part of Body

Part of Body	Number[a]	Percent[b]
Head, Face	42 (5)[c]	17.9
Eyes	6	2.6
Neck	3	1.3
Shoulders	20 (1)[c]	8.5
Arms, Hands	53 (2)[c]	22.6
Chest	11	4.7
Back	18	7.6
Hip	6 (3)[c]	2.6
Legs	15 (1)[c]	6.4
Knees	24 (2)[c]	10.2
Ankles	9 (2)[c]	3.8
Feet	28 (1)[c]	11.9
Total	235	100.1

[a] Numbers include several cases of multiple injuries to the same person

[b] Because of rounding, the total does not equal 100%

[c] Hospitalized injuries

3. Where did the injuries occur?

As shown in Table 18, about one half of the injuries (52.9%) happened in or near the home. The home locations included 12 injuries in apartments, 4 in mobile homes and 1 in a parked moving van. Two (2) of the mobile home injuries happened during an aftershock, and the 4 injuries included falls (3), and being bumped against a railing (1). Only 7 injured persons were located above the ground floor, including 5 on the second floor, 1 working on the roof, and 1 on an oil rig. The 12.7% in "business" locations were in offices (3), shops (16), a restaurant(2), the post office(1), and a hotel (2) which later burned. Generally, these were more vulnerable locations than homes. The 31 "outside" injuries happened at different times: 8 during the main earthquake, 1 during an aftershock, and 22 during the recovery period. The 8 "outside" injuries on May 2, 1983, included a fall off a bike, jumping out of a parked moving van, outside an old building which collapsed, jumping off a tractor in an open field, hit by bricks from a collapsing building while parked in a car (2 people), falling near a shaking tree, and escaping from the top of an oil rig, 65 feet in the air.

TABLE 18: Coalinga Injuries: Location

Location	Number	Percent
Home	100	52.9
Business	24	12.7
Outside	31	16.4
Unknown	34	18.0
Total	189	100.0

4. When did the injuries occur?

Table 19 shows the times of injury. As expected, most happened during the main earthquake (69.4%). However, there were also 10 injuries (5.2%) during aftershocks. Three (3) of them (33%) included 2 anxiety attacks, and 1 person jumping out of a window, possibly indicating an increased level of nervousness. Table 20 presents some details related to the 49 persons injured during the recovery phase.

TABLE 19: Coalinga Injuries: Time of Injury

Time of Injury	Number	Percent
Main Earthquake (May 2, 1983)	134[b,c]	69.4
Aftershocks[a]	10[b,d]	5.2
Recovery Phase	49[c,d]	25.4
Total	193	100.0

[a]Mainly during the aftershock of July 25, 1985

[b]Two people were injured both during the main earthquake and an aftershock

[c]One person was injured both during the earthquake and the recovery phase

[d]One person was injured both during an aftershock and the recovery phase

5. How did the injuries occur?

Table 21 summarizes the manner of injury. In this case numbers include injured in more than one category. To clarify the manner of injury, further analysis was conducted.

TABLE 20: Coalinga Injuries: During Recovery Phase

Item	Number[a]
Injury to Volunteers, National Guardsmen, ARC, and Police Force	20
Cleanup Work	26
Anxiety, Stress	6
Cuts	10
Dog Bites	2
Illnesses	6
Falls on Wet Surfaces	2
Total	72

[a]Numbers include multiple entries. The total number of people involved is 49.

TABLE 21: Coalinga Injuries: Manner of Injury

Manner of Injury	Number	Sex: Male	Sex: Female
Hit by Objects	47	15	32
Falls	54	14	40
Leaving Buildings	23	9	14
Bumped into Something	14	6	8

[a]Numbers include injured in more than one category

Table 22 shows the location of the 47 injured who were hit by objects, in decreasing order of frequency. The hour of the day explains the large number of women preparing dinner in the kitchen, a place most vulnerable to be hit by a variety of objects. Three (3) injuries by objects happened while standing in a doorway. One family had established a "predetermined" safe location for earthquakes, an alcove in the house some 4 feet by 4 feet, and four people used it, with only one of them being hit by a falling stereo. This happened in an old building built in 1899.

TABLE 22: Coalinga Injuries: Hit by Objects: Location

Location	Number	Sex		Percent[a]
		Male	Female	
Kitchen	8	1	7	17.0
Shop, Store	8	3	5	17.0
Exiting	7	3	4	14.9
Living Room	5	1	4	10.6
Doorway	3	1	2	6.4
Family Room	2	1	1	4.3
Bathroom	2	1	1	4.3
Bedroom	1	0	1	2.1
Dining Room	1	0	1	2.1
Hall	1	0	1	2.1
Predetermined Alcove	1	0	1	2.1
Second Floor	1	1	0	2.1
Roof	1	1	0	2.1
Outside: Parked Car	1	0	1	2.1
Cleaning (recovery phase)	1	0	1	2.1
Unknown	4	2	2	8.5
Total	47	15	32	99.8

[a]Because of rounding, the total does not equal 100%

Table 23 provides some details of the 54 falls, in decreasing order of frequency. The average age of the 21 knocked down by the earthquake was 62 years (8 out of 21 were over 70 years), and 18 were female (85.7%). Of those 11 listed as "trying to leave buildings," 8 actually exited and 3 only intended to do so. None of these 11 injured was hit by an object. Their average age was 30 years. The 8 injured listed as "hit by objects" experienced subsequent falls.

TABLE 23: Coalinga Injuries: Types of Falls

Type of Fall	Number	Sex: Male	Sex: Female	Percent[a]
Knocked Down	21	3	18	38.9
Trying to Leave Buildings	11	3	8	20.3
Hit by Objects	8	3	5	14.8
Recovery Phase	4	1	3	7.4
In Mobile Home	3	0	3	5.6
Off Bicycle	2	2	0	3.7
Panic in Business	1	0	1	1.9
On Wet Surface (Shop)	1	0	1	1.9
Off the Roof	1	1	0	1.9
Unknown	2	1	1	3.7
Total	54	14	40	100.1

[a] Because of rounding, the total does not equal 100%

Table 24 shows the details of a variety of events associated with people who were injured while exiting or close to exiting buildings during the earthquake. The most common occurrences were falls and being hit by an object (34.8% each). Those leaving buildings included 8 people from old unreinforced buildings. They suffered 4 "hits by objects" and 4 falls.

TABLE 24: Coalinga Injuries: Leaving Buildings

Event Associated With Leaving Building	Number	Sex: Male	Sex: Female	Percent[a]
Falls	8	2	6	34.8
Hit by Objects	8	3	5	34.8
Back Pain	2	0	2	8.7
Ankle Injury	1	0	1	4.3
Bumped Against Railing	1	1	0	4.3
Cut Arm on Screen Door	1	1	0	4.3
Jumped Out of Window	1	1	0	4.3
Jumped Out of Second Floor	1	1	0	4.3
Total	23	9	14	99.8

a Because of rounding, the total does not equal 100%

Table 25 describes the variety of objects that injured people "bumped into". The most common were doorways and walls (21.4% each). One of the 2 doors was a screen door which caused cuts on exiting.

The number of injuries involving glass was 30, representing 15.9% of the total.

TABLE 25: Coalinga Injuries: Bumped Into Something

Object	Number	Sex: Male	Sex: Female	Percent[a]
Doorway	3	1	2	21.4
Walls	3	1	2	21.4
Door	2	1	1	14.3
Stove	2	2	0	14.3
Table	2	0	2	14.3
Railing	1	1	0	7.1
Armchair	1	0	1	7.1
Total	14	6	8	99.9

[a] Because of rounding, the total does not equal 100%

Beyond the above discussion of the who, what, where, when and how of the Coalinga injuries, and certainly overlapping with this discussion, we will consider next four specific items, namely the hospitalized injuries, disabilities, doorways, and old buildings.

6. Hospitalized Injuries

We have already mentioned the fact that there were 17 people hospitalized (8% of the total of 211 injuries, or 9% of the 189 analyzed), representing an incidence of 2.3 per 1,000 population. Their age and sex distribution is shown in Table 15. Also Table 17 gives the distribution of the parts of body involved among the hospitalized. Table 26 presents some information on the circumstances of their injury. The largest percentage was of 5 people in or near old buildings (29.4%). There was a similar number of 5 "vulnerable" people, namely an oxygen tank collapsing on 1 sleeping partly disabled man, 3 falls by people partly paralyzed, old, or with previous hip surgery, and 1 disabled person hit on the head by an object in the kitchen. The accident involved a volunteer in a truck, and the illness was a sunstoke suffered by a National Guardsman. Out of the 17 cases, we know that the hospitalized were in 6 buildings with minor damage, and in 5 buildings which were destroyed. The periods of hospitalization ranged from 3 days to over 2 months.

TABLE 26: Coalinga Injuries: Hospitalized

Circumstances	Number	Percent[a]
In or Near Old Buildings	5	29.4
Vulnerable	5	29.4
Anxiety	2	11.8
Accidents, Illness	2	11.8
Brick Facing of Fireplace	1	5.9
Fall on Wet Surface	1	5.9
Unknown	1	5.9
Total	17	100.1

[a] Because of rounding, the total does not equal 100%

7. Disabilities

Of the 133 injured interviewed, 50 persons (37.6%) had some "disability" or vulnerability that could have made them more prone to injury, or to a more severe injury. In fact, 5 of the 50 (10%) were hospitalized, not significantly higher than the overall (8-9%). Only 3 of this group of 50 people seem to be officially listed as "disabled". The multiple disabilities present in the group are summarized in Table 27. Heart conditions and high blood pressure predominate (40.9%). The injuries include 2 cases which also emphasize the vulnerability of the group. Two (2) people, 1 diabetic and 1 with high blood pressure, had to go to the hospital to pick up essential medication lost during the earthquake. Listing pregnancies as "disabilities" does not, in any way suggest them to be such! The 5 women were over 6 months pregnant, and it is suggested that this may have influenced their normal mobility. It may have also made more vulnerable once injured. In addition to this group of 50, there were 2 cases relevant here. One (1) injured person suffering a fall had a cast removed 3 weeks earlier, and 1 person was injured when grabbed on a recent surgical incision causing an hemorrhage.

8. Doorways

Standing in a doorway figures prominently among recommended actions in an earthquake. In fact, on questioning, 7 people (5.3%) stated that they had such instruction. A total of 13 people (9.8%) acknowledged some prior "training" for earthquakes. Of these, 1 stated that he was told to "get out" of a building, 1 told to take cover, and 4 were not explicit about their earthquake training. Nine (9) people questioned (6.8%) indicated that at the beginning of the earthquake, taking cover under a doorway was the "first thing" they tried to do.

Table 28 lists various events that were associated with injuries at doorways, involving 10 people. As described previously, 3 people were bumped at doorways (Table 25), and 3 were hit by objects (Table 22). One of those hit, by glass, escaped more serious injury from a falling bookcase. Two (2) people reached the doorway after injury, by glass and by a dresser. An additional person reached a doorway and left it before the earthquake was over, moved outside and got cut.

TABLE 27: Coalinga Injuries: Disabilities

Disability	Number [a]	Percent [b]
Heart Condition, High Blood Pressure	27	40.9
Diabetes	13	19.7
Arthritis	10	15.2
Pregnant	5	7.6
Back Problem	4	6.0
Artificial Hip Joint	2	3.0
Mild Brain Damage	1	1.5
Partially Paralyzed	1	1.5
Prior Knee Surgery	1	1.5
Asthma	1	1.5
Partially Blind	1	1.5
Total	66	99.9

[a] Numbers include several cases of multiple disability of the same person

[b] Because of rounding, the total does not equal 100%

TABLE 28: Coalinga Injuries: Injuries At Doorways

Circumstances	Number	Percent [a]
Bumped into Doorway	3	27.3
Hit by Objects at Doorway	3 [b]	27.3
Reached Doorway after Injury	2	18.2
Fall at Doorway	3 [b]	27.3
Total	11	100.1

[a] Because of rounding, the total does not equal 100%

[b] One person was both hit and fell

9. **Old Buildings**

Eighteen (18) injuries were associated with people in or near old buildings. Fifteen (15) buildings were involved. Out of these 2 had minor damage, 3 had major damage, and 10 were demolished. Of these 18 injuries, 5 were hospitalized (27.8%), and 8 occurred while exiting an old building.

We are now in the process of interviewing the two types of controls, people who were in the same physical setting as the injured (we aim at about 60 interviews), and friends of the injured (about 150 to 200). These will be then thoroughly analysed to obtain further insights into injury and behavior under earthquakes. We have already mentioned the 22 minor injuries indentified recently in a randon short questionnair. in conjunction with another study. The total number of such one-page questi. aires so far has been 116. These were random interviews stratified with respect to building damage, i.e. the distribution of the interviews is aimed to mirror the distribution of the building damage as noted by the American Red Cross. It is very interesting to note that not only have we found 22 of them (19%) to have sustained minor injury, but of the 94 uninjured interviewees, 39 (41%) mentioned being struck (10) or bumped (9), having been trapped (5), or having fallen (9), having had some problems with animals (14), or having been **almost** struck or bumped (15). There were thus 62 **missed opportunities** for injury among these 39 people. These observations underscore at least two points. There are probably a large number of minor injuries which are never recorded in the official numbers, and which may add up to a lot of suffering and loss. Also, there is probably much knowledge that can be usefully gained from studying the near misses, as well as the injured.

Chile, 1985

On Sunday, March 3, 1985, at 19:48 P.M. local time, a destructive earthquake struck near the coast of central Chile. It has a magnitude of 7.8, and was felt widely through South America. Maximum MM intensity VIII was in the Valparaiso Vina del Mar area. Damage to structures, particularly unreinforced adobe buildings but reinforced concrete as well, occurred over a wide area. Bridges, lifelines, and industrial facilities were also damaged.(32) Some 51,000 housing units have apparently been destroyed and 220,000 damaged, total of over 16% of the housing stock of a large area.(33) The official number of homeless, as of March 12, 1985, was 370,000. It is now believed that 180 people died and 2,572 sought hospital treatment for injuries within the first 3 days. However, health officials estimate that there may have been as many as 10,000 people with minor injuries who did not seek hospital treatment during this period.(2) Although, most of the injuries occurred in Santiago with its huge population, the ratio of those treated to population was highest in the most heavily damaged areas of Milipila, San Antonio, Valpariso and Vina del Mar.

Reasons given for the light casuality rate include the timing of the earthquake which occurred on a late summer Sunday evening when commercial districts were closed and there was still light enough to enable people to successfully evacuate buildings. We believe that many of the earthquake fatalities were due to the partial collapse of structures or the failure of non-structural elements like ornamentation or ceilings. Of the 86 cases of death that have so far been established 72 died of head injuries or multiple trauma. For exmple, a 67 year old man was killed in a movie theater in San Antonio when the structure collapsed. A 45 year old man died in a Santiago restaurant when a wall fell in on him. However, most of the non-fatal injuries were due to non-structural elements, building contents and inappropriate actions (e.g. being hit by a falling roof tile or being thrown against something or being cut by a piece of glass).

The injured apparently had time to engage in many actions during the earthquake, at lest in low-rise structures. And, our preliminary interviews suggest that some of these actions were detrimental. An example is the woman who fell and fractured her ankle while trying to run down the stairs of an undamaged home. There appears to be a rough correlation between level of damage and injury. For example we have so far identified 3 injuries in the heavily damaged Acupulco Apartment and expect to find more. However, many injuries occurred in buildings with no apparent structural damage. It will be interesting to uncover the cause of these injuries. Our speculation is that the relatively greater motion of some mid-rise apartment buildings may have prevented occupants from taking self-protective actions.

As part of our NSF study, we are now in the process of conducting interviews in Chile, in collaboration with local people, of about 1,000 to 1,200 of the injured and some 500 to 700 controls.

OCCUPANT BEHAVIOR

Previous Studies of Occupant Behavior in Past U.S. Earthquakes

1. Occupant Behavior in Damaged Hospitals[14]

The first study was a documentation of the actions of patients and staff inside the five hospitals most heavily damaged in the 1971 San Fernando Earthquake. These facilities suffered different levels of damage--ranging from minor, in the case of Kaiser Panorama City, to complete collapse of some buildings, in the case of the Veterans Administration Hospital. This investigation of occupant behavior during and immediately after the earthquake was part of a larger, more comprehensive study of the immediate response and the long term recovery process that each hospital experienced. Administrators, department heads, physicians and key members of the nursing staff were interviewed, those present at these hospitals when the earthquake began or arriving soon after to assist in relief operations. Each person was asked to recount his sequence of actions from the time that the shaking began until the time that he had reached a place of safety. In this manner the initial response as well as the evacuation was covered.

Although some time had elapsed since the earthquake, making it difficult to locate all of the key informants, a good general picture of what people did was gained. Contrary to the popular assumption of general panic, little irrational behavior was found among hospital staff on duty at the time of the earthquake. Staff members first engaged in a combination of behavior aimed at self protection and safeguarding their patients. For example, one nurse herded the moving incubators together and sheltered them with her body until the shaking stopped. The first action after the shaking was to look in on patients and reassure them. This reaction is not surprising since the hospitals were hierarchically organized with clearly defined lines of authority.

It was also found that, surprisingly, with the exception of those 47 killed in the collapsed VA Hospital and the 3 at Olive View, there were few reported injuries among staff and patients. This low injury rate is attributed to timing and location, the fact that at 6 A.M. few people were at work or moving about or on the lower floors where damage was greatest.

2. Occupant Behavior in a Mid-Rise Public Office Building[15, 16]

The 1979 Imperial County Earthquake provided a unique opportunity to study systematically the response of office workers in the Imperial County Services Building, a 6 story office building sustaining enough damage to render it inoperative and require subsequent demolition. The building did sustain considerable non-structural damage.

Using a very structured approach, 112 individuals were surveyed who were in this building during the earthquake, 96% of those actually in the building. This instrument included questions about people's actions, their perceptions of risk, and their previous training.

Once again it was found that prior training and expectations played an important role in the way that people responded. For example, 79% of the occupants followed the prearranged evacuation plan. However, because of several previous evacuations necessitated by bomb threats, occupants evacuated down only one of two possible firestairs, even though both were intact. This could have created conditions of overcrowding had the building been more fully occupied or if there were injured to be transported. The evacuation pattern was prearranged because this stairway was thought less hazardous than the other in case of a bombing. This finding supports the position that different emergency situations call for different organizational responses. It further illustrates the need to have contingency plans developed and in place.

A second finding questions the general usefulness of advice and prior training. Although 36% of occupants reported getting under their desks, 9 of these received minor injuries when their desks struck them or they bumped the desks while trying to get underneath. Of the 47 injuries that occurred, half happened to people engaged, unnecessarily, in evasive behavior (getting under a desk or standing in a doorway).

A third finding is that building contents caused more injuries than non-structural elements, with the ratio of injuries from building contents to non-structural elements 3 to 1. People were hurt mostly by moving desks, filing cabinets, and furniture located in the immediate vicinity of where those people were located. This suggests that in addition to the traditional non-structural abatement measures of securing suspended ceilings and lighting fixtures, we need to secure or reposition building contents in areas where people spend most of their time. This finding illustrates the necessary interplay of engineering and preparedness practices.

Present Study-Occupant Behavior

The preliminary information presented in this paper, on the earthquakes investigated as part of our study, contains much about occupant behavior. Indeed it substantiates recent emphasis in the literature that there is a great deal of activity that occupants can engage in during the terrifying seconds of shaking. Some of this activity can be detrimental and some beneficial. We shall review some observations, within the conceptual framework and its groups of factors.

I. **Human factors** seem to play a significant role in earthquake injuries. Coalinga has shown the greater risk of injury with age, and with its associated factors of vulnerability. With age people seem to become more vulnerable to earthquake falls. On the other hand, younger people seem to have a greater tendency to exit the built environment. It is as if, for earthquake safety, one is at greater risk when either young and energetic, and seemingly using this energy to exit violently and quickly, or old and weak and unable to prevent being knocked down.

Exiting, contrary to popular impulse, and particularly evacuating an unreinforced masonry buildng during an earthquake is not necessarily beneficial and may prove harmful. So far, we have analyzed the actions of those inside the unreinforced masonry buildings in the Coalinga Central Business District. By interviewing the proprietors of the small businesses located in these buildings, and selected occupants, we were able to determine the number of people inside, whether they stayed inside or attempted to exit during the shaking, and whether they suffered any injuries.(34) Our initial results indicate that the relative risk of injury while exiting one of these buildings versus staying may be over 3 to 1. It is unfortunate that this apparently is the first thing that people try to do. Table 29 shows responses to one of our questions in Coalinga, namely "if you moved, what was the **first thing** that you tried to do?" We obtained 70 responses (52.6% of 133 interviews). Out of those 44 (62.9%) gave "leaving the building" as their answer! Taking cover in a doorway is another high priority, with both advantages and possible dangers.

How important is the human factor of sex in occupant behavior? In Santa Barbara, possibly stimulated by the pattern of aftershocks, those injured while exiting buildings were mainly young men (8 to 1, as shown in Table 7). One example comes to mind. A young couple were in their living room next to the exit door. Outside there was a long railing that forced a person going out to walk next to the house for some 15 to 20 feet before turning into the garden. The husband dashed out, dived over the railing, fell and suffered wrist injuries. His wife followed more quietly, walked around and was uninjured.

When it comes to interactions with other people in an earthquake, what are the human motivations, and are there differences between the sexes? We have observed a few examples of injuries as a result of "panic" in a crowded place. In Coalinga, we have also observed behavior originating in a desire to protect children, friends, and spouses. In one case an injured woman used pieces of a ceiling, that fell on her in a store, to protect herself and a friend. Out of 16 cases of observed behavior aimed at helping or reaching others, 14 (87.5%) were women.

TABLE 29: Coalinga Injuries: If You Moved, What was the First Thing that You Tried to Do?

"First Thing"		Number	Percent [a]
Leave Building		44	62.9
Take Cover:		14	20.0
In Doorway	9		
Under Table	3		
Predesignated Place	1		
Unknown	1		
Move to Another Room		5	7.1
Dodge Objects		2	2.9
Try to Help (daughter, mother)		2	2.9
Try to Find (children)		1	1.4
Other		2	2.9
Total		70	100.1

[a] Because of rounding, the total does not equal 100%

2. **Socio-Economic factors** may possibly explain the great difference in anxiety and emotional reactions between Santa Barbara (2.0%), Coalinga (5.4%), and Imerial County (12.6%). On the other hand, the difference between those "leaving buildings" does not seem significant (Santa Barbara: 10.6%, Coalinga: 14.0%; and Imperial County: 10.0%).

3. **Circumstantial factors** were much in evidence. The timing of the Santa Barbara earthquake was very lucky. In Coalinga many women were caught in their kitchens.

Questions arose on why there were so few injuries and no fatalities in the business district of Coalinga, in spite of the great amount of damage to the old buildings. At least two factors seem to be responsible. The time of day, and day of the week, found relatively few people downtown. Also, a number of the heavily damaged unreinforced buildings were unoccupied, have been vacated, and had vacant second floors because of previous fire code violations.

CONCLUSIONS

We have discussed many issues of earthquake injuries and occupant behavior. We have argued that there is a great need for much empirical data on the subject, which could, eventually, begin to make a difference in saving lives, pain and suffering. The evidence suggests that there are possiblities for significant benefits to be achieved. There is a need for a comprehensive approach to the problem of earthquake injuries and we have described and suggested a suitable conceptual framework.

We have presented preliminary results from our epidemiological study of the role of the physical environment and occupant behavior in earthquake injuries. This deals with four earthquakes, from 1978 to 1985, in California and Chile, and we are continuing the field work of collecting data.

In terms of future research needs on the subject, three items come to mind:

-- A new approach seems to be needed, to develop the eventual mass training and education of the public towards a safer behavior during earthquakes. It should be aimed at reducing detrimental immediate reactions, and increasing the ability of adapting to earthquake situations, and using the short time available during the event for both decisions and actions aimed at maximizing the probability of safety and survival.

-- The accumulation of empirical data and theoretical insights, should proceed parallel with new engineering developments towards safety from injuries, that go beyond the needed work on structural design. This should include, for example, the safety of building contents, non-structural elements, and the development of cheap and effective safe "earthquake shelters" within our buildings, old and new.

-- International cooperation should be particularly valuable here, since earthquakes are relatively rare and simultaneous efforts in many parts of the world can create a most valuable synergism.

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I.2
THE ROMANIA EARTHQUAKE OF MARCH 4, 1977:NOTES ON THE EFFECTS THE POST-EARTHQUAKE REACTION, AND THE FUTURE ACTION NEEDS

Horea Sandi*

1. INTRODUCTION

The social awareness about the real dimensions of earthquake risks is increasing, given the general social development followed by an increase of the elements at risk and in many cases of their vulnerability, the experience of recent destructive earthquakes, and the higher degree of social education and of effectiveness of mass media. This awareness generates an increasing pressure for mitigation of risks, by adopting sound coordinated strategies in this view.

The control and limitation of the risk of adverse effects of strong earthquakes is determined essentially by two major factors:

(a) the control and limitation of the vulnerability of the elements at risk, leading to a limitation of immediate eventual earthquake effects;

(b) the social ability to promptly and efficiently react in the aftermath of strong earthquakes in order to prevent potential severe chain effects, as well as losses due to a prolonged paralysis of the social mechanism.

Any attempt to develop an integrated plan of mitigation of seismic risks must consider the two factors mentioned. The development of such protection plans must rely, on the other hand, on a sound filtering of the experience of destructive earthquakes. This paper is intended to present, in this relation, a view on some significant aspects pertaining to these factors in the light of the experience of the Romania earthquake of 1977 and, thus, to contribute to the development of improved general earthquake protection strategies.

2. SOME DATA ON THE ROMANIA EARTHQUAKE OF 1977 AND ON ITS EFFECTS

2.1. General

A summary of some data on the destructive Romania earthquake of 1977 and of its effects, based on the detailed infor-

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mation provided in /1/ and /12/, is given in this section.

The earthquake of Friday, 4 March 1977, occurred at about 22.00 o'clock, local time. Its mechanism was of multi-shock type and the last, and strongest, shock was at a depth of 109 km and had a Richter magnitude of 7.2. The earthquake affected with intensities of VII MSK or more (practically up to VIII or VIII +), an area of more than 10^5 km^2, of which some 80% belongs to Romania. A few instrumental data obtained, as well as the interpretation of the results of the post-earthquake surveys, have shown that, for extensive areas, the ground motion was characterized by relatively low predominant frequencies, of 0.5 to 1.0 Hz. This was the case for the Southern part of Romania, characterized by thick and relatively soft sedimentary geological layers. On the other hand, it appears that, for zones with different geological conditions, located in the mountain area or in the North-East of the country, predominant frenquencies were of 2.0 to 4.0Hz. There was apparently a strong correlation between the predominant frequencies produced during the 1977 earthquake and the predominant frequencies corresponding to the records of weak earthquakes at various seismological stations. The area affected by intensities VII or more is inhabited by some 50% of the population of Romania and covers centers of important social and economic activities where a wide variety of artifacts of man were located. Among other, it must be mentioned that more than one million apartments built during the post-war era and designed to resist earthquakes were located in the area referred to.

The total number of victims was 1570 (of which some 90 % were in Bucharest) and the overall economic losses were higher than 2 x 10^9 US dollars /1/.

The earthquake had effects on the natural environment at numerous places (rockfall, small landslides, modification of the water table, formation of new wells, subsidence etc.), but nowhere did these effects play a significant role in relation to social and economic losses.

2.2. Some Data on the Performance of Old Buildings

The diversity of structural behavior during the 4 March 1977 earthquake has been put especially to evidence by the case of buildings (apartment buildings, hospitals, schools, etc.). The degree of damage undergone by buildings, especially by apartment buildings, ranges from apparently little or no damage to heavy damage and even collapse.

The heaviest consequences on old buildings have been recorded in Bucharest, where 27 apartment buildings located in the central area have collapsed, totally or partially. Most of these buildings had been heavily affected by the 1940 earthquake, so that they did not possess the necessary resistance capacity for a second strong earthquake. Most of the buildings that collapsed were located at street corners, which has confirmed the unfavorable effects of overall torsional oscillations, of whipping behavior and of shock transmitted by adjacent buildings. The buildings which collapsed were six-to twelve-story high and relatively flexible, such that their fundamental natural periods fell in the range of predominant periods of the ground motion, in some cases after some flexibilization due to post-elastic strain was produced during the earthquake.

The old, relatively low, bearing masonry buildings, characterized by a higher stiffness, have shown a better performance as a rule especially in Bucharest. Collapses were noticed in isolated cases, outside of Bucharest.

An overall picture of the damage distribution for the old buildings is given by the statistical damage spectra derived on the basis of an extensive survey of the performance of buildings /1/, which was under other at the basis of the vulnerability analysis summarized in /11/.

The category referred to as having the most affected buildings in the City of Bucharest, presented some specific construction features. Their bearing structure was in most cases composite, consisting of bearing brick masonry and reinforced concrete members oriented horizontally and vertically. There was no concern during design for earthquake protection and, in most cases, even for their protection against other horizontal loads, like those due to wind.

The pattern of behavior of some of the old tall buildings, which represented the most heavily affected category, may be understood in connection with some features of their urban and architectural planning, of their structural design, of the quality of construction works, of their history of service, overloading, maintenance and intervention of man during service. One may emphasize in this respect:

(a) lack of concern for providing adequate conditions for neighboring buildings (neighboring buildings with strongly different dynamic characteristics and unsufficient separations were bound to undergo heavy hammering/pounding efforts);

(b) prevalence of architectural planning over structural design, that led rather often to unsuited structural solutions, characterized by lack of dynamic symmetry, discontinuities in the vertical bearing members, etc;

(c) lack of concern in the structural design for protection against earthquake loading and even against static lateral loading (like that induced by conventional wind);

(d) poor quality of buildings materials and sometimes of construction works;

(e) overloading by the strong earthquake of 10 November 1940 any by blast effects due to war bombing;

(f) negative effects of corrosion, of traffic induced vibration etc.;

(g) lack of appropriate maintenance, rehabilitation and upgrading works;

(h) unsuited interventions that sometimes weakened load-bearing members of crucial importance.

2.3. Some Data on the Performance of New Buildings

The new apartment buildings, built after 1950, present a wide diversity of architectural planning and of structural solutions. The solutions adopted have appeared especially in standardized design. The standardized solutions of large-series apartment buildings adopted in Romania may be divided into two categories: low-rise buildings (up to 5 stories) and high-rise buildings (8 to 18 stories, the most frequent being that of 10-11 stories). Various construction technologies have been used for the various structural systems, including prefabrication and industrialized forms for cast-in-place concrete. The structural solutions have varied considerably.

Three cases of partial collapse of new apartment buildings have been recorded. Two of them were related to tall buildings in Bucharest where the groups of apartments located around one of the stair cases (placed in each case at the extremity of the building) were affected. For one of the buildings (a four-staircase one), the two lower stories collapsed, such that the upper six stories moved downwards without reversal. For the other building (an eleven-story six-staircase one), the group of apartments was reversed and destroyed.

The behavior of bearing wall masonry structures, designed and erected according to the code provisions, has been generally a good one, these buildings having undergone only rather slight damage. An exception, characterized by heavy damage, was represented by a few buildings, located in the cities of Ploiesti and Craiova, for which no satisfactory earthquake protection measures had been adopted during the design.

The most characteristic damage produced by the earthquake to masonry buildings may be characterized as follows :

- inclined cracks, especially in longitudinal walls, starting as a rule from the corners of openings;these cracks have been some times wide, and their orientation has been as a rule at 45° (sometimes X-shaped), putting to evidence the failure of masonry under principal stresses (due to lack of tensile strength), this kind of failure affected either only the mortar,or even the bricks ;

- horizontal cracks in walls, under the floor over the first story, or at the lower and upper ends of vertical members located between windows;

- long vertical cracks in walls, sometimes along the whole height, as well as in window benches, or at the zone of crossing of longitudinal and transverse walls, due to local low quality of work (weak mortar, absence of mortar, etc.);

- tilting of walls;

- dislocation and expulsion of masonry at corner zones.

The large panel construction represented an increasingly important share of the new construction. The performance of these buildings was good or fair in almost all zones for five-story as well as for eight-or nine-story buildings. Some features of the specific damage pattern must be nevertheless emphasized. The localization of damage was in most cases at the joints, seldom at the lintels and more seldom at the vertical bearing parts of panels. Damage was observed also at the corners of panels for one solution characterized by connections at the corners, by means of thick reinforcement bars. The lower rise, five-story panel buildings presented more extensive damage in areas where the predominant frequencies of ground motion are believed to have been higher, as in the City of Iasi. The opening of cracks at joints or at other parts showed a tendency to increase in time,apparently in connection with the increase of ground settlements. This generates, of course, scare, given the risk of corrosion of reinforcement, especially of the reinforcement crossing the joints.

The cast-in-place, reinforced concrete, shear wall buildings that present the greatest weight among the structural solutions adopted in seismic zones, especially for high-rise buildings, have shown various behavior patterns, as a function of their overall number of stories, and of the structural solution, as well as of the intensity of ground motion. Thus, five-story buildings have shown a good or fair behavior, independent of the structural solution adopted (smaller or large intervals between shear walls).Little important damage has been recorded for these buildings on the other hand the eleven-story buildings have shown, in some cases, heavy or medium damage, for various types of structural solutions, especially at the lower stories of buildings. The damage has been apparent especially for structures with larger intervals between shear walls, with a free first story, used for shopping areas.One of the principal causes of the damage recorded has been the general design, not always adequate for resistance to lateral loads. One must mention in this relation following shortcomings of solutions adopted:

- a) stiffness discontinuities in case of open stories,which have led to concentration of high deformation at the first story, or to insufficient connections to assure a spatial interaction of floors and transverse shear walls;

- b) non-uniform distribution of stiffness at a given story in case of some structures with short intervals between parallel shear walls, where tendencies of separation have occurred in the staircase zone;

- c) low ductility.

The following typical damage must be mentioned for high - rise buildings with cast-in-place reinforced concrete shear walls:

- vertical cracks at various stories of the buildings, due to shrinkage effects, favored by the absence of corresponing continous reinforcement, which have increased their width after the earthquake;

- horizontal cracks, especially in the zones of casting interruption, at buildings erected by means od sliding-form technologies, as well as in the zones of joints of floors with the bearing walls;

- cracks and concrete crushing in lintels of the lower stories and wide cracks, ranging up to destruction of the extreme zones of horizontal sections, where buckling of reinforcement and concrete expulsion have occurred.

The new reinforced concrete framed structures,with five,or eleven to twelve stories, for which a regular pattern of columns and beams has been provided, have shown generally a much better performance than old buildings with reinforced concrete framed structures, for which the pattern of columns has been an irregular one. The damage has been localized generally at the lower two or three stories in case of tall buildings and consisted of damage of the main structure, as well as of the non-structural parts (infill masonry, cladding, etc.) placed between structural members. A main cause of damage of framed buildings consisted of the important gap between actual stiffness and stiffness estimated in design. This is due to the lack of attention paid in design to the contribution of non-structural members to the overall stiffness of buildings. The presence of infill masonry modifies considerably the stiffness. This can lead to non-symmetry, significant torsional effects due to the irregular pattern of walls that can produce an overloading of some of the columns. It must be mentioned that most damage in framed structures has been observed at the first, open stories, used for shopping areas.

The service systems (sanitary, electrical, heating, etc.) of apartment buildings did not undergo directly damage. They were damaged in cases when the building elements supporting them were affected more seriously (expulsion and collapse followed by indirect damage).

As in the case of old buildings, an overall view on the damage distribution pattern in Bucharest is given by the statistical damage spectra of /1/.

The effectiveness of earthquake resistant construction may be evaluated, keeping in view the proportion of apartments collapsed :out of almost 400,000 apartments built in Bucharest, from 1950 to 1977, only some 40 apartments, located around two staircases of two different buildings, were destroyed, leading directly to victims, i.e. the proportion of collapsed apartments was slightly higher than 1×10^{-4}. There were no collapses to affect people in inhabited apartments built during the same period outside of Bucharest. In case one considers the total number of earthquake resistant apartments built in zones affected by intensities of VII or more, the proportion of collapsed apartments reduces to about one third of 1×10^{-4}.

2.4. Some Data on the Effects upon Urban Systems

The experience of destructive earthquakes shows that, in many cases, the overall losses are due to an important extent not directly to collapses, but to chain effects that can significantly magnify (in some cases very many times) the initial losses (the effects of fire during the San Francisco, 1906, or the Tokyo, 1923, earthquakes dramatically confirm this statement).

The second order, or chain, effects did not play an important role during the Romania earthquake of 1977. Given the limited height of buildings and the width of streets, the collapses of buildings, even in Bucharest, where some 30 taller buildings (around 10 stories high) collapsed, did not affect the posibility of evacuation of neighboring buildings when that became necessary. The rescue teams did not encounter significant access difficulties when tackling buildings where people buried or injured needed help. Earthquake induced fire was limited in Bucharest to two collapsed buildings and it was quickly confined and extinquished. Summarizing these effects, it may be stated that chain effects were practically negligible and the cases of damage or failure could be analyzed and dealt with each of them by itself.

The number of apartments affected by heavy damage or collapse totalled 32,900.

It is useful to mention also some facts in relation to the risk of loss of life generated by earthquake induced damage. The central part of the small town of Zimnicea, consisting of o l d, low quality, low rise buildings, was to a great extent destroyed or damaged beyond repair by the earthquake, such that afterwards it was fully replaced. In spite of the numerous cases of collapse or damage beyond repair, the number of victims was extremely low, since almost all inhabitants managed to flee from the collapsing one-story houses. Of course, the chances of escape from collapsing tall buildings are much lower. In some other towns, none of the cases of collapse of low rise buildings led to victims,but several casualties were due to collapse of chimneys or of other non-structural parts of buildings, that hit persons walking on the streets.

3. SOME DATA ON THE POST-EARTHQUAKE REACTION AND ON THE FACTORS HAVING INFLUENCED IT

3.1. General

The society must defend itself, as far as possible,in order to minimize the losses inflicted by strong earthquakes.Moreover, the society should turn the earthquake impact to i t s advantage, using the opportunities created in this dramatic way to l e a r n and open new development paths, because have been relaxed t h e otherwise almighty constraints of the status quo.The way in which the earthquake impact was used after the Skopje earthquake o f 1963 represents a remarkable example in this sense.

The post-earthquake reaction, in the narrow sense of actions undertaken in the aftermath of an earthquake during a limited time period,includes two main and complementary components :

(a) the pragmatic aspects, aimed essentially to limit the earthquake induced losses;

(b) the scientific aspects, aimed essentially to convert the earthquake impact into an opportunity of societal learning.

The two apsects referred to, together with a discussion of the favorable factors having prompted the post-earthquake reaction, are briefly reviewed further on.

3.2. The Post-Earthquake Reaction.Pragmatic Aspects.

Turning now to the post-earthquake reaction,it may be noted that small scale rescue operations were started in Bucharest during the night of the earthquake by some small teams. A large scale reaction started the next morning, following the decisions taken during the night by the government. The active participation of the head of state, who personally coordinated the acquisition of specific information, as well as making and carrying out decisions in relation to the reaction strategy, must be mentioned. It also must be mentioned that the resources of the whole country were thoroughly coordinated and used in order to overcome the earthquake, which affected more or less half of the territory of the country, and to accelerate the recovery at a national scale. Strong teams, in which army units played the major role and which were supported by a massive deployment of building equipment, started to work the next morning, in order to rescue survivors and to remove the debris. During the same time numerous design and research engineers were sent to do a free inspection and survey of the central area of Bucharest, that was the most affected. The headquarters of civil engineering work were organized by the People's councils of counties and towns, including the People's council of Bucharest. Numerous institutes with competent design and research engineers existed at the earthquake time in Bucharest and in several other major towns strongly affected by the earthquake. Nevertheless, numerous competent design engineers from the institutes located in towns not affected were mobilized and brought to the most affected areas, particularly to Bucharest, in order to participate in the recovery effort. The headquarters defined the most urgent tasks for teams of engineers asked to evaluate the components of the building stock (residential buildings, industrial structures, etc.) and to propose solutions of emergency intervention (introduction of provisional supports to damaged buildings, adoption of emergency strengthening solutions, adoption of emergency demolition solutions, in accordance with the specific situations). Sufficient quantities of buildings materials were made available for these emergency interventions.

Some 35,000 families lost their shelter due to the earthquake. New shelter could be provided promptly primarily because of the fact that the construction of new residential buildings ran at that time at a rate of more than 150 000 apartments a year (almost all of them being built from centralized funds, to be rented or sold to the population after completion of construction). It was also possible, without significant difficulties, to provide provisional shelter to residents of buildings requiring more serious repair and strengthening operations. In spite of the fact that several hospitals were damaged, some severely, the medical network was able to cope with the burden of injured persons.

In relation to the design regulations in force at that time, it may be noted that, by means of decisions at the government level, some major corrections were promptly brought to the basic documents defining the seismic zones of Romanis as well as the design forces (particularly the spectral factors).

The headquarters referred to, together with the Central

Institute for Research, Design and Guidance in Civil Engineering, distributed the tasks of design of repair and strengthening to various institutes of design and research and assigned some tasks also to faculty staffs, which were particularly active in all centers. The repair and strengthening solutions used were adopted specifically for steel, reinforced concrete or masonry members or parts of buildings. The research laboratories were very active in testing, under emergency conditions, various jacketing, coating, injections solutions. Among others, a repair solution for reinforced concrete members, consisting of fiberglass coated with epoxy, was developed in INCERC.

The repair and strengthening work, which started under emergency conditions, was not governed at the very beginning by a unique philosophy on the decision making about the nature of technical solutions, the level of safety to be provided to buildings affected, etc. Within a few weeks it was decided to restore the general look of Bucharest, replacing the collapsed buildings and those to be demolished, with buildings to fit into the existing pattern of the central area. It was also decided to build a new political and administrative center in Bucharest (that major investment is currently under construction). As concerns the level of safety to be provided by repair and strengthening work it was decided to restore the initial earthquake strength of buildings. That philosophy, which was intended to limit the consumption of buildings materials, is being reexamined at present basically in research work, keeping in view the needs of deciding on the adequate level of earthquake resistance with consideration of the importance of the structures and of the duration of subsequent service, given the general plan of renewal of the building stock. It may be noted here that the older (pre-war) building stock, which was not designed initially to resist earthquake will be replaced almost entirely by the end of this century, given the rate of new construction.

The replacement of collapsed or demolished buildings proceeded at a fast pace. At the locations of most of the thirty buildings collapsed in Bucharest one could see at the end of 1978 new inhabited buildings adapted to the architectural environment and built at a qualitative level exceeding the average.

The rate of new construction was slowed to some extent during 1977 and partly during 1978, due to the recovery effort. Measures were taken nevertheless in order not to reduce the total construction output during the five-year plan period 1976 - 1980. This represented a very intense construction effort during the second half of the five-year plan period.

The earthquake resistant design regulations were replaced in early 1978: the seismic zoning standard STAS 2923-63 was replaced by the new standard 11100/1-77, which basically endorsed most of the corrections made shortly after the earthquake. The earthquake resistant design code P.13-70 was replaced by the new code P.100-78 (slightly revised subsequently as P.100-81), which introduced significant improvements in the design rules. The spectral factor was adapted to the knowledge provided by the earthquake. The qualitative recommendations on providing ductility were replaced by a consistent approach specifying quantitative conditions required by the providing of ductility(limitation of gravity induced compressive stress in columns, limitation of conventional tangential stresses in sections of columns and of shear walls, specification of minimum and maximum reinforcement ratios, detailing rules, etc.). Strong emphasis was put on the selection of structural solutions intended to provide a high degree of resistance under economical conditions, (limitation of masses and height, avoiding of asymmetries, providing of stiffness and ductility etc.). The concern for adoption of economical solutions was constantly present.

According to a program of drafting design regulations for various categories of structures, which was endorsed by the National Council on Science and Technology, work was started already in 1977 in order to draft new codes for the earthquake resistant design of bridges, of dams, of some specific industrial equipment, etc.

There was a constant concern for learning from the earthquake and for improving the design and construction work quality. The government agencies involved in this field (primarily the National Council on Science and Technology and the Central Institute for Research, Design and Guidance on Civil Engineering) supported strongly the general effort to improve the technical knowledge. Some specific details on this subject are given in next section. The important contribution of highly qualified foreign specialists of the field of earthquake engineering must be kept in view here too. The discussions with well-known specialists of several countries or international agencies (Japan , Soviet Union, United States of America, UNESCO etc.) helped the design and research engineers considerably in better understanding of the earthquake consequences, of the needs of rehabilitation and upgrading of the buildings affected, the needs of improvement of the design regulations, the specific problems raised by construction techniques ,etc. The government agencies referred to actively contributed in making these contacts as efficient as possible under the post-earthquake circumstances. These government agencies consistently coordinated also the efforts to acquire specific scientific information (see in this respect next section too). The improvement of the research facilities was also a constant concern at the government level during the post-earthquake period. Besides other directions kept in view, the most important direction is represented by the seismic testing station which is currently under construction at INCERC-Bucharest (this station will include two shaking tables and a testing hall endowed with strong reaction walls and hydraulic actuators). The research branch at Iasi will also considerably extend its facilities in the near future.

The international aid received during the post-earthquake period must be mentioned here too. Three UNDP/UNESCO national projects have contributed considerably in improving the research equipment and the technical level of the Romanian civil engineering community by means of consultants, services and of fellowships. The important aid provided by the government of the United States of America must be mentioned here too (the fraction of it devoted to research equipment played a particularly important role). The aid provided by Soviet Union, Japan, China and other countries must be mentioned here too. The positive effects of the regional UNDP/UNESCO/UNDRO and UNDP/UNIDO projects, in the period from 1981 to 1983, which were organized to a great extent due to the impact of the earthquake of Romania and of Yugoslavia (15 April 1979) should be noted too at this place.

3.3. The Post-Earthquake Reaction. Scientific Aspects.

A strong earthquake affecting a populated and developed area may represent, if this opportunity is properly used, a source of information of inestimable value. It is a task of highest priority for all the professions and professionals that may subsequently contribute by means of their knowledge to the mitigation of seismic risks, to collect the highest possible amount of relevant information during the shaking and thereafter.

Some main categories of information to be considered are:

a) Data on the ground motion, as well as on the motion and performance during the earthquake of various artifacts of man;

b) Data on the damage inflicted of various buildings, engineering structures, infrastructure components etc.;

c) Data on the complex losses due to the erathquake;

d) Data on the actual reaction of individuals and of various protection systems designed to more or less automatically react during the earthquake.

The post-earthquake activities carried out in Romania after the 1977 earthquake are presented to a large extent in /1/. Some data on this subject are nevertheless summarized here.

A standard macro-seismic survey was undertaken by the Center of Earth Physics and Seismology. Some 12.000 copies of a questionnaire were distributed to practically all localities (some 2,000 copies of them were distributed in Bucharest). The methodology adopted was that of the MSK scale, which was standar -

dized in Romania prior to the earthquake. The isoseismals compiled on this basis within three months after the earthquake were in very good agreement with the isoseismals compiled in Bulgaria and USSR following the same methodology. This activity was related exclusively to the category of data (a).

The information provided by the macroseismic survey was completed by the instrumental information, which was of seismological and of engineering nature. The seismological information contributed to the localization of the source and furnished thereafter a considerable quantity of information on the sequence of aftershocks. The recording of aftershocks (the strongest of which were in the range of Richter magnitudes 4 to 5) was and is carried out as a routine activity. The strong motion recors obtained durring the earthquake provided information on the motion of the ground at several places, as well as of a type of tall buildings. All the instrumental information of this category pertained to the category of data (a).

A survey of the damage distribution was organized in Bucharest under the auspices of the National Council on Science and Technology. More than 18.000 buildings of the City of Bucharest were investigated according to a methodology derived on the basis of the MSK-scale methodology. The buildings were categorized according to two criteria : structural system and fundamental natural period. The following categories of buildings were considered :

1) buildings made of weak materials (adobe like);
2) brick masonry with flexible floors (built before the war);
3) the same (built thereafter);
4) brick masonry with rigid floors (built before the war);
5) the same (built thereafter);
6) buildings with r.c. frame strcture;
7) buildings with r.c. bearing walls (with small inter-wall distances);
8. the same (with wider inter-wall distances).

The basic result of the survey was represented by the statistical damage spectra derived for 1 km x 1 km squares of Bucharest, for each of the structural systems considered. The processing of data made it possible to compile maps of macro-seismic intensities related to some definite intervals of periods of oscillation /1/. The statistical damage spectra were used thereafter also in order to derive conditional damage histograms characterizing the vulnerability of the various structural systems /11/, /16/. The information obtained on the basis of this survey was thus related to both categories (a) and (b) referred to in this section. The basic information was obtained within two or three months after the earthquake, leading to the intensity maps. The additional processing required to derive vulnerability characteristics was carried out in 1982.

Another survey of the damage distribution was organized in Bucharest under the auspices of the Central Institute for Research, Design and Guidance in Civil Engineering. The study was carried out by a staff of the Design Institute of Bucharest and of INCERC (Buildings Research Institute). More than 800 buildings built according to several standardized design solutions, were investigated, so that any of the types of standardized buildings considered was exhaustively dealt with. For any types to which more than 100 buildings pertained, the fundamental natural periods for three directions (oscillations in the longitudinal plane, oscillations in the transverse plane, oscillations of overall torsion) were exhaustively determined, using the ambient vibration technique. The statistical distribution of damage was investigated and correlations with the location and with the azimuthal orientation were derived. This study, which provided essentially data of category (b), but which was of interest also in order to confirm the data related to category (a), was completed within about one year after the earthquake. The results are summarized in /1/.

Another survey was carried out for a sample of several tall buildings for which pre-earthquake experimental data on the natural periods were available. This survey included the determination of the post-erathquake natural periods, along with the investigation of damage undergone. The increase in natural periods was correlated with the damage observed as well as with the azimuthal orientation of buildings /1/. The data obtained have shown that light damage involved an increase of natural periods of no more than 20 to 25%, while moderate damage involved as a rule an increase of no more than 40 to 50%. The data obtained in this way pertain again to the information of category (b). This work was carried out within some six months.

Several individual structures were analyzed in depth, using experimental and/or computational methods. The structure for which the study was the most detailed is that of the main hall of the Exhibition of Achievements of the National Economy, which consists of a 96-meter span steel dome supported by 32 radially oriented couples of columns. The latter study included an experimental biography :natural periods of translation along two directions, of rotation in a horizontal plane and of ovalization of the main ring supporting the dome, determined before the earthquake (July 1976), immediately after the earthquake (March 1977) after the provisional strengthening (April 1977), before the final upgrading work (July 1982) and after this work (July 1984). This experimental work was accompanied by engineering calculations (analysis in the linear range, non-linear time-history analysis on a simplified model, estimate of the ultimate static acceleration on several simplified models, corresponding to the different directions of oscillations and to the pre-and post-upgrading stages). The experimental data obtained up to 1982 are given in /1/. Another case to be mentioned here is that of a tall building for which a strong motion record at the top floor was obtained during the earthquake. The experimental data were used for response spectrum and Fourier analysis tecniques and engineering calculations were carried out for both cases referred to.

It must be noted also that, besides the projects referred to which were very directly linked to the earthquake experience, important research efforts, sponsored from centralized funds or from funds of several clients (design institutes, industrial enterprises etc.), were and are being currently devoted to the general improvement of knowledge in the fields of seismology, engineering seismology, earthquake engineering, general structural engineering etc. The topics dealt with cover a wide field, ranging from fundamental problems to studies of specific site conditions, specific structural solutions, etc.

3.4. Remarks on the Factors Having Prompted the Post-Earthquake Reaction.

It is important to underline some major factors that made possible at that time (1977), a strong reaction to the seismic event. These are:

(a) the general strength of the national economy, which made it possible to carry the heavy burden of losses and of the immediate post-earthquake recovery efforts;

(b) the existence of a large earthquake-resistant building stock, which could be kept in service and house people and current activities, and also represented a good operational basis for the intervention activities;

(c) the high rate of construction for new apartment buildings, which created a buffer residential area and made it possible to promptly house practically all inhabitants whose dwellings were damaged beyond repair or needed important rehabilitation works;

(d) the availability of a satisfactory amount of building materials, building equipment and other material resources of good quality, which made it possible to put into practice the solutions of intervention adopted;

(e) the existence of a satisfactory number of engineers, technicians and workers of good qualifications, which made it possible to develop technical solutions required by the intervention works and to put them into practice;

(f) the existence of good operational bases, in the proximity to sites requiring intervention and also the existence of good access possibilities to the sites repairing intervention;

(g) the existence of a sufficient number of scientists and technicians who were able (with the very precious assistance of highly qualified experts of several countries, having visited Romania in the aftermath of the earthquake) to summarize the experience of the earthquake and to develop some technical proposals of action and of important improvements of the regulatory basis of technical activities;

(h) last and not least, the ability of the government to keep the general social order, to mobilize and coordinate the existing resources of various natures, to make and implement prompt decisions.

4. SOME NEEDS OF FUTURE ACTION

4.1. Control and Mitigation of Seismic Risks

A first major factor mentioned in the introduction in relation to the limitation of overall risks is represented by the limitation of the vulnerability of the various elements at risk and/or of their exposure(the seismic risks are determined also by the factor of seismic hazard, but the possibilities of man in relation to this factor are practically limited to the appropriate selection of sites of human activities, i.e. avoiding the particularly hazardous sites).

The limitation of vulnerability and exposure of elements at risk must represent a constant concern in the development of strategies related to various fields of work. In relation to the consideration of new buildings and of other strctures, it is necessary to apply the rules of earthquake resistant buildings. This implies the development of the corresponding regulatory basis, the education of people as well as organizing the appropriate framework for application in practice of the knowledge accumulated. The general social and economic development spontaneously adds new elements at risk, some of them characterized by a potential for heaviest consequences fo eventual earthquake induced damage. Romania has entered the nuclear age and the nuclear power plants, as well as some components of the auxiliary nuclear industry, which will strongly increase their number in the near future, represent the most significant examples in this sense. On the other hand, the tendency to increase building density increases also the seismic vulnerability, if not at the level of individual buildings, then at the level of urban systems. Other factors increasing the vulnerability could be mentioned too. All these facts require a rethinking of the paths of general development as well as of the criteria of earthquake protection and the adoption of appropriate measures . The experience of the Romania earthquake as well as of numerous other strong earthquakes, has put to evidence the importance of the contribution of the high vulnerability of older buildings and other artifacts of man to the overall seismic risks. The control and mitigation of seismic risks must include therefore, as an essential component, the concern fot the gradual rehabilitation and upgrading of the existing building stock, especially of the older buildings, not engineered to resist earthquakes. Similar attention must be paid also to the more recent buildings which , in spite of having been built according to some earthquake resistant design rules, may not correspond any more, due to the adverse effects of the previous earthquake(s) or of other sources of overloading, of corrosion and aging, or, simply, to the modifications of the earthquake resistance requirements involved by the more recent knowledge. The intervention on the existing building stock must be planned in connection with the general activity of planning of the development of urban systems. The seismic risk analysis must lead in principle to the specification of categories of buildings of different priorities and correlate the general development plans with the deadlines set for intervention for the various categories requiring action. A more analytical view in this respect is given in /7/.

Considering now the technical support that must be provided to the design activities, it may be mentioned that Romania benefits from design regulations for conventional buildings and engineering structures that are generally up-to-date and for which a constant concern of periodic revision and improvement exists. Nevertheless, the task of completing and updating the regulatory basis with some important components is far from being fulfilled. The most stringent present needs are :

(a) the improvement of provisions related to the seismic conditions of the country;

(b) the development of a regulatory basis concerning the earthquake protection of the existing building stock;

(c) the development of design codes for non-conventional structures, like nuclear power plants and other components of the nuclear industry;

(d) the development of a regulatory basis concerning the earthquake protection of various built systems, like urban systems, lifelines, etc.

4.2. Earthquake Preparedness

The earthquake preparedness measures must make sure that a prompt and efficient reaction will take place in the event of a strong earthquake. This reaction must cover both main aspects referred to in section 3.1., namely the ability to limit the adverse earthquake effects and the ability to gather a great amount of significant information of scientific and technical nature.

A comprehensive view of earthquake preparedness needs must consider the various post-earthquake phases and the corresponding need for action. One can consider in this respect the following post-earthquake periods [9] :

a) the occurrence phase, lasting for seconds and minutes;

b) the short phase, lasting for hours and a few days;

c) the medium phase, lasting for weeks and a few months;

d) the long phase, lasting for some years.

Generally speaking, the shorter the phase referred to, the more one must rely on appropriate preparedness measures adopted prior to the earthquake. For the occurrence phase it is possible to rely only on the action of some automated systems (e.g.: the strong motion accelerographs, the safe-shut-down devices, the emergency supply systems, etc.), or on the appropriate reaction of some specially trained people. During the occurrence phase some private persons may turn off gas or electric power, may flee, etc. The authorities could, in case appropriate preparation is made, alarm the inhabitants of a definite territory, especially against some chain effects like flooding due to collapse of a dam, leakage of gas or some chemical substances, etc. From the scientific view point it is possible to randomly observe some aspects of the natural phenomenon and their influences on the natural and man-made environment. Gathering of information during this period is highly dependent on automated systems, like the components of strong motion arrays. During the short phase, it is time to fight against fire, leakage of natural gas or of various chemical substances, flooding of basement floors from which people cannot escape at the moment; to save people from underneath the debris, and to take some emergency measures aimed to avoid further collapses of buildings. From the scientific view point it is possible mainly to observe visually some earthquake effects and to take pictures of them. During the medium phase, it becomes possible to take technical action in order to tackle some of the highest risk sources of possible destructive aftershocks or other possible strong actions (e.g. strong winds, flood, industrial accidents etc.). This time should be intensively used in order to collect the highest possible amount of relevant information of scientific and technical nature. The long phase is a time for full recovery, and systematic abatement of risks in connection with the adoption of new development plans in the frame of physical and urban planning activities. This must be a time of summary of earthquake lessons, of adoption of comprehensive preparedness plans from all the pragmatic and scientific view points, of course and of thorough and qualified implementation.

The importance of proper functioning of automated devices during the occurrence phase was emphasized already. Besides that, it was also mentioned that a prompt and well oriented reaction of individuals during the occurrence period as well as thereafter may have far reaching positive effects too. Such reactions will be much more likely in case appropriate educational activities are organized. The educational activities should be at the level of public and, separately, at the level of individuals who are assighed some precise tasks, in relation to the components of urban or regional infrastructure, of some branches of industry, etc. The educational activities must be periodically repeated and this must include exercises announced in advance or, eventually, not announced, under conditions intended to simulate some features of the earthquake occurrence. It is useful in this connection to have an idea of the eductional activities organized in Japan in relation to an anticipated earthquake [15]. Another aspect of preparedness is represented by the concern for emergency interventions on damaged structures in order to avoid accidents during the post-earthquake period. This includes analysis of solutions and preparation of guidelines for installing emergency supports or strengthening members, for carrying out emergency demolition works, etc. This aspect of preparedness must be considered in correlation with the various requirements of construction works : education of qualified workers, making sure that building materials and equipment can be made available under emergency conditions, etc.

In spite of the fact that prediction activities are still in their infancy, attention should be paid to the chances offered by eventual successful prediction activities, which, in case of adequate preparedness, may lead to a spectacular decrease in the losses of various natures, first of all in the losses of lives. According to the estimates, a correct prediction and warning would eliminate, in the event of a magnitude 8 Tokai earthquake, 10,900 deaths, 16,700 cases of heavy injury and 98,300 cases of light injury, 275,000 houses destroyed by fire, that would represent the toll of earthquake under standard conditions of no prediction and no warning. These impressive positive effects of prediction and warning can materialize only in case of complex preparedness activity thoroughly carried out.

The scientific preparedness includes two main aspects:

(a) installation and maintenance of various instruments intended to record automatically some significant parameters of the seismic phenomena;

(b) methodological preparation of surveys to be carried out in the event of an earthquake.

The instruments (standard strong motion network, master-slave systems or central recording systems, eventually systems intended to record strain parameters, etc.) represent the object of some specialized projects, requiring a certain investment and, besides this, a qualified maintenance activity. This is not a proper place to discuss details of that direction of work, but it is nevertheless necessary to mention the importance of proper maintenance in order to provide a high probability of proper functioning of the automated instrumentation.

The second main aspect referred to deserves more attention here. A proper methodological preparation of the surveys to be carried out in the event of an earthquake is of primary importance for efficient work under the emergency and stress conditions created by a destructive earthquake. The surveys to be carried out may be categorized as follows:

(a) surveys on the physical effects of earthquake ;

(a.i.) effects on the natural environments;

(a.ii.) effects on the artifacts of man;

(b) surveys on the losses due to the earthquake;

(b.i.) hard losses;

(b.ii.) soft losses.

The surveys on the effects of category (a.i.) wiil cover a wide range of aspects, like surface faulting, landslide, rock fall, subsidence, liquefaction, modification of the water table, etc. In order to derive a high quality of significant information on such effects, it is necessary to collect a comprehensive set of data, including:

- a description of the pre-earthquake environment and conditions such as: features of the landscape (including quantification of the free slopes), data on the geological conditions (including, if possible, geotechnical and geophysical characterization of the ground), data on the effects of some natural or artificially generated events (possible strong rains, flood, effects of quarry blasts etc.);

- a precise description of the earthquake effects, such as direction, length and amplitude of faulting, slope, mass of rocks and distance of sliding related to a landslide, area affected by liquefaction, possible quantitative characterization of earthquake generated wells, subsidence amplitude, quantification of water table modification, etc.

It may be hoped that precise data could be useful for more accurate assessments of the characteristics of ground motion at places where no other information is available.

The surveys on the effects of category (a.ii.) may represent the most significant and accurate source of information, provided the built environment exists and proper surveys were carried out. Such surveys may cover samples of works pertaining to a certain category (in most cases, residential buildings), the investigation of which should provide basically information to be statistically processed, or they may cover individual works, which could represent the object of in depth case studies. The samples representing the object of statistical studies must fulfill two requirements that are unfortunately conflicting : the samples must be sufficiently homogeneous (e.g.: masonry buildings of a rather homogeneous height, quality, age), architectural layout, structural layout, etc.) and they must be sufficiently numerous in order to make possible a statistical processing of data. The individual structures, like industrial engineering structures, dams, bridges, etc., represent cases per se, deserving a special preparatory work, which cannot be tackled at this place. As for the surveys devoted to subsequent statistical analyses and interpretation, it will be necessary again to collect a comprehensive set of data, including :

- a description of each of the buildings and other works dealing with, position and orientation, age, architectural characterization and layout (including a schematic drawing), structural characterization and layout (material, type of structure, quantification of areas of resistant sections along the various directions, description of connections etc.); characterization of soil and foundation conditions (nature of soil, geotechnical characterization, type of foundation, estimate of ultimate overturning moment etc.); and corresponding quantifications (at least in a "broad" way); quantification (at least in a broad way) of some characteristics like dynamic characteristics, static acceleration corresponding to the limit of linear behavior, ultimate static acceleration, ductility; data on the biography (past cases of strong loading and of damage occurrence, past repair and strengthening works, past changes of layout etc.);

- a description of the damage induced by the earthquake (the description must be made in terms that are specific to the materials and structural type, as given in the form of fig. 2.2. of [11]).

Some discussions on the damage evaluation methodology may be mentioned here. A main goal in organizing the damage survey should be that of making possible a most accurate assessment of the ground motion characteristics, given the concepts and methods of structural analysis used in earthquake engineering. The task is of course not easy, given the inherent high scatter of earthquake effects (consider the scatter of damage probability matrices or the scatter of damage degree plotted against static critical accelerations). Nevertheless, efforts must be made in this direction.

The surveys on the effects of category (b.i.) will refer to losses of life and injury, to losses of property and costs of recovery actions (direct losses, as those corresponding to buildings collapsed, cost of rehabilitation works, etc.; indirect losses, as those related to the property lost due to collapse or damage in buildings) monetary losses due to interruption of productive activities, interruption of transportation etc.; description of chains of events having resulted in losses (e.g. immediate cases of casualties and injuries) etc. It is desirable here to describe in qualitative terms and to quantify all aspects that may be quantified, in spite of the difficulties of quantifying some indirect losses which may sometimes hardly be classified exclusively into only one of the categories (b.i) and (b.ii) . A possibly accurate quantification of losses will be of paramount importance in order to carry out subsequent cost-benefit analyses related to the reasonable degree of earthquake protection.

The survey of the effects of category (b.ii) will refer mainly to psycho-social effects, to cultural implications of the deterioration or change of the environment etc. These aspects may be hard to quantify, but their importance should by no means be underestimated. It is most desirable to identify, in the surveys of this nature too, the immediate causes and sources of stress , of psychonervous ailments, etc.

The results of scientific nature referred to in this section will be of use, primarily, for the community affected directly by an earthquake. They can represent no less, a most valuable national or regional contribution to the international effort devoted to the comprehensive understanding of the earthquake problem. It may be suggested that more efforts be devoted at an international scale to the preparatory work for surveys related to all categories of data referred to in this section.

4.3. Some Present Research Needs

We should emphasize some research needs of general interest for the earthquake protection. The directions mentioned here are of primary importance for the analysis of seismic risk, as well as for the decision on the appropriate level of earthquake protection. They are important in Romania and elsewhere and may represent a subject of cooperative research. They are related to:

(a) the analysis of seismic hazard;

(b) the analysis of the mechanical behavior of members, bearing structures, non-structural components and buildings or engineering structures as a whole;

(c) the analysis of the complex losses due to strong earthquakes;

(d) development of an appropriate framework for the miti -

gation of seismic risks related to the existing building stock

The analysis of seismic hazards should encompass the expectancy of activity of various seismic sources affecting the Romanian territory, the specific attenuation laws and the features of local amplification phenomena. This should lead to a better code basis, expressed in terms of maps of the activity of sources and of expectancy of various seismic intensities and, if possible also of maps specifying the expected predominant frequencies of ground motions.

The analysis of the mechanical behavior of various structures, structural components and systems should be directed primarily such as to provide realistic and at the same time operational, full models of the force-deflection relationships. Such developments are needed not only for proper structural components, but also for the analysis of ground-structure interaction phenomena. It must be noted in this respect that, according to several engineering analyses, the ground-structure interaction phenomena played a key role in the behavior of relatively rigid and resistant buildings during the earthquake of 1977. It must be kept in view, also, that the development of non-linear force- deflection models for various structural components and for the ground is a primary condition for a rational calibration of some factors used in code formats and, in the future, for a replacement of the conventional linear structural analysis methods by means of more realistic ones.

The analysis of the complex losses inflicted by earthquake is a necessary step in order to provide the basis for a comprehensive seismic risk analysis. The various loss components (losses of lives, losses of property, disruptions of human activities etc.) must be considered in a coordinated and comprehensive manner. The importance of this direction is supported by an apparent gap in the estimates of overall losses due to different earthquakes. As it was mentioned in section 2.1., according to official estimates, the losses due to the earthquake of Romania, which affected with intensities of VII to VIII (+) an area of some $10^5 km^2$, (which some 10^7 inhabitants), were higher than $2x10^9$ US dollars. For comparison, the earthquakes of Friuli, 1976, and Montenegro, 1979, which affected with intensities of VII to IX (+) areas of a few thousands of square kilometers, with no more than some 10^5 inhabitants, were estimated to have produced losses of 5×10^9 and 4×10^9 US dollars respectively. It is hard to believe that the diference between the Romania earthquake and the two other destructive earthquakes which covered limited areas could be so great. The opposite seems to be more credible. That is why the methodologies of estimation of losses must be rethought and revised. Note in this respect also that an eventual underestimate of losses, or of vulnerability, will lead to a distorted outcome of any cost-benefit analyses and consequently to systematic underdesign and exaggerated subsequent losses in the event of future strong earthquakes.

The development of a framework for the mitigation of seismic risks related to the existing building stock should put into value the preparatory work already carried out in this field. This preparatory work has encompassed topics and methods of evaluation of the earthquake resistance of existing buildings /3/, of analysis of seismic risk and decision methodology /5/, /7/, /8/ etc., of methods of repair and strengthening of various types of structures /6/, /13/ etc. It is time now to start pilot studies on extended samples of buildings and, then, to systematically tackle the various components of the existing building stock, in order to reach within a limited time interval of less than two decades, an acceptable level of seismic risks (note here that according to /2/ a future destructive earthquake of a similar nature to that of the 1977 is predicted for 2004 $\pm$ 4 years and that one of the authors of /2/ participated in the successful prediction in 1974/4/ of the 1977 event).

4.4. A Stringent Task: an Integrated Earthquake Protection Plan

The evaluation of the current state-of-the-art of Romanian activities related to earthquake protection permits to us state that Romania is an at least fairly developed country from this view point. Such a statment is supported by several facts: the number of trained experts, the level of technical solutions adopted in practice (which have at least fairly dealt with the 1977 earthquake, inspite of having been among the most economical in the world), the export of know-how, the number and quality of scientific contributions.

The 1977 earthquake led to a significant progress from almost all viewpoints. In spite of the progress achieved, it is time now to develop an integrated earthquake protection plan at the national level. Such a plan, which should become a component of the national, economic and social development plan, should include sections related to the development and design of new buildings, industries, facilities, etc.; the development and protection of urban systems, lifelines, etc.; the control and mitigation of risks related to the existing building stock; the development of research activities; the preparedness measures of various natures (including correlation with the earthquake prediction activities); the education of population and specialists, etc. A joint effort to answer in the closest future this challenge is certainly justified, it not already overdue.

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I.3 SEISMIC BEHAVIOR OF SOME INDUSTRIAL AND TELECOMMUNICATION STRUCTURES: SOLUTIONS FOR SEISMIC RISK REDUCTION

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The structural layout of some industrial and telecommu - nication buildings, damaged by earthquakes are presented.

Structure seismic response and strengthening measures are described together with conclusions upon the building sa - fety degree, in case of a future event.

1. TELECOMMUNICATION CENTER

1.1. Structural framework. Damage description.

The steel structure is built of columns and girders with stiff fixed joinings, made of riveted rolled steel sheets and profiles. The reinforced concrete floors have been made using steel joints, supporting shuttering for concrete work,all joints remaining embedded in concrete as a protection measure against fires.The masonry work has been carried-out at the same time with the structural framework, using cement mortar, so that the outer walls of the building actually acted as shear- resistant walls of the building actually acted as shear-resistant walls, preventing the free deformation of the respective steel frames and taking over shear forces, due to the seismic loading.

The building structure underwant two major earthquakes of about the same magnitude in 1940 (M=7.4) and 1977 (M=7.2)without interrupting the operation of the telecommunication equipment for 12.000 connection lines.

In the present situation, the 53 columns of I shape steel,

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forming a concrete section with stiff reinforcement, are subjected to high compression stresses (of about 1500-2000 daN/cm^2) under fundamental loading. The additional stresses that might appear during a future earthquake were intended to be taken over mostly by certain additional elements. The floors were calculated for live loads varying between 200 and 600 daN/cm^2, depending on the technological function of the respective floor.

The strength of the building under seismic stresses has been obtained in an empirical way, at the time when it was designed (1933), considering the hyperstatic nature of the structure, which favors its post-elastic behavior. The whole structural assembly, participates in the absorbtion adn dissipation of the energy by using suitable infill materials and by careful execution, providing the necessary ductility. However the structure does not have uniform and symmetrical layout in horizontal and vertical planes, this being subordinated to the architectural outlook (the arrangement of columns presented no regard to continuous alignement and to symmetry, their orientation was different. There existed also indirect transmission of vertical forces). This fact led to rotation effects around a vertical axis, beside the translation ones, due to the seismic forces. Consequently, some additional horizontal forces had been introduced in certain vertical members of the structure, especially in the recessed angles of the courtyard. Moreover, the vertically non-uniform distribution of masses, the great height of the construction (52.50 m, the highest in the country, at the time of its erection, as well as the height of stories- all led to the failures of non-structural members (brick masonry, panels, windows) under the seismic forces of the 1977 earthquake. These were due to the fact that the relative displacements of the storys was considerable, exceeding the values provided by the present norms (for example Hn/200 for brick infill masonry). However at present, part of the ductile damping of some structural members (columns and the absorbtion capacity of other elements (masonry) have been more or less exhausted.

The position of the stiffness center and the position of mass centers respectively for each level of the three distinct zones of the building, as well as their respective excentricities, have been calculated, being known as the plane U shape arrangement of columns and the related loads on each level (basement, ground floor, mezzanine, 10 stories and 2 towers)

In order to estimate the general torsional effects for each zone, the horizontal seismic force has been considered as it applies to each level with an eccentricity against the centroid.

The construction arrangements have been considered for each zone and as a result, the seismic force may be taken over by each respective zone and the assymetry effect calculation for each zone may be neglected.

1.2.Strengthening measures

The strengthening solution should ensure the building against a severe earthquake of II class (according to the earthquake resistant design principles of K.Muto/2/: 0.2 g acceleration), so that in the frame of structure response, the stresses and deformations of structural members should exceed the elastic range limit and the non-structural members could be somewhat damaged. Consequently, a linear calculation method has been devepolled for the new situation, considering also the possibility of stresses within the post-elastic range.

The value of total possible seismic force, to be taken over by the strengthening work (F=16.000 kN), calculated according to Romanian Code P 100-81, has been estimated at about 5-7% of the building weight, which would provide protection for a seismicity degree of 8.0-8.5. Also, the achievement of strengthening work for such an important building, which should comply with the strength and stability requirements stipulated by the present technical regulations, is rendered difficult by the fact that the operation of the telecommunication center may not be interrupted. Consequently, there have been considered only those variants of strengthening solutions which required no indoor interference.

1.2.1. Strengthening of the steel structure by steel reinforcing members, aiming to create strengthened vertical stiffening diaphragms, made of steel trusses, inserted between two rows of columns depending on the restrictions imposed by the equipment.

1.2.2. Compound bracing of the steel structure by reinforced concrete and steel elements. The same bracing principle as the previous one is taken into consideration but the bracing of steel columns is performed by embedding them inside a B 300 reinforced concrete section.

1.2.3. Bracing of the structure by inserting B 300 reinforced concrete elements, building frames, using stiff reinforcement in columns and steel rods in girders. The disadvantage of unaesthetic diagonals is obviated but there are great difficulties to perform concrete casting in narrow spaces, near equipment under operation.

1.2.4. Strengthening of the structure through a reinforced concrete control tubular construction, five levels high, built outside the existing building (inside the courtyard). The connection between the building and the tubular construction is to be made in front of the existing knots of the floors. Taking into consideration the U shape of the building, if a seismic force is developed perpendicularly upon the two aisles, the tubular structure position has a great eccentricity towards the existing building and therefore, besides the translation action, it is stressed also by a general torsional effect. In order to diminish this rotation effect, it is also necessary to perform the stiffening of a front panel of the same height. As the tubular structure has a small weight, in comparison with the building weight, it is necessary to weight the basement of this construction with a 3 m slab foundation, to avoid columns pulling out of the ground.

The new tubular structure, together with the stiffening panels of the existing building, might take over the conventional, additional stresses, due to seismic effect, and the building's natural vibration period would be approximately equal for the two directions and would diminish to about 1.2. - 1. 4. seconds.

1.2.5. Strengthening of the structure by building some diaphragms on the outline of the building, made of B 250 reinforced concrete, cast from the outside. The masonry and the stone plating of the facades are to be removed, preserving the present architecture.

1.2.6. Strengthening of the structure through two tubular structures, made of reinforced concrete, one located inside the courtyard and the other outside the existing building.

2. THERMAL POWER STATION

2.1. Structural framework. Damage description

The structural framework is made of frames with reinforced concrete columns, the span being of 42 m and 9 m, respectively disposed in bays of 9 m. The 10 bays have a central expansion joint of 0.75 m. Between the A and B rows of columns the 12 m high machine-hall located accomodating two turbine-generator units of 125 MW and a revolving speed of 3,000 rpm which subject the hall to some technological vibrations. Between the B and C rown the 28 m high auxiliary building, are located.

The columns of the AB hall have a gradual variable rectangular cross-section with the lower part of 180 x 70 cm.

These columns support on hinged supports, roof trusses made of rolled profile bars, forming simple or compound sections The truss is placed on top of columns, on support devices fixed by M 36 screws in ovalized holes and welded pegs, preventing horizontal displacement but allowing limited rotations on supports. Two travelling cranes with a lifting capacity of 1,000kN operate on rails, laid at +20.50 m level on prefabricated reinforced concrete beams with prestressed reinforcement, The beams are simply supported by the columns. The whole structure is covered with prefabricated concrete caissons.

The structural framework of the BC building is made of frames with cast-in-place reinforced concrete columns and girders, longitudinally fastened by reinforced concrete 35 x 80 cm beams and prefacricated floor. This building is much more rigid as the AB hall structure, being lower, with a much smaller span and having several longitudinal connections.

Therefore, during the braking of the travelling crane which is generating longitudinal inertia forces along the running rails, the measured displacement of a column from the A row was about 4.5 times greater than that of the correspondend column of the B row. Also, the natural fundamental vibration period of the longitudinal frame of the A row, separately calculated, was about 70% greater than the natural period of the longitudinal frame of the B row also taking into account the masonry panels) These results have been obtained after the replacement of the concrete caissons from the level of AB hall roof by thermo - insulated and waterproof, corrugated pressed steel sheets, of ROMCOR type.

The following damage of the structural framework has been produced by the earthquake of March 4, 1977; the columns of the A row suffered large remanent strains at roof level and consequently the trusses slid off the bearings causing the sliding of caissons of one bay and the sliding of the beams supporting the crane rails, off the columns brackets.

2.2. Bracing Measures

- jacketing of columns of the A row, after removing the concrete layer covering the steel reinforcement, by putting on additional longitudinal reinforcing ø 16 mm bars, at 15 cm, joined by ø 10 mm, ties, welded at ends to the existing reinforcement.

- Bracing of columns of the A row, between levels 10.0 m and 16.9 m. in two bays, where the technological conditions permitted. A steel truss, made of two U 24 profiles, has been used.

- Strengthening of front concrete columns by jacketing them with additional reinforcing ∅ 20 mm bars, joined by ∅ 10mm ties, at 10 cm, along a height of 12 m, increasing the cross-section from 60 x 140 cm to 60 x 165 cm.

- Laying of a light cover made of corrugated steel sheet and bracing horizontally, vertically, longitudinally and transversally the roof framework at the upper and lower soleplates of the girders. Taking into account the height of the struc - ture of the machine-hall, it is absolutely necessary to put lengthwise additional longitudinal bracings, at the roof level, made of continuous beams, so that the large deformations of frames might be reduced by passing part of the loadings from the more flexible structures (AB) to the more rigid ones (BC).

3. BARS DRAWING MILL

3.1. Structural framework. Damage description.

The production hall has 4 spans of the same height(H = 7.5 m) and a 5^{th} span of a different height (H=11,7 m) with an area representing about 1/6 of the whole area. Travelling cranes of 50 kN run along the first spans while the last one has no lifting mechanism The current bay is 6 m. The supporting structure xonsists of reinforced concrete columns, embedded in pre - fabricated concrete foundations, prestressed reinforced concrete beams for roof and 3.0 x 6.0 m caissons made of reinforced concrete. At the higher hall, (H=11.7m) the transverse prestressed beams for roof have not been fixed on columns by monolith joints and the roof longitudinal caissons have not been monolith with the supporting beams. Only the joint weldings between the steel plates, embedded in beams and columns, have been performed.

The weldings without seismic effects, but the embedded anchors of the plates were pulled out of the roof beam concrete, and cracks were found in the concrete between the strands, in the supporting area.

3.2. Strengthening measures

Jacketing of the upper and of the columns in order to increase roof beam support with about 10 cm, was provided.

4. WELDED PIPES FACTORY

4.1. Structural framework. Damage description.

The production hall has 3 spans of 18 m each and a bay length of 6m The hall is provided with travelling cranes for 180-200 kN.

The upper part of the columns of the outside row, where the reinforced concrete beams are supported, were damaged. These effects were probably caused by the existence of an adjoining building (the electrical power station) of a smaller height which rests on some brackets of the damaged columns. Cracks and dis - placements of the building front were also caused.

4.2. Strengthening measures

Strengthening columns jacketing them with B 300 reinforced concrete, between the bracket supporting the crane track and the upper end. It has been found that similar damage occurred in the structures with the cross-section of frame type with dis - levelments, such as columns supporting the roof (reinforced concrete stiff plates) at different levels. Thus, large cracks a-ppeared at the level where the column section changes, In this row of columns, cracks and displacements of the concrete also occurred at the upper part of the column.

Considering these fundings, it is recommended for the future, to give up the case of small dislevelments, it is ad-visable to provide uniform heights and in case of large ones, to provide longitudinal joints.

5. ELECTRICAL AND MECHANICAL MAINTENANCE WORKSHOP

5.1. Structural framework. Damage description.

For this purpose a prefabricated reinforced concrete hall with two spans of 24 m and bays of 6 m, with two expansion joints was provided. The hall is equipped with two travelling cranes of 50 kN, and 160/50 kN respectively. The reinforced concrete co - lumns are embedded in prefabricated foundations. The beams have been made of panels assembled by prestressing; purlin and pre - fabricated plate are laid on these beams.

Following the 1977 earthquake, the front longitudinal walls, made of 37.5 cm thick masonry, were displaced and brick crushes and breakings occurred. Because no straps were made, the brick plating has been detached over large surfaces. At median co - lumns detachments of the concrete at the level of the crane track beams, have been found uncovering the reinforcements along a height of 10-20 cm. The steel reinforcement did not buckle up.

Within the joints of trusses with the column ends, some harmless crushes of the monolith concrete took place.

5.2. Strengthening measures

Partial insertion of walls in the facade.

Insertion of longitudinal steel bracings between column upper ends and the level of the crane track beams.

6. PIPE DRAWING MILL

6.1. Structural framework. Damage description.

Hall with 2 spans of 30 m and 24 m and bays of 12m. Pre - fabricated reinforced concrete pillars spatial steel trusses,steel girders for the tracks of travelling cranes of 50 kN with ro - tating jib, roof made of prefabricated reinforced concrete plates.

Following the earthquake, fine cracks appeared at the upper part of columns, at the level of the crane track beams.

6.2. Strengthening measures

The cracks have been grouted with epodur.

7. DRAWING MILL

7.1. Structural framework. Damage description.

The hall has a span of 15 m and bays of 10.77 m.

The structure is made of poured-in-place reinforced con - crete frames. The track beams for travelling cranes of 50kn are made of monolith reinforced concrete. The roof is made of rein - forced concrete shell.

Following the earthquake, the marginal columns, common to an adjoining auxiliary hall; broke in the zone over the adjoining hall. This one has represented an element which caused stiffness variation along the column height.

7.2. Strengthening measures.

Removal of damaged concrete, casing of columns with steel profiles and recasting of concrete for the disleveled part above the adjoining hall.

8. EXTRUDING HALL

8.1. Structural framework. Damage Description.

The hall has a span of 36 m and bays of 12 m.

Reinforced concrete columns supporting steel trusses and steel purling. The foof is built prefabricated reinforced concrete caissons. Travelling cranes of 50 kN and 125 kN are operated.

After the earthquake, cracks appeared at the upper part of the columns, in the recess for the walkway.

8.2. Strengthening measures.

Jacketing of columns, along the damaged parts with angle steel profiles. put on the adges and tightened by steel yokes.

Taking into account the damage caused by the dynamic effect of the earthquake upon the reinforced concrete columns with variable sections, within the zone of the quick change of the sections a gradual transition that should be made between their upper and lower parts is suggested.

9. MACHINE-TOOL FACTORY

9.1. Structural framework. Damage description.

The halls are made of steel frames with 5 disleveled spans of 30 m each and 9 bays of 12 m. Within the first 2 spans there is the heavy mechanics hall, provided with 4 travelling cranes of 150 kN and 300 kN. The steel columns have the lower part made of Vierendeel system caisson and the upper part of I profile, joined rigidly by girders with variable cross-sections of velded compound double T profile. Within the following 3 spans, the light mechanics hall is provided with travelling cranes for 100 kN lifting capacity. The steel columns,of slender simple caisson type, support crossed trusses made of pipes. Upon the rigid columns of Vierendeel type, common for both halls, the trusses rest on rollers. During the earthquake the rollers sprang out, because they had not been provided inside boxes and with blackings for limiting vertical motions and therefore the trusses slid off the bearings.

9.2. Strengthening measures.

Remplacement of the rollers with classical hinged sup - ports, giving up the possibility of horizontal displacement, taking into account the flexibility of columns in the light mechanics hall.

NOTE: The designers of the structures described in this paper were:

1. Arch.L.S.Weeks, New-York for
   - The Telecommunication Center (steel frame made by "Reşiţa" factory, 1931).

2. Energetic Design and Research Institute for
   - Thermal Power Station - West Bucharest.

3. IPROLAM - Bucharest, for
   - Bar Drawing Hall - C.O.S.Tîrgovişte
   - Welded Tube Factory - I.T.Republica, Bucharest
   - Electrical and Mechanical Maintenance Workshop- I.T. "Republica", Bucharest
   - Tube Drawing Hall - I.T."Republica", Bucharest
   - Drawing Hall - "Laromet" plant, Bucharest
   - Extruding Hall - "Laromet" plant, Bucharest
   - Machine and Tool Factory, Bucharest

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I.4 SOME CONCLUSIONS ON THE BEHAVIOR OF THE STEEL AND COMPOSITE STRUCTURES SUBJECTED TO SEISMIC ACTION

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Elena Dragomirescu[xx]

ABSTRACT

The steel or composite structure halls reacted favorably during the March 4, 1977 earthquake; nevertheless damage was noted especially in the cases of heavy roof deck solutions or of inadequate structural conception.

The paper presents an analysis of the damage causes and also the conclusions cencerning the improvement of the Romanian Aseismic Structure Design Code P 100-78 published in 1978.

This paper presents conclusions of the analysis of the steel and composite industrial halls'behavior during the March 4, 1977, earthquake in Romania.

The majority of the steel and composite industrial buildings (over 80 %) were built after 1950.

The design of these structures ensured an earthquake resistance complying with the code provisions in force in the period 1950-1977.

The usual structural systems were:
(a) Steel halls with light roof deck, consisting of trusses or plate girders (rafters) and purlins that support corrugated steel or asbestos-cement sheeting.

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(b) Steel halls with heavy roof deck including trusses or plate girders and reinforced concrete, reinforced cement or light cell concrete plates, resting directly or on steel purlins, or prestressed concrete large panels supported directly by trusses. The roof framing was provided with steel bracings. An effect of shear diaphragm was ensured by the concrete roofing and by the corrugated steel sheeting. Usually the steel halls have been provided with heavy travelling cranes, running on steel girders, or with transport equipment overhanging the roof framing. The built up or latticed steel columns generally had hinged connections to trusses at the roof deck level. The longitudinal stiffness of the structure was ensured by adequate vertical bracing systems.

(c) Mixed halls had the same types of roof deck (see items a and b) but they had reinforced concrete columns. To ensure the necessary longitudinal stiffness of the structure two schemes were used: transversal slender columns and vertical steel bracings, or columns developed on both directions, without bracings.

The most important general aspects concerning the behavior of these structures are:

1. The steel and composite halls with light roof deck had a very good behavior during the earthquake.

No damage to require repair loading to interruption of the production process occurred although in some areas of the country the earthquake intensity exceeded by 1 - 2 grades the values mentioned by the existing code. This better behavior of the steel structures in comparison with other types of structures is explained by the higher strength and ductility of the steel, by the lower weight of these buildings as well as by the excess-strength of the columns (The columns have been checked for the most unfavorable combination of the loads-vertical and horizontal crane forces, wind loads etc.). The high deformation capacity of the steel in the inelastic range allowed the dissipation of the induced energy during the earthquake without damage to structural members, even when the design values of the loads were significantly exceeded.

2. The importance of the bracing systems and of the roof diaphragm in order to assure the general stability of the structures and limit the deformations due to the earthquake was already noted. Thus, when the overloading of some main structural members occurred, the roof diaphragms of corrugated sheet ensured the redistribution of the seismic loads, avoiding the partial failure of the roof framing.

3. The steel and composite halls with heavy roof deck generally had a satisfactory behavior during the earthquake. Nevertheless some cases of damage to members or details producing the failure of some parts of the building have been noted.

The main cause of these damages was the fact that the design values of the seismic forces provided by the code available before 1977 were exceeded by the earthquake.

The failure occurred in some members or member sections without supplementary resistance capacity, where the seismic loading played the main role. In this category are included the upper parts of the columns and bracings over the runway girders and also the connections between the truss and the column. Other parts of the columns, as the bottom parts under the runway girders, designed for the load combinations that considered the horizontal and vertical forces induced by cranes and wind loads, presented enough supplementary strength to support the seismic action without damage. In some cases, the failureswere caused by indequate design of the structures, or of some members only, that could lead to the amplification of seismic loadings effect. For all these analyzed cases one must underline the adequate behavior of the horizontal diaphragm made of concrete plates, that ensured the redistribution of the stresses in the roof plane, from the more stressed members to the less stressed ones, thus limiting the extension of the damage.

4. A number of damaged halls with heavy roof decks, in Bucharest, where the seismic lateral forces were significantly higher a comparison with the provisions of the code available before 1977, are presented in the following:

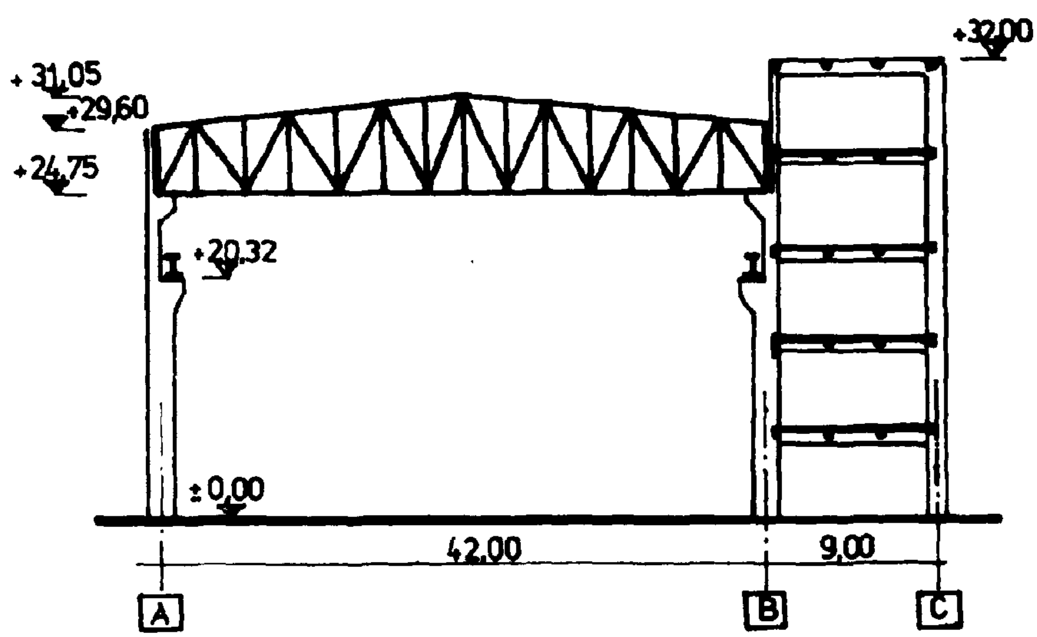


Fig.1

- The engine room of a thermoelectric-power station in Bucharest (fig.1) with a span of 42 m, a 9 m bay and a clear height of 31 m was provided with a reinforced concrete panel roof, supported by steel trusses. The columns, supporting the mounting heavy cranes, were made of reinforced concrete. The columns on one of the sides are common with those of a multistored cast-in-place concrete building. During the earthquake the reinforced concrete columns failed and the anchorage zones of the trusses were damaged. That led to the collapse of the roof framing. Under the conditions of an excessive seismic action the main causes of the collapse may be considered as follows: a heavy roof located at a high level, an important difference between the lateral stiffness of the columns and the multistory building and a poor quality of the cast concrete.

The new roof deck was made of corrugated steel panels.

- The turbo-units hall in the Heavy Engine Plant-Bucharest (fig.2) with two central spans of 36 m, the clear height of 30 m and two lateral spans for each side, of 24,50 m with 15 m height, provided with very heavy travelling cranes, had a steel structure and a heavy roof deck made of reinforced concrete panels and thermal insulation consisting of light cell concrete, having a total weight of 400 kg/m^2. During the earthquake lateral displacements of the roof framing with 8-85 cm (an average of 50 cm) occurred, due to the shearing of the connection into the plastic deformation of the top side of the structural steel columns and to the buckling of vertical bracing bars, located over the runway crane girders. It has to be noted that for the above mentioned reasons (item 3), the damage was limited at the zone over upper runway girders. Repairs to the damaged building included also the replacement of the heavy concrete deck by a light roofing made of corrugated sheeting. A hall for welded assemblies at the 23 August Plant in Bucharest (two bents of 24 m and 12 m bay), with composite structure and heavy roof deck,underwent the collapse of the trusses(b) from two contiguous bents (fig.3). The collapse was caused by the failure of the central column (a), whose free deformation during the earthquake was impeded by a rigid interior building, built without expansion spacing around the column; the whole deformation of the structure under seismic action was developed along the top part of the column that becomes thus a "short column" with strong shear forces, but without an adequate reinforcement. As a result of the analysis of these cases, after 1977, for halls with large spans and heights ($L > 24$ m and $H > 16$ m), solutions for halls with light roof decks have been adopted.

5. The roof bracings generally had a good behavior, an important contribution being the interaction with the roof deck diaphragm. The vertical bracings between columns made of members with a low slenderness, resulting from the crane and wind loadings,suffered no damage. Some bracings with very slender cross diagonals of the halls failed through buckling of compressed diagonal,followed

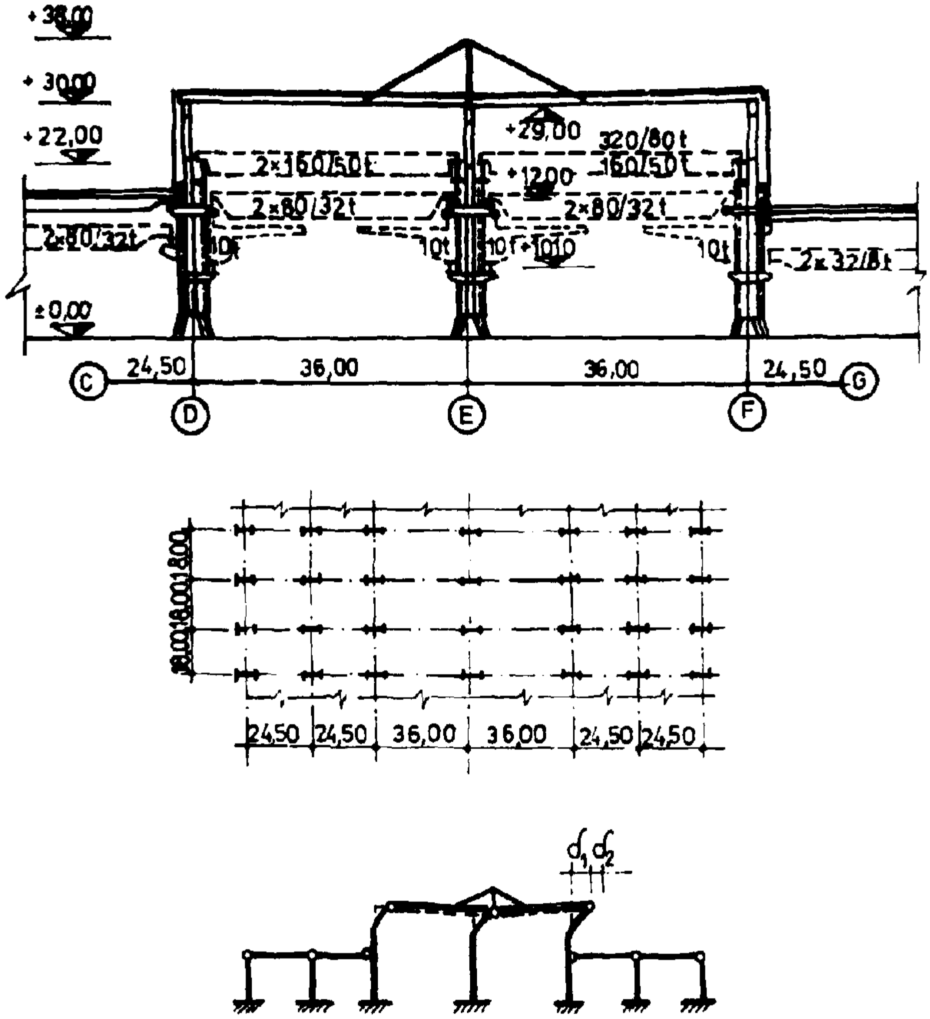


Fig.2

by the fracture of the tensioned diagonal.

6. The connections between steel members and concrete members sometimes suffered damage resulting from overstressing during

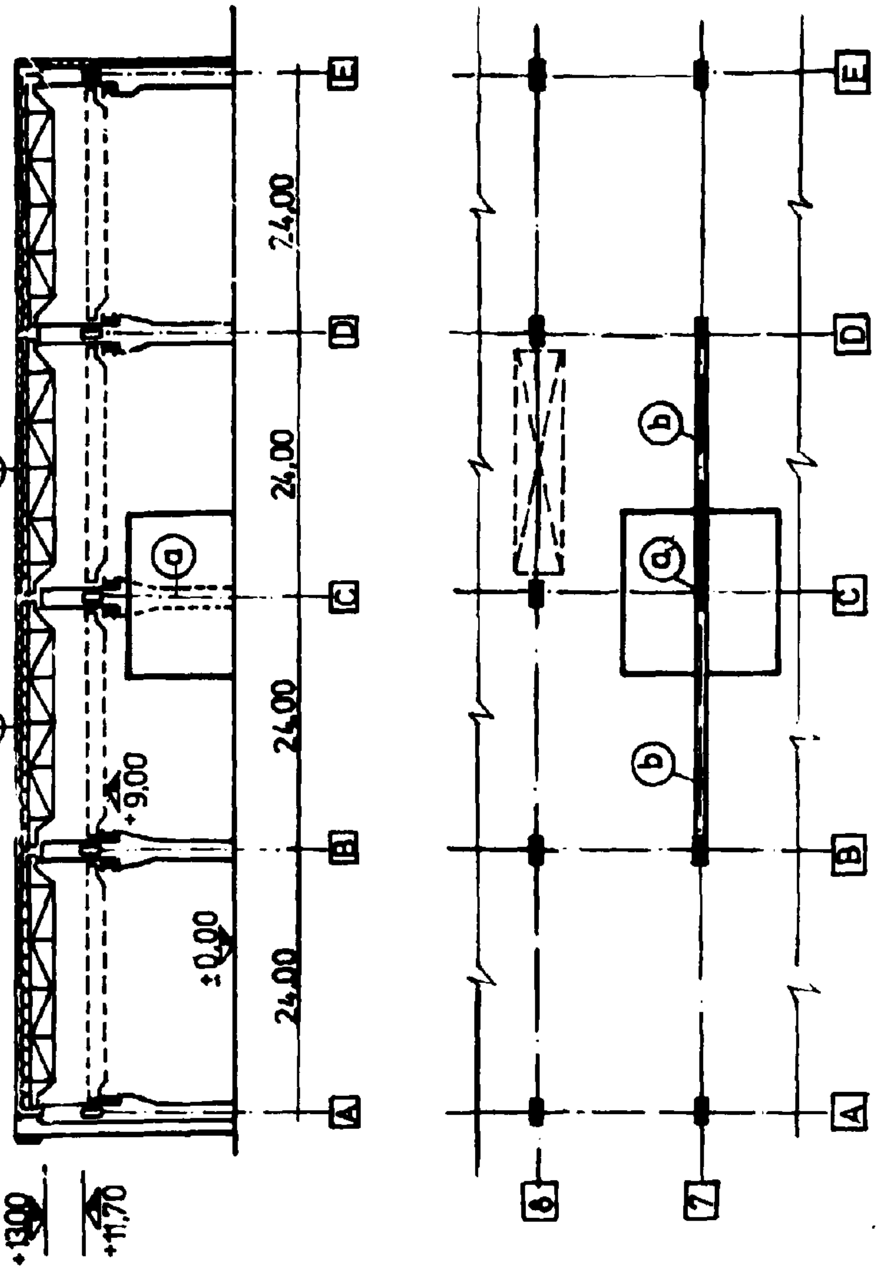
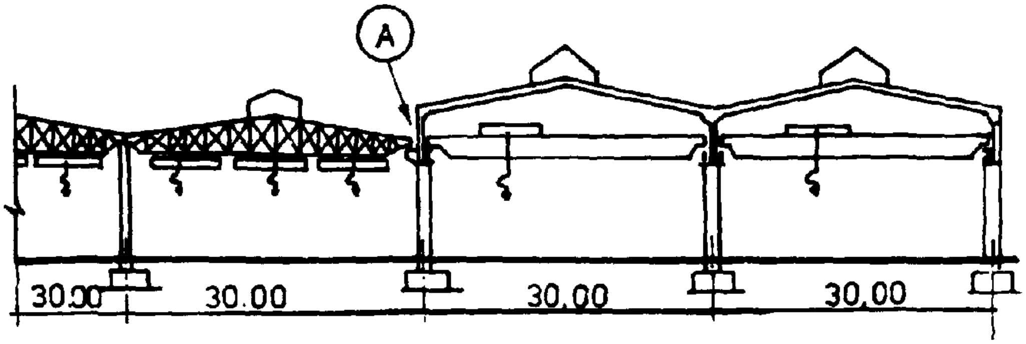


Fig.3



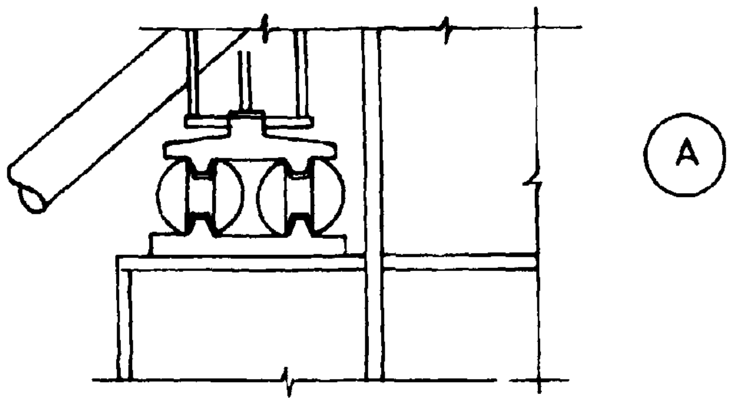


Fig.4

the earthquake. The ordinary bolt connections subjected to shear behaved unsatisfactorily both at the truss-to-column connections and at the crane girder-to-column connections. The transmission of the shear forces by the keeper plates had a high load-bearing capacity. Another weak structural detail was the anchorage of the bearing plates supported by the reinforced concrete columns for the connection with the trusses, with the crane girders and the vertical bracings. The "pulling out" phenomenon was encountered. This effect was caused also by the damage of the concrete around the anchor bolts.

Some damage appeared in the roller expansion bearings. At the Tool and Units Plant in Bucharest (fig.4) the expansion joint between two steel halls with a light roof deck having very different rigidity to lateral displacements, was carried out by roller bearings. Because of the different vibration natural periods of the two parts of the hall (one with stiff columns supporting heavy cranes, the other one with elastic columns having transport equipment hanged to trusses) important relative displacements occurred, the rollers facturing the keeper plates and falling from the bearings. The trusses remained supported by the steel brackets of the columns that were provided with high bearing capacity and could therefore support the shock produced by the collapse of trusses. Special provisions have been adopted for different types of fixed connections and of the roller expansion bearings being subsequently included in the aseismic structural design.

7. The March 4, 1977, earthquake confirmed the special importance of the layout and also of the aseismic protection provisions; it was underlined on this occasion, that these measures have to be compulsorily observed for all the buildings located in seismic areas, whatever the accepted intensity of the earthquake. The analysis of many cases showed that the structures provided with an overall aseismic layout and adequate detailing solutions behaved more satisfactorily during an earthquake with higher intensities that those considered in design.

The conclusions of the analyses carried out on the damaged structures and also the analysis of some significant damage of the steel structural members and connections of the proposals for the improvement of the specific design were the basis provisions drafted by the authors, provisions that were subsequently included into the Romanian Aseismic Structural Design Code P 100-78.

I.5 DYNAMIC BEHAVIOR OF SPATIAL STRUCTURES AND SHELLS DURING THE MARCH 4, 1977, EARTHQUAKE IN ROMANIA

Mircea Mihailescu x)

The effects of the March 4 1977, earthquake on the Romanian area, appear in fig.1, in which the principal intensity zones are depicted and triangles mark the spots where the structures discussed later on are located.

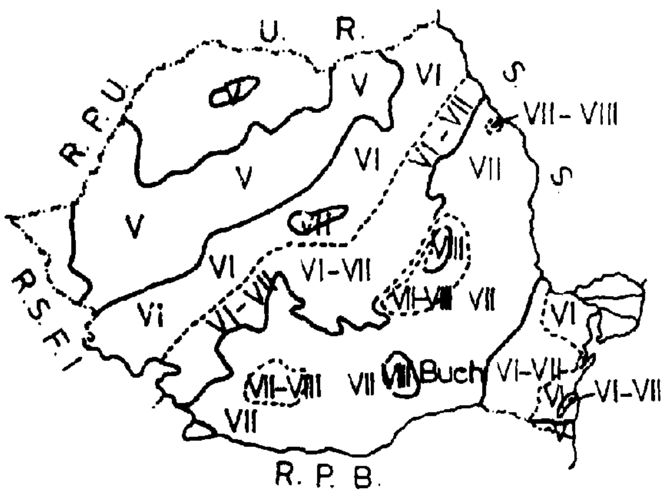


Fig.1

x) Professor at the Polytechnic Institute Cluj-Napoca.

The behavior of spatial structure , during this earthquake, was analyzed, taking into account two time periods for the erection of buildings, namely: (i) in the 1920 - 1940 interval before the first great earthquake occurred in Romania, and between 1944 - 1977 until the second big earthquake.

These categories will underline that the earlier buildings of were erected without any seismic protection, while those of second class were conceived in agreement with the Romanian seismic code.

The spatial structures dealt with belong especially to shell roofs and containers, only one case is reffering to a double layer grid covering.

It also should be mentioned that all the considered structures are made of reinforced or prestressed concrete. Generally, spatial structures, shells in particular, behaved rather, well during the March 4 1977, earthquake, in their fields (curved plates within their boundaries). The explanation of this good behavior has to be seen, in the intimate layout of spatial structures, able to produce a similar response for actions oriented in any direction (fig.2).

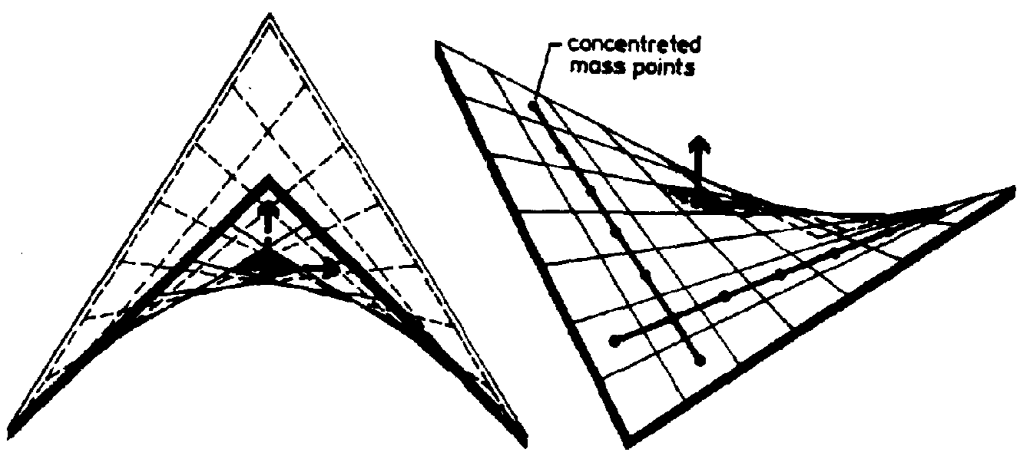


FIG. 2

Damages occurred mostly at the edge or support elements , then when they were wrongly conceived, or when later introduced partition walls or other functional elements made the entire structure to rotate during the earthquake motion around an eccentric vertical axis.

Failures happened at many water towers not to their container shells, but to the support cylinders, due to shear forces greater than the section capacity or due to some foundation deficiencies.

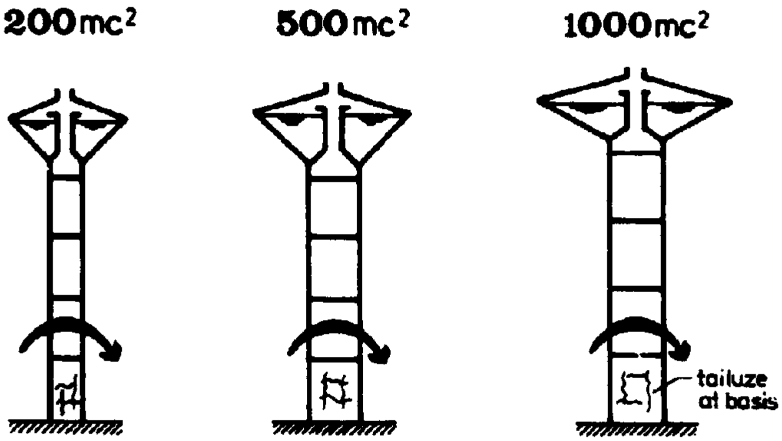


FIG.3

Two umbrella shells, as fig.4 shows, overturned about their bases, because they were not well enough fastened at the rigide level with the adjacent house. (fig.4).

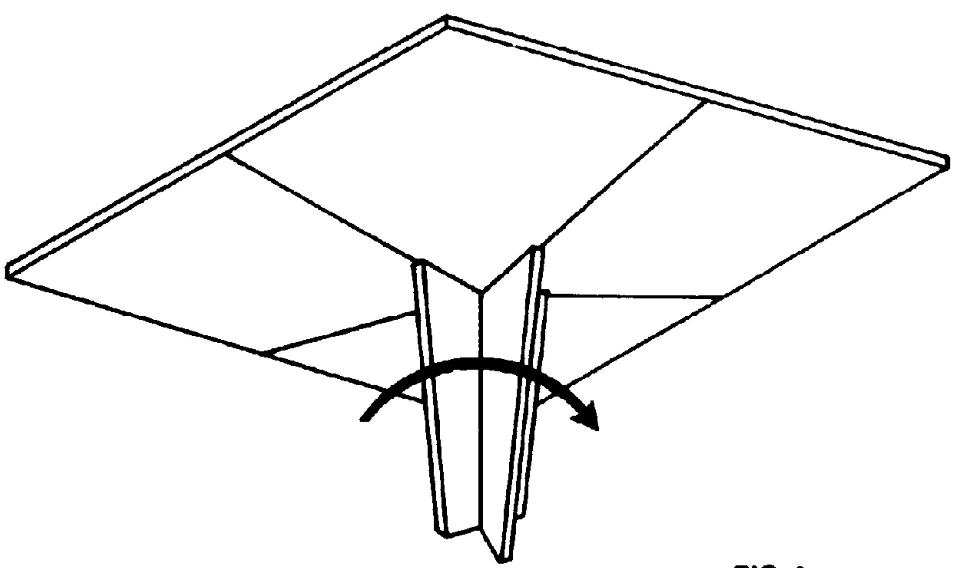


FIG. 4

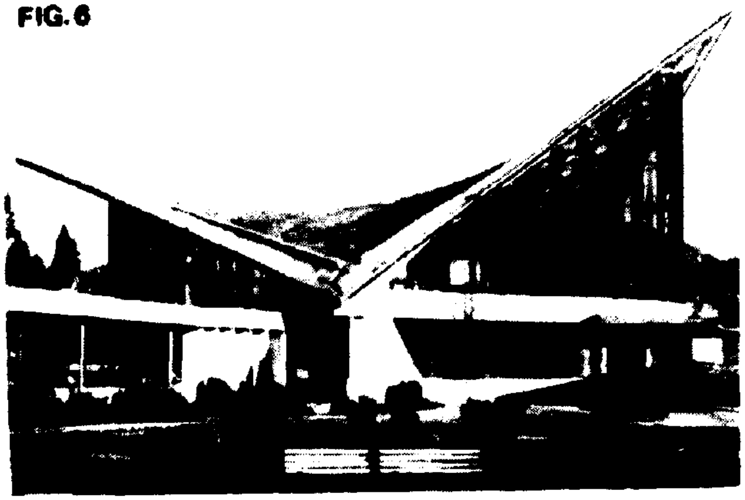
In the second part of this report 7 shells located in various intensity zones, as fig.1 shows, will be presented.

At a locomotive depot erected in 1974 (marked 1 on the map), whose roof is made of 40 conoidal shells of 6.5 cm thickness (fig.5) each resting on arches supported by columns, no trouble was recorded, in spite of the VIIth degree intensity of the earthquake.

FIG. 5

In the center of this depot there are five elliptic shells of 8cm thickness, each covering a 27x16.50 m^2 area. These also have suffered no damage. The second shell belongs to the Predeal railway station (fig.6) whose roof is an asymmetric hyperbolic paraboloid made of precast elements assembled by prestressing.

FIG.6



The covering of about 800 m^2 area, rests on two abutements, trough steel hinges, and it suffered apparently no damage during the earthquake referred to. The shell, marked on 3 on the fig.1 map (placed in an 8th degree intensity zone) is a sport hall, whose roof is made of five shells, described by a 4th degree equation covering an almost 1200 m^2 area (photo fig.7). The shell was made of precast units, made of two 4 cm thick plates including, between these a 5 cm thermal insulation sheet. The assembling was performed by longitudinal prestressing. This shell, which rests on the foundation through ste

FIG. 8

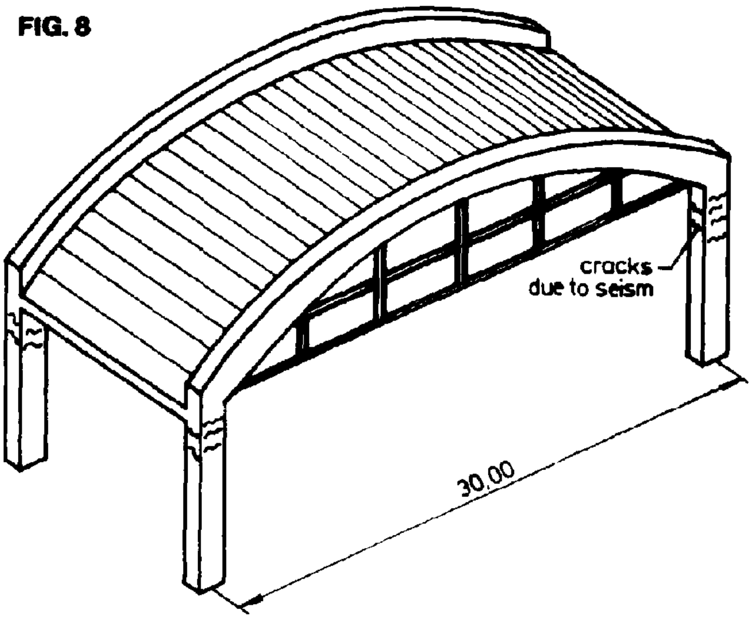


FIG. 9

el hinges, suffered no damage.

In Bucharest a big depot (4 on fig.1) erected in 1933, whose roof is made of cylindrical shells, supported by tied arches, was affected only by cracks in columns it their upper parts as drawn in fig.8.

Also in the capital, a big covered area of about 10,000 m^2 has to be mentioned. Its roof is made of a 7 cm thick cylindrical shells shaped in an original S form (photo fig.9). Cracks occurred,in this case, to the supporting frames, especially to some columns over loaded by the torsion created through some partion walls which have displaced the rigidity center. Fig.10 shows the cracks and some repairing details.

A similar S-shaped shell (photo 11) was built at Savineşti (location 5 on fig.1). The shells of 6.5 cm thickness rest on frames with curves spandrals, placed at the outer shell side (fig.12).

In this case, no damage was observed,

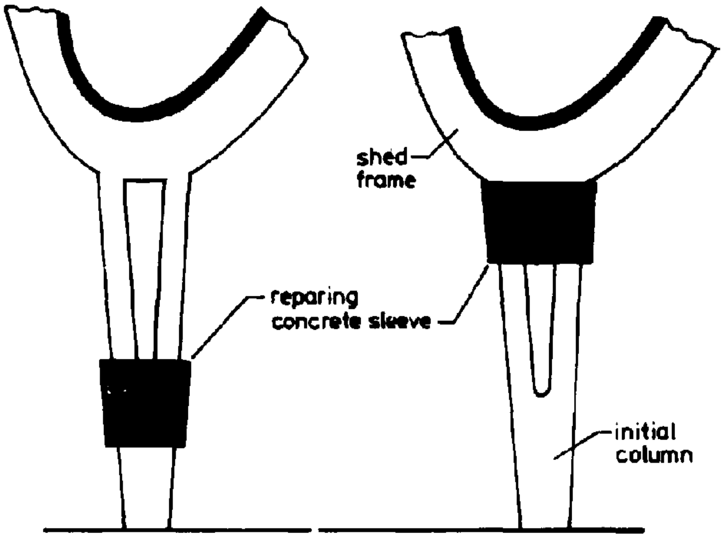


FIG.10

as was the case with a veladroidal erected in Cluj (location 7 on fig.1), whose inner view appears in fig.12. This shell, made of precast elements, had a very good performance during the last earthquake.

The structure made of precast thin lens elements (location 8 on fig.1, photo in fig.13) has suffered no damage either.

Also undamaged was a double-layer grid structure made of single-type precast elements (location 9 on fig.1), which was assembled at the ground through post-tensioned cables (fig.14).

In this case, no damage of any kind could be seen in spite of the fact that intensity eight was suffered during the 1977 earthquake.

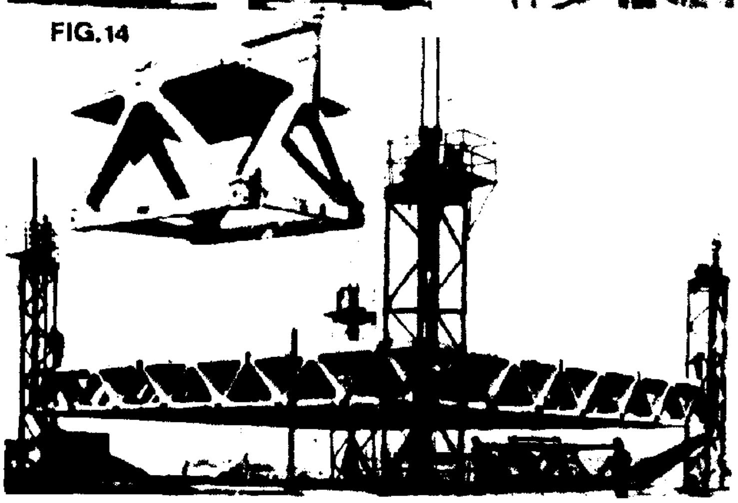
Reference:

/1/ STAS 9165-72


FIG.12

FIG.13

FIG.14



I.6
CONSIDERATIONS OF THE CAUSES OF COLLAPSE OR DAMAGE OF SOME DWELLINGS IN BUCHAREST DURING THE MARCH 4, 1977, EARTHQUAKE

Gheorghe Săndulescu[x]

ABSTRACT

The collapse of 30 dwellings, having 6 to 12 stories, during the March 4, 1977, earthquake imposed the need for serious analyses in order to explain and eliminate the causes of the collapse. The in situ determinations of the dynamic characteristics of the severely damaged buildings correlated with the on-site investigation concerning the actual response of the buildings under strong seismic loads. They revealed the main causes of failure for several characteristics structural elements, and for the whole structural assembly.

The paper presents the errors both in the seismic design and in the execution of some reinforced concrete characteristic details, corresponding to the available level of knowledge on a national and international scale in the period these buildings were constructed.

The report also presents a number of the author's considerations about safety as a function of time, a new method of seismic design; the design of conventional elements has to be replaced by realistic determinations of the available load-bearing capacities of the building at a given moment.

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The March 4 1977 earthquake affected a very large area of the territory of our country. As a consequence, this earthquake subjected to seismic actions a large number of buildings essentially different from the point of view of the structural conception, structure material, level of engineering knowledge available for design and execution, building age at the time of the earthquake, number of preceding micro and macro earthquakes, consequences of the service period concerning the preservation of the stiffness characteristics, loading level, etc.

As a matter of fact, taking into account the above mentioned items, almost every building can develop a specific strength during a local earthquake. The specific value of this strength is determined by the interplay between all the events the building have previously suffered and the loading characteristics imposed by the respective earthquake. Nevertheless is possible to establish some common general features for the response of different building categories with identical or similar design conceptions during a certain type of earthquake - especially when the age of these buildings is almost the same and the loading antecedents similar. Such a situation obviously occurred during the 4 March 1977 earthquake, when the great majority of the buildings in Bucharest behaved satisfactorily, excepting the 8 - 10 story dwellings located in the center of the city and erected in the period between the two World Wars, which did not behave properly and led to 30 of the 33 collapses recorded in Bucharest.

The main cause of these failures both in structural and non-structural elements was the lack of an aseismic design conception of the buildings built before 1940. Also these buildings were not designed for conventional mass loads, as the specialists concluded subsequently. As a matter of fact, the basic design conception of these buildings provides only a load bearing capacity for the gravity load. We assign that fact not to an insufficient training of the specialists but to an insufficient knowledge of the earthquake engineering problems in that period of time.

For tall buildings with reinforced concrete framed structure, the lack of an aseismic design conception that led to failures and collapses can be expressed by:

1. The functional design lay-out presenting asymmetrical shapes with respect to the main axes. That led to overloadings as a result of the general torsioned effects (e.g.Bd.N.Bălcescu, No.24).

2. The partition of the functional space (flats) in asymmetrical shapes led to the impossibility of an organized lay-out of the structural members (beams and columns). Thus they are not able to form structural subassemblies (frames) providing the required stiffness and capacity to resist the overloadings, by dis-

sipating the absorbed seismic energy, in post-elastic regime.

3. Due to the random arrangement of the main structural members as mentioned, as a consequence of the total subordination of the structural to the functional solution, the structural system withstanding the horizontal loads has a causal character and consists of concrete structural elements provided to resist gravitational loadings and non-structural elements-interior and exterior walls whose contribution to the load-bearing systems was fortuitous.

4. The contribution of the interior and exterior walls to the load-bearing system for horizontal loads along with the structural members provided to carry the vertical loads to the ground, led to a random development of structural schemes (weak frames). Due to these schemes the building, by the general box-effect, could resist the effect of the seismic base shear force and the deformation imposed by the earthquake. Therefore stress redistributions, in some cases even total changes of the stress types and thus exceeding the load-bearing capacities of some specific sections of the main structural elements, failed.

5. The general box-effect, made the partitioning walls and especially indeed those framed by columns and beams, to act as shear walls in the transfer of the horizontal inertia mass loads to the ground. The inability of the masonry walls to take over tensioned stresses, caused cracks and diagonal failure planes at X shape, out of plane displacements and even their collapse. At the same time, in the concrete structural elements that framed the walls, overloading resulted due to the partial shear wall effect (in fact compression diagonal truss mechanism). These overloadings are represented by additional shear forces in columns causing the damage of these elements.

6. The lack of designed stiffening elements, corresponding to the vertical profile and to the mass of the building, provides a high flexibility featured by natural vibration periods of more than 2 sec. The above mentioned general box-effect reduced the building flexibility, providing an additional rigidity in comparison with that of the concrete structural elements. Even if this "selfcorrection" of the dynamic characteristic of natural vibration had been realized, the constructive assembly remained in the "flexible" range. Thus, considering the shape of the response spectra curves determined for the March 4 1977 earthquake, a higher amount of seismic energy was absorbed by these buildings in comparison with more rigid buildings and so the structural and non-structural elements were more intensively stressed.

7. At the same time, the general box-effect, gave these buildings a general adapting process under the action of real seismic loads. This occurred less by bending inelastic lateral deformation-the building was not provided for but more by shear forces, whereby

the absorbtion of the seismic energy by an irreversible but limitted stress-strain deformation process was achieved.

8. In most of the cases, the vertical profile of the building does not present a constant stiffness or a linear variation due to the large free spaces of the shop ground-floor. The general box-effect did not include the height of the ground floor of the building. So, the transfer of the seismic forces towards the ground, by shear forces especially, overloaded the ground floor columns, the unique elements available to withstand the base shear force at this level. Therefore these elements suffered such damages as: cracks and failure planes due to the exceeding of the capable stresses by the compressive principal stresses corresponding to the stress combination and by this may determine the beginning of the collapse process.

9. Plastic hinges in the column-beam joints developed when the number, nature and distribution of the interior walls couldn't significantly ensure the general box-effect. The response of the building is performed by the balance between the induced seismic energy and the potential energy corresponding to the inelastic deformation developed by the whole building. The plastic hinges developed in most of the cases in beams (joint area) without reaching the limit deformation of the concrete in compression. Thus vertical cracks due to exceeding the elastic deformation in reinforcement have occurred.

10. The reinforced concrete slabs, which were not designed as horizontal diaphragms able to provide common deformations of the adjacent vertical load-bearing structures, overloaded by the general torsion effect of the buildings (as a result of its plane and vertical symmetry), suffered such damage as: cracks and failure planes because the reinforcement elastic limit were exceeded (generally the reinforcement disposed along the secondary direction.

11. A reciprocal influence was noted between adjacent buildings with different dynamic characteristics when aseismic joints were not provided or when the existing ones were too small. The example of the building Calea Victoriei 95 was significant. In this case, one of the adjacent buildings having a lower height, started a process that impeded the deformation tendency of the building during its post-elastic adapting. Thus, the structural elements at the respective level have been drastically damaged.

12. It is assumed that one of the causes of the damage, besides the above mentioned lack of an aseismic design, is the poor quality of the concrete used for columns and beams. In that period of time the B 120 - B 140 Grade was used in building. The loadings caused by the 1940 earthquake, the bombing during the 1941-1944 War and especially the progressive decay of the material

due to the pollution and to the corrosion of the concrete led to a decrease of the initial strength value under 100 kg/cm^2 (in some cases 50-60 kg/cm^2).

The decay of the concrete by concrete mass microcracks and by the loosening of the covering-layer also involved a significant corrosion process of the load-bearing reinforcements. That decreased obviously the load-bearing capacity of the structural elements, especially for overtaking the shear forces, leading to cracks and failure planes and finally on severe states of total collapses.

We do consider that the correct understanding of the causes that produced severe damage in this category of buildings during the March 4 1977 earthquake, represents only the starting point for the assessment of a new design philosophy for buildings under severe seismic loading. This philosophy has to take into account the complexity of the safety relationships in order to balance a very uncertain loading state with a required load bearing capacity available at the moment of hazard.

Thus we intend to present here some considerations regarding the future development of a more realistic analysis of the buildings under seismic loadings.

It is well known that in the common design practice, building safety is determined by a "designed safety" corresponding to an ideal case. In this case the building would perform in strict conformity with the design condition from all points of view: dimensions, physical and mechanical characteristics of the materials, ground and loading conditions. The specialist checks the required safety conditions both for the common service loads and for exceptional loadings as the seismic loads. The specialist has to check the building design as if it would be already in service, unchanged, for the whole service period.

Thus the building is provided with a load-bearing capacity that exceeds the normal service loadings and is close to the limit value of the extraordinary loadings that determine the proportioning of the structure. Under normal service loadings the building has to remain in the elastic stress-strain range, while under extraordinary loadings an inelastic behavior is allowed, as a survival method.

From this point of view we can ask the question:
Has the building under extraordinary load the same survival whatever its age, or, in other words, can we accept the idea of a constant post-elastic adapting capacity of the main structural elements for the whole service period?

That is under the question because it can be assumed that a building is featured by:

- life: The building is a living mechanism that permanently records the consequences of the service conditions.
- age: The building presents a variation of the available load-bearing capacity during the service period.
- reaction: The building can periodically adapt itself to a loading state.

As paradoxical as it seems, the actual design philosophy does not consider the safety problem under the above mentioned aspects, although it is obvious that the total resources the building has, during a peak-loading (the seismic loading) are directly determined by the age of the building and also by irreversible loading antecedents of the building.

The exceptional seismic load that can bring the building close to collapse will have values different from the design value. It should be also assumed that these values are different for different moments in time. The building is influenced by:

- The state of the building loads that preceded the event and could lead to a variation of the stiffness distribution as regards the design assumptions, to a lower or higher "micro out-of - use state".

- The characteristic of the process of the building adapting to the ground, under normal service loads and the nature of the existing soil-structure equilibrium at the event moment.

- Variation in time of the mechanical characteristics of the construction materials that may prove to be a favorable or unfavorable development from the point of view of the load-bearing capacity and especially from the point of view of the non-linear deformation characteristics.

- The number and value of the seismic loading antecedents of the building, taking into account the increasing importance which has to be paid to the micro-earthquakes. Even if these are not directly perceived by people, they subject nevertheless the buildings to the ultrasound effects.

- The influence of the environment on the building (corrosion, bio-degradation).

Considering the above mentioned, the safety philosophy of the actual design consisting of the check-up of a theoretical load-bearing capacity as regards one or several presumed loadings, has to be approached from a more realistic view. This new philosophy should accept the necessity to verify the building's functionality by the safety criterion considered as a variable function in time, considering the building permanently influenced by the complex action of the environment. The analysis of the safety dynamics of a structure is necessary when an anticipated possible behavior of the building is required for a future earthquake, due to the fact that the available load-bearing capacity to resist the seismic loads is a direct result of the previous loading suffered by the buildings.

Two different ways may be adopted in design practice to determine the variation of the safety in time, namely:

1. A check of the whole safety of the building even in the design phase for several distinct stages of the service-period.

For each of these stages, taking into account on one hand the probability of the strength and deformation characteristics changing and on the other hand, that of the loading and adapting process to the environment, it would be possible to assess again the allowable functionality conditions, keeping the same safety level for a possible seismic load action of a given type of earthquake.

2. The periodic checking of building safety for seismic loadings, at significant time-intervals (20-25 years), taking into account the real situation of the building, on the basis of the mechanical and stiffness characteristics by nondestructive analysis methods.

Depending on the above mentioned data, in order to maintain the same safety level, the building may suffer functional limitations, as for example the reduction of the loading level, the partial shutdown of constructive assembly (reduction of the level number or of the additional structural elements in comparison with the initial design, harmoniously integrated into the constructive assembly).

By these two methods essentially, it would be possible to adopt a new philosophy, passing from the static assessment of safety, to a main realistic approach, implemented now in all the fields of technology, concerning a new assessment of the available potential as a function of the physical and moral wear of the respective unit, because the building can be easily assimilated to a complex mechanical unit.

Transpositing this approach into current design requires the development of an appropriate calculus method based on a research technique able to supply complete data concerning the actual building stiffness, the recorded stress state and the available strength capacity of the structural elements.

I.7 EARTHQUAKE PROTECTION OF EXISTING LOW-RISE RESIDENTIAL AND MIXED-OCCUPANCY BUILDINGS

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Radu Marinov[x]

Most of the existing building stock found both in Romania and elsewhere consists of low-rise (2-5 stories) residential and mixed-occupancy buildings. Depending on the date of construction, the construction of such buildings range from plain or reinforced masonry walls with wooden floors or small brick vaults with steel members, or RC floors to construction or prefabricated reinforced concrete frame or shear wall structures.

Except for buildings with the latter type of construction, buildings constructed before earthquake-resistant design became common practice (and before the first Romanian code,P13-1965,for earthquake-resistant design was issued), have been found to show major shortcomings as regards their global and local seismic behavior.

Thus, for example, such buildings are often irregular in shape and asymmetric in the horizontal and vertical distribution of volumes, masses, and stiffnesses. This is partly due to the sometimes decisive influence of the architectural layout of a building. The unfavourable response of older buildings to earthquake motions is aggravated by their wooden floors, whose inadequate stiffness in their own plane and jointing to the surrounding walls prevents the benefical interaction between the vertical members of the structure. Moreover, the stiffness, bearing capacity and ductility of the structural elements providing protection against earthquake effects and diminished due to inappropriate detailing and the lack of engineering analysis. The generally low density of masonry walls spaced sometimes as much as sixteen to twenty meters span (a spacing far in excess of the present code provisions), the reduced dimensions and poor transverse reinforcement of the columns of reinforced concrete frames, the presence of too many openings for doors, etc. in the walls all may cause severe damage under an earthquake. In addition, inappropriate locations of the load-bearing members cause the torsional stiffness to be greatly reduced as compared to the

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translational one and determine a pronounced eccentricity of the center of stiffness. This, in turn, causes the lateral bearing members, especially those located at some distance from the center of stiffness, to be overstressed, an effect that increases with the duration of the earthquake, causing greater damage and loading, implicitly, to an even greater loss of torsional stiffness. The effect of structural torsion was noticed in many low-rise buildings after the earthquake in Bucharest on March 4, 1977.

The picture of structural deficiencies that make buildings sensitive to earthquakes is completed by the use of low-grade building materials (bricks, mortars and concrete), made worse by the influence of environmental factors like vibrations, physical and chemical pollution, etc.; faulty execution on site; and last but not least, subsequent alterations in the structure, particularly in buildings where the ground floor has special functions, different from those of the superstructure above.

In many cases, low-rise buildings have insufficient stiffness. However, the second-order (P-Δ) effect due to the dead load of the structure is less pronounced in them than in taller, often more flexible buildings constructed before 1960.The total collapse of low-rise buildings was encouraged by the lack of ductility in the vertical load-bearing members (low-grade concrete, poor transverse reinforcements, insufficiently anchored longitudinal reinforcement).

Recent earthquakes of high intensity(including the 1977 earthquake in Romania) have clearly shown that older low - rise buildings without earthquake protection have a different response to ground motion than newer ones (built after 1960) which had been designed according to modern concepts of earthquake protection. Thus, the structural deficiencies described above, particularly those affecting the general structural layout, caused various degrees of deterioration in the structural members such as horizontal, vertical or inclined (diagonal) cracks in masonry walls and concrete columns and beams, culminating in the local or total collapse of the building.

Earthquake-resistant design therefore became a major design consideration with the staff of IPROTIM, the Timişoara/Romania Design Institute, when they were charged with strengthening buildings in Bucharest with moderate-to-severe damage from the 1977 earthquake, and in Ouled-Fares in Algeria after the devastating earthquake on October 10, 1980 (intensity X-XI on the MKS scale).

Correction of the damage to these buildings required, above all, adequate bracing of the structural ensemble by means of new, high-stiffness elements (masonry or RC diaphragms); this operation was sometimes difficult to carry out because of the functional restrictions due to the architectural layout both in the superstructure and in the ground floor (which had larger spaces with fewer columns). One example is a block of flats (consisting of basement, ground floor and four storeys) that was built in the 1930's, with masonry walls and reinforced concrete frames and floor slabs; another example is the reinforced concrete-framed structure of the Ouled-Fares hospital mosque that was under construction when the earthquake occurred.

In the Bucharest building, the two main masonry shear-walls that are oriented in almost orthogonal directions were overstressed due to their eccentric position, and the floor slab rotated as a result of the general torsional effect, superimposing this displacement and stressing intensely the main frontal longitudinal frame of the ground floor facade.

To provide a uniform bracing of the structure (by creating similar stiffness along the principal direction and dampening the general effect of torsion), median shear walls running in the direction of the two firewalls were introduced. This was done by lining the existing, 15 cm thick partitions of the other floors with a 10 cm, coat of reinforced concrete and contianing these on the ground floor and basement with 15 cm thick reinforced concrete or 25 cm thick reinforced masonry shear-walls (fig.1).

In badly damaged structural members (masonry firewalls, cracked in a diagonal directions, and columns in the facade frame) the brickwork was repaired and its cracks grouted with cement mortar before bracing the walls and columns with reinforced concrete. Less badly damaged walls were also repaired by grouting.

While the general layout of the Bucharest building permitted new stiffening members to be introduced inside the building, this was impossible in the case of the Ouled-Fares hospital mosque. The only solution functionally admissible was to provide RC shear-walls along the contour of the building, playing the role of external "tubes", as they are specific of multistorey buildings(fig.2).

Since the structure had not been protected against earthquakes in accordance with the provisions of an adequate code, the short columns at the level of the hollow "sanitary" space suffered visible damage owing to base shear. A peripheral RC diaphragm was therefore provided at this level, and both the ground floor columns and the newly provided shear-walls were jointed to this diaphragm.

Before deciding whether a structure is worth strengthening and, if so, how the strengthening should be carried out, the degree of deterioration has to be assessed. This can be done in one of the following ways:

1. Knowing the natural period of vibration of the building in its initial stage (say, on commissioning) To and at some later time, T1, the degree of deterioration may be expressed as

$$g = 1 - \exp\left[\frac{T_0 - T_1}{T_0}\right]; \ (0 \leq g \leq 1) \tag{1}$$

in which To is assumed to have been determined experimentally.

2. A survey of damages in the structure gives the degree of deterioration as:

$$g = \frac{\sum c_i S_i}{\sum S_i} \tag{2}$$

in which Si is the bearing capacity of a structural member "i" under horizontal loading, and ci is the rate of decrease of this bearing capacity due to damage.

The decision of whether a building should be pulled down, or, on the contrary, strengthened, and the extent of strengthening, should be based on the following parameters: reliability of the structure, the estimated life expectancy of the building.

The reliability of the structure can be assessed from Eq.3:

$$R = R_0(1 - g)^n \tag{3}$$

where R is the structural reliability at the considered time, Ro is the initial reliability of the structure, g is the degree of deterioration of the structure, and n is the number of earthquakes of maximum intensity likely to occur during the life of the building. For a period of 40 years, between two earthquakes of maximum intensity, $n = 1 \div 2$.

Since considerable social, economic and financial efforts are required for reconditioning all the buildings damaged after a disastrous earthquake, a "strengthening strategy" should be worked out by grading damages and scheduling repairs. "Graded strengthening" really means that the measures required the damage done by the earthquake are differentiated, drawing a distinction between :

(a) full strengthening (100%), which involves all the measures required to restore at least the initial reliability of the structure so that it satisfies current earthquake code provisions, and

(b) partial strengthening (75 or 50%), respectively), which means that the measures indicated for (a) are achieved to an extent of 75 or 50%, respectively.

"Sheduled strengthening" refers to the planning in time of strengthening works, usually in the following order:

1 - strengthening of vertical load-bearing members(columns, walls)

2 - restoring horizontal rigidity by providing shear- walls

and bracings, and strengthening longitudinal walls
3 - repair and strengthening of floors.

Full strengthening requires all these three operations to be carried out, in case of partial strengthening, steps 1 and 2 must be carried out to achieve 75% - strengthening, while step 1 is compulsory for a 50% strengthening of the structure.

Table 1 proposes a way of grading and sheduling strengthening works depending on the decrease in reliability and the life expectancy of the building at a given time:

Table 1

R/R_o and D/D_o - dependent grading and scheduling of strengthening works

D/D_o \ R/R_o	$>0{,}8$	$0{,}8>\frac{R}{R_o}>0{,}4$	$0{,}4>\frac{R}{R_o}>0{,}2$	$<0{,}2$
$>0{,}8$	$\frac{100\%}{(1)}$	$\frac{100\%}{(1+2)}$	$\frac{100\%}{(1+2+3)}$	$\frac{100\%}{(1+2)+(3)}$
$0{,}8>\frac{D}{D_o}>0{,}6$	$\frac{100\%}{(1+2)}$	$\frac{100\%}{(1+2)+(3)}$	$\frac{75\%}{(1+2)+(3)}$	$\frac{75\%}{(1+2)+(3)}$
$0{,}6>\frac{D}{D_o}>0{,}4$	$\frac{100\%}{(1+2)+(3)}$	$\frac{100\%}{(1+2)+(3)}$	$\frac{75\%}{(1)+(2)+(3)}$	$\frac{50\%}{(1)+(2)+(3)}$
$<0{,}4$	$\frac{75\%}{(1)+(2)}$	$\frac{50\%}{(1)+(2)}$	$\frac{50\%}{(1)+(2)}$	demolition

In this table, Do is the initial life expectancy of the building and D is the life expectancy at the time of consideration.

The disastrous damage caused by recent high-intensity earthquakes, especially in old buildings that had not been given antiseismic protection after earlier earthquakes, have clearly shown that adequate strengthening of such buildings is essential. Action must also be taken in areas of potential seismicity where , so far, structures have not gone through earthquakes of the intensity for which they have been designed. One example is provided by Timişoara/Romania, a town located in an earthquake area of intensity 7(MSK).

Of the 170 earthquakes recorded in this area (the Banat) between 1766 and 1976, six had an epicentral intensity of 6 - 7.5 on the MSK scale. These were generated in the fault running west to Timişoara. Owing to the small depth of the foci (4 to 10 km), the intensity of such earthquakes decreases rapidly with the distance from the epicenter; for an earthquake of intensity 7, the radius of the corresponding isoseismal line is 3 to 8 km, depending on the depth of focus. This explains why the city of Timişoara has never been affected by previous earthquakes; the most intense of more recent earthquakes (on 27 May 1959), of a maximum intensity assessed to be 7,5 in the epicentral area (near the village Sag, on the southern section of the fault), occurred at 12 to 15 km from Timişoara, a distance that did not affect the present perimeter of the town.

It is obvious that in case of future, stronger, seismic activity in the central section of the fault, the isoseismal line of intensity 7 will include all of the present built-up perimeter of Timişoara. This makes an analysis and strengthening of existing unprotected low-rise buildings in the town essential, moreover because their foundations are absolutely inappropriate. In a considerable number of buildings located on difficult grounds (dumpings, alternating clay and sand layers with a rather high water table), unacceptable differential settlement occurred, causing cracks and other damage that was than aggravated during subsequent earthquakes.

CONCLUSIONS

Earthquake-resistant protection of existing unprotected low-rise buildings for housing and social-cultural activities requires urgent action to be taken. This involves:

1. making a survey of the structures of such buildings, to provide a history of previous interventions and a distinct classification of the latter;
2. determining the dynamic and material characteristics of the more important members of the structure; determining the natural frequency of vibration of the buildings at different times (intervals) to assess the progressive decrease in the stiffness of the structure due to environmental effects;
3. mapping and micro-zoning the areas of seismic hazard, i.e. determining the spectra of seismic response for potentially seismic town areas. Distinction must be made between static and dynamic responses of buildings, according to the depth of the focus.
4. determining the actual level of structural safety of the analyzed buildings and categorizing structures according to this criterion;

5. deciding in favor of strengthening, depending on the assessed structural safety level of the building, on the seismicity of its location and on the planned future service expectancy correlated with general city planning. Scheduling or strengthening work depends on the financial state of the owners. In very old buildings in the first stage, the structure can be stiffened (braced) by means of additional transverse walls, cappings, ties etc. and wooden floors can be clamped and stiffened by means of orthogonal steel reinforcements, at a later date, they can be replaced by RC floors;

6. working out a consolidation design at the assessed level of structural safety. Elements newly introduced into the structure should be integrated as completely as possible, and structural requirements should be correlated with the functional requirements of the existing architectural layout. Also, considering the close interaction between structure and soil, the negative effects of this interaction should be eliminated or at least diminished;

7. issuing codes concerning both the methods of evaluation and the techniques of strengthening for buildings; working out detailed as well as standard designs to ensure maximum efficiency of the earthquake-resistant protection of existing low-rise buildings.

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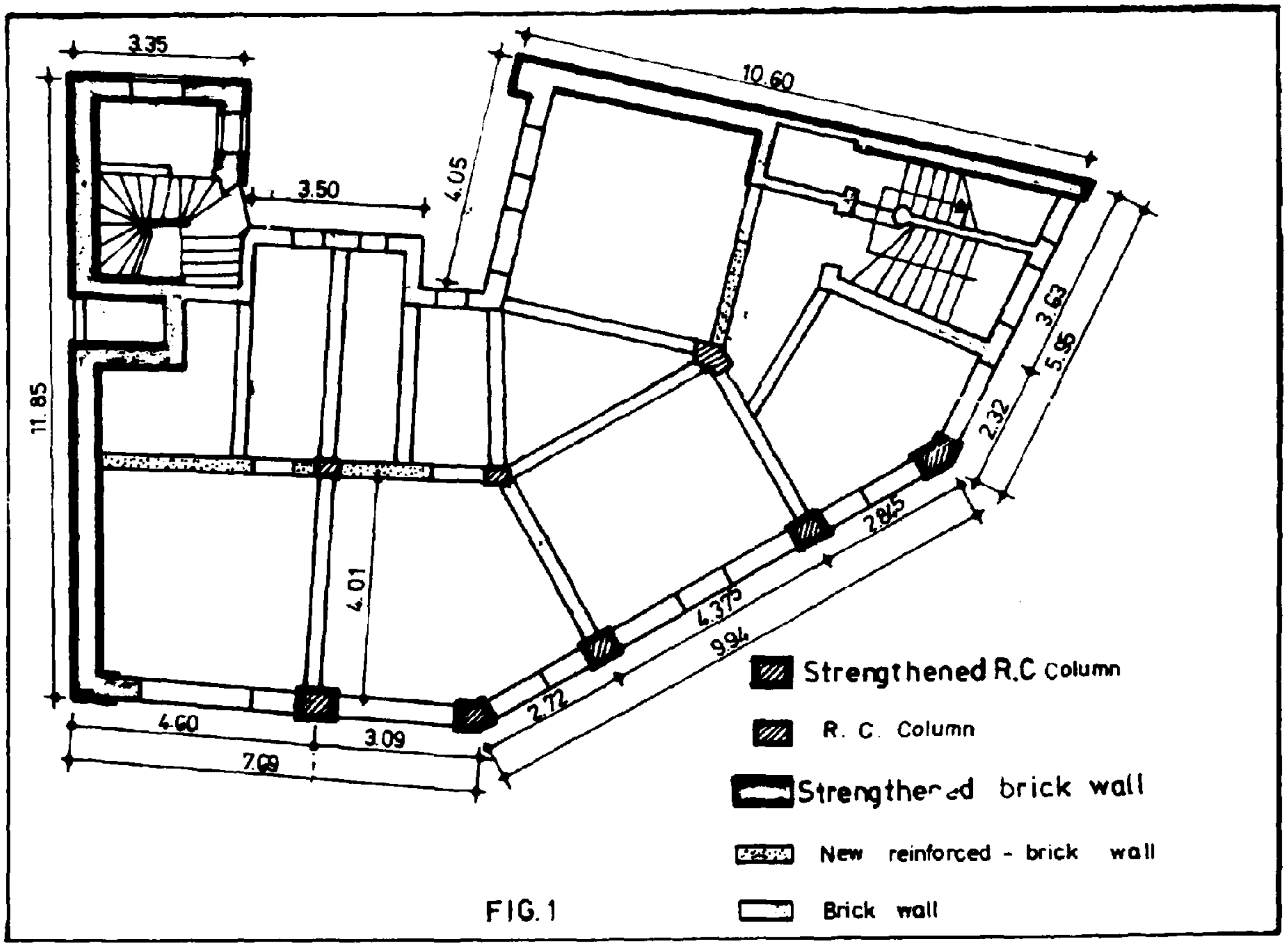
3.35
3.50
4.05
10.60
11.85
3.63
5.95
2.32
2.865
4.375
9.94
2.72
4.01
4.60
3.09
7.09
Strengthened R.C Column
R. C. Column
Strengthened brick wall
New reinforced - brick wall
Brick wall

FIG. 1

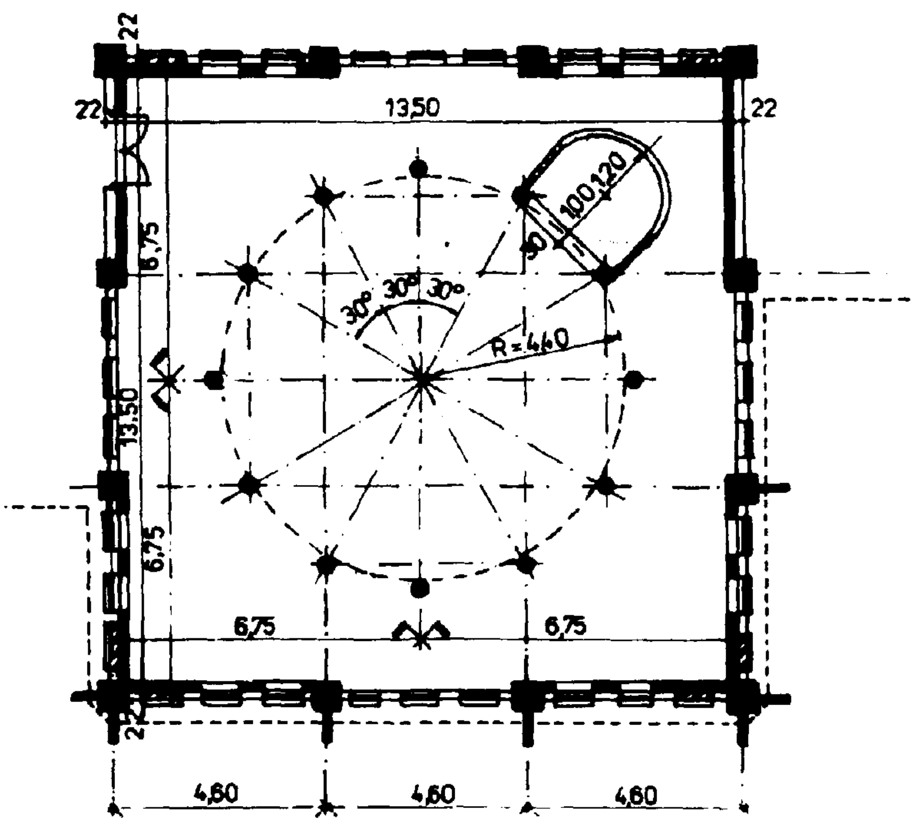
22
22
13,50
22
100,120
30
6,75
30°
30°
30°
R=4,40
13,50
6,75
6,75
6,75
22
4,60
4,60
4,60

FIG. 2

I.8

REMARKS AND CONCLUSIONS CONCERNING THE EFFECTS OF THE 1977 EARTHQUAKE IN THE PRAHOVA COUNTY: THE BEHAVIOR OF PEOPLE AND BUILDINGS, SOCIAL AND TECHNICAL ACTIONS FOR RECOVERY AFTER THE EARTHQUAKE

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1. GENERAL DATA

The Prahova district is Southwest of Vrancea seismic zone. The capital of district, Ploieşti city (227,000 inhabitants) is 60 km North of Bucharest and 130 km from Focşani, the capital of Vrancea district. The Prahova district has 852.000 inhabitants (1984) about 3,8 % of the country population. In the North of the district are mountains (the Carpathians); in the center a hilly zone (with great salt mines Slănic-Prahova, Telega-Cîmpina and oil fields); in the South the Romanian plain. From a tectonic point of view, the Prahova district is situated on the moesic sub-plate, including the contact zone with the inter-alpine sub-plate. Fault system exist in the Drajna-Vălenii de Munte zone (to the West) and the major fault Tinosu (10 km Southest of Ploieşti), Fierbinţi - Călăraşi to the North (V.Cîmpina).

2. SEISMIC HISTORY OF THE PRAHOVA DISTRICT

The Prahova district has felt all the earthquakes whose epicenters have been located in the Vrancea zone. The epicenters of many earthquakes of low intensity before 1977 were located at Bucov (8-10 km east of Ploieşti). Existing documents lack information concerning the previous earthquakes, though we have more information about the November 1940 earthquake. Withnesses of both the 1940 and 1977 earthquakes were very useful. Here are some excerpts from the official documents of the 1940 earthquake:

"The ninth degree (sometimes 10th degree) of seismic intensity had been felt... here and there... in Plopeni (Prahova), 70% of the residential buildings were destroyed, the rest being damaged. The railway bridge fell. The church... suffered serious damage. Brick, stone, and wooden buildings have collapsed.

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"The eigth degree was felt in large areas.... In Valenii de Munte... walls of brick buildings on stone block foundations were split... At Soimari (Prahova) 22% of the houses collapsed and 68% suffered heavy damage. In Cimpina the walls of weak and solid houses were slpit." Similar failures were felt in Ploieşti and other towns.

Here are some facts about the 1977 earthquake for comparison:

- In Plopeni a bricked, 4 story building collapsed, and a water tower too. A great number of reinforced concrete buildings (schools) and bricked houses suffered serious damage.
- At Vălenii de Munte; only several historical buildings (the memorial N.Iorga) have been kept.
- At Cîmpina: a similar situation . The other localities on the list from 1940 are to be found in 1977.

As a result of an inquiry among the witnesses of the 1940 and 1977 earthquakes the following conclusions have been drawn:

- the recallections grew blur with a great deal of subject.
- the few actual comparisons seem to demonstrate a stronger earthquake effects in 1977.

The same conclusions are drawn from the analysis of photographic documents, which show the effects of 1940 earthquake on the Culture Palace in Ploieşti. This was more seriously damaged by the 1977 earthquake, but not in the same zones where it had been repared after 1940 seism.

The 1963 regulations included the Prahova district in the 8^{th} degree seismic zone and the buildings erected after this period took into account this regulation.

3. THE PLACE OF PRAHOVA DISTRICT IN THE SEISMIC EVENT FROM 1977

The 1977 earthquake had a multiple focus. The final shock S3, the strongest, took place near Pătîrlagele(Buzău district) 90 km Northeast from Ploieşti (fig.1). The main direction of energy radiation was South-West, the faults from the synclinal side Drajna-Vălenii de Munte served to propagate the energy towards the central zone of the district. The salt mountain mass Telega-Băicoi - Slănic, directed the seismic wave, towards the fault Tinosu(Ploieşti) -Fierbinţi-Călăraşi and its extension to the North.

For the Prahova district we have no records of seismic intensity. This is assessed by CFPS Bucharest at I-VIII, on the Medvedev map at I-VIII-IX, and by Despeyroux (France) at I-VIII. As compared with Bucharest (registered by the above mentioned at I - VIII), we must take into account the following aspects:

- In the Prahova district, there were no framed old buildings similar with those that fell in Bucharest.

- The buildings with the same structures, designed for the 8k seismic grade in Ploieşti, Cîmpina etc., suffered similar failures with those in Bucharest, designed for the 7^{th} seismic degree.

- The buildings designed after 1964 in the Prahova district were near the level of actual regulations.

- A number of old buildings (churches) suffered greater damage as compared with those in Bucharest, The analysis of these aspects lead to the conclusion that the seismic intensity in Ploieşti and in the most affected zone in the district was at least equal with that recorded in Bucharest. The under estimate of intensity was generated, probably, by the smaller number of victims.

4. HUMAN BEHAVIOR DURING THE EARTHQUAKE

Between the 1973-1976, the population felt some faint, short tremors. These events did not produce damage. Therefore, in its initial phase, the 1977 earthquake did not alarm the population. But because of the duration of the oscillations and their increasing amplitude, together with loud noises and the cutoff of electric power, the panic-stricken population left the buildings in haste. In crowded places (restaurants, theaters) the panic was general.

Lacking seismic education, some inhabitants in the street were injured by falling fire walls, chimneys, etc.

The following were also noticed:

- strong oscillations, first vertically and then horizontally, which gave the impression of general instability and insecurity;
- loud noises, produced by the falling of furtniture,walls and floors;
- large furniture, shelves, and cupboards overturning, plates and dishes breaking; pictures falling;
- outside, large and strong oscillations on the surface of asphalt (with the bending of columns), which later resumed their position;
- the visible illumination of the sky.

5. THE REACTION OF EMERGENCY SERVICES

The breaking off of electric power supply (due to the shifting of transformers) caused the interruption of radio and television emission and also of the telephonic connections. Though with dificulty, traffic went on. The telephone connections were replaced by radio-telephones, which kept working in the medical assistance network.

Medical assistance was assured without interruption i n the whole district, though of the 16 hospitals, 5 were strongly affected, some pavillions needed evacuation, and another 4 were greatly damaged. The fire engines succeded in stop several small fires. All the available means were directed to the salvation of victims and helping those who had suffered from the disaster.The population was continuously supplied with food.

6. SEISMIC PERFORMANCE OF BUILDINGS

a) GENERAL ASPECTS

The housing stock in the Prahova district was deteriorated more or less. Some buildings fell down; many were strongly da - maged. The industrial installations, the bridges and the roads were damaged. Some factories interrupted their activities. Some fires broke out.

b) EFFECTS ON RURAL BUILDINGS

Buildings built according to traditional methods (wooden buildings) suffered little damage.

Raw stoned buildings were strongly damaged; the mortar did not secure the band between the stone blocks. The stone buildings suffered little damage. The brick buildings behaved differently:

- those with simple brickwork walls suffered some collapses and much damage;
- those with brickwork with concrete cores and floors suffered only slight damage, even if the cores were only at the corners.

c) THE EFFECTS ON URBAN BUILDINGS

We took into account representative public buildings o 3 stories.

The buildings built during 1920-1935 generally of brick massive masonry, suffered important damage, destruction and dislocation. Some of them needed demolishing, other ones repairing.

For public buildings with reinforced concrete frames(Central Halls Ploiești) the damage was limited, but the cost of repairing were very high.

Five-story load-bearing masonry buildings constructed before 1964 suffered heavy damage, such as failure of corners,windows, longitudinal elements, etc., and a general shear failure at the ground level. Many of these buildings were evacuated and reoccupied after major repairs.

There were few framed buildings in the district. These suffered heavy damage in joints, cracks at the end of beams, and shear failure in columns.

A special case was the collapse of a 4 story building with floor shops. The block fell down by the general shearing of columns under the floor beam. The buildings with monolithic reinforced concrete walls had little damage: cracks and concrete crushing. The large panel buildings were affected very little. The preexisting cracks opened and the slabs slipped over lintels.

7. THE SPECIALISTS ACTIVITIES

In evaluating the post-earthquake situation, the specialists were confused. They had to carry on both a psychological and a technical activity. As compared with the problem, the number of engineers was insufficient. Some of them derived subjective conclusions and delayed them decisions. Technical analysis involved many specialists: design engineers and technical boards from administration. In case of another earthquake, the situation may be repeated, because many of the specialists who had gained experience in 1977, should be retired by then. In order to avoid future confusions and delays, it would be useful to develop regulations in order to establish a consistent way to analyze the effects of earthquake and to evaluate the vulnerability of buildings. In a first step, the methodology must be very simpl in order to involve medium trained specialists; Later, complex analyses may be done, on the base of data from the first step.

8. THE REPAIR AND PROTECTION OF BUILDINGS

By sustained efforts of all the working people, in a short period of time, the damaged buildings were ready to be used again. Top priority were the hospitals, the schools, administrative buildings, as well as the evacuated apartments. In the masonry buildings - showing a longitudinal deficit - the bearing walls were provided with reinforced netting. Other buildings were repaired in the displacement areas, by introduction of structural steel braces, or of some webs in the corners. The shear wall buildings were infected, strengthened with concrete newly grouted, The damaged columns and nodes, were repaired and strengthened with concrete. To protect affected areas, a sustained action of judicious verification from the view point of antiseismic strength, is necessary the repaired or strengthened buildings included. For solving the problem as a whole a permanent program is necessary because the passing of time sends us away from the passed earthquake and draws us near the next.

9. CONCLUSIONS

We again underline the following aspeots:

- the Prahova district was strongly affected by the 1977 earthquake;
- continuous activity is necessary in order to protect the existing buildings. A useful tool may be a regulation which establishes a constant way of work.
- for the next earthquake training of specialists body is necessary.

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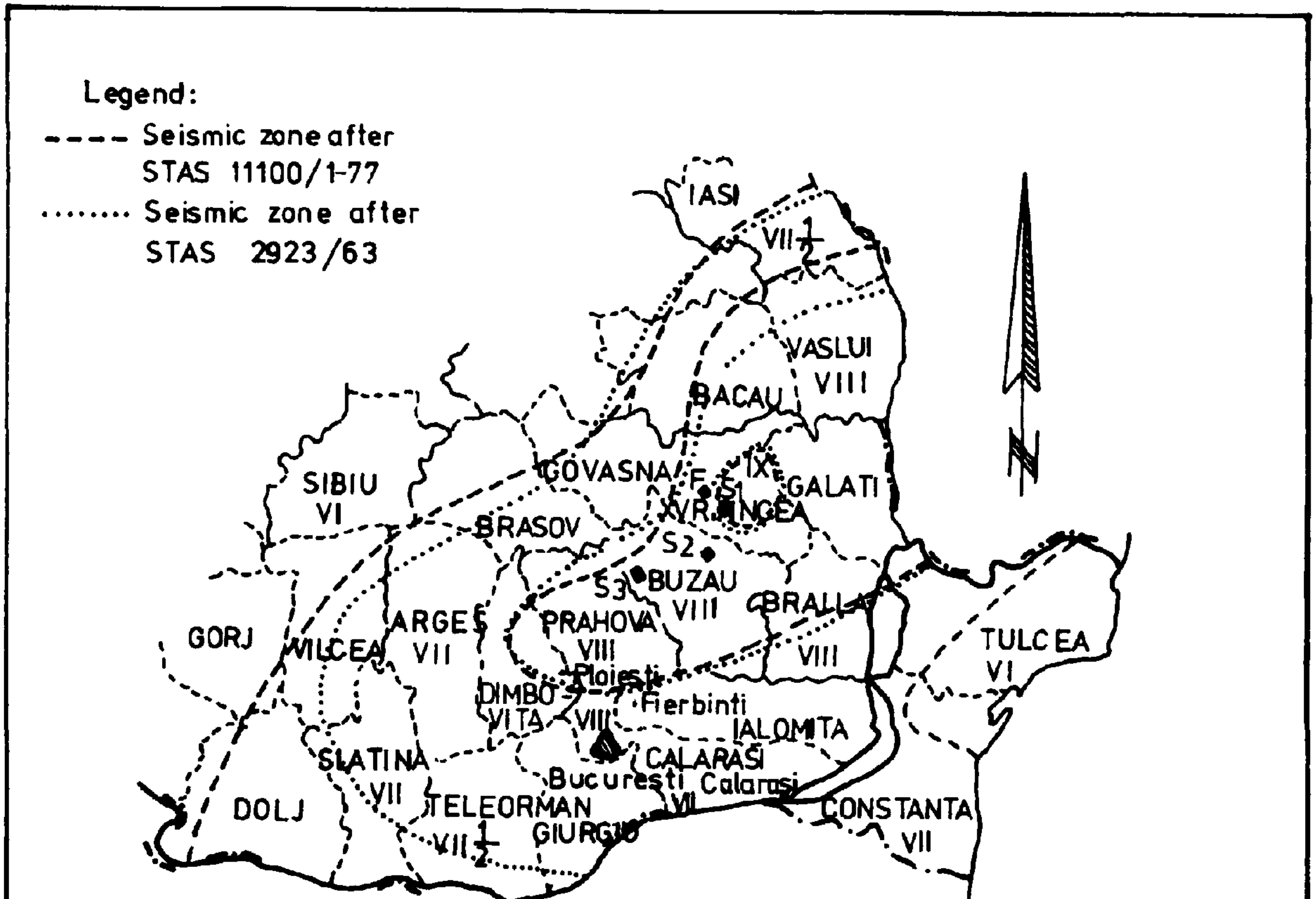
Legend:
Seismic zone after STAS 11100/1-77
Seismic zone after STAS 2923/63
IASI
VII 1/2
VASLUI
VIII
BACAU
SIBIU
VI
COVASNA
F.S.
IX
GALATI
VRANCEA
BRASOV
S2
S3
BUZAU
VIII
GORJ
VILCEA
ARGES
VII
PRAHOVA
VIII
BRAILA
VIII
TULCEA
VI
Ploiesti
DIMBOVITA
VIII
Fierbinti
IALOMITA
SLATINA
VII
CALARASI
Bucuresti
Calarasi
DOLJ
TELEORMAN
VII
CONSTANTA
VII
VII 1/2
GIURGIU

Fig. 1

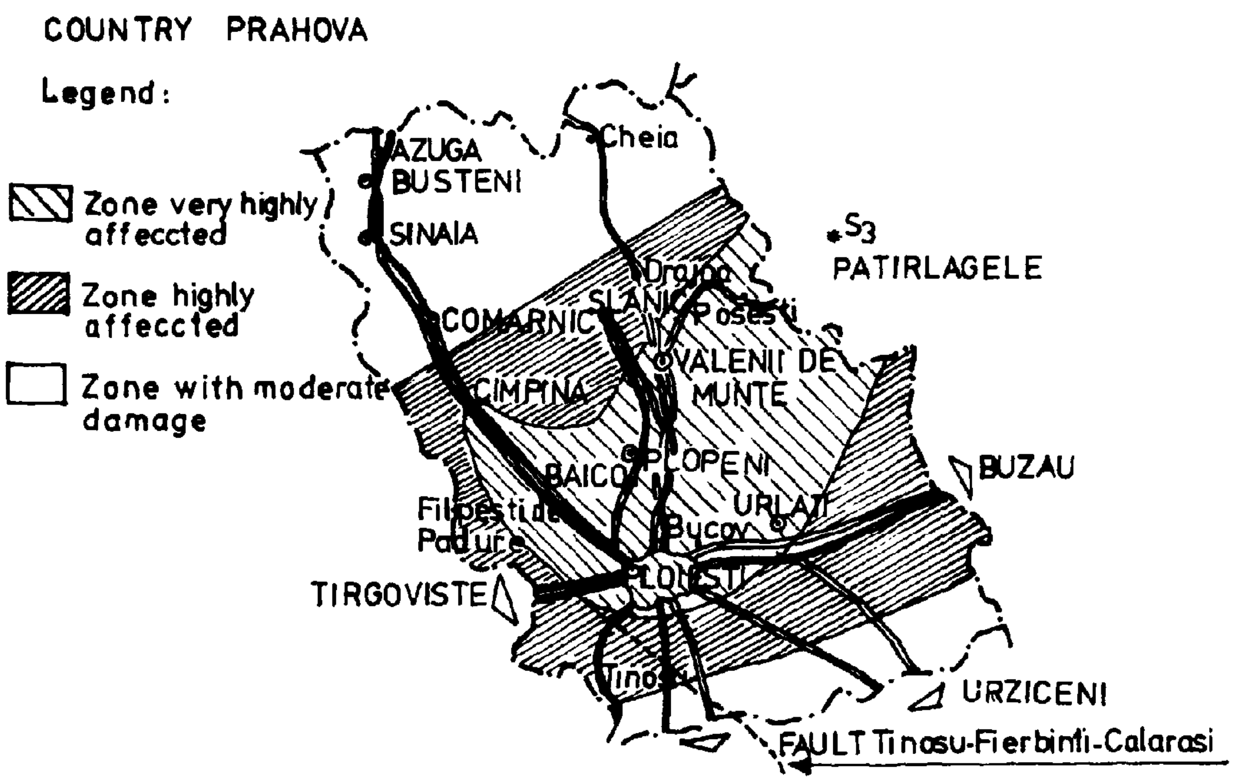
COUNTRY PRAHOVA
Legend:
Zone very highly affeccted
Zone highly affeccted
Zone with moderate damage
AZUGA
BUSTENI
SINAIA
Cheia
S3
PATIRLAGELE
COMARNIC
SLANIC
Posesti
VALENII DE MUNTE
CIMPINA
PLOPENI
BAICOI
BUZAU
URLATI
Bucov
Filipestii de Padure
TIRGOVISTE
PLOIESTI
Tinosu
URZICENI
FAULT Tinosu-Fierbinti-Calarasi

Fig. 2

I.9 BEHAVIOR OF RESIDENTIAL BUILDINGS IN JASSY DURING THE 1977 VRANCEA EARTHQUAKE

Constantin Mihai x)
Silvia Covali xx)
Gheorghe Palamaru xxx)

The original monuments in the city of Jassy which mark its different historical stages, such as Saint Sava church, Three Hierarchs and Barboi church or The Palace of Culture (which is built on the site of the old princely palace), are guarded by Cetăţuia, Galata, Frumoasa and other monasteries which are located on neighboring hills.

Due to its suitable conditions for human life and its location at the crossing of commercial routes, the city of Iassy quickly developed and in a short time became the capital of Moldavia country, a political, and economic, cultural center of the country. It is noteworthy to mention that in the city of Iaşi functioned the first university as well as the first academy in Romania.

It may be said that in the city of Jassy every place reminds one a historic moment. That is why the city may be considered as a true national museum where each stone and house is talking about its past.

Generally, the historical monumental buildings of Iaşi have stone masonry foundations, stone or brick masonry walls and columns, timber floors and various kinds of small vaults and arched walls.

The roofs were made of wood frame, works, and special fillings of heavy materials were used as floor insulation. Both exterior and interior walls had appreciable thickness (generally more than 1 m) and sometimes they had no weaving connections at corners and intersections. The older buildings were provided with steel ties or more rarely, with wood ties as well as with iron-clamps. The quality of stone and brick masonry as well as that of the mortar varies appreciably from one building to another.

The historic monuments to which references are made in this paper have resistant structure made of massive bearing

x, xx, xxx, Senior researchers, ICCPDC - branch of Jassy, Romania

masonry and floors with variable stiffness and heavy masses. Church buildings are generally provided with towers symmetrically or asymmetrically distributed about the main structure. Arrangement of the resisting elements and their connections, as well as general arrangement of the masses, often induce, due to seismic input, complex actions that are difficult to evaluate by calculations. Also difficult to estimate is the bearing capacity of different elements due to the fact that they underwent various degrees of damage in time, which are difficult to be evaluated For example, an old building constructed 500 years ago in Romania has survived 31 earthquakes of which, seven had the intensity degree of 7, five of 7/2, ten of 8, five of 8/2 and four of 9 in the epicentral region.

It should be noted that a part of the old buildings, due to their structural equilibrium and accuracy of execution underwent minor damage from the earthquakes, However, the number of these building is relatively small (fig.12).

SOME REMARKS REGARDING THE PERFORMANCE OF SEVERAL HISTORICAL MONUMENTS IN THE CITY OF JASSY

Twelve photos have been selected from various views of the monumental buildings in Jassy taken after the March 4,1977,earthquake to emphasize specific damage of monuments that were affected. Figures 2 to 4 show the old University and Academy buildings while Figs.5 - 12 show images of different churches.

The most important observations are mentioned below:

- the effect of masses upon inertia forces even for low rise buildings (Figs.2 - 11);
- weak mechanical qualities of some materials degraded during the years;
- unfavorable effects of some foundations without spatial connections and made of unadequate materials;
- unfavourable effect due to the absence of some continuous intermediare connections at large height walls;
- unfavourable behavior of appendages located at the upper part of building, especially when asymmetrically placed (Figs.7; 8; 11);
- unfavourable behavior of short columns, especially when they are made of stone work or brick masonry (Figs.9; 10);
- any modifications performed to bearing elements of the building during its life are evidenced during earthquake action and generally have detrimental effects (fig.11);
- lack of walls weaving at corners and intersections presents particular dangers (Fig.2; 3);
- timber floors do not provide the required interaction of the building walls. These floors can present a great danger when the ends of wood beams are degraded (Fig.3; 6);
- in some cases the interior or facade ornaments under-

went important damage. Their repair is expensive and difficult to perform.

The restoration of historic monuments is a special and very difficult problem. A restoration design should be based upon numerous studies and research projects, from which those referring to structural strength are of a special importance.

Fig. 1

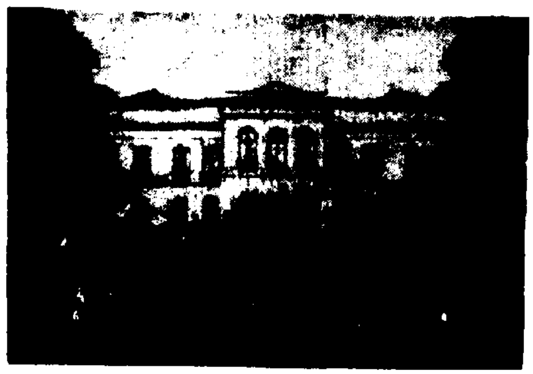

Fig. 2

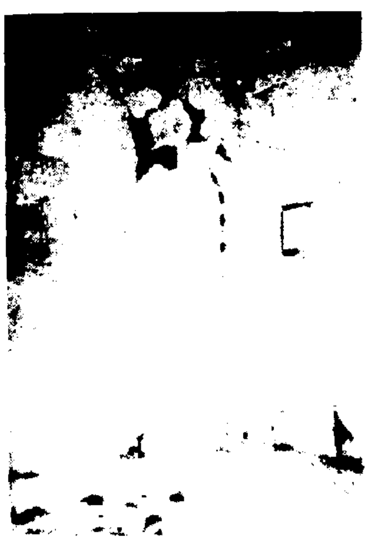

Fig. 3

Fig. 4

Fig. 5

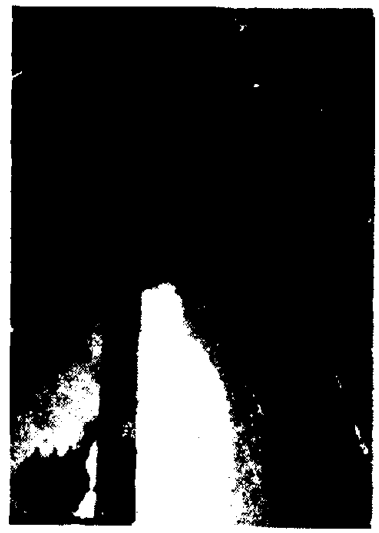

Fig. 6

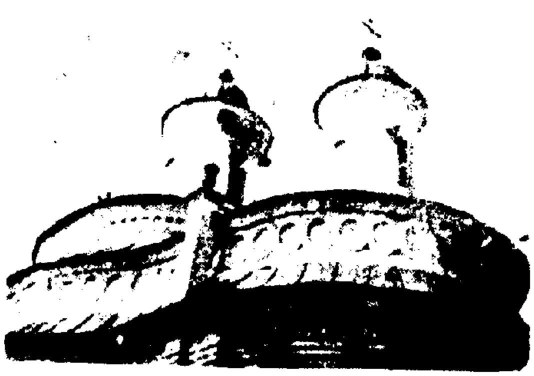

Fig. 7

Fig. 8

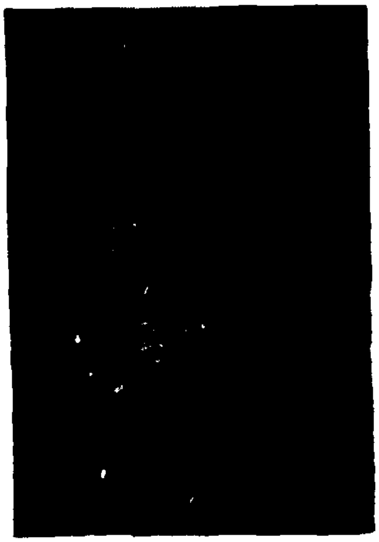

Fig. 9

Fig. 10

Fig. 11

Fig. 12

I.10 DESIGN AND PERFORMANCE OF PREFABRICATED BUILDINGS LOCATED AT SITES WITH DIFFICULT GROUND CONDITIONS DURING THE MARCH 4, 1977 EARTHQUAKE

Adrian Mihalache[x]
Aurel Liulica[xx]
Victor Mihailovici[xx]

ABSTRACT

The paper presents some aspects concerning the design (layout, construction peculiarities, material, consumption) and behavior of dwellings having five stories with large-panel structure built in the city of Jassy, in high seismic zones with difficult foundation conditions.

The main features of the behavior of these structures in service are described, under the action of both permanent and special loads. References are made to the behavior of such buildings in the earthquake of March 4, 1977. Also shown are some aspects concerning the general degradation of the buildings, the components, and the damage due to external agents;as well as implications about the stability and resistance capacity.

Finally, necessary conclusions are presented for the design of fully precast buildings on grounds with difficult foundation conditions and a high degree of seismicity, in order to achieve, from the socio-economic point of view, a satisfactory architectural solution and general seismic protection.

Buildings with large panels.with five stories erected after 1960 in the city of Jassy, generally have a rectangular shape in the plan, with the structure of coupled shear walls along the two directions (fig.1).

The buildings with reinforced concrete shear walls erected with large panels represent about 40% of the dwellings built till the year 1977. The high percentage of these buildings is

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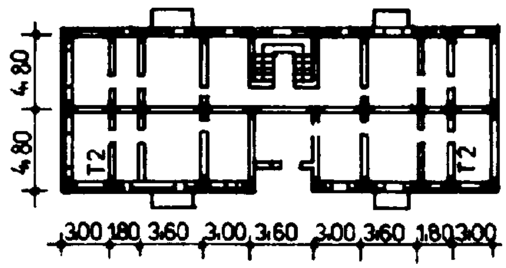
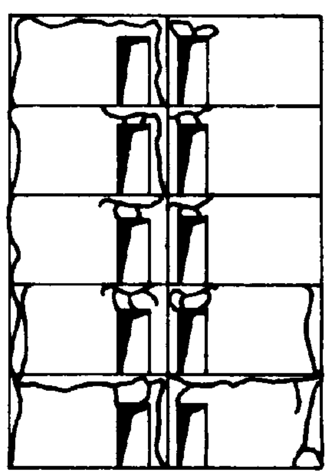
x Dr.eng.
xx Engineers

Research and Design Institute - ICPROM Jassy
Romania



SHEAR WALLS
FLOOR SLABS > PREFABRICATED
FOUNDATION – continous
DESIGN CODE P13-63



SURVEY OF THE CRACKS

FIG. 1. PREFABRICATED LARGE PANELS

due to peculiarities of layout, low material consumptions, and high speed in erection.

The region where the city of Jassy is located has two characteristic zones:

- zones on the plaine (river Bahlui) with grounds consisting of contractile clays with variable level of the underground water, which can rise to the ground level;
- zones on the slopes of the hills with ground consisting of loessical dusty calys, ground that is sensitive to moisture (consolidated).

The layout and the analysis for seismic loads have been performed in accordance with the Romanian standard STAS 2923-52 (63) and the Romanian earthquake resistant design - code P13-63 (version P13-70) taking into account that the town of Jassy is located in a zone of degree 7 (MSK).

The designation of large pannel system is applied to multistory structures composed of large concrete panels which are connected vertically and horizontally so that the wall panels enclose appropriatelly size spaces for the rooms of the building. Prefabricated wall panels are usually one story in height and in general both horizontal and vertical joints exists between the panels. The horizontal floor and roof panels usually consist either of one-way spanning prefabricated slab elements or of two-way spanning elements of the size of the relevant room. When properly connected together the horizontal elements act as diaphragms, transferring the earthquake loads to the walls in addition to resisting the gravity loads.

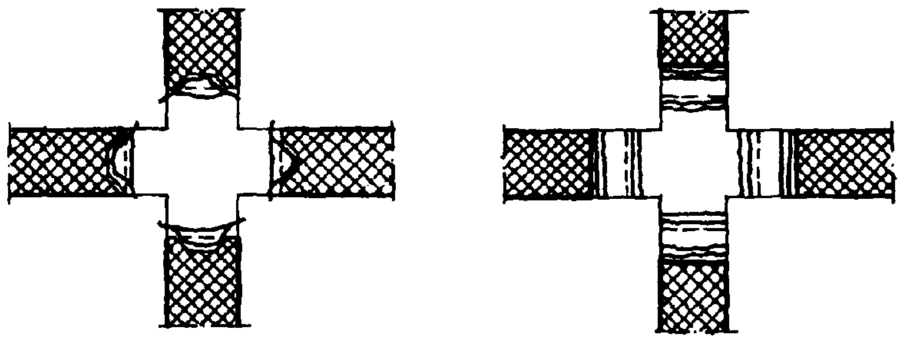
The load bearing walls are placed both perpendicular and parallel to the longitudinal axis of the buildings. The walls provide resistance to horizontal seismic loads in both directions and support the gravity load from two-way spanning floor and roof elements.

The joints between the panels are constructed to assure the continuity of the shear-walls and of the slabs realising the linkage of the precast members. The joints have been designed to take the shear force that occurs between the ends of the precast elements, so that finally the entire structure shall behave like a monolithic reinforced concrete coupled shear wall structure.

The lateral end shape of the large panels and the linkage elements have been designed so that the joints shall have the capacity to take the shear force and the local compression force of the concrete.

Technologically, the joints have been designed of the

CASTELLATED JOINTS



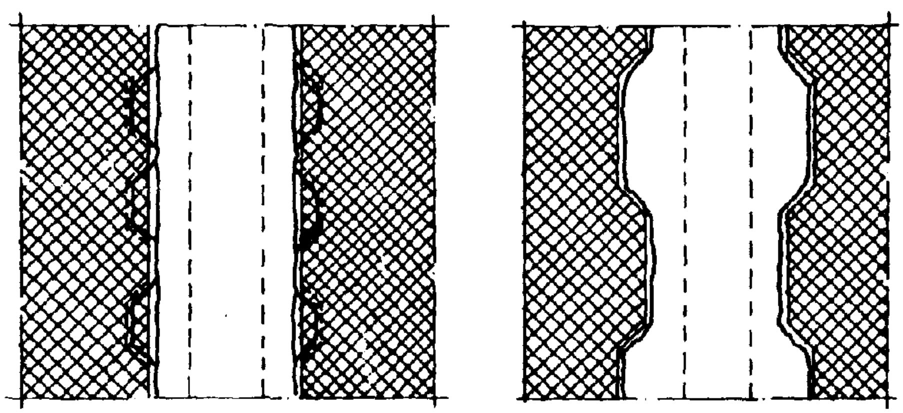


FIG. 2 VERTICAL JOINTS

closed or open type, undercasting the concrete or casting it at the view.

Depending upon the type of joints, the end preparation of the panels is with shear keys of the castellated type.

The general structural behavior of those buildings during the earthquake of the March 4, 1977 was been good. Special phenomena was not observed, but there have been noted cases of damage in the structural elements characteristic for the shear walls of reinforced concrete as well as specific damage due to the execution technology of the large panels.

In the dwellings with the structure of reinforced concrete shear-walls erected large panels cracks of the following kind could be noted:

. in the joints between the large panels (walls) that occur in general at all types of vertical joints. These cracks occurred on the entire height of the joints, with a current opening of 0,1-0,2 mm and accidentally of 0,5-1,0 mm and have the trace of the contact zone between the panel and the concrete cast in the joint(fig.2).The seismic action has also amplified the opening of some of these cracks that existed with a very small opening before the earthquake.

. under the floor panels - in the contact zone with the vertical wall (0,1-0,2 mm) which have longer openings in the zone of the coupled beams where occurs also a local crushing of the mortar.

. in the coupled beams frequently at the doors, vertical or inclined in the fixing zone (0,1-0,2mm) and of recuded frequency at the windows.

. in the wall panels - rare enough, along a vertical or inclined direction, at the ground level, the second and the fifth level.

Also noticed were cracks between the prefabricated stairs and the horizontal stair panels due to the layout of thejoints between these elements.

The lack of plaster at the entirely precast buildings with five stories in the city of Jassy means that the damage is not impressive at the first view but the damage at the large panels is at least of the same nature and the same size at the buildings with 9-11 stories, with the structure of reinforced concrete shear walls. The foundation grounds in this zone have a natural vibration period computed as 0,25-0,41 sec.in the zone of the plain (Bahlui river) and 0,26-0,26 sec.for the zone of the slopes and this fact makes the buildings with a high stiffness sensitive (reinforced concrete shear walls structure) with small periods (0,3-0,4 sec.) and leads also to a specific spectral composition of the seismic motion of this zone. That is because the effects of the earthquake have been so obvious for these types of structure with the decrease of the lateral rigidity. Thus, an aspect that must be pointed out is determined by the influence of the foundation ground, characterized by the nature, thickness, declination and succession of the ground layers, the presence of the underground water, factors that can determine overload effects on the buildings.

Taking into account the importance of the joints of the residential buildings with large panel structure in transferring of the forces and in the dissipation of the induced earthquake energy, it is necessary to estimate the bearing capacity reserve of the structure. Because of the peculiarities of this constructive system for which the shear walls are realized of separate elements (prefabricates panels) and zones of cast insitu concrete, the cracks develop generally between the two types of concrete. These cracks, as have been observed at the investigated buildings, are amplified as number, length and opening by the action of the earthquake, and this leads to a lack of protection of the reinforcement in the joints and also to a separation of the structural elements.

From the analysis carried out it turns out that this construction system is an earthquake resistant system. This system has some technological aspects that require its use in our country.

Together with the increasing of the reliability degree, fulfilled some social-economic demands concerning the reduction of the fuel and of the material consumption, the realization of some urban enssemble and agreable volumetric aspect with lasting finishing (executed in the factory)

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I.11 SEISMIC BEHAVIOR OF DWELLINGS WITH COUPLED SHEAR WALL STRUCTURE LOCATED AT SITES WITH DIFFICULT FOUNDATION CONDITIONS DURING THE EARTHQUAKE OF MARCH 4, 1977

Adrian Mihalache[x]

ABSTRACT

Presented are the characteristics of the foundation ground on which buildings with coupled shear wall structure (nature of foundation ground, the presence of the phreatic water, vibration period of the ground) of the zone of the city of Jassy, that can determine the local amplification of the seismic oscillations with the effect of overloading the buildings. Also pointed out are some aspects concerning the design of these buildings and their behavior during the earthquake of the March 4, 1977. The remarks are related to the general structural behavior and to the behavior of the structural members depending on the execution technology, the seismic intensity, etc. Also pointed out are the possibilities of assessing the influence of the foundation ground on the seismic loading and also the parameters that characterized the working stages of the building. In conclusion elements are presented that must be taken into account in the design of structures located in such zones.

To make evident the capacity of earthquake resistant of structures with coupled shear walls located on grounds with difficult foundation conditions, their behavior at the earthquake of March 4, 1977, and especially the comparative behavior of the buildings with different locations are examined.

The earthquake of March 4, 1977 represented a natural testing laboratory of particular importance the structures designed to resist earthquakes erected by means of industrialized techniques.

An aspect that must be pointed out is determined by the local influence of the ground, characterized by the nature, thickness, inclination and succession of the layers of earth, the presence of the phreatic water, as well as the depth and the

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thickness of the underground water layers.

The region where the city of Jassy is located has two characteristic zones (fig.1):

- zones on the plaine (river Bahlui) with grounds consisting of contractile clays with variable level of the underground water which can rise to ground level (fig.1a);
- zones on the slopes of the hills with grounds consisting of loessical dusty clays, grounds are sensitive to moisture and consolidated (fig.1b).

The structural and economic efficiency of the coupled shear walls has decided upon their choice, about 30 years ago, structural elements of multi-story dwellings and administrative buildings situated in seismic areas.

Dwellings in Jassy with 8-11 stories with coupled shear walls of reinforced concrete were erected until 1977 with:

a. plywood panels formworks (fig.2.1.-2.4.);
b. gliding formworks (fig.2.5.-2.11);
c. plain steel forms (fig.2-12).

The layout and the analysis for seismic loads have been performed in accordance with Romanian standards STAS 2923-52(63) and Romanian earthquake resistant design code P 13 - 63 (version P 13 - 70) taking into account that the Jassy is considered located in a zone of 7th degree (MSK),

In determining the horizontal loads at all these buildings the calculated values of the factor (tab.1) which introduces the influence of the fundamental vibration period and of the ground, has been increased by 50%. The increase of the factor (was required because the ground in this zone have an elevated underground table (zones on the plaine). This value was maintained also for the grounds sensitive to the moisture, which represents a protection at the 7V2 degree (MSK).

a) Behavior of structural members

In the full shear-walls the following kinds of cracks have been observed:

- vertical, generally at the upper levels of the building, resulting from the accentuated shrinkage effect because of the absence of a continuous reinforcement with network along the vertical direction of the building. These cracks have been located especially in the long shear-walls;
- horizontal, straight along the casting joints, both two consecutive stories and in the joining zone of the shear-walls with the level-floors, cracking opened due to the seismic actions;

- inclined, due to the principal tensile stresses from the seismic shear forces accentuated also by some interruptions in casting concrete in these areas, as well as by the absence of a continuous corresponding reinforcement with network. The presence of the inclined cracks even at the upper levels of a slender shear-wall also indicates the shearing force effects due to the action of the superior modes of vibration.

The coupled shear-walls have presented cracks both in the structural members -like these mentioned previously and in the horizontal connection elements:

- vertical, in the rigid fixing zones of the coupled beams, in the vertical structural elements, due especially to the seismic bending moments;
- horizontal, straight along the casting joints (under the slab) uncorrespondingly tested and opened due to the seis - mic action;
- inclined, or "X" shaped, due to the main tensile stresses from the seismic shear force. These cracks have appeared more accentuated, even leading to the concrete dislocation in the lower zones of the stiff shear walls with one single row of openings (0,2-0,4) H. At the flexible shear walls,the cracks have extended almost to all the levels.

b) General structural behavior

One of the main causes of the structural damage was the absence of an adequate antiseismic layout with regard to the general geometry, of the story stiffness distribution and of the connections between the structural members. The damage appeared in the coupled beams and especially in the staircase shear-walls. In buildings with a honey comb type of structure (fig.2.10) the damage was located in the staircase elements connecting the two building bodies, which have a separation. In the building (fig.2.11, fig.2.12) the tendency to damage was located especially in the internal longitudinal shear - wall taking most of the seismic loads along this direction.

In the two distinct and different zones of the city of Jassy buildings with indentical structures and layout are located. Thus, for building A(fig.2.11) in the plain zone, the seismic loads have been determined (tab.) with the seismic factor c = 6,75% (increasing the factor for grounds with high underground water table) while building T (fig.2.12)in the slope zone in the tătăraşi residential district have been also pro - tected for the seismic factor c = 6,75%, although the calculated seismic factor was 5,4%. The analysis performed have pointed out that in the buildings located in the plain zone (river Bahlui) where the underground water table is high, the damage was stronger (A.2.11) is compared with that in the buildings of the Tătăraşi zone (T.2.12.). Thus the values of the funda - mental vibration periods measured after the earthquake(tab.2.) are longer for the buildings in the plain zone, as a result of the amplification by the ground of the seismic loads. The observations carried out require improved construction solutions in the design of such structures, as follows:

- to adopt an adequate layout (symmetry of the general geometry, distribution of the story stiffness and connections) in view of ensuring a best spatial cooperation;
- to use structures with at least two longitudinal shear walls;.
- to determine the stiffness of the structure and hence the number of necessary shear - walls in terms of the limits in which it follows to maintain the seismic response, in the case of different kinds of earthquakes.

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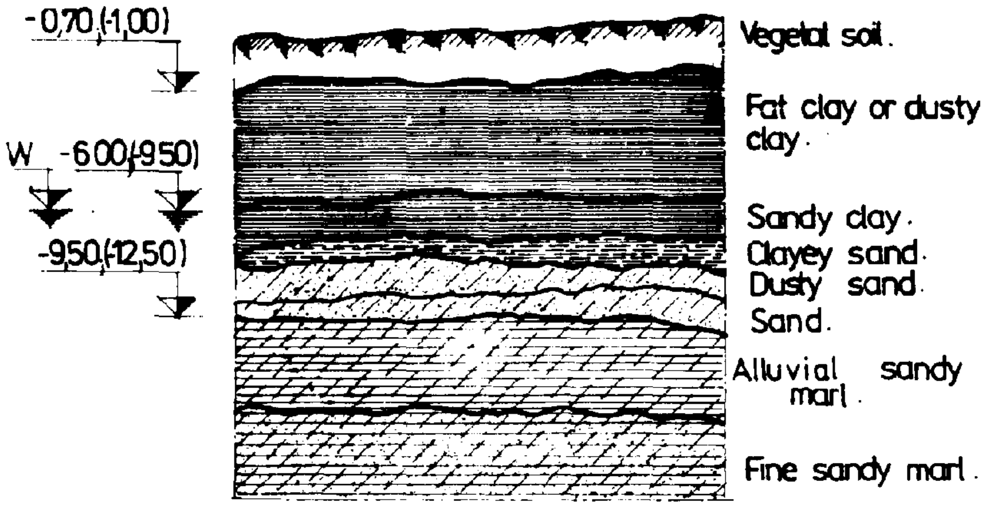
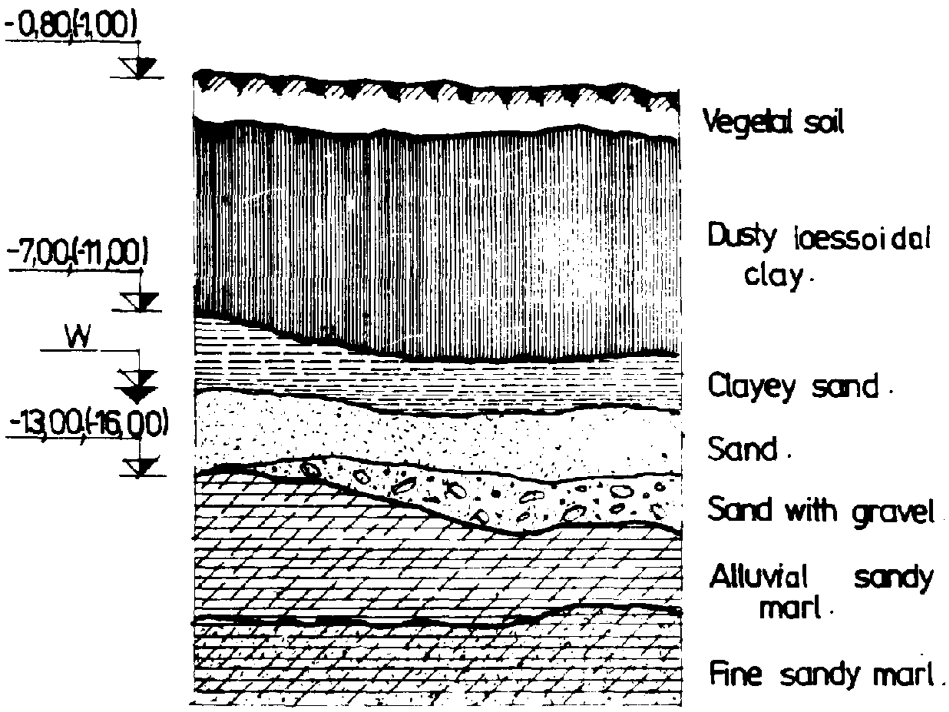


1a. Zone on the plains of the river Bahlui



1b. Zone on the slopes of the hills

FIG.1 STRATIFICATION OF THE FOUNDATION GROUND

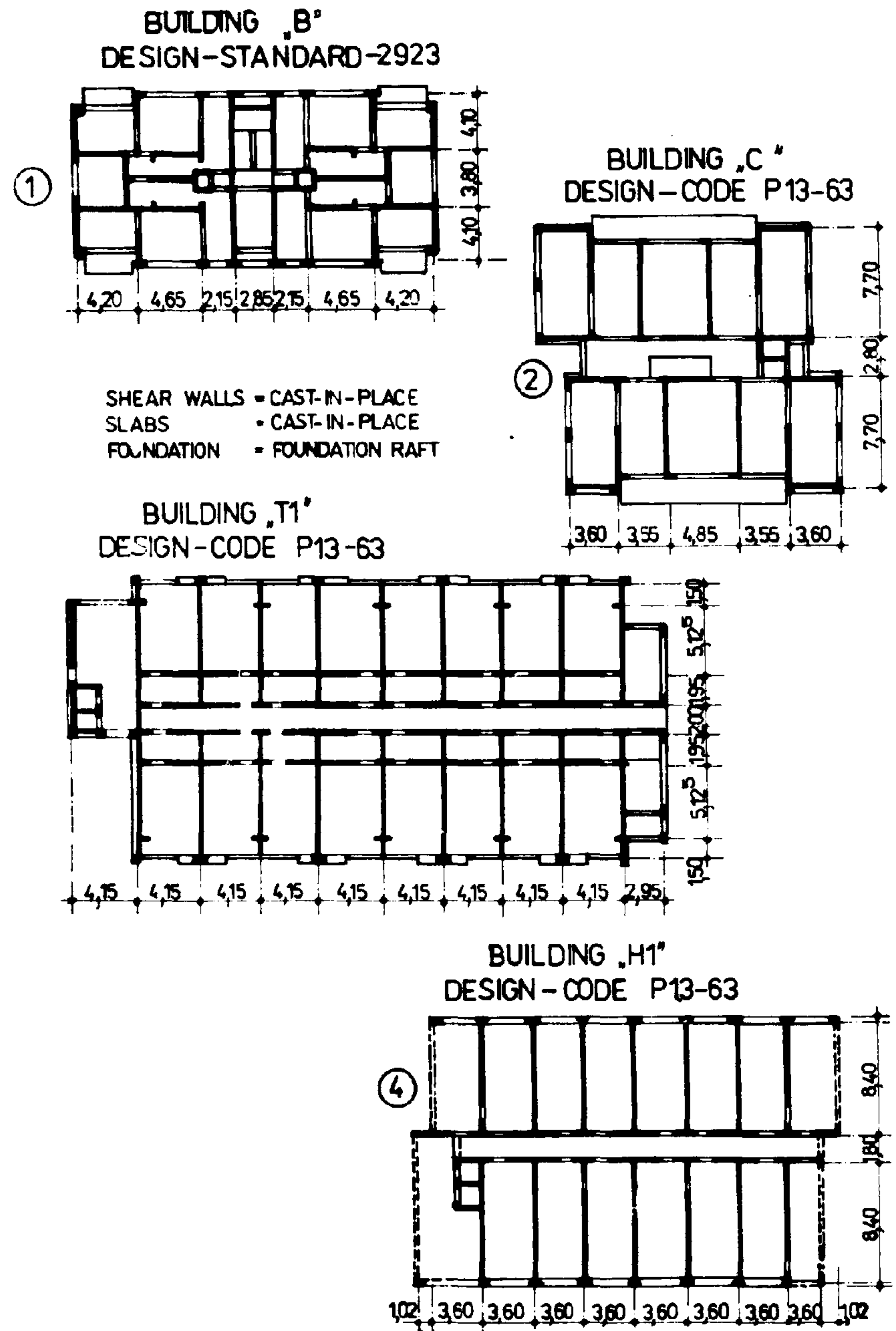
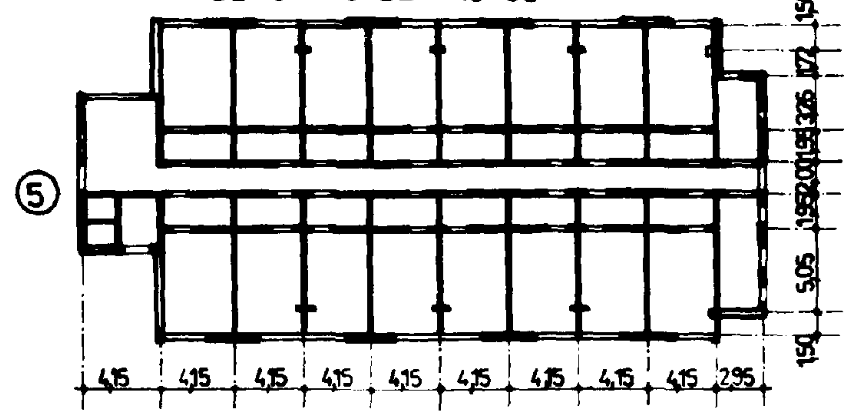

BUILDING „B"
DESIGN-STANDARD-2923
1
4,10
3,80
4,10
4,20 4,65 2,15 2,85 2,15 4,65 4,20
BUILDING „C"
DESIGN-CODE P13-63
2
7,70
2,80
7,70
3,60 3,55 4,85 3,55 3,60
SHEAR WALLS = CAST-IN-PLACE
SLABS = CAST-IN-PLACE
FOUNDATION = FOUNDATION RAFT
BUILDING „T1"
DESIGN-CODE P13-63
150
5,12,5
195
200
195
5,12,5
150
4,15 4,15 4,15 4,15 4,15 4,15 4,15 4,15 4,15 2,95
BUILDING „H1"
DESIGN-CODE P13-63
4
8,40
1,80
8,40
1,02 3,60 3,60 3,60 3,60 3,60 3,60 3,60 3,60 1,02

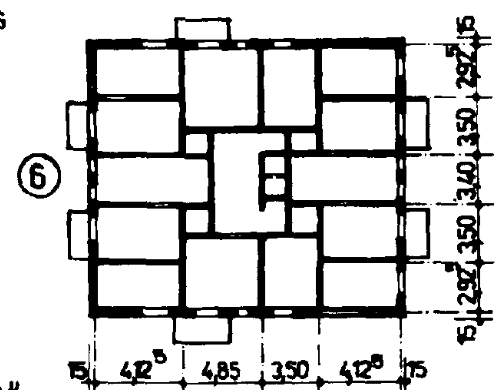
FIG. 2

BUILDING „A"
DESIGN – CODE P13-63



BUILDING „X"; „G"
DESIGN-CODE P13-63

SHEAR WALLS = CAST-IN-PLACE, GLIDING FORMWORKS
SLABS - CAST-IN-PLACE
FOUNDATION - FOUNDATION RAFT



BUILDING „H"; „A"
DESIGN-CODE P13-63

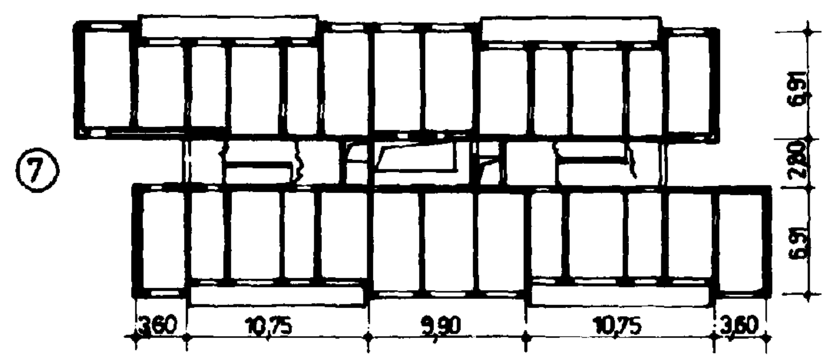
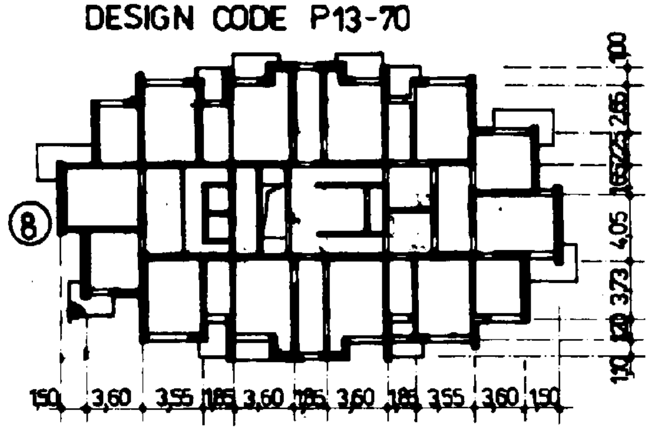


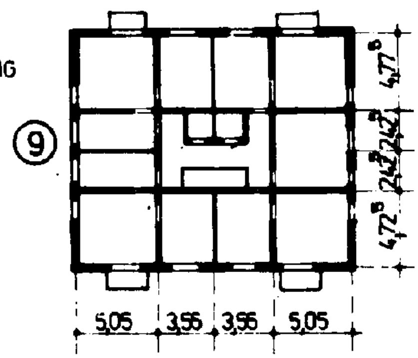
FIG. 2

BUILDING „Z"
DESIGN CODE P13-70



BUILDING „B"
DESIGN CODE P13-70

SHEAR WALLS - CAST-IN-PLACE GLIDING FORMWORKS
SLABS - CAST-IN-PLACE
FOUNDATION - FOUNDATION RAFT



BUILDING „T"
DESIGN CODE P13-70

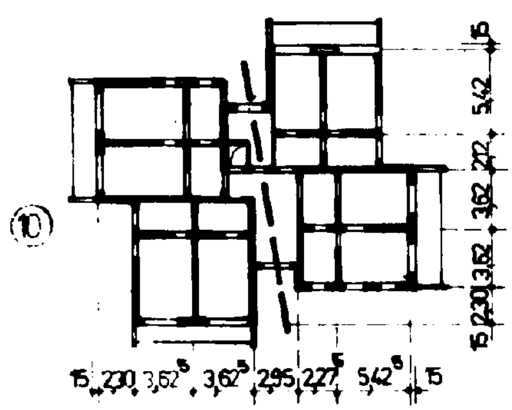
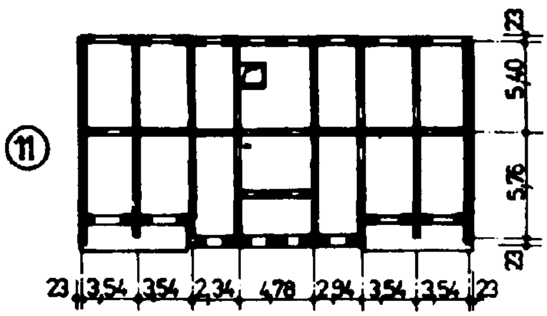


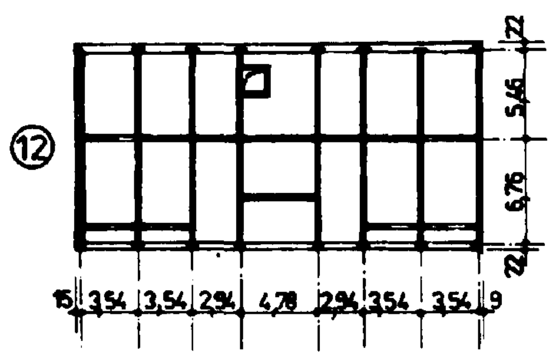
FIG. 2

BUILDING „A'
DESIGN-CODE P13-70



SHEAR WALLS - CAST-IN-PLACE, GLIDING FORMWORKS
SLABS - CAST-IN-PLACE
FOUNDATION - FOUNDATION RAFT

BUILDING „T'
DESIGN-CODE P13-70



SHEAR WALLS - CAST-IN-PLACE PLAIN STEEL FORMS
SLABS - PRECAST
FOUNDATION - FOUNDATION RAFT

FIG. 2

DYNAMIC CHARACTERISTICS

TABLE 1.

	BUILDING	FUNDAMENTAL VIBRATION PERIOD (SEC)		DYNAMIC FACTOR β AND SEISMIC DESIGN FACTOR C β 0,8/T							
				TRANSVERSAL				LONGITUDINAL			
		TRANSV	LONG	β	C %	β incr	C %	β	C %	β incr.	C %
1.	„B" Mal Sfing Bahlui	0,567	0,455	—	—	—	—	—	—	—	—
2.	„C" Tătăraşi	0,460	0,518	1,74	4,70	2,50	6,75	1,55	4,18	2,32	6,26
3.	„T1" Socola - Nicolina	0,408	0,325	1,96	5,29	2,50	6,75	2,00	5,40	2,50	6,75
4.	„H" Socola - Nicolina	0,414	0,371	—	—	—	—	—	—	—	—
5.	„A" Tătăraşi	0,410	0,325	1,95	5,27	2,50	6,75	2,00	5,40	2,50	6,75
6.	„X", „G" Tătăraşi	0,484	0,551	1,65	4,45	2,47	6,67	1,45	3,91	2,18	5,88
7.	„H", „A" Tătăraşi	0,482	0,365	1,66	4,48	2,49	6,72	2,00	5,40	2,50	6,75
8.	„Z" Socola-Nicolina	0,468	0,414	1,71	4,62	2,50	6,75	1,93	5,21	2,50	6,75
9.	„B" Mircea cel Batrin	0,518	0,533	1,55	4,19	2,33	6,29	1,50	4,05	2,25	6,08
10.	„T" Tătăraşi	0,365	0,473	2,00	5,40	2,50	6,75	1,69	4,55	2,50	6,75
11.	„A" Alexandru cel Bun	0,365	0,473	2,00	5,40	2,50	6,75	1,69	4,55	2,50	6,75
12.	„T" Socola-Nicolina	0,471	0,501	1,70	4,59	2,50	6,75	1,60	4,32	2,40	6,48

$C = K_s \cdot \beta(T) \cdot \psi \cdot \varepsilon$

$K_s = 0{,}03$ $\quad \varepsilon = 0{,}75$ $\quad \psi = 1{,}2$

C = seismic design factor (related to the fundamental mode)

K_s = ratio between the maximum acceleration of the seismic ground motion and the acceleration of gravity depending on the seismic zone.

$\beta(T)$ = dynamic factor depending on the fundamental vibration period

β_{incr} = increased dynamic factor for grounds with difficult foundation conditions.

ψ = correction factor considering the influences of the damping characteristics of the strength reserves and of the ductility

ε = equivalence factor.

THE VALUES OF THE FUNDAMENTAL VIBRATION PERIODS „T"(SEC) FOR THE BUILDINGS „A" AND „T"

TABLE 2

	TRANSVERSAL		LONGITUDINAL	
	BUILDING A	BUILDING T	BUILDING A	BUILDING T
The value determined according to the P13-70 with the relation $T_t = (0,045 - 0,055)\,n$ $T_l = (0,040 - 0,045)\,n$ (n = number levels)	0,495...0,605	0,495...0,605	0,44....0,495	0,44....0,495
with the relation: $T = (0,065 - 0,075)\frac{H}{\sqrt{B}}$ (B = dimension along the direction in which we consider the vibration).	0,552...0,637	0,552....0,637	0,390...0,450	0,390...0,450
The value determined by means of dynamic analysis.	0,365	0,365	0,473	0,473
The value measured after the earthquake.	0,524	0,425	0,727	0,652

SESSION II:EVALUATION OF THE EXISTING BULDING STOCK.
VULNERABILITY AND RISK ANALYSIS.
REPAIR AND STRENGTHENING OF STRUCTURES.

II.1

DETERMINATION OF BUILDING STOCKS FOR URBAN VULNERABILITY ANALYSIS AND EARTHQUAKE LOSS ESTIMATION*

Barclay G. Jones,** Donald M. Manson,***
Charles M. Hotchkiss,**** and Michael J. Savonis*****

ABSTRACT

Estimating potential losses from earthquakes involves determining the elements at risk, establishing the vulnerability of each type of element to an event of a given magnitude, and assigning a probability that an event of that severity will not be exceeded at the site of the element within a specified period of time. Engineers and scientists have devoted much research to vulnerability and hazard assessment. Estimating elements at risk has received less attention.

This paper reports on a series of studies concerned with developing methodologies for making indirect estimates of building stocks. The results of recent research using a complete enumeration of the building stock of a moderate-sized metropolitan area in the United States, Wichita, Kansas, are compared to previous and less complete studies. The number and area of buildings disaggregated by use are given and replacement costs calculated. The spatial distribution of buildings by rings outward from the center is also determined. The existing building stock of a metropolitan area is shown in greater detail than previously available, and regularities in the composition and distribution suggest the techniques are generally applicable.

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INTRODUCTION

In considering the impact of potential earthquakes, a prominent earthquake engineer wrote,

> . . .it must be considered that buildings of the past will still abound--buildings which can neither be abandoned, nor strengthened. Thus the fact remains that for the foreseeable future strong earthquakes are still going to be natural catastrophies. [Turnsek, 1982]

Another expert in the field has discussed the problem at greater length.

> In the U.S. and many other countries there exists a tremendous amount of building stock which was constructed before seismic design requirements were established or before the extent of potentially active areas was established. The range of hazard potential in these buildings can range from acceptable to totally unacceptable. Because this stock of existing structures represents a huge economic investment, it is not feasible to remove and replace any substantial part of these structures. Thus a serious problem exists in being able to assess the level of hazard presented by an existing structure. . . .
>
> An additional problem with existing structures is that for many regions of the world, including the U.S., there do not exist any good inventories of what the actual building stock is. An actual survey of existing building stock can be very expensive and will not be feasible for most regions. Thus indirect methods are needed to estimate building stocks and the potential hazard level or possible economic losses for a region. [Gaus, 1980]

The building stock in most regions has been the result of a slow accretion over time with structures remaining from all phases of the settlement of the locale, even the earliest one. The survivability of buildings over centuries and millenia is phenomenal. As a consequence, the inventory of buildings at risk from an earthquake is enormously complex in its characteristics.

Risk assessment consists of establishing the elements at risk from a particular natural hazard. The vulnerability of each of these elements to an event of a given magnitude must be assigned by expressing the degree of loss on a scale from 0 to 1. Hazard assessment establishes the probability that an event of a given magnitude will not be exceeded at the site of the element within some specified period of time.[UNDRO, 1980]

Vulnerability functions are then calculated in a matrix listing hazards of various magnitudes on one axis and building types by categories of vulnerability on the other. The percentage of buildings in each category expected to sustain various degrees of damage in terms of percentage loss in value is estimated for each magnitude of event. The cell entries are the sum of the products of the percent loss of value times the percent of buildings suffering various degrees of damage for each category for each magnitude of event. The potential loss from a particular event can then be readily calculated by multiplying the cells of the matrix by the total value of structures in each category and summing for each magnitude of event. Risk can then be determined by multiplying the aggregated vulnerability by the probability of a particular event. Aggregate losses for a specified period of time can be calculated quite simply.

Vulnerability assessments for urban areas are useful for a variety of purposes. They establish the potential magnitude of a problem and provide a basis for determining whether or not and to what extent vulnerability reduction measures are justified. They provide a basis for preparing for catastrophic events by indicating the scale of effort that will be necessary to recover from a disaster. If they could be made quickly and easily, they could help in planning for and undertaking relief and reconstruction immediately following catastrophies.

Much vulnerability analysis has been undertaken and many loss estimates have been prepared. Perhaps the most extensive of these was a monumental study which projected annual building losses from 1970 to 2000 for each of the states in the U.S. for each of nine natural hazards (earthquake, landslide, expansive soil, hurricane wind-storm surge, tornado, riverine flood, local wind, local flood, and tsunami. [J. H. Wiggins Company, 1980] Another notable example estimated the losses from a recurrence today of the most severe earthquake in U. S. history, that at New Madrid, Mississippi, which began December 16, 1811. [Liu, et al., 1979] Recent research which was limited to residential structures used approaches similar to those reported here. [Shodek, Gauchat and Luft, 1984] Scores of other studies have been made. Many of them have been very sophisticated and have considered other economic losses such as the contents of structures, loss of production and income, injuries and loss of life. They are too extensive to review here.

Risk, or the probable loss of all kinds, is therefore a function of three variables: hazard, vulnerability, and elements at risk. We have improved our hazard assessment capability greatly in recent years. We are making considerable progress in vulnerability assessment for particular kinds of structures. We have not done nearly as well in devising methods for determining the elements at risk. Regardless of the sophistication and scientific knowledge that has been used in vulnerability analysis in arriving at loss estimates, we really do not know how good such assessments are because they are based on estimates of the elements at risk the accuracy of which is unknown. Most of these studies carefully make disclaimers at the outset indicating some degree of uncertainty about the assumptions that have had to have been made about the elements at risk. The research that will be reported here is part of a continuing series of studies directed towards improving our capacity to make indirect estimates of one of the major elements at risk from natural disasters, the building stock existing in an urban region.

ESTIMATING EXISTING BUILDING STOCKS

In previous attempts to develop methods and techniques for making indirect estimates of existing building stocks we have posited that inventories of buildings would be characterized by various empirical regularities, as is the case with the results of so many kinds of human spatial behavior. [Jones, Manson, Mulford and Chain, 1976; Jones, 1978] These regularities would make it possible to build models which would describe the characteristics of building stocks that would be generally applicable to many different regions in a wide variety of cultures and countries. The parameters of the models would need to be fitted and modified to produce accurate estimates in each situtation, but the general structure and architecture of the model would remain constant.

The basic model indicated there would be a stable relationship between people and buildings. The Population/Building Ratio would increase generally with the size of the urban area under study. The size distribution of the building stock as measured by area would be skewed to the right, as would the distribution of buildings by height. The joint distribution of building by area and height would be approximated by a bivariate lognormal distribution. Further, the classification of buildings by residential and non-residential use would show stable relationships. Total building area per person and residential building area per person are expected to be relatively stable also. Remarkably robust empirical regularities were found in the studies cited above. Further study confirmed the earlier findings and indicated building area per person relationships were stable also. [Bauman, 1983]

None of the early studies covered a complete urban region. Most of them were restricted to central city areas. Whether or not there would turn out to be major differences between central and peripheral areas was unknown. The data used permitted only very crude disaggregation of buildings by use, and study in detail was not possible.

ENUMERATING A METROPOLITAN BUILDING STOCK

To address some of these questions we set out to study the building stock of a complete, self-contained moderate sized metropolitan area. It was desirable that it be isolated and not part of some larger conurbation. The study area selected for detailed empirical analysis was Wichita, the largest city in the state of Kansas, located approximately at the geographical center of the country. For purposes of analysis the metropolitan area can be considered to consist of the county in which it is located, Sedgwick County, throughout the period of study.

Spatial Analysis

Wichita, a relatively young urban center barely more than a century old, has been a major city for fifty years, but much of its growth has taken place since World War II (Table I, Figure 1). Centrographic measures have proven to be useful techniques for summarizing location, dispersion and shape of variables distributed in space. [Jones, 1980] A limited number of these spatial statistics was used in this study of the Wichita building stock: the Center of Population, the Average Radius, and the Standard Radius. Historical analysis of the Center of Population (center of gravity of population by place of residence) indicates the location of Wichita has been remarkably static measured in thousands of meters in Universal Transverse Mercator coordinates. In 1980 it was 910 meters south and 190 meters west of its location in 1950 (Table II). Like many cities in the world, Wichita has become more dispersed and spread out during the study period. The average distance of the place of residence of a member of the population from the Center of Population (Average Radius) has increased from 5.4 kms. in 1950 to 8.3 kms. in 1980. The Standard Radius (root mean second moment) has increased also. The increasing dispersion is not closely related to the rate of change of the population and must be considered independent of growth (Table II).

Detailed data were acquired from the computer tapes maintained by the County Tax Assessor for 166,000 parcels of land and 248,000 buildings for 1983. Non-building structures, while extremely important in terms of use, function and magnitude of value, were excluded from consideration. Since the purposes of the tax assessor in compiling and maintaining the tapes did not coincide with the objectives of the study, it was necessary to make access to the assessor's parcel

Table I. Population of Wichita and Sedgwick County, KA, 1870-1983.

Year	City of Wichita			Sedgwick County			Wichita as Percent of County
	Population	Change		Population	Change		
		Number	Percent		Number	Percent	
1870				1,095			
1880	4,911			18,753	17,658	1612.60%	26.19%
1890	23,853	18,942	385.71%	43,626	24,873	132.63%	54.68%
1900	24,671	818	3.43%	44,073	447	1.02%	55.98%
1910	52,450	27,779	112.60%	73,095	29,022	65.85%	71.76%
1920	72,217	19,767	37.69%	92,234	19,139	26.18%	78.30%
1930	111,110	38,893	53.86%	136,330	44,096	47.81%	81.50%
1940	114,966	3,856	3.47%	142,999	6,669	4.89%	80.40%
1950	168,279	53,313	46.37%	222,290	79,291	55.45%	75.70%
1960	254,698	86,419	51.35%	343,229	120,939	54.41%	74.21%
1970	276,554	21,856	8.58%	350,694	7,465	2.17%	78.86%
1980	279,272	2,718	0.98%	366,531	15,837	4.52%	76.19%
1983				381,300	14,769	4.03%	

Figure 1

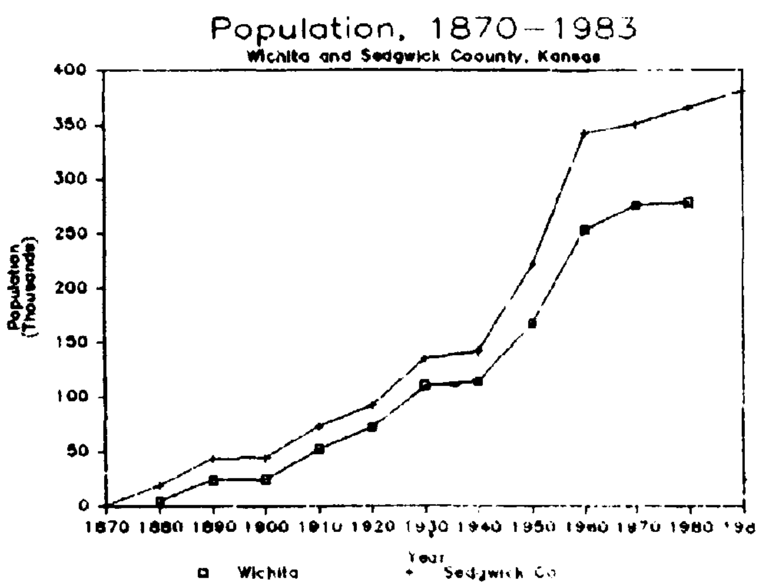


Table II: Centrographic Measures, Population, Sedgwick Co., KA, 1940-1980.

Measures	1940	1950	1960	1970	1980
Total Population	142,999	222,290	343,229	350,694	366,531
Change		79,291	120,939	7,465	15,837
Percent Change		55.45%	54.41%	2.17%	4.52%
Center of Population: X Axis (meters 000s)	645.70	646.75	647.04	646.77	646.56
Center of Population: Y Axis (meters 000s)	4172.63	4171.96	4171.18	4171.06	4171.05
Average Radius (kms.)	5.51	5.41	6.60	7.38	8.27
Change		-0.10	1.19	0.78	0.89
Percent Change		-1.73%	21.90%	11.88%	12.11%
Standard Radius (kms.)	8.80	7.87	8.60	9.35	10.26
Change		-0.93	0.73	0.75	0.90
Percent Change		-10.60%	9.30%	8.75%	9.65%
Coefficient of Circularity	0.81	0.88	0.89	0.90	0.91

records to supply missing data items. Field surveys were also made to clarify ambiguities and verify information. A large scale effort made it possible to determine the number of buildings with considerable accuracy, classify the buildings into primary and secondary (or auxilliary) structures, and categorize them into detailed types by use. Area, height, and year of contruction were other attributes associated with the buildings. The results are summarized in Table III which provides more information about the building stock of a metropolitan area in the United States than has been previously available.

The spatial distribution of the building stock within an urban region is of particular interest. Many of the constructs of urban economics and urban geography are concerned with specifying spatial relationships in metropolitan areas. Centrographic analysis indicates while the number of structures is slightly more dispersed than the population (an average radius of 8.6 kms. vs. 8.3 kms.) the total area of structures is slightly more concentrated (an average radius of 8.2 kms.) as shown in Table IV. Therefore, the larger buildings are closer to the center, but only to an extremely modest degree and not to the extent usually assumed. Analysis of mean area of buildings by annule shows fluctuation from ring to ring that is not systematic. The area of single family homes is more dispersed than the number, but not nearly as much as urban economic rent models would lead one to expect. Annular analysis of mean area shows a tendency for average size to vary with distance but with considerable variation. Mobile homes, an increasingly popular form of low cost housing, are highly dispersed reflecting both their exclusion from central areas and their recent growth. But this indicates the presence of substantial numbers of households with modest incomes at considerable distance from the center instead of close in where many theories suggest they are concentrated.

The Average Radius of industrial buildings by area is larger than that for total population and that for any other building type except mobile homes and agricultural structures. This finding does not conform either to current theoretical constructs or conventional wisdom about the distribution of manufacturing activity. Hospital and college and university buildings have the smallest Average Radius and are the most concentrated types.

Distribution by Rings or Annules

Buildings by number and area by use were analyzed by annules from the Center of Population. The annules were derived from the centrographic measures for greatest generality of application and comparison with other areas. The first annule has a radius 1/4 of the Average Radius, about 2 kms., the second through the seventh annules have a width of 1/2 of the Average Radius, about 1 km., the eighth

Table III. Number & Area Total Bldgs., Sedgwick Co. KA., 1983.

Use of Structure	Number Primary	Number Secondary	Number Total	Area in Sq. Ft. Primary	Area in Sq. Ft. Secondary	Area in Sq. Ft. Total
Total	133,210	114,560	247,770	272,865,477	44,769,088	317,634,565
Residential	120,319	107,404	227,723	163,728,294	38,500,546	202,228,840
Single Fam. & Mob. Homes	118,389	106,878	225,267	146,612,218	37,728,152	184,340,370
Single Family	110,763	106,516	217,279	140,046,280	37,583,424	177,629,704
Mobile Homes	7,626	362	7,988	6,565,938	144,728	6,710,666
Multi-Fam. & Group Qtrs.	1,930	526	2,456	17,116,076	772,394	17,888,470
Non-Residential	12,891	7,156	20,047	109,137,183	6,268,542	115,405,725
Commercial	7,534	2,009	9,543	52,597,989	2,478,061	55,076,050
Industrial	1,326	512	1,838	26,163,996	743,854	26,907,850
Agricultural	2,196	4,305	6,501	3,629,480	2,554,247	6,183,727
Other	1,835	330	2,165	26,745,718	492,380	27,238,098
Government	442	203	645	5,849,484	341,323	6,190,807
Institutional	780	112	892	9,255,899	96,175	9,352,074
Fratern. & Charitable	217	36	253	1,046,531	46,982	1,093,513
Religious	495	72	567	4,569,423	44,263	4,613,686
Hospital	68	4	72	3,639,945	4,930	3,644,875
Education	613	15	628	11,640,335	54,882	11,695,217
School	543	12	555	9,086,519	54,452	9,140,971
College	70	3	73	2,553,816	430	2,554,246

Table IV. Average & Standard Radii: Bldgs. by Use, Sedgwick Co., KA, 1983.

Use of Structure	Number of Structures Average Radius	Number of Structures Standard Radius	Area of Structures Average Radius	Area of Structures Standard Radius
Total Primary Structures	8.55	10.70	8.20	10.32
Residential	8.68	10.53	8.69	10.66
Singl.e Fam. & Mob. Homes	8.50	10.56	8.87	10.90
Single Family	8.30	10.37	8.73	10.77
Mobile Homes	10.11	12.36	10.29	12.56
Multi-Fam. & Group Qtrs.	7.13	8.69	7.09	8.05
Non-Residential	9.28	12.01	7.68	9.80
Commercial	6.77	9.41	6.10	8.46
Industrial	7.80	9.74	8.82	10.25
Agricultural	17.35	18.50	17.87	18.94
Other	8.82	11.43	6.86	9.28
Government	12.09	14.44	7.73	9.96
Institutional	7.63	10.11	4.94	7.05
raterm. & Charitable	6.85	9.29	6.65	8.37
Religious	8.36	10.84	6.30	8.70
Hospital	2.34	2.76	2.35	2.61
Education	7.84	10.35	7.91	10.31
School	8.16	10.86	8.81	11.30
College	3.65	3.92	2.71	3.35
Total Population (1980)	8.27	10.26		
Total Housing Units (1980)	7.71	9.70		
Housing Units Built 1970-1980	10.46	12.06		

and ninth annules have a width of 1/2 Average Radius, and the tenth annule extends from twice the Average Radius to the boundaries of the county. The annules were selected in such a way as to minimize the differences in the number of buildings in each ring while maintaining a simple progression in width. The number of primary buildings by use by annule is shown in Table V and Figure 2. In the study region, 90% of the structures are residential and 10% non-residential, which is quite comparable to findings of previous studies. The percentage composition by these major categories by annules, summarized in Table V and Figure 3, is remarkably constant except for the first and tenth rings. However, the mixtures by use within residential and non-residential categories vary considerably by annule.

The area composition of the building stock is quite different from the building count since non-residential structures tend to be much larger than residential ones. For the study region 60% of the area of primary buildings was in residential use and 40% in non-residential. The variation by annule is shown in Table VI and Figures 4 and 5. Except for the first annule which was dominated by non-residential area and the ninth annule where residential area predominated, the distribution is remarkably constant. Again, there are major differences from ring to ring within the residential and non-residential categories.

Replacement Cost

We have now derived the primary data to permit analysis of value of the building stock by use and by distribution in space. It was decided to estimate replacement cost of buildings since that would reflect cost of repairs of partially damaged structures and restitution of destroyed ones. In other words these estimates provide the basis for approximating recovery costs rather than any other measure of value or loss. Depreciated value of buildings is not considered. Value of contents and losses due to inability to use structures are not calculated. Replacement cost or value of non-building structures or site improvements are not included. Land value is omitted. Only primary structures have been included here, and replacement costs of secondary buildings are not shown.

Construction costs vary by use of building, type of construction and size. The building stock was disaggregated by use and by size class interval. Construction costs per square foot were taken from a standard estimating guide. [Means, 1985] Normally, square footage costs decline with size. Consideration was given to the fact that the use categories, while quite detailed, contain heterogeneous building types. For example, government structures include buildings as different as simple fire stations and imposing court houses. For some building types assumptions were made that quite different kinds of buildings would characterize

Table V. No. of Primary Bldgs. by Use by Annule, Sedgwick Co.,KA,1981.

Use of Structure	First Annule	Second Annule	Third Annule	Fourth Annule	Fifth Annule	Sixth Annule	Seventh Annule	Eighth Annule	Ninth Annule	Tenth Annule	Total
Total	8,181	10,949	15,106	17,120	14,944	10,500	10,896	14,022	15,998	15,494	133,210
Residential	6,126	10,096	13,433	16,030	14,089	9,683	10,091	12,824	14,796	13,151	120,319
Single Fam. & Mob. Homes	5,981	10,007	13,226	15,691	13,860	9,544	9,912	12,465	14,650	13,053	118,389
Single Family	5,890	9,931	13,092	15,333	13,133	8,927	[illegible]	11,138	13,360	11,611	110,751
Mobile Homes	91	76	134	358	727	617	1,564	1,327	1,290	1,442	7,628
Multi-Fam. & Group Qtrs.	145	89	207	339	229	139	179	359	146	98	1,930
Percent Residential	74.88%	92.21%	88.92%	93.63%	94.28%	92.22%	92.61%	91.46%	92.49%	84.88%	90.32%
Non-Residential	2,055	853	1,673	1,090	855	817	805	1,198	1,202	2,343	12,891
Commercial	1,676	666	1,211	756	614	430	450	526	561	644	7,534
Industrial	213	71	166	130	11	261	107	183	34	150	1,326
Agricultural	12	10	14	35	16	45	85	238	459	1,282	2,196
Other	154	106	262	169	214	81	163	251	148	267	1,835
Government	32	11	27	17	8	15	12	111	56	113	442
Institutional	69	50	153	94	90	44	52	86	69	73	780
Fratern. & Charitable	16	7	60	28	5	17	14	50	5	17	217
Religious	49	41	61	[illegible]	61	[illegible]	35	35	64	56	495
Hospital	4	2	32	5	24	0	0	1	0	0	68
Education	53	45	102	58	116	22	59	54	23	81	613
School	53	33	93	56	73	16	55	54	23	81	543
College	0	12	9	2	43	4	[illegible]	[illegible]	0	0	70
Percent Non-Residential	25.12%	7.79%	11.08%	6.37%	5.72%	7.78%	[illegible]	8.54%	7.51%	15.12%	9.68%
Population-Building Ratio	2.58	2.53	2.87	2.90	3.14	2.98	3.15	3.25	2.74	2.43	2.86

Figure 2

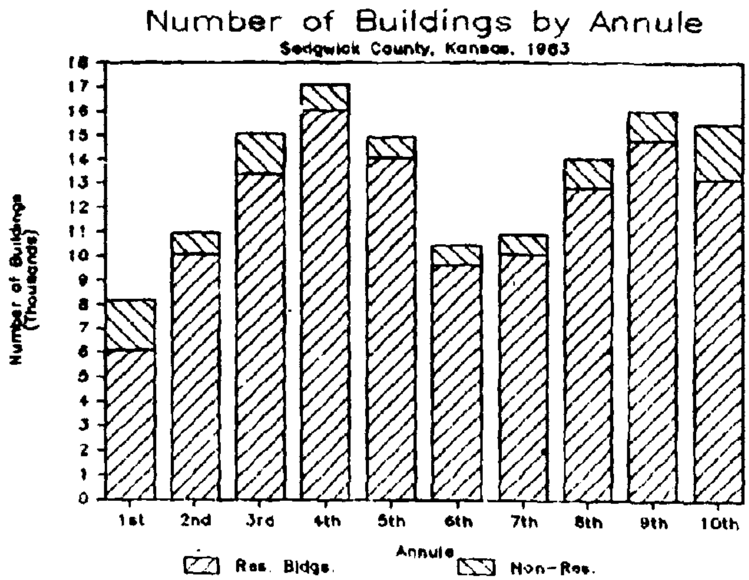


Figure 3

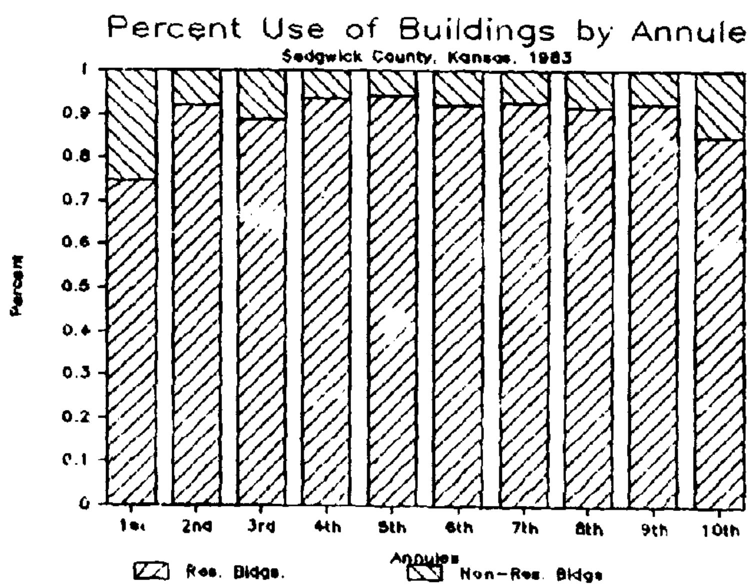


Table VI. Area of Primary Bldgs. by Use by Annulo, Sedgwick Co., KA, 1983.

Use of Structure	First Annulo Sq. Feet	Second Annulo Sq. Feet	Third Annulo Sq. Feet	Fourth Annulo Sq. Feet	Fifth Annulo Sq. Feet	Sixth Annulo Sq. Feet	Seventh Annulo Sq. Feet	Eighth Annulo Sq. Feet	Ninth Annulo Sq. Feet	Tenth Annulo Sq. Feet	Total Region Sq. Feet
Total	20,038,503	16,222,118	29,121,207	30,276,043	26,907,202	22,760,012	23,399,010	40,031,923	24,610,487	31,466,993	272,865,471
Residential	7,633,134	10,489,107	17,372,629	21,196,626	17,704,659	14,033,396	14,331,623	22,333,687	19,889,428	18,832,196	183,728,196
Single Fam. & Mob. Homes	6,159,995	10,003,661	15,833,876	19,096,054	16,169,777	13,220,533	13,435,754	17,267,055	19,358,279	17,998,354	166,612,218
Single Family	6,107,164	10,036,185	15,754,180	18,828,540	15,366,474	11,683,431	11,079,355	16,093,730	18,243,968	16,668,025	140,046,280
Mobile Homes	52,831	67,456	78,696	265,514	803,303	543,102	1,356,199	1,173,327	1,113,311	1,330,329	6,565,936
Multi-Fam. & Group Qtrs.	1,473,139	483,466	1,539,753	2,102,572	1,534,692	1,026,063	1,885,868	4,947,732	530,149	853,843	17,116,076
Percent Residential	37.22%	64.66%	59.66%	70.01%	65.80%	61.72%	61.20%	55.49%	80.83%	59.87%	60.00%
Non-Residential	12,405,369	5,733,111	11,748,578	9,079,417	9,202,813	8,715,616	9,078,186	17,816,236	4,721,059	12,636,796	109,137,183
Commercial	13,800,019	3,260,754	6,006,003	5,254,993	4,903,156	3,677,881	6,126,599	5,453,145	2,374,170	3,670,321	52,597,989
Industrial	1,968,616	763,762	1,689,637	1,829,316	105,849	4,383,277	2,277,256	8,464,035	243,841	4,256,173	26,163,996
Agricultural	6,390	4,263	8,896	61,750	6,630	64,179	110,721	283,803	733,650	2,366,210	3,629,488
Other	4,560,344	1,704,350	4,063,226	1,932,164	4,186,770	810,339	2,563,813	3,635,253	1,187,370	2,344,092	26,765,718
Government	1,676,919	57,876	293,240	75,453	131,356	97,167	733,836	2,037,259	161,007	389,963	5,649,404
Institutional	2,169,430	1,166,060	1,615,115	1,044,334	672,228	387,312	547,563	672,796	313,329	662,915	9,255,899
Fratern. & Charitable	193,488	30,306	151,625	116,307	19,590	135,169	70,332	201,271	13,720	112,323	1,046,531
Religious	1,163,279	298,563	494,158	472,842	370,882	252,043	477,030	390,615	299,609	336,592	4,569,423
Hospital	810,672	838,000	1,273,332	434,985	281,756	0	0	81,000	0	0	3,639,965
Education	713,906	480,405	1,930,063	812,377	3,380,986	325,860	1,305,014	785,190	692,634	1,291,212	11,640,333
School	713,906	135,621	1,657,698	795,697	1,405,535	282,126	1,305,014	785,190	692,634	1,291,212	9,086,539
College	0	344,784	273,365	16,680	1,975,451	43,736	0	0	0	0	2,553,826
Percent Non-Residential	62.78%	35.34%	40.34%	29.99%	34.20%	38.28%	38.80%	44.51%	19.16%	40.13%	40.00%

Figure 4

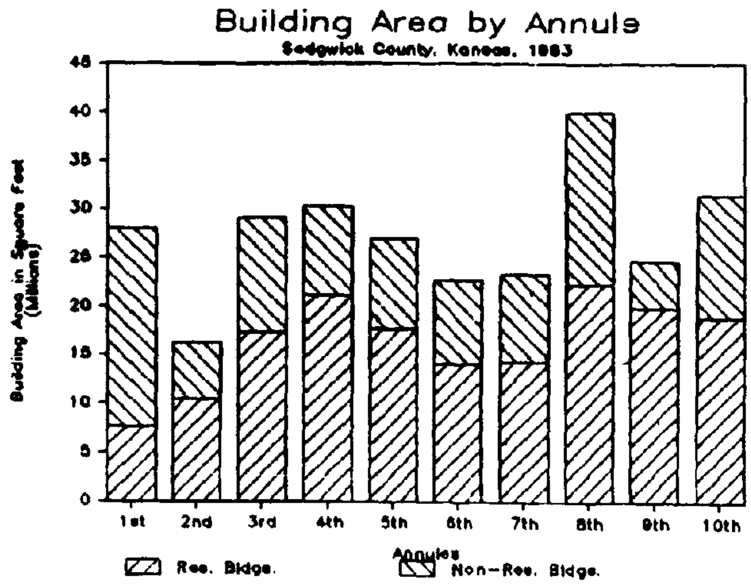
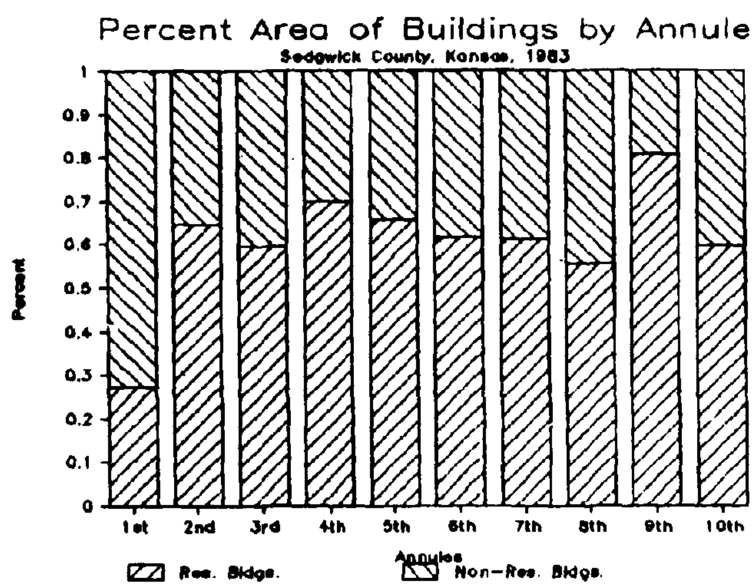


Figure 5



different size classes. For that reason the square foot costs do not decline monotonically within each use category. In each case the type of construction selected was neither the most nor least expensive. The estimates are shown in Table VII. The total replacement cost of the 1983 building stock in Wichita is estimated at $13,615,000,000, or approximately $35,700 per person. Of the total 54% is for residential and 46% for non-residential structures.

Because of the considerable variation of uses within the major categories by annule, replacement costs vary more from one ring to another than other measures as shown in Table VIII and Figure 6. However, the replacement cost by residential and non-residential use is relatively constant from one ring to another except for the first annule where non-residential buildings represent an overwhelming percentage of the total cost and the ninth annule where residential costs predominate (Figure 7).

Density Distributions

It is extremely useful and illuminating to analyze spatial distributions in standardized terms. The most common means of doing this is by expressing variables in terms of units of area by distance from some reference point. Strong empirical regularities have been found from place to place and from one point in time to another through such analysis. Values per square mile were calculated for annules outward from the Center of Population for resident population, and number, area and replacement cost for total, residential and non-residential buildings. The results are shown in Table IX and demonstrate for most series the expected monotonic declines outward from the center. Population and buildings per square mile are shown in Figure 8. For 35 years we have known that the decline in population density outward from the center is well described by a negative exponential function. [Clark, 1951] The log of population and building density are plotted against distance in Figure 9. The resulting lines are reasonably close to the straight ones we would get if the negative exponential fitted precisely.

Building area per square mile is plotted in Figure 10, and the log of the value in Figure 11. Building area conforms even better than number of buildings to the negative exponential function. One of the findings graphically portrayed in this diagram is that non-residential building area conforms quite well to the model. If we assume constant area per worker across the annules as a result of the interaction of many variables, this would indicate employment is negative exponentially distributed. We have had little evidence of the spatial distribution of employment previously, but centrographic studies have indicated similar degrees of dispersion as found here. [Jones and Manson, 1982] While the number of square feet per worker seems to be somewhere between 400 and 500, there is tremendous variation

Table VII. Replacement Cost, Buildings by Use, Sedgwick Co., KS, 1981.

Area Class Interval	Total Residential Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	Single Family Residences No.	Area	Sq. Ft. Cost	Replacement Cost	Mobile Homes No.	Area	Sq. Ft. Cost	Replacement Cost	Multi-Family Dwellings No.	Area	Sq. Ft. Cost	Replacement Cost	Group Quarters No.	Area	Sq. Ft. Cost	Replacement Cost
0-1,000	47,100	800	$38.84	$1,463,362,800	40,664	800	$41.75	$1,358,177,600	6,363	800	$20.70	$101,808,000	61	800	$56.40	$2,752,320	12	800	$65.10	$624,960
1-2,000	62,167	1,500	$38.33	$3,568,690,830	60,355	1,500	$38.50	$3,485,385,750	1,254	1,500	$18.00	$33,858,000	554	1,500	$56.40	$46,868,400	6	1,500	$65.10	$585,900
2-4,000	10,132	3,000	$43.04	$1,308,373,430	9,369	3,000	$41.95	$1,179,086,850	8	3,000	$11.00	$180,000	707	3,000	$56.40	$119,624,400	48	3,000	$65.10	$9,374,400
4-6,000	438	5,000	$45.34	$99,735,000	106	5,000	$40.00	$62,424,000		5,000			130	5,000	$56.40	$36,660,000	2	5,000	$65.10	$651,000
6-10,000	187	8,000	$54.00	$80,900,000	65	8,000	$49.60	$25,792,000		8,000			121	8,000	$56.40	$54,595,200	1	8,000	$65.10	$520,800
10-15,000	95	12,500	$53.23	$63,208,750	6	12,500	$53.45	$2,671,300		12,500			46	12,500	$49.20	$41,820,000	23	12,500	$65.10	$18,716,250
15-25,000	69	20,000	$49.43	$88,320,000		20,000		$0		20,000			37	20,000	$46.40	$52,096,000	12	20,000	$65.10	$15,624,000
25-50,000	40	37,500	$54.47	$56,840,000		37,500				37,500			44	37,500	$54.00	$89,100,000	4	37,500	$59.60	$8,940,000
50-100,000	52	75,000	$51.55	$201,045,000		75,000				75,000			51	75,000	$51.55	$201,045,000	0	75,000	$56.70	$0
100-500,000	28	250,000	$51.61	$361,287,500		250,000				250,000			27	250,000	$51.53	$347,962,500	1	250,000	$53.30	$13,325,000
500-1,000,000	0	750,000		$0		750,000				750,000			0	750,000		$0	0	750,000		$0
	120,316			$7,311,100,630	110,761			$6,113,540,300	7,625			$135,956,000	1,821			$993,323,820	109			$68,362,310

Area Class Interval	Total Non-Residential No.	Area	Sq. Ft. Cost	Replacement Cost	Commercial Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	Industrial Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	Agricultural Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	Government Buildings No.	Area	Sq. Ft. Cost	Replacement Cost
0-1,000	2,822	800	$53.92	$121,729,360	1,199	800	$56.40	$53,863,200	109	800	$51.55	$4,495,160	1,078	800	$51.55	$44,456,720	38	800	$59.75	$1,816,400
1-2,000	1,479	1,500	$58.31	$194,504,325	1,551	1,500	$50.45	$117,371,925	141	1,500	$51.55	$10,902,825	556	1,500	$51.55	$42,993,700	91	1,500	$59.75	$8,155,875
2-4,000	2,765	3,000	$50.41	$418,137,450	1,833	3,000	$47.65	$264,886,330	223	3,000	$48.40	$32,379,600	399	3,000	$48.40	$57,934,000	84	3,000	$59.75	$15,057,000
4-6,000	1,402	5,000	$49.44	$346,552,300	1,011	5,000	$47.00	$237,810,000	159	5,000	$45.13	$35,896,250	90	5,000	$45.13	$20,317,500	50	5,000	$59.75	$14,937,500
6-10,000	1,374	8,000	$48.90	$315,517,200	915	8,000	$43.35	$318,362,400	204	8,000	$42.40	$69,325,200	36	8,000	$42.40	$11,884,000	61	8,000	$54.00	$26,742,400
10-15,000	840	12,500	$45.13	$361,022,500	379	12,500	$40.40	$192,342,500	134	12,500	$41.65	$69,765,750	7	12,500	$41.20	$3,605,000	36	12,500	$55.75	$25,093,750
15-25,000	631	20,000	$49.24	$423,335,000	334	20,000	$49.40	$329,992,000	134	20,000	$39.00	$104,520,000	7	20,000	$39.55	$5,537,000	38	20,000	$52.43	$39,842,800
25-50,000	421	37,500	$60.41	$169,295,625	161	37,500	$46.00	$282,555,000	111	37,500	$55.05	$169,225,625	2	37,500	$38.55	$2,891,250	25	37,500	$67.55	$61,070,115
50-100,000	181	75,000	$44.47	$603,661,250	89	75,000	$41.50	$216,762,500	55	75,000	$34.00	$140,250,000		75,000		$0	6	75,000	$80.30	$36,135,000
100-500,000	134	250,000	$53.46	$1,685,725,000	61	250,000	$67.75	$1,042,375,000	53	250,000	$38.05	$504,162,500		250,000		$0	9	250,000	$67.25	$151,312,500
500-1,000,000	4	750,000	$38.79	$160,300,000	1	750,000	$67.15	$50,437,500	2	750,000	$38.05	$57,075,000		750,000		$0	1	750,000	$67.25	$50,437,500
	12,880			$6,303,780,010	7,550			$3,104,780,375	1,323			$1,178,191,910	2,195			$196,819,770	437			$430,238,050

Area Class Interval	Fraternal & Charitable Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	Religious Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	Hospital Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	School Buildings No.	Area	Sq. Ft. Cost	Replacement Cost	College & University Buildings No.	Area	Sq. Ft. Cost	Replacement Cost
0-1,000	64	800	$47.00	$2,406,400	33	800	$70.40	$1,661,360	8	833	$82.65	$320,960	309	800	$69.03	$12,321,920	3	800	$65.10	$156,240
1-2,000	46	1,500	$47.00	$3,243,000	74	1,500	$90.40	$10,303,600	1	1,500	$82.65	$123,975	21	1,500	$69.03	$822,323	6	1,500	$65.10	$585,900
2-4,000	49	3,000	$47.00	$6,909,000	135	3,000	$90.40	$36,611,000	9	3,000	$82.65	$2,231,550	9	3,000	$69.03	$1,345,930	4	3,000	$65.10	$781,200
4-6,000	15	5,000	$45.10	$3,382,500	57	5,000	$99.13	$28,251,050	6	5,000	$82.65	$2,479,500	10	5,000	$69.03	$2,491,300	3	5,000	$65.10	$976,500
6-10,000	15	8,000	$62.20	$7,464,000	85	8,000	$84.15	$57,221,000	6	8,000	$82.65	$3,967,200	16	8,000	$69.03	$6,389,000	13	8,000	$65.10	$6,770,400
10-15,000	8	12,500	$59.95	$5,995,000	47	12,500	$75.00	$44,331,500	2	12,500	$82.65	$2,066,250	24	12,500	$49.03	$14,931,000	5	12,500	$65.10	$4,068,750
15-25,000	12	20,000	$55.80	$12,276,000	41	20,000	$60.00	$55,780,000	10	20,000	$81.65	$16,330,000	34	20,000	$69.03	$53,038,000	4	20,000	$63.00	$5,040,000
25-50,000	8	37,500	$53.15	$16,005,000	53	37,500	$60.40	$52,095,000	10	37,500	$79.40	$29,850,000	67	37,500	$47.03	$219,720,675	15	37,500	$62.00	$34,875,000
50-100,000	1	75,000	$50.00	$3,750,000	5	75,000	$55.00	$20,625,000	10	75,000	$76.05	$57,037,500	25	75,000	$46.85	$87,843,750	10	75,000	$57.65	$63,137,500
100-500,000		250,000		$0	3	250,000	$50.00	$37,500,000	4	250,000	$71.15	$71,250,000	18	250,000	$41.30	$283,830,000	7	250,000	$53.30	$93,275,000
500-1,000,000		750,000		$0		750,000		$0	2	750,000	$70.90	$106,350,000		750,000		$0		750,000		$0
	217			$61,430,900	495			$344,567,510	68			$292,414,935	543			$683,372,070	70			$169,786,690

Table VIII. Replacement Cost, Bldgs. by Annulo, Sedgwick Co., KS, 1983.

Use of Structure	Total Replacement Cost	First Annulo Replacement Cost	Second Annulo Replacement Cost	Third Annulo Replacement Cost	Fourth Annulo Replacement Cost
Total	$13,614,960,640	$1,611,424,467	$844,003,607	$1,520,424,126	$1,507,020,360
Residential	$7,311,180,630	$356,407,711	$477,634,366	$792,633,014	$968,427,108
Single Family	$6,113,340,500	$263,326,611	$449,684,796	$608,102,939	$822,383,272
Mobile Homes	$135,954,000	$1,104,034	$992,086	$1,445,170	$5,550,613
Multi-Fam.& Group Qtrs.	$1,061,686,130	$91,977,066	$26,959,464	$182,085,704	$140,493,163
Percent Residential	53.70%	22.10%	56.59%	51.86%	64.26%
Non-Residential	$6,303,780,010	$1,256,006,956	$366,367,263	$735,790,212	$538,593,252
Commercial	$3,104,700,375	$883,734,044	$203,312,624	$376,644,056	$315,649,480
Industrial	$1,170,191,910	$95,915,050	$36,578,760	$82,476,138	$96,053,137
Agricultural	$196,819,770	$353,446	$334,001	$491,948	$3,470,850
Government	$450,220,050	$169,326,535	$4,278,965	$20,033,380	$5,524,686
Frat. & Charitable	$61,630,900	$13,462,430	$1,370,072	$8,175,416	$6,035,913
Religious	$344,567,310	$73,649,510	$23,391,990	$38,716,302	$37,046,504
Hospital	$291,614,935	$61,195,226	$63,570,412	$99,456,722	$36,040,239
School	$483,572,070	$38,369,097	$11,624,998	$88,836,516	$42,641,633
College & Univ.	$189,766,490	$0	$18,902,633	$20,759,433	$1,330,993
Percent Non-Residential	46.30%	77.90%	43.41%	48.14%	35.74%

Use of Structure	Fifth Annulo Replacement Cost	Sixth Annulo Replacement Cost	Seventh Annulo Replacement Cost	Eighth Annulo Replacement Cost	Ninth Annulo Replacement Cost	Tenth Annulo Replacement Cost
Total	$1,389,883,206	$1,091,899,998	$1,125,848,511	$1,886,310,391	$1,135,720,366	$1,492,385,409
Residential	$812,938,219	$628,114,342	$620,461,230	$999,180,711	$845,419,618	$801,042,634
Single Family	$696,641,932	$510,390,236	$483,927,096	$693,878,907	$786,720,836	$718,682,854
Mobile Homes	$12,112,286	$11,353,758	$18,331,867	$23,936,159	$23,274,199	$27,115,769
Multi-Fam.& Group Qtrs.	$103,884,[illegible]	$106,370,367	$116,182,266	$281,366,646	$35,424,300	$56,143,012
Percent Residential	58.49%	57.52%	55.03%	52.97%	74.44%	53.70%
Non-Residential	$574,946,987	$463,805,656	$497,307,282	$887,110,679	$290,306,069	$691,302,775
Commercial	$281,720,492	$201,841,131	$231,343,993	$302,075,343	$181,494,172	$219,362,830
Industrial	$5,110,150	$285,881,996	$104,883,762	$336,567,907	$12,274,791	$189,567,357
Agricultural	$488,031	$2,443,444	$6,124,239	$13,487,811	$40,560,843	$116,943,335
Government	$8,674,456	$6,897,390	$40,379,102	$132,224,437	$11,847,112	$43,195,790
Frat. & Charitable	$1,035,138	$9,665,027	$3,634,872	$10,640,475	$740,741	$6,451,387
Religious	$32,459,416	$19,747,214	$33,646,034	$27,333,951	$27,632,667	$30,683,643
Hospital	$24,793,747	$0	$0	$6,320,308	$0	$0
School	$73,323,018	$13,119,107	$69,936,079	$37,781,745	$35,730,621	$40,196,413
College & Univ.	$143,283,484	$3,489,947	$0	$0	$0	$0
Percent Non-Residential	41.51%	42.48%	44.18%	47.03%	25.56%	46.30%

Figure 6

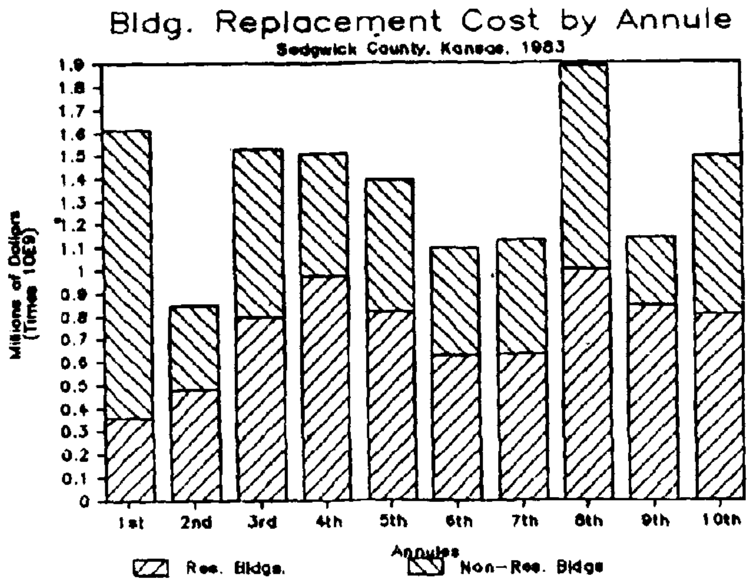


Figure 7

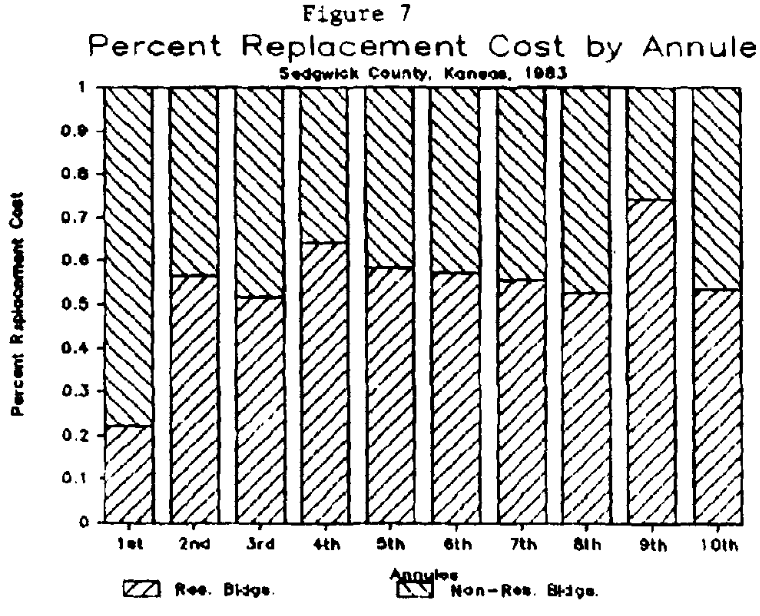


Table IX. Density by Annuli: Pop., Bldgs, Area, Cost, Sedgwick Co. KA, 1983.

Ann.	Area Sq. Mi.	Pop.	Total Bldgs.	Res. Bldgs.	Non-Res. Bldgs.	Pop. per Sq.Mi.	Bldgs. per Sq.Mi.	Resid. Bldgs./ Sq.Mi.	Non-Res. Bldgs./ Sq.Mi.
1	5.19	20,199	8,181	6,126	2,055	3,892	1,576	1,180	396
2	6.49	26,457	10,949	10,096	853	4,078	1,688	1,556	131
3	9.08	41,389	15,106	13,433	1,673	4,557	1,663	1,479	184
4	11.68	47,443	17,120	16,030	1,090	4,063	1,466	1,373	93
5	14.27	44,816	14,944	14,089	855	3,140	1,047	987	60
6	16.87	29,857	10,500	9,683	817	1,770	622	574	48
7	19 46	32,768	10,896	10,091	805	1,683	560	518	41
8	103.81	43,541	14,022	12,824	1,198	419	135	124	12
9	145.33	41,863	15,998	14,796	1,202	288	110	102	8
10	674.82	35,949	15,494	13,151	2,343	53	23	19	3
	1,007.00	364,262	133,210	120,319	12,891	362	132	119	13

Ann.	Total Bldg.Area per Sq.Mi.	Resid. Bldg.Area per Sq.Mi.	Non-Res. Bldg.Area per Sq.Mi.	Tot. Bldg. Repl. Cost per Sq.Mi.	Res. Bldg. Repl. Cost per Sq.Mi.	Non-Res.Bldg Repl. Cost per Sq.Mi.
1	5,402,134	1,470,664	3,931,470	$310,661,362	$68,668,505	$241,992,857
2	2,500,407	1,616,736	883,671	$130,090,263	$73,620,346	$56,469,917
3	3,206,135	1,912,661	1,293,474	$168,273,736	$87,266,007	$81,007,730
4	2,592,550	1,815,076	777,474	$129,046,760	$82,926,803	$46,119,957
5	1,885,157	1,240,397	644,760	$97,377,079	$56,955,458	$40,421,620
6	1,349,806	833,122	516,684	$64,736,584	$37,236,243	$27,500,341
7	1,202,241	735,820	466,421	$57,844,117	$32,289,233	$25,554,884
8	385,644	214,013	171,631	$18,171,621	$9,625,711	$8,545,910
9	169,345	136,859	32,486	$7,814,963	$5,817,343	$1,997,620
10	46,663	27,937	18,726	$2,212,914	$1,188,375	$1,024,539
	270,969	162,590	108,379	$13,520,318	$7,260,358	$6,259,960

Figure 8

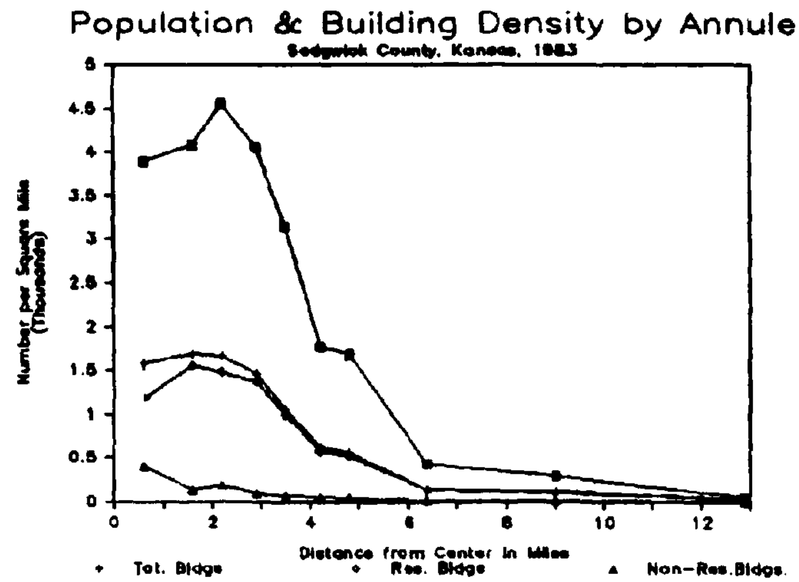


Figure 9

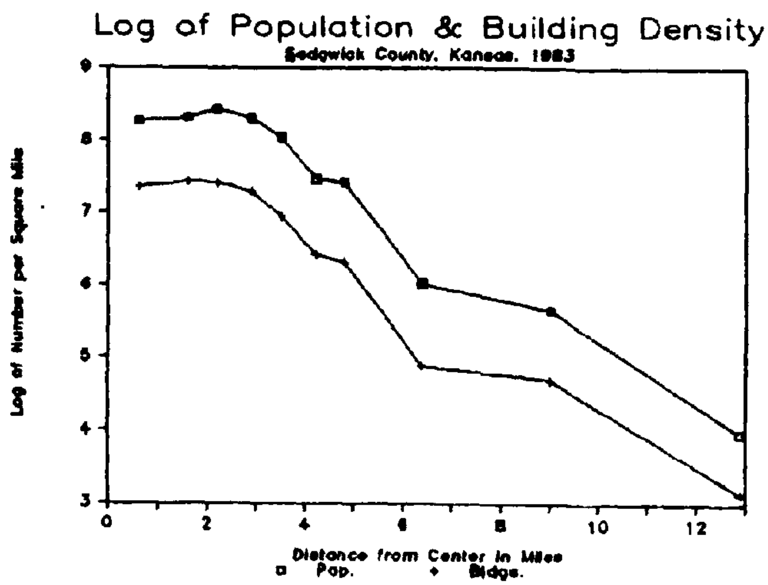


Figure 10

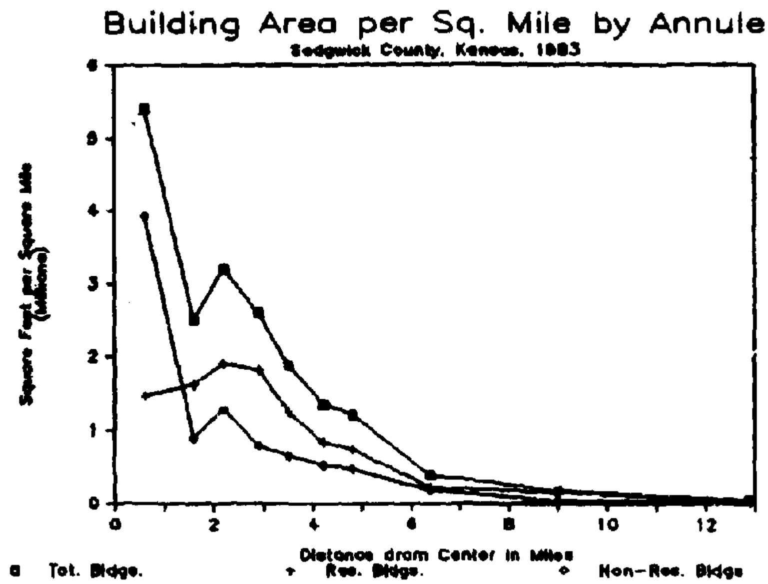
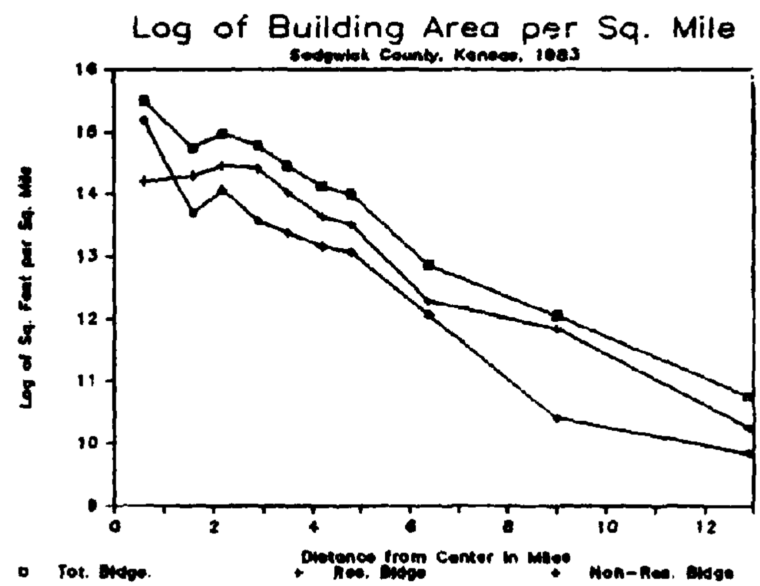


Figure 11



from one use to another and even within uses. Church buildings, grain elevators, and petroleum tank farms have low worker densities. There is a vast range in area per worker from one type of retail activity to another. Central city hotels have lower worker densities than nearby office buildings. In the absence of better information, the evidence suggests a negative exponential function reasonably describes the distribution of employment density in a metropolitan area. Admittedly, this hypothesis requires further investigation. Traditional competing models indicating the central concentration of employment and conversely the sub-urbanization of jobs are not supported. The dispersion of employment activities seems to conform to general patterns in the dispersion of metropolitan areas.

Replacement cost of buildings per square mile is also reasonably approximated by the negative exponential as shown in Figures 12 and 13. The values are averages and mask a great deal of important detail. However, they provide a useful basis for making loss and damage estimates for sub-areas within an urban region. Residential costs exceed non-residential ones for every annule except the first.

CONCLUSIONS

Careful enumerations of building stocks and their characteristics such as size and use provide a basis for studying the morphology of the structures which make up urban regions. Preliminary analysis indicates empirical regularities characterize their distributions over categories and by categories over space. If these relationships are found to be general ones, the task of making indirect estimates of existing buildings in areas of interest will be greatly facilitated. Better replacement cost estimates for elements at risk from earthquakes and other natural disasters can be made. Determining risk or the probable loss will be made earier. Urban vulnerability analysis can be simplified and the results of such analysis more readily used in establishing appropriate mitigation measures.

Figure 12

Replacement Cost per Sq. Mile by Annule

Sedgwick County, Kansas, 1983

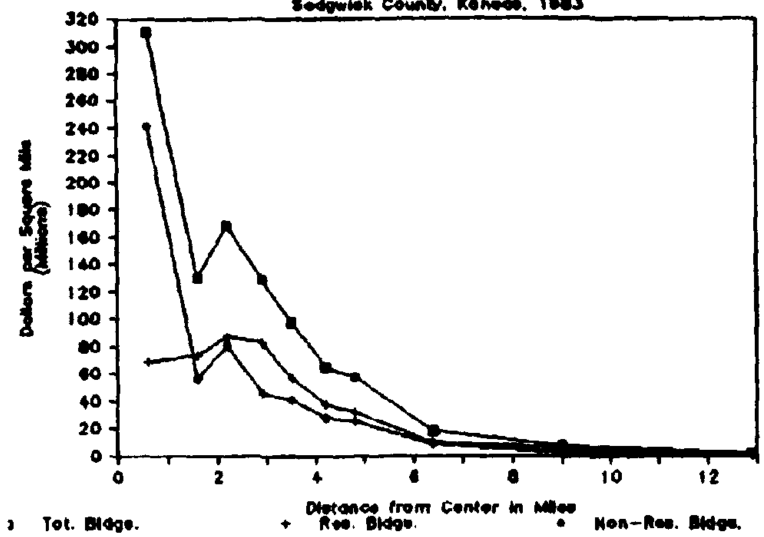
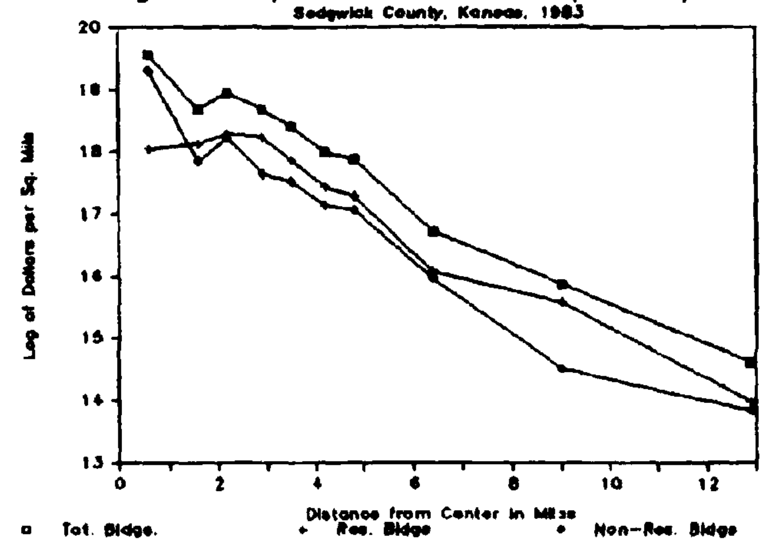


Figure 13

Log of Replacement Cost per Sq. Mile

Sedgwick County, Kansas, 1983



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II.2 ROMANIAN TECHNOLOGY FOR STRENGTHENING REINFORCED CONCRETE STRUCTURES BY SHEATHING WITH FIBERGLASS TISSUES GLUED WITH EPOXY RESINS

Dr.eng.Romulus Constantinescu[x]

The March 4, 1977 Vrancea earthquake posed great problems concerning the repairing and strengthening of damaged buildings. The structure of residential buildings, hotels, commercial buildings, schools as well as hospitals were designed according to various design solutions for handling seismic loads, such as : masonry bearing walls, monolith reinforced concrete and precast frames structures and reinforced concrete shear walls. The degree of damage to these structures was different according to the construction type and the action of the seismic load.In the case of reinforced concrete frame structures, with special reference to girders, damage occurred especially in the supporting zones where the shearing force caused cracks or specific failures.

INCERC had to provide immediate strengthening solutionsand techniques according to the kind and degree of damage.

This paper deals with an original method provided and used by INCERC to strengthen the reinforced concrete buildings by sheathing them with fiberglass tissues glued with epoxy resins. This method can also be used for strengthening beams damaged due to shearing forces.

The experiemnts sought to finalize the technological strengthening procedures and to control the efficient use of this method with a view to recover the initial bearing capacity of the damaged members by strengthening.

A large rumber of static tests were performed on full scale beams and reinforced concrete shear walls models initially located up to failure (a loading similar to that caused by the earthquake). Afterwards the element is repaired and then loaded to a new failure in order to obtain comparable results.

At the same time, a thorough analysis has been performed on the efficiency of the strengthening solution in the recovery ofthe member-bearing capacity in accordance with the conditions of failure under initial loading. Special attention was paid

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to zones damaged by predominant stresses due to shear forces.

During the first stage, three beams G_1, G_2 and G_3, all having the length of 4 m, the T-shaped cross section of 25x50cm, the slab width of 125 cm and the slab thickness of 10 cm, were tested.

According to the initial estimates, G_1 would fail at a P overall loading of about 42 tf, at shearing force, G_2 would fail at a P overall load of about 40 tf and at a bending moment due to the better quality of concrete and due to the low quality of longitudinal reinforcement; beam G_3 fail at shearing force at a loading of P_{tot} = 55 tf, as a result of the fact that during the test of this beam efforts were transmitted towards its ends.

During the tests of this first stage the average resistance of concrete cubes - corresponding to each beam was as follows:

- G_1 beam - 160 Kgf/cm^2
- G_2 beam - 170 kgf/cm^2
- G_3 beam - 195 kgf/cm^2

During the two testing stages all the beams were simply supported.

After failure the beams were strengthened and the second test was conducted under the conditions of the first one. The strengthening followed a different solution for every beam.

G_1 beam: The beam strengthening was performed by covering with reinforced concrete during two successive stages:
- the internal strengthening of the beam was done by injection the cracks which occurred after failure at loading during the first stage with epoxy resins.

- the external strengthening of the beam was done by covering it with reinforced concrete; then the two lateral sides of the beam web were roughed (bush hammered) of concrete and cleaned and finally a primar coating of epoxy putty layer made of epoxy resins, hardener, and cement was brushed on.

Soon after brushing on the epoxy lute layer the covering reinforcement was mounted on both sides of the beam web using spacing means, and then the shuttering was tightened by means of a polyethylene leaf. The shuttering was fixed on the beam with two steel bars introduced in holes already existing in the beam web. The steel bars were provided at the ends with a screw thread and a screw nut for fixing the shuttering on the exterior side of the beam, and with plastic pipes in the coating area. After mounting the shuttering, the covering was performed by casting concrete manually from the upper side of the shuttering with a space of 5 cm height left along the whole length of the beam. Initially the concreting was done by pumping but during the preliminary tests the results were not those anticipated.

The concrete compaction was performed with interior vibrating devices. After concreting, the upper side of the covering coat in the unshuttered zone was filled with concrete. The whole concreting work, including the mounting of shuttering, was done in a very short period of time because after no more than 30 minutes the epoxy resin of the primar coating layer is polymerized, meaning a considerable decrease of the adherence because of the formation of a glassy surface. For providing a better adherence at coating, viscous concrete was used.

During testing the resistance per cube in the concrete coating of the strengthened beam was of 539 Kgf/cm^2.

It has been established by calculation that if the jac - keting and steel work fully, an increase of 70 % can be obtained in the bending capacity and another increase of 16 % can also be obtained in the shearing force capacity. In such conditions the failure should occur by bending moment at a total force of about 71-72 tf.

G_2 beam: The beam was strengthened by coating it with a fibre glass tissue stuck on an epoxy resin. The coating was done with fibreglass tissue anchored with acronal and with resisting fibres along one direction.

The strengthening was performed in two stages:

- interior strengthening of the beam was accomplished by injecting epoxy resins into the crack openings over 0.5mm which occurred at failure during the testing of the first stage;
- exterior strengthening of the beam was done by coating it with a fibre glass tissue stuck on the epoxy resin.

During the second stage the two sides of the beam web were brushed with a wire brush and cleaned with compressed air, afterwards applying the first epoxy lute layer. Then, a fibre glass tissue previously cut to adequate dimensions(and with cross and horizontal resistance fibres) was applied over the lute. The tissue was applied by means of a steel cylindrical wheel moving along the two directions for a better penetration of the epoxidic lute. The second layer of epoxidic lute and fibreglass tissue was applied according to a similar technology.

Finally, three layers of fibreglass tissue were applied on each side of the web. On the last tissue layer one more layer of epoxidic lute pressed with a steel cylindrical wheel was applied. The covering coating was applied along the whole length of the G_2 beam.

After applying this covering coat the calculated bearing capacity was increased by 20 tf at shearing force.

Due to the fact that the tissue was applied with the resisting fibres along a cross direction, there was no increase in the bearing capacity at flexure after coating. The bearing capacity was exclusively recovered by injecting epoxy resins into the cracks.

G_3 beam - The strengthening of the beam was performed by coating it with fiberglass epoxy resin tissue. This strengthening method was similar but not identical with the one used for the G_2 beam. The difference between these two methods are as follows:

- use of a fibreglass tissue with roving fibres, the resistance fibres being orthogonally placed along the two directions of the plane;
- the coating was performed by applying discontinuous strips on the longitudinal line of the beam and in zones of maximum shear forces at both ends of the beam (trying to avoid the possible reductions of the post-elastic strains when coating is continuous); there were three layers on each side of the beam web.

After coating, the bearing capacity at minimum shear force was increased by 20 tf. The bearing capacity was exclusively recovered by injecting epoxy resins into the cross.

The beams were tested before and after strengthening and the following could be observed:

- cracking condition (occurrence, development and crack openings at various stages);
- strains (thread comparators in openings, in the middle and near the transmission force points, link comparator on supports);
- the overall behavior, possible disturbance and local damages;
- bearing capacity and method of cracking.

The tests results of the three beams may be synthetized as follows:

G_1 beam - During the two tests no cracks or initial damages occurred due to casting or handling.

During the first test (before strengthening), the beam cracked under a load of P = 10,600 Kgf between the points where forces were applied, with a maximum crack opening of 0.10 mm (mean value 0.065 mm).

In proportion to the increase of load, the cracks increased in number towards supports and enlarged in openings up to failure in the plate.

The beam failed at a total load of P = 40.600 Kgf after an inclined crack had occurred in the maximum shear force zone; at breaking, the opneing of this crack was 12 mm.

During the second stage (after strengthening), the covering coat cracked on both sides of the beam web at a load of P=2.600 Kgf in an area close to the extremities of the beam (including zones of maximum shear force); the minimum opening of the cracks at this stage was of 0.08 mm (mean value 0.05lmm) The heavier the load is the larger the number of cracks (on supports) and their openings will be up to failure at the upper side of the plate.

The failure of the coated beam occurred at a total loading of P = 60,000 Kgf when uncovering the web (on both sides) and as a result of breaking old concrete (concrete of a low quality as compared with that one used for coating) and at the same time with the yielding of the longitudinal reinforcement anchorage at the same extremities.

After uncovering, no interaction was seen between the coat and the beam and the latter was broken due to the shearing force along an inclined crack that occurred in the plate, too. The maximum deflection at breaking was of 14.80 mm; than means 1 : 260 out of the calculation span.

G_2 beam - The beam had no cracks or initial degradation at casting or handling. During the first stage (before the strengthening), the beam cracked at a load P = 10.600 Kgf between the points were forces are applied, with a maximum crack span of 0.02 mm.

With the increase in load, the cracks increased in number (towards supports) and enlarged in openings. The beam failed at a total load P= 54.600 Kgf; after the occurrence at failure of a normal crack in the middle of the beam; the crack opening was of 2.50 mm.

During the second stage (after strengthening) the coat cracked at a load P = 37.600 Kgf on both sides in the central zone of the beam; the maximum opening of the crack was 0.10 mm and with the increase of the load, the normal cracks increased in number (towards supports) and enlarged in openings up to failure at the upper side of the plate.

The strengthened beam failed at a total load P=58.6000 Kgf when a normal crack occurred in the middle of the beam and closer to one of the points where the forces are applied; the opening of the crack after failure was of 3 mm.

In zones of maximum shear force and close to the supports there was no damage in the beam or in the fibreglass epoxy resins tissue coating.

Under a similar load, beam G_2 was broken (before strengthening) at a bending moment, not at a shearing force as beam G_1 did, and this is due in the main to the high quality of concrete.

It is possible that the rigidity of the supports should have had a bad influence on the behavior of the beam at a shearing moment. It is a noteworthy fact that both before strengthening and after it, the beam broke at a bending moment. After strengthening the additional at breaking was unsignificant.

The test proved the efficiency of strengthening in the case of beams damaged at a bending moment and cracks injected with epoxy resins. The coat made of tissue impregnated with epoxy resins, increased the rigidity of the beam, its deformations being in general very low in comparison with the uncovered beam.

G_3 beam - This had no cracks or initial damages at casting or handling. As concerns the analysis on the efficiency of the covering coat made of fibre glass tissue and used for strengthening those parts of the beam damaged by shearing force, a certain laoding scheme was chosen that may cause the breaking of the beam under shearing force (for this purpose the application points of loads were close to the supports).

During the first stage (before strengthening), the beam cracked at a load of P = 7,600 Kgf between the application points of forces, and with a maximum opening of the crack of 0.08 mm. At the same time, with the increase of the load, the normal cracks increased in number (towards supports) and enlarged their openings. At a total loading of 45,000 Kgf, an inclined crack occurred in the zone of maximum shear force close to one of the supports.

The beam was broken at a total load P= 64,400Kgf after an inclined crack occurred between the application point of a force and the adjacent support (the crack occurred in the plate, too); the breaking crack had occurred before. At breaking, the opening of the inclined crack was of 2.7 mm and the normal crack with the maximum opening in the beam was of 2.2mm.

During the second stage (after strengthening) the beam recracked at a load p = 20.000 Kgf in the central zone of the beam; the maximum opening of the crack was 0.12 mm and when the loading increased the cracks increased in number (towards the supports) and enlarged in openings.

The failure of the strengthened beam occurred at a total load of P = 86,600 Kgf in the central part of the beam after a normal crack occurred in the middle of the beam; the opening of this crack was of 0.65 mm at breaking. The covered zones had no damages in the area of maximum shear force.

As compared with the growth of the bearing capacity that should be obtained by coating (and based on calculations), the real growth was superior, although smaller than that one adequate for a load of 40 Kgf/cm obtained on samples, and as it stands, the break from the first stage somehow damaged the initial resisting capacity. It is worth mentioning that the value of the bearing capacity after strengthening is limited by the concrete resistance in the beam, respectively by its capacity to ensure the link between the non-strengthened element and the covering layer.

The results of the tests showed the efficiency of the strengthening method in beam zones damaged by the shearing force occurred at sheating with fiberglass tissue covered with epoxy resins. The advantage of this strengthening method are as follows:

- low consumption of manual labor, as compared to those required by other strengthening methods obtained by removing the hammering of concrete surface, the concrete forming, reinforcement and concrete casting in narrow spaces;
- the short execution time, because execution is easy and simple;
- avoidance of section thickening in strengthened zones and therefore to observe the initial service and aaesthetical conditions as well as the inconvenience generated by changing the stiffness of strengthened units.

We may conclude that by using fibreglass tissues embedded in epoxy resin in strengthening, the bearing capacity of section under predominant stresses due to shear force may be restored or even improved by including a corresponding number of embedded tissue layers determined by calculus according to "C 184 - 77 Technical Prescriptions".

The efficiency of a large number of layers is limited by the quality of concrete in the units to be plated namely by concrete ultimate stress.

ASPECTS FROM THE TESTING OF G_1 BEAM BEFORE STRENGTHENING

PHOTO 1

Overall view of the beam at failure

PHOTO 2

Detail of the end where the failure occur

ASPECTS FROM THE TESTING OF G_1 BEAM AFTER STRENGTHENING

PHOTO 3

Overall view of the beam at failure

ASPECTS FROM THE TESTING OF G_1 BEAM AFTER STRENGTHENING

PHOTO 4

General view of the beam in an intermediate phase of strengthening (injection of the more opened cracks)

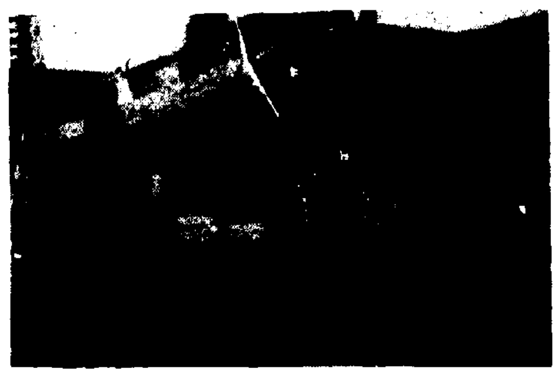

PHOTO 5

General view of the beam strengthened with fibre-glass tissue

PHOTO 6

End detail of the strengthened beam

ASPECTS FROM THE TESTING OF G_3 BEAN AFTER STRENGTHENING

PHOTO 7

General view of the strengthened beam

PHOTO 8

Detail of the midspan of the beam where the failure crack between two injected crack can be observed

PHOTO 9

End detail of the partial strengthening with fiber glass

II.3. CURRENT POSSIBILITIES FOR ANALYSIS AND MITIGATION OF THE SEISMIC RISK AFFECTING THE EXISTING BUILDING STOCK

Horea Sandi*

1. INTRODUCTION

Earthquake protection of the existing building stock represents a problem of large dimension which increasingly concentrates the attention of experts of various fields. The experience of numerous countries demonstrates dramatically the high seismic risk connected with older buildings, which were not designed to resist earthquakes. The experience of the Romania earthquake of 1977 confirms fully this statement.

The works involved in the rehabilitation and upgrading of existing buildings are at the same time technically difficult and costly and this fact clearly limits the possibilities of tackling the existing building stock. On the other hand, in many countries, particularly in Romania, the existing building stock is gradually replaced, sometimes at a rapid pace, by new buildings, related to the general plans of urban renewal and development. It is obviously necessary to correlate the two major actions referred to, namely that of mitigation of seismic risks related to the existing building stock on one hand, and that of urban development on the other hand. This paper is intended to contribute to a review of some conceptual and methodological aspects, of basic data required by the estimation of seismic risks and by the adoption of the most suitable decisions for intervention, of the practical possibilities of risk estimation and decision.

2. BASIC CONCEPTS AND METHODOLOGICAL ASPECTS

2.1. General

Seismic risk represents the expectancy of losses of various natures due to the occurrence of seismic actions (which is considered in this paper only in relation to the artifacts of man, potentially affected by destructive earthquakes). The impossibility of deterministic prediction of the losses to occur (moreover even of the basic factors which generate such losses) makes necessary the use of probabilistic concepts and of corresponding methodologies in this connection. Some basic definitions and relations on this subject are presented in this section. For more details it is possible to consider the developments of /4/, /9/, /14/.

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2.2. Basic Concepts

Considering the developments of /14/, the risk analysis can be carried out using the following terminology:

Seismic Hazard (H) :expectancy of occurrence of cases of seismic action at the sites of buildings dealt with,expressed in probabilistic terms.

Seismic Vulnerability (V) : distribution (in probabilistic terms) of damage or losses, conditional upon the magnitude of seismic action.

Specific Risk (S) : expectancy of damage or losses, as a function of service duration, assuming the elements at risk fully exposed, expressed in probabilistic terms.

Elements at Risk (E): people, property, human activities that might be adversely affected by earthquake (and for which it is necessary to quantify, besides values, the degree of exposure).

Risk (R) : expectancy of losses, as a function of service duration, considering the real exposure of elements at risk, expressed in probabilistic terms.

This list of basic concepts may be usefully detailed and completed, considering the quantifications of damage, loss and exposure. On the other hand, it is necessary to keep in view the crucial fact that the losses inflicted by earthquakes are connected with various systems (urban systems, regional systems,lifelines etc), due to various interactions and chain effects. A comprehensive approach should consider this fact. Besides some basic aspects considered in this connection in /2/, it may be useful to apply the developments of /6/ in order to analyze the risk to complex systems.

The limits of the extent of this paper makes it possible to deal only with individual buildings and to reduce, as far as possible,the number of basic concepts. An abridged approach may deal only with three basic concepts, namely hazard,vulnerability and risk.

2.3.Basic Expressions and Relations

Hazard is quantified by the parameter $\bar{n}_j^{(h)}$, which denotes the expected rate of occurrence of seismic action at intensity q_j (discretization of intensities is assumed)for a unit time interval . A second hazard measure is represented by the parameter $\bar{N}_j^{(h)}(T)$, which denotes the expected number of events of intensities equal to or higher than q_j, for a time interval of duration T,

$$\bar{N}_j^{(h)}\ (T) = T \sum_{j'}^{j' \geq j} \bar{n}_{j'}^{(h)} \qquad (2.1)$$

Possible losses due to earthquakes are quantified into several loss components, L_c, which are not comensurable (losses of lives, losses of property etc.). It is also assumed that the losses are discretized into a finite number of degrees, L_{ck}. Under these conditions, the vulnerability of a building dealt with is related to one seismic event and may be quantified by means of the conditional discrete probabilities $p_{ck/j}^{(v)}$, where where

$$\sum_k p_{ck/j}^{(v)} = 1, \text{ (for any c or j)} \qquad (2.2)$$

which denote the distribution (in a probabilistic sense) of losses, conditional upon the intensity q_j. The approach adopted in this paper includes the characteristics of exposure of various elements at risk in the characteristic $p_{ck/j}^{(v)}$ referred to.

The seismic risk may be quantified in several ways, all of which rely on the assumption (or model) of immediate and full post-earthquake rehabilitation of buildings and other elements at risk affected. A basic characteristic is the parameter $\bar{n}_{ck}^{(r)}$, which denotes the expected rate of occurrence of losses of severities L_{ck} (for a unit time interval). This parameter is given by the convolution

$$\bar{n}_{ck}^{(r)} = \sum_j p_{ck/j}^{(v)}\ \bar{n}_j^{(h)} \qquad (2.3)$$

A second risk measure is represented by the parameter $\bar{N}_{ck}^{(r)}(T)$, which denotes the expected number of cases of losses of severities L_{ck} or higher, for a time interval of duration T,

$$\bar{N}_{ck}^{(r)}(T) = T \sum_{k'}^{k' \geq k} \bar{n}_{ck'}^{(r)} \qquad (2.4)$$

Another risk measure is represented by the probability of non-occurrence and non-exceedance of losses of severity L_{ck}, for a time interval of duration T. In case one adopts a Poisson model for the process of earthquake occurrence and one accepts the assumption on full and immediate rehabilitation mentioned above, the probability referred to, $H_{ck}^{(r)}(T)$, will be given by the expression

$$H_{ck}^{(r)}(T) = \exp\ [-\bar{N}_{ck}^{(r)}(T)\ ] \qquad (2.5)$$

A last measure of risk, used in this paper, is represented by the cumulated expected losses, $\bar{L}_c(T)$. In order to derive an expression for these losses, it may make sense to consider, for some loss components, discounting of losses by means of discounting functions z_c (t). The expression of $\bar{L}_c(T)$ will be in this case

$$\bar{L}_c\ (T) = \int_o^T \sum_k L_{ck}\ \bar{n}_{ck}^{(r)}\ z_c(t)\ dt \qquad (2.6)$$

2.4. Some Methodological Aspects

The risk estimation may be made, in case of individual buildings without any major difficulties if one uses the relations presented and one makes the necessary data available (the difficulties of risk analysis are considerably increased in case one tackles explicitely systems and one wants to adopt a consistent approach, according to the developments of /6/.

At the level considered, the main difficulties raised by the risk analysis will be related to the input data, which are discussed in next section.

3. PROBLEMS RAISED BY THE INPUT DATA

3.1. General

According to the approach developed in section 2,the input data are of two categories:

a) the hazard characteristics $\bar{n}_j^{(h)}$

b) the vulnerability characteristics $p_{ck/j}^{(v)}$.

These two categories of basic data raise specific and difficult problems, so some specific comments are in order.

3.2. The Data on Seismic Hazard

The basic data on seismic hazard, represented by the characteristics $\bar{n}_j^{(h)}$, may be derived in principale on the basis of official documents, like existing zoning maps (and eventually microzoning data), or of special studies, intended to investigate the seismicity of a site. The hazard maps provide nevertheless limited information, (e.g. macroseismic intensities, or peak ground accelerations with one or two return periods). This makes it necessary to use additional assumptions in order to determine the full set of parameters $\bar{n}_j^{(h)}$ required by a full risk analysis. It is possible here to use classical recurrence laws like a Gumbel type distribution for macroseismic intensities (which is similar to the well known Richter magnitude recurrence law),or a Frechet type distribution for peak ground accelerations, or effective peak accelerations /11/, or other kinematic parameters considered in these analyses. Given the constant geometric ratio of kinematic parameters corresponding to macroseismic intensities ordered according to a constant arithmetic ratio, there is compatibility between the use of the classical Gumbel and Frechet distributions for the two categories of parameters. A more difficult problem is raised by the limited validity of the classical laws, given especially some physical limitations to earthquake magnitudes (given the features of geo-tectonic processes) and to ground motion intensities (given also the features of local site conditions). It must be recognized that the current state of the art of seismological knowledge is characterized by serious limitations and the need of extensive future studies.

3.3. The Data on Seismic Vulnerability

The basic data on seismic vulnerability, represented by the characteristics $p_{ck/j}^{(v)}$, may be derived on the basis of statistical analyses of the outcome of post-earthquake surveys and /or on the basis of engineering and social-economic analyses. The first source referred to, represented by the observation, quantification and statistical treatment of post-earthquake studies, leads to characteristics of observed vulnerability. A comprehensive set of data of this kind is given in /5/, /12/ where the most significant and detailed results have been obtained on the basis of the post-earthquake surveys carried out in Romania in 1977/1/, /7/. The second source referred to, represented by the engineering and, afterwards, socio-economic analyses, leads to characteristics of predicted vulnerability. The comprehensive data of /9/, which are based on data observation and analysis and judgement, must be mentioned here as a valuable source of information, which puts into evidence, among other things, the influence of earthquake resistant design strategies on the vulnerability of buildings.

The vulnerability characteristics of the sources referred to are affected by a high scatter of damage or losses for a definite intensity of action. This fact is particularly significant for the current level of knowledge in the field of earthquake engineering. On the other hand, this high scatter of vulnerability data leads to severe limitations of the posibilities of control and management of seismic risks.

3.4. Addenda

The limited amount, accuracy and certainty characterizing the input data referred to raises the need for extensive future research of various kinds. This must be oriented along directions corresponding to both of the categories of data (a) and (b) mentioned in section 3.1. This research must include at the same time some fundamental conceptual and methodological developments as well as applied research, related to the seismicity of some sites and regions and to the vulnerability of specific classes of buildings or other works.

4. RISK ANALYSIS AND CRITERIA FOR DECISION TO INTERVENE ON EXISTING STRUCTURES

4.1. General

Risk analysis is not a final goal in itself, but represents an intermediate step required by a cost-benfit analysis, which represents, in turn, a main support of decision making. Decision making is a necessary activity in principle any time when a design, or earthquake protection strategy is to be adopted. This may happen either in the case of the design of new buildings, or in the case of rehabilitation and upgrading activities related to the existing building stock. It is necessary to consider, in any case, various possible protection strategies in order to adopt, after corresponding analyses and judgement, the most reasonable of the possible strategies.

4.2. Cost-Benefit Analyses

A central goal of activities related to the building stock, whether this concerns new developments or existing ones, is represented by the maximization of the overall utility. One must consider in this relation the following main factors, or contributors (in a positive or a negative sense) to the overall utility: the benefits B resulting from the normal service, the investment costs, C', the maintenance costs, M, and the costs of damage (or the losses), L. As in the case of earthquake induced losses, for which it was necessary (see section 2) to consider various non-comensurable components L_C, it is necessary to consider homologous components for the other terms, B_C, M_C etc. (e.g.: the lives expected to be lost, corresponding to a definite quantity L, may be algebraically added to the lives expected to be saved by the activity of a hospital or of another building providing emergency shelter or rescue). The overall utility, U, will be quantified under these conditions by a system of components U_C, with the expression :

$$U_C = B_C - C'_C - M_C - L_C \tag{4.1}$$

The expression (4.1) is useful in order to judge a posteriori the utility of a given building, provided full information on this subject is available. In case one wants to carry out a cost-benefit analysis related to the future utility, it will be necessary, of course, to consider the random nature of at least some of the terms of expression (4.1) and try to maximize the overall utility. A first, perhaps sometimes too rough, approach, consists of the use of average, or expected, values, for the random quantities of the expression (4.2). The expected values of the utility components $U_C, \bar{U}_C$, will be in this case

$$\bar{U}_C = \bar{B}_C - C'_C - \bar{M}_C - \bar{L}_C \tag{4.2}$$

(where by it was assumed that the investment costs may be estimated in a deterministic way). The conditions

$$\hat{U}_c = \max \tag{4.3}$$

cannot be satisfied simultaneously, as a rule, since different strategies may prove to be the most suitable in case one single component (c) is considered. The multi-parameter (with respect to the index (c) problem may reduce to a one parameter optimization problem, in case one considers some (positive) weights w_c to be attached as factors to the condition (4.3) and one rewrites

$$\sum_c w_c \bar{U}_c = \max \tag{4.4}$$

The values w_c of the weights referred to should represent a matter for social-economic analyses and, ultimately, of judgement by an interdisciplinary experienced team. Such a team should consider various possible strategies S_i, to which various values $\bar{U}_{ci}$ (4.2) will be associated and select the most reasonable one.

4.3. Cost-Benefit Analysis versus Code Provisions

The cost-benefit analysis represents a difficult task, given the huge quantity of information to be gathered and processed. This activity is avoided in practice by using the provisions of design codes, which are assumed to have adopted on the basis of a sound socio-economic judgement, the most suitable calibration of some conventional design parameters defining the strategy to be adopted. This represents an important limitation of design effects, and, also, a source of safety, since possible coarse errors due to lack of information or experience in cost-benefit analysis will be avoided. On the other hand, it must be recognized that the code provisions specifying the level of earthquake protection present serious shortcomings, since they do not lead to some reasonable differentiations of design strategies or they do not cover some specific cases. To illustrate this statement, one must emphasize the very limited specifications of codes of many countries (including Romania), with respect to the protection (including eventual rehabilitation and upgrading) of existing developments, the lack of differentiation of seismic coefficients as explicit functions of the desired service duration of buildings (whether they represent new developments or existing ones), etc.

These facts lead to the conclusion that the use of codes must be completed, in several cases, by using more or less developed cost-benefit analyses, which are able to provide an insight of highest importance into the problems raised by the adoption of earthquake protection strategies.

4.4. Some Specific Conditions and Restrictions to Be Considered

in Relation to the Existing Building Stock

A specific question to be considered in relation to intervention on existing buildings is represented by the formulation of decision alternatives. These are related to two main aspects:

(a) the function of a building ;

(b) the structural characteristics of a building.

From the view point of the function of a building, one may consider, as alternative solutions:

- functional upgrading in correlation with structural upgrading;

- unchanged function;

- adoption of a function of lower importance;

- abandoning the use.

From the view point of the structural characteristic, one may consider, as alternative solutions:

- structural upgrading;

- unchanged structure;

- removing of some hazardous parts (e.g.some balconies or upper stories);

- demolition.

While man is practically unable at present to influence in a positive sense the seismic hazard, it is possible to influence considerably the vulnerability of buildings. This may refer either to the primary vulnerability, i.e., the proneness to damage conditional upon seismic intensity, which is influenced by the intervention on the structural characteristics, or to the secondary vulnerability, i.e., the proneness to losses conditional upon the degree of damage, which is influenced by the intervention on the function of buildings.

The adoption of the decision on intervention on the existing building stock is subjected to specific restrictions of which the most important are related to:

(a) the feasibility of some of the solutions considered initially;

(b) the need of correlation of solutions considered for individual buildings presenting some kinds of interaction and of correlation with the general plans of urban development.

The feasibility conditions may be of various type, such as:

- making sure that specific materials or manufactured parts required by a definite solution may be made available;
- making sure that design engineers, construction technicians and workers possessing the expertise required by some specific solutions are available;
- making sure that there exists the necessary room for construction works of various natures (including eventual demolition);
- making sure that the duration of intervention works is compatible with some specific service conditions.

Without going into analytical details that are specific to complex systems /6/, some important qualitative aspects connected with urban systems must be taken into account. The experience of behavior and analysis of urban systems suggests the importance of a high degree of safety to be provided to buildings located at some critical places, like crossings of streets, narrow portions of streets etc. The eventual collapse of such critically located buildings may lead to the heaviest post-earthquake consequences, due to the inability of rescue teams to gain access, of evacuation of people, etc.

Another practical aspect not to be neglected is that of differentiation of the required earthquake resistance of buildings, as a function of their subsequent service duration. Conversely, assuming that the resistance of a building was evaluated and does not fulfill the requirements for normal buildings of the same category, a deadline for intervention (including eventual abandoning and demolition) can be set in order to limit the seismic risk to an acceptable value. Under some simplifying assumptions: in case one considers the ratio

$$r = \frac{S_{eff}}{S_{req}} \qquad (4.1)$$

(S_{eff} :effective earthquake resistance, expressed in terms of forces, accelerations, seismic coefficients ;S_{req} :required earthquake resistance according to code provisions, expressed in homologous terms), the acceptable subsequent service duration is given by an expression

$$T_{acc} = r^2 \, T_{nom}, \qquad (4.2)$$

where T_{nom} is the nominal, or normal, service duration for new buildings of the same category (the exponent 2 in relation (4.2) approximately corresponds to the exponents of the extreme value laws characterizing the recurrence of seismic actions of various intensities).

5. ILLUSTRATIVE ANALYSES OF SEISMIC RISK AFFECTING SOME CATEGORIES OF BUILDINGS

5.1. Input Data

An illustrative analysis of seismic risk affecting some categories of buildings was carried out in the form of a research project of INCERC/10/. The analysis referred to was concerned with several categories of buildings, for which data on vulnerability were obtained from American /9/ and Romanian /5/,/8/ sources. The data on seismicity were adopted according to a Gumbel distribution, corrected in order to account for possible intensity.

The American categories of buildings referred to in /9/correspond to four design strategies, related according to UBC to zones 0,2,3 and S (the latter zone implies doubling of forces determined for zone 3). These strategies were considered equivalent to protection against nominal intensities VI, VIII, IX and X according to the MSK intensity scale. Additional data hypothetically corresponding to a design strategy associated with zone 1 were determined through interpolation. Interpolation was used also in order to adopt a finer discretization with respect to intensities, going from integer intensity degrees to halves of degrees. The damage degrees were defined according to /9/ in the following manner (Table 5.1) :

Table 5. 1.

Damage degree (k)	Characterization	Losses, as a percentage of replacement cost
0	none-0	0
1	light-L	o to o.05 %
2	medium-M	0.05 to 1.25 %
3	heavy-H	1.25 to 20 %
4	total-T	65 to 10C % (building condemned)
5	collapse-C	100 %

The Romanian categories of buildings were the classes A. 2 (bearing wall masonry without r.c. floors, built before the 194 earthquake) and A.6 (framed structures, built before the 1940 earthquake, basically not engineered to resist earthquakes). The damage degrees and vulnerability matrices are described in /8/.

The seismic hazard was represented by means of an analytical law

$$\bar{N}^{(h)}(q,T) = T \int_{q}^{q_{max}} \bar{n}^{(h)}(q')\, dq' \qquad (5.1)$$

where

$$\bar{n}^{(h)}(q) = \begin{cases} b' \exp\left[a' - b'q - \dfrac{c'}{q_{max} - q}\right] & (q < q_{max}) \\ 0 & (q \geq q_{max}) \end{cases} \qquad (5.2)$$

The values of parameters (a') and (b') were a' = 2 ln 10, b' = 0.5 ln 10. The parameters (c') and (q_{max}) were assigned several values aimed to lead to a parametric study. The results presented here in correspond to the values c' = 0.04 ln 10, q_{max} = 9.5 (IX 1/2). A graphic representation of the seismological data is given in fig.5.1.

5.2. Results Obtained

The results obtained were expressed by means of the risk characteristics $\bar{N}_k^{(r)}$ (1) (one simple loss component, c= 1, with index (c) committed, was considered). The representation of results given in following figures is related to the natural cologarithm H_{2k} of this parameter,

$$H_{2k}^{(r)} = - \ln \bar{N}_k^{(r)}(1) \qquad (5.3)$$

It is easy to determine on this basis the survival probabilities for any time interval, using the relation

$$H_{ok}^{(r)}(T) = \exp[-T\, \bar{N}_k^{(r)}(1)] = \exp[-\bar{N}_k^{(r)}(T)] \qquad (5.4)$$

derived from the more general expression (2.5).

The illustrative results obtained for the input data referred to are represented in fig.5.2 and 5.3 for the American buildings and in fig.5.4 for the Romanian buildings.

In order to judge the results obtained, it must be kept in mind that the values of the parameter H_2 correspond to following values of the parameters $\bar{N}^{(r)}(1)$ and $H_o(T)$ (Table 5.2)

Table 5.2

H_2	$\bar{N}^{(r)}(1)$	H_o(10 years)	H_o(100 years)
4	1.83×10^{-2}	0.8326	0.1602
6	2.48×10^{-3}	0.9755	0.7805
8	3.35×10^{-4}	0.9967	0.9670
10	4.54×10^{-5}	0.9995	0.9954

The results obtained put to evidence the strong dependence of the parameter H_2 on the earthquake protection strategy. There exists no absolute protection, given the vulnerability characteristics assumed. The efficiency of increased protection is obvious especially in connection with the risk of greater damage. Lighter damage is practically unavoidable in case of a longer service duration.

6. FINAL CONSIDERATIONS

1. The seismic risk must be considered as an objective reality for regions which are likely to be affected by strong ground motions. It is possible to quantify it and to analyze it, provided input data of satisfactory quality are at hand. It is possible to mitigate it by means of some appropriate measures, positively influencing the vulnerability of buildings and of ather artifacts of man.

2. The seismic risk must be analyzed and controlled, in order to limit losses inflicted by future earthquakes. While the new developments, designed according to modern code provisions, are in most cases provided with a satisfactory degree of earthquake protection, the old buildings, not engineered to resist earthquakes, represent a major source of risk and, consequently, a major threat to society. Interventions intended to mitigate the risk connected with these buildings, according to a sound strategy, are therefore necessary.

3. While code provisions related to the design of new buildings are well developed in practically all technically advanced countries, the regulations related to the protection of the existing building stock are missing in most countries and, where they exist they do not cover the whole range of interest. It is necessary to concentrate important technically qualified forces in order to gradually bridge this important gap. It must be noted that, besides design methodologies and formats that are similar to those specified by codes related to the design of new buildings, it is necessary to consider different methodologies and formats for specific purposes. Explicit cost-benefit analyses, based on explicit risk analyses, may be justified in order to adopt reasonable decisions with respect to the interventions on the existing building stock. The explicit consideration of intervention alternatives related to the function and the structural characteristics, a primary filtering considering the feasibility criteria and thereafter a proper cost-benefit analysis, are required in this context.

4. The intervention on the existing building stock must be based on the consideration of the building stock as a system, since the seismic risk is related essentially to various systems, the components of which may strongly interact. The analysis of risk which could affect existing systems must put into evidence the most

significant risk sources, like buildings located at critical places, buildings or facilities that are highly hazardous or likely to be involved in chain effects, etc.

5. The problems of intervention on the existing building stock must be analyzed in correlation with the strategy of new development of urban systems, lifelines, etc. The earthquake resistance required for existing buildings must be derived, keeping in view the expected subsequent service duration. The required resistance level must be thus differentiated. Conversely, in case a definite existing resistance level is considered in the category of the problem data, it is possible and necessary to set a corresponding intervention deadline. The consideration of the deadlines specified for various individual developments must be summarized in a list of intervention priorities. A sound list of priorities, followed by appropriate interventions on the buildings presenting the highest intervention priorities, may result in a spectacular reduction of seismic risk, obtained by means of moderate efforts.

6. The most difficult technical problem raised by the analysis of seismic risk is represented by the analysis of vulnerability of buildings and other works. High attention must be paid to the task of gathering information of this subject in relation to all aspects of vulnerability.

7. Engineers and other specialists who should be involved in the mitigation of seismic risk affecting the existing building stock need a special training, exceeding the fields dealt with in classical education.

8. An important step to be considered in relation to the mitigation of seismic risk affecting the existing buildings is represented by the planning of pilot studies, to precede mass action.

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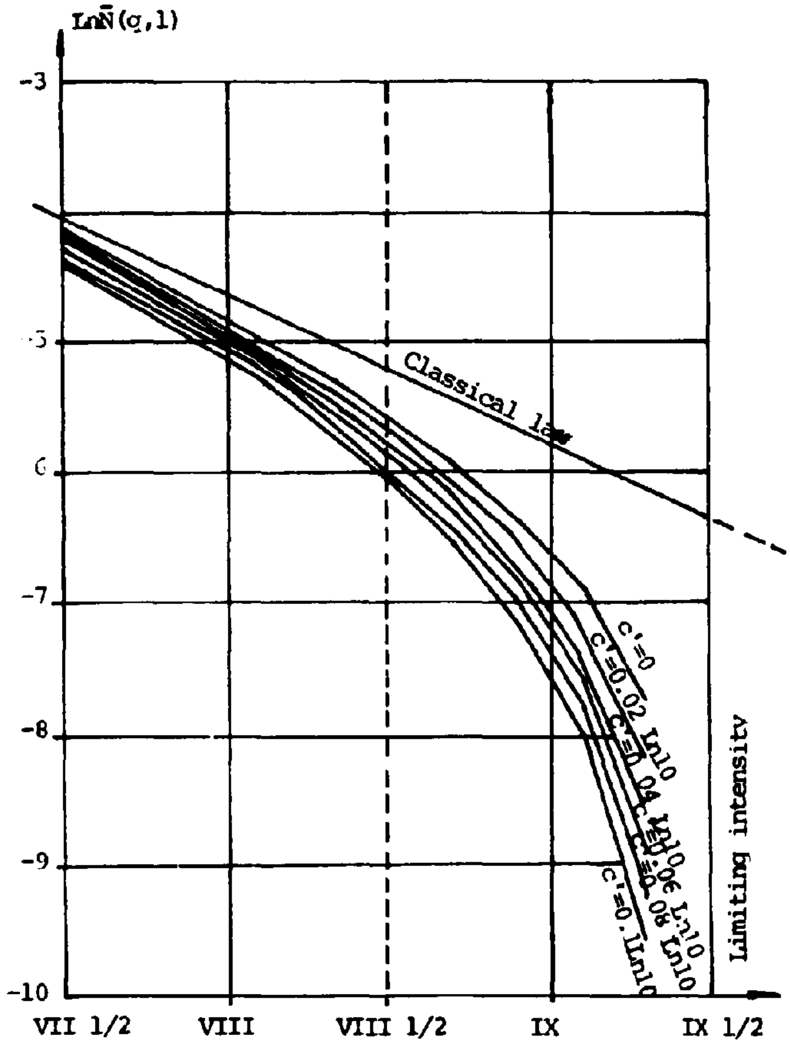

Fig.5.1

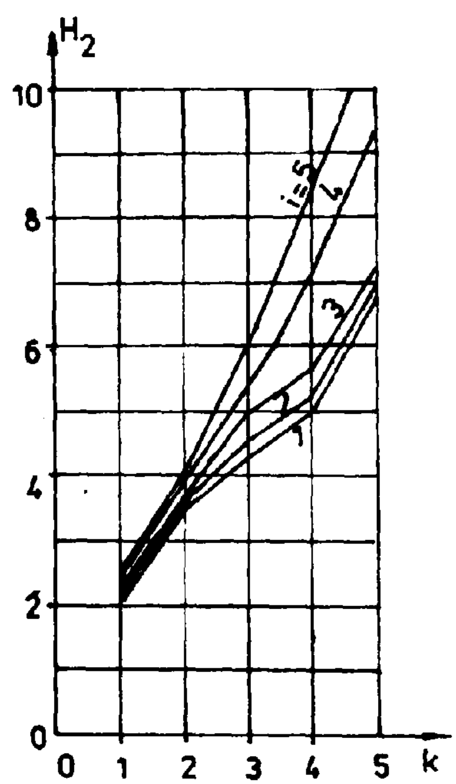

Fig.5.2

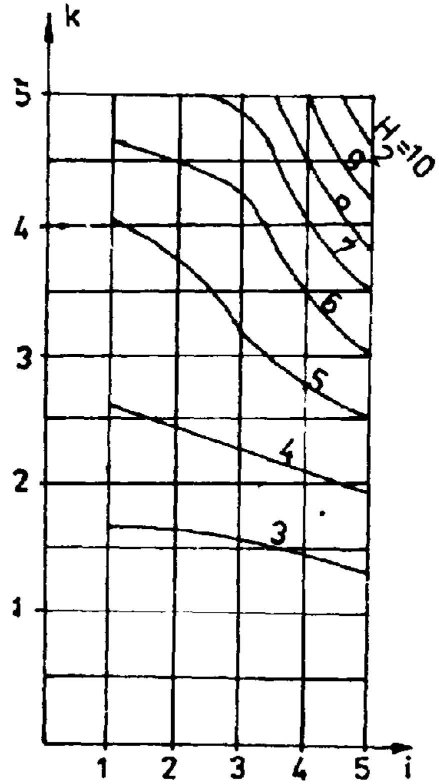

Fig.5.3

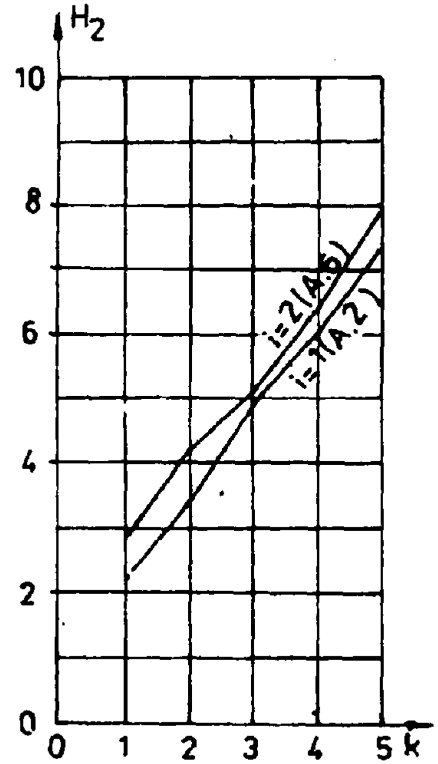

Fig.5.4

II.4 A SUMMARY OF STUDIES ON THE SEISMIC VULNERABILITY OF BUILDINGS, CARRIED OUT IN BUCHAREST SUBSEQUENT TO THE MARCH 4, 1977 EARTHQUAKE

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1. INTRODUCTION

The Romanian, March 4 1977, seismic event represented not only a destructive earthquake, but also an opportunity of exceptional value to improve of the technical knowledge in earthquake engineering. This fact was underlined by many of the foreign experts who visited Romania after the earthquake. The main reasons for the scientific value of this experience are the features of the natural phenomena (type of earthquake, extent of the area subjectedto strong shaking, spectral content of ground motion) and the number of artifacts of man that were subjected to a direct natural testing (this includes, among other, various categories of modern structures, most of them standardized and erected by means of industrialized methods and, at the same time, engineered to resist earthquakes).

The seismic event of March 4 1977 was followed by numerous analyses of its nature and effects. One of the important directions of work was the detailed post-earthquake survey of buildings in Bucharest. This survey, designed initially to provide detailed information on the features of ground motion, provided basic information that was used subsequently for other purposes too. Among other things, this information was used, in the framework of an international project [13], to derive data on the seismic vulnerability of buildings.

The present paper is intended to present a summary of stu-

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dies on the vulnerability of buildings, parts of which were presented previously in [5],[6],[7],[8],[13]. The content is devoted to a presentation and discussion of basic concepts and of methodological aspects, of the results of observation, of the correlation between survey results and results of some engineering analyses as well as to comments on the state-of-the-art.

2. CONCEPTUAL AND METHODOLOGICAL ASPECTS.

2.1. General. Conceptual and Methodological Aspects.

The basic concepts and terminology were used in agreement with the developments of [14], of which some references are given in [6]. VULNERABILITY was understood, in this frame, as the distribution of damage (expressed in physical, observable, terms,the distribution being understood in probabilistic or statistical terms), conditional upon the intensity of seismic ground motion affecting definite buildings.A distinction was made between OBSERVED VULNERABILITY, or VO, (i.e., conditional statistical damage distribution, as derived from post-earthquake surveys for some definite classes of buildings or other structures) and PREDICTED VULNERABILITY, or VP, (i.e. conditional probabilistic damage distribution, as derived from engineering analyses for some types of buildings).

The vulnerability analysis requires basically definitions and quantifications for three categories of data :

(a) the object of analyses (type or class of buildings or of other works dealt with);

(b) the effects of seismic action (damage or loss actually, or potentially, inflicted to the object);

(c) the seismic action.

The bulk of primary data for the VO-analyses carried out for various classes of buildings in the city of Bucharest were provided by the extensive and detailed damage survey organized in April and May under the auspices of CNST (National Council on Science and Technology of Romania). This survey, which is described in more detailed terms in [1], is briefly summarized here too.

The territory of the city of Bucharest was divided into squares of 1 km x 1 km. Some 300 buildings were analyzed within each of the squares (the buildings were selected, so as to have a representative sample of the damage distribution). The buildings investigated included dwellings, schools and hotels. A damage survey form as given in fig, 2.1.was used.

Classification of buildings.

The buildings investigated were divided into five basic categories pertaining to different construction systems:

C_1 Low quality construction (adobe-like).
C_2 Masonry bearing walls and flexible floors.
C_3 Masonry bearing walls and rigid floors (R.C.).
C_4 Reinforced concrete framed structures.
C_5 Reinforced concrete bearing wall structures (cast-in-place or precast solutions).

The categories C_2 and C_3, which represented the bulk of the survey sample, were subdivided according to age (pre-1940 and post 1940). The buildings pertaining to each of the categories were subdivided according to the criteria of dynamic characteristics; using simple empirical formulae, the fundamental natural periods were estimated and the subcategories were related to following oscillation period intervals : 0 to 0,15s; 0.15 to 0.25s.;0.25 to 0.35s.;0.35 to 0.50s.; 0.50 to 0.70s.;0.70 to 1.00 s.;1.00 to 1.30 s.;and more than 1.30s.

Quantification of damage.

Rules were established in order to quantify the observed degree of damage for various types of structural elements and for some types of non-structural elements. These rules were inspired by the MSK scale (the MSK scale experience shows a fairly linear correlation between DD's (damage degrees), as given by the methodology adopted on one hand, and the MSK intensity degree on the other hand). The statistical survey referred to was organized at an unprecedented level of detail and this fact required some iterative testing of the damage degree calibration. The final quantification methodology represented the third variant adopted.More details on this aspect are given in [1]. The DD's ranging from 0 (not affected) to 5 (collapse), were finally defined as follows :

I.Masonry

Bearing walls:
- not affected :0
- slightly affected :1 ;
  (for lack of specification, 0.5) ;
- cracked :1.75;
- strong cracking :2.50;
- strong cracking , 45°:3;
- out of vertical direction :4 ;
- collapsed :5.

Non-bearing masonry :
- not affected :0;
  (for lack of specification :0.5);
- cracked :1 ;
- partially collapsed :2 ;
- completely collapsed :3;

The DD assessed for a building was the maximum of the DD's assessed for bearing walls and for non-bearing masonry respectively.

II.R.C. frames

Columns :
- not affected :0.5;
- cracked :2;
- strong cracking :4;
- crushed concrete :4;
- buckled reinforcement :4.

Beams :
- not affected :1;
- slightly affected :1;
  (for lack of specification :0.5);
- cracked :2;
- strong cracking :3;
- failed :4.

Infill masonry :
- not affected :0;
- boundary cracks :1;
- cracking :1.5;
- strong cracking :2;
- dislocation :2.5;
- collapsed :3.

The DD assussed for a building was the maximum of DD's assessed to columns and to infill masonry for pre-1950 buildings,and the maximum of DD's assessed to beams and to infill masonry for post-1950 buildings.

III.R.C. bearing walls
- not affected :0;
- slightly affected :1;
  (for lack of specification 0.5);
- cracked :2;
- strong cracking :3;
- failed :4.

The same values as in case of R.C. frames were used for infill masonry and for columns.

The DD's assessed for a building was :the maximum of DD's assessed for R.C. bearing walls and for infill masonry, respectively, in case of buildings with homogeneous layout :the maximum of DD's assessed for the former ones and for the columns,in case of flexible-first storey buildings.

IV. Buildings done of low quality materials

Walls :- not affected :0;
- slightly affected :1;
(for lack of specification :0.5);
- cracked :2;
- strong cracking :3;
- collapsed :5.

The DD assessed for a building was measured as the DD as - sessed for the walls.

Statistical damage spectra. The outcome the of investiga - tion was represented by the statistical damage spectra derived for the various squares of the territory and for the various ca - tegories of buildings. As an example, the statistical damage spec- tra derived for one central square of Bucharest are reported in fig.2.2. (complete results are given in [1]).

The degree of confidence of results may be checked on the basis of variance of results (in case of large samples, the r.m.s DD did not exceed 0.7, which represents a statisfactory result from the view point of the MSK scale; the coefficient of varia - tion of DD was decreasing for increasing DD).

The statistical damage spectra were used in order to derive MSK - intensity assessments for the various squares of the ter- ritory of Bucharest. These assessments were related to oscillation period intervals. The observed intensity maps developed were re- lated to the intervals of periods 0 to 0.15 s. 0.15 to 0.25 s and 0.7 to 1.0s. These maps are given in [1] , where the tables of conversion of DD into MSK in- tensities, for various classes of buildings, are also given.

2.2. Methodological Features Related to the Deriving of Vulnerability Characteristics.

The task of deriving vulnerability characteristics (more precisely, according to the formulation adopted in the Project documents, vulnerability functions), was novel in Romania and work was started primarily due to the requirements of the UNDP/ UNESCO Project. The methodologies adopted in this connection were derived by using the concepts, definitions and guidelines adopted by the first meeting of the Working Group and, besides this, by the developments of [4].

The main features of the approach to the vulnerability analysis can be summarized as follows :

1. Input data . The input data were the assessments on ground motion characteristics and on the distribution of damage degree for some samples buildings.It must be mentioned again, in this respect, that the assessments of the ground motion charac - teristics were themselves the result of a comparison of the sta - tistical data on damage distribution with the assessments of the

MSK scale and of a least error approach that led to the assessment for a zone of more or less homogeneous conditions of ground motion (and in Bucharest, also for a specific interval of oscillation periods), of a certain degree of MSK - intensity (the intensity assessments were made, in Bucharest, in halves of MSK degrees, e.g. ... VII, VII$^{1/2}$ or VII to VIII, VIII, ...).

2. Processing. It was possible, on this basis, to consider, for classes of buildings that were homogeneous from the view point of construction system (and of fundamental natural periods) the statistical distribution of damage, as related to samples corresponding to more or less homogeneous conditions of ground motion intensity (and oscillation period intervals). To be more specific, considering the statistical information given by damage spectra in Bucharest for various categories of structures as illustrated in fig.2.2, it was possible to consider all the squares of approximately equal intensity (for a given interval of oscillation periods) and to draw up a histogram of damage distribution for one of the structural types considered.

3. Representation of results. The results of processing were represented in the from of series of histograms of DD (damage degree) distribution for various MSK intensities for each of the constructive systems considered. It was possible to derive, on this basis, data on the average DD and on the r.m.s. DD and to plot the data on the same graphs as those representing the histograms. The presentation of results obtained in this way is given in Chapter 4 of the Report.

4. Additional information. It was possible for a class of structures that were highly homogeneous to go one step ahead in an analytical direction that might help in characterizing the vulnerability in a more analytical manner. This was the case of some standardized buildings, erected series-wise in Bucharest. The consideration of damage in bearing walls oriented longitudinally or transversally respectively, in conjunction with the consideration of the azimuthal orientation of buildings, permitted researchers to consider the distribution of damage in longitudinally oriented and in transversally oriented bearing walls, as a statistical function of the azimuthal orientation of buildings. It was possible, in this way, to eliminate the influence of the directivity of ground motion and to derive some conclusions on the longitudinal and, respectively, transversal "V" 's of the given type of building.

3. VULNERABILITY CHARACTERISTICS DERIVED FROM THE POST-EARTHQUAKE SURVEY.

3.1. Data on the Characteristics of the Building Stock.

The building stock that is relevant from the viewpoint of vulnerability analysis presented in this paper consists primarily of residential buildings in service for not more than approximately one century, i.e. having been built not earlier than the last quarter of the 19-th century.

The stock of residential buildings used as the basis for the statistical sample in the post-earthquake analysis may be categorized according to various qualitative and quantitative criteria, among which the most significant are :age, basic construction materials, construction system, degree of engineering, azimuthal orientation, features and characteristics of ground conditions, dynamic characteristics, characteristics of the degree of earthquake protection and ductility characteristics.

It is possible to define some classes with respect to any of the criteria enumerated. In order to describe the building stock referred to, it is useful to present a brief on the development of earthquake protection in relation to the development of building activities.

Romania had entered the stage of modern development by mid nineteenth century, when the development of Europe gradually became significant or prevalent for the social, economic and urban development of the country. The brick masonry constructions of residential buildings had become prevalent especially in urban areas. Most of the buildings built during the nineteenth century were one-or two-story buildings. The quality of materials and workmanship was different for different geographiy areas, due to the nature of basic materials and the tradition of manufacturing and workmanship. The early twentieth century witnessed the gradual development of taller buildings and the introduction of reinforced concrete, at first mainly for slabs, lintels, girders, etc. The systems of vertical and horizontal loadbearing members were nevertheless not designed to resist horizontal loads, since the concern and know-how of design engineers in relation to earthquake resistance problems was practically absent up to the second world war.

The fifth decade of the 20-th century represented a turning point in building activities as they related to earthquake protection, due to several reasons, of which the most significant were :

(a) the occurrence of the destructive earthquake of November 10 1940 (the first strong earthquake withstood by modern construction in Romania);

(b) the occurrence of the Second World War which led to destruction or damage due to bombing and to a slowdown in construction activities;

(c) the first direct contacts with the Soviet school of earthquake engineering, one of the leading schools in the world.

The period after 1950 witnessed a major improvement in engineering education in universities, mainly by the courses on theory of structures of a modern level and the organization of earthquake engineering research activities that tackled an increasing range of problems of experimental, theoretical or computational nature.

The development of construction systems during the third quarter of the twentieth century may be characterized as follows: the trend toward industrialization of construction has led to a gradual decrease in the use of masonry as compared with other building materials. Cast-in-place reinforced concrete has been used increasingly, for low-rise buildings (up to five stories high) and for high-rise buildings (more than seven stories high). The most widely used solution became that of bearing walls. Various technological solutions were adopted, among which, for some years, were sliding forms. The technological solution proving to be the most advantageous, primarily due to the increased certainty of a satisfactory quality, was that of industrialized, dismountable forms (consisting of two-or three-dimensional components). The cast-in-place bearing wall reinforced concrete structures represent by far the bulk of high-rise buildings built during last two decades. The solutions adopted are characterized by low steel consumption indices, which are in the range of 20 kg/m^2 for low-rise buildings and in the range of 30 kg/m^2 for high-rise buildings.

An industrialized system which was introduced in about 1960 and which gradually increased its share until it became prevalent for low-rise buildings and now tends to become prevalent for high-rise buildings too, is that of large panel construction. Besides this, prefabricated construction was adopted also by using three-dimensional (room-size) precast elements ("bell" or "tunnel" shaped).

It is also useful to give some broad quantitative characterization of the building stock from the view point of parameters related to earthquake behavior and resistance. The most significant parameters from this viewpoint are :

- the fundamental natural periods ;

- the seismic design factor C_I;

- some earthquake resistance indicators defined further on;

- some ductility characteristics.

The factor C_I may be considered primarily in case of engineered construction.

It is convenient to use also the following simple indicators :

- the area indicator,

$$I_{ar} = \frac{A_{active}}{m}$$

and

- the acceleration indicator

$$I_{ac} = \frac{A_{active} R_s}{m} = I_{ar} R_s$$

where :

A_{active} (m^2): the total area of horizontal sections of shear resisting members, oriented along a direction considered;

m(t) : the mass generating seismic forces to be transmitted through A_{active};

R_s : the ultimate shear strength of the material

I_{ar} : has not a direct physical sense, but

I_{ac} (m/s^2) : has the sense of ultimate static acceleration , corresponding to the shear strength of the structure considered (critical acceleration).

The building stock of Romania is generally rigid from the viewpoint of dynamic characteristics. The fundamental periods of low-rise buildings belong to the interval (0.2-0.3)s and for high-rise buildings belong, for the same conditions, to the interval (0.04n-0.06 n)s (n:number of stories) [9].

Engineered structures, designed according to the provisions of earthquake resistant codes, may be easily characterized by means of the seismic design factor C_I (related to the fundamental mode). According to the codes in force after 1963, this factor was to be determined according to the relation

$$C_I = k_s \cdot \beta(T_I) \cdot \psi \cdot \varepsilon_I \qquad (3.1.)$$

where

k_s :ratio of conventional design acceleration to the acceleration of gravity, depending on the fundamental natural period T_I;

$\beta(T_I)$:dynamic factor, depending on the fundamental natural period T_I;

ψ :correction factor, accounting for the influences of damping characteristics, of strength reserves and of ductility;

ε_I :equivalence factor, given by the relation :

$$\varepsilon_I = \frac{(\Sigma_k m_k \cdot V_{kI})^2}{(\Sigma_{k'} m_{k'})(\Sigma_{k''} m_{k''} V^2_{kI})} \tag{3.2}$$

(V_{KI} :fundamental natural shape).

As a result, the value of C_I ranged between 0.06 and 0.09 for I = VII MSK and between 0.10 and 0.15 for I = VIII MSK for rigid buildings and respectively between 0.02 and 0.03 for I= VII MSK and between 0.03 and 0.05 for I = VIII MSK, for relatively flexible buildings (T_I = 1 s).

Non-engineered structures may be characterized by means of the parameters I_{ar} and I_{ac}. These parameters may be useful,on the other hand, also in order to characterize the ultimate horizontal static force carried by a structure. The nature of building materials, the building tradition, as well as some non-structural requirements, have led to some typical solutions that may be characterized as follows. The lowest, traditional, masonry buildings (one or two stories) are characterized by relatively high values of I_{ar}, such as 0.02 to even 0.05 m^2/t. This leads, even in case of lower quality masonry (e.g. R_s= 0.2 MPa), to high values of I_{ac}, that can exceed 5 m/s^2. The more modern masonry buildings (non-engineering), which might be three-to five-stories high, may lack internal walls (replaced by reinforced concrete columns not designed to resist earthquakes);may be characterized by low values of I_{ar}, ranging between 0.005 and 0.01 m^2/t and leading to values of I_{ac} that are even as low around 1 m/s^2. The values of I_{ar} and I_{ac} were even lower for high-rise buildings not designed to resist earthquakes, the resistant structure of which was composite, resulting from the assemblage of reinforced concrete members and masonry walls (sometimes lightweight masonry). Engineered high-rise bearing wall buildings (ten to twelve stories) were characterized (for different solutions and directions) by values of I_{ar} sometimes lower than 0.001 m^2/t, but as a rule in the range of 0.002 to 0.003 m^2/t. This leads, in the most unfavorable cases,to values of I_{ac} around 1 m/s^2, but as a rule in the range of 5 m/s^2.

The ductility characteristics of the buildings stock were in most cases low. The traditional low-rise masonry buildings, lacking reinforced concrete members, have few ductility sources. The ductility characteristics were low in many cases even for framed structures designed to resist earthquakes. Due to this fact, the ductility characteristics of buildings with reinforced concrete framed structures were to a great extent the result of interaction of reinforced concrete frames with masonry (the latter one playing the role of non-linearly behaving trusses).

3.2. Vulnerability Characteristics for Various Classes of Buildings.

1. Buildings Made of Low Quality Materials

These buildings were relatively old (most of them built before 1900) and made of low quality materials (adobe type, wooden structure with earth infill, etc.). They were in no case engineered. Their height was as a rule one story. The floors usually were made of timber. The living conditions in such buildings were, in most cases, precarious. They represented a small part of the building stock and were disapearing due to the demolition work carried out in relation to the activities of reshaping the city.

The most characteristic damage noticed in such buildings consisted of a wide vertical crack at the intersection of walls, with a tendency to lose stability, and expulsion, inclined cracks tendencies of sliding of floors and roofs, collapse of chimneys, etc.

The "V" characteristics are given in fig.3.1.

2. Old Buildings with Masonry Bearing Walls and Flexible Floors (built before 1940)

These buildings represented the major part of the older building stock of Bucharest. Their share in construction activities started to decline after the First World War, when reinforced concrete floors became a usable solution in Romania. Their height was in most cases of one or two stories but were built to a height of four or five stories in relatively rare cases. Their shape was in many cases irregular and highly non-symmetric. The quality of masonry was variable, ranging from poor to excellent. It is possible to meet high quality masonry even in some poor small houses. The poorer houses often lacked a proper foundation, especially when a basement was not built. This shortcoming led in several cases to the development of specific cracks or of tilting of walls even before the earthquake, due to uneven settlements development mainly as a result of the freezing-defreezing phenomena. The floors were made as a rule of wood or masonry vaults supported by steel beams.

The damage due to the earthquake was in some cases heavy, including some collapses (more numerous collapses occurred outside Bucharest). The heaviest damage involved dislocation and expulsion of walls. Beside that, it was possible to observe in many cases cracks of various orientations and width. The inclined cracks sometimes let to failure planes. X-shaped wide cracks were also observed. The lack of a satisfactory horizontal tie, due to the lack of rigid and strong floors, raised the risk of loss of stability after the occurrence of damage due to earthquake.

The vulnerability characteristics are given in fig.3.2.

3. New Buildings with Masonry Bearing Walls and Flexible Floors (built after 1940).

The share of these buildings in the building stock tends to decrease in time, given the increasing use of reinforced concrete in construction of floors.

The general pattern of observed damage was similar to that of the previous class of buildings.

The "V" characteristics are given in fig.3.3.

4. Old Buildings with Masonry Bearing Walls and Rigid Floors (built up to 1940).

The general features of this class of buildings are similar to those of the class discussed in the previous two paragraphs, except for the floors, which were made of reinforced concrete.

The general damage pattern was qualitatively similar to that of the previous classes referred to, but the presence of rigid floors did not permit the development of some unfavorable types of behavior, like sliding of wooden or steel beams and tendency for walls to lose stability.

The "V" characteristics are given in fig.3.4.

5. New Buildings with Masonry Bearing Walls and Rigid Floors (built after 1940)

The use of reinforced concrete members (horizontal and vertical) with a confining role contributed seriously to increased strength and also gave the masonry buildings a significant degree of ductility.

The "V" characteristics are given in fig.3.5.

6. Buildings with Reinforced Concrete Framed Structure.

The increase in height of buildings led, especially after [illegible], to the introduction of reinforced concrete vertical bearing [illegible]rs. This construction system was used for numerous taller buildings in Bucharest which shaped the modern center of the city. The height of these buildings is 6 to 12 stories. It must be mentioned that, until the destructive earthquake of 1940 (and then during the war and early post-war period up to 1950), these buildings, in spite of being built on the basis of engineering design, were not designed to resist earthquakes. There was no concern for an adequate structural layout and detailing. In many cases, the vertical bearing members were not continuous over the entire height of a building, so that second or even third order supporting solutions were adopted. There was no concern for including moment-resisting nodes and the reinforced concrete structure played the role of transmitting part of the gravity loads to the ground and, also, of confining the infill masonry, which represented an essential element for the lateral load-resisting capacity. The (apparent of hidden) effects of the destructive earthquake of 1940 upon these buildings must be noticed.

The reinforced concrete framed structures designed to resist earthquakes were built after 1950 to a limited extent, given the economic and technological advantages of the bearing wall reinforced concrete structures used in mass construction of high rise buildings. Such structures were used for buildings of the health care network, schools, etc., but relatively seldom (especially before 1970) for high-rise residential buildings.

The older buildings of this class represented the major portion of buildings which collapsed during the 1977 earthquake.Many buildings of this class were affected by heavy damage. The main reasons for their poor performance was the lack of concern for earthquake resistant design, the frequently low construction quality, the effects of the previous overloadings (1940 earthquake, bombing during war), the corrosion phenomena, some cases of unsuitable interventions that reduced their ability to withstand earthquakes and the lack of survey and adequate repair and stregthening measures.

The new buildings of this class, designed to resist earthquakes, shawed a considerably better performance. They were nevertheless affected by damage mainly to non-structural elements. In some cases specific damage due to collision (pounding)also occurred. In some cases structural members were also affected by damage

The damage in the reinforced concrete members of older buildings was in many cases heavy. Cracks, sometimes of considerable width, as a rule appeared close to the ends and had various orientations (horizontal or inclined,due to shear force effects). Concrete crushing also appeared. Most affected were the external columns (especially corner columns). Concrete expulsion and reinforcement buckling were also noticed. The beams were also affected by cracks at the ends (vertical or inclined). Concrete crushing and reinforcement building appeared also. The floors were affected as a rule by frequent, small cracks. The nonstructural members (mainly infill masonry, separation walls, etc.)were frequently affected by heavy damage like wide cracks (as a rule inclined or X-shaped), dislocations or even collapses and reversals of walls.

The damage observed in newly constructed buildings was qualitatively similar, but statistically slighter, due to the e fficiency of earthquake protection measures.

The "V" characteristics are given in fig.3.6.

7. High-Rise Buildings with Closely Spaced Reinforced Concrete Bearing Walls.

The bearing wall reinforced concrete structures represented the bulk of the high-rise building stock built during the last two decades.

The layout for long-in-plane buildings was based, for different design solutions, on one or two longitudinal internal bearing walls.

The heaviest damage observed for such buildings was a crushing of concrete due to bending compression and by inclined wide cracks, followed in some cases by concrete dislocation and expulsion. Other characteristic damage types were inclined or X- shaped cracks in lintels, vertical cracks (sometimes wide) or inclined cracks in bearing walls, especially in the more rigid ones. The cases of slight damage were characterized by fine cracks of various orientations (vertical, inclined, horizontal) in bearing walls, especially in relation with some local construction defects.

The "V" characteristics are given in fig.7.

8. High-Rise Buildings with Reinforced Concrete Bearing Walls at Larger Intervals.

Besides the solutions characterized by small intervals between the transverse bearing walls, several solutions were characterized by larger intervals between bearing walls, aimed to permit more architectural freedom in shaping the apartments. These solutions were designed and erected under conditions that are similar to those referred to for the previous class of buildings.

The damage pattern was qualitatively similar to that described for the previous class of structures.

The "V" characteristics are given in fig.3.8.

4. CORRELATION WITH SOME PARAMETERS OF ENGINEERING ANALYSES.

4.1. Methodological Aspects.

In order to perform a first attempt at correlating survey data on damage distribution with the outcome of engineering analysis, was decided to use a sample of buildings of Bucharest ,for which post-earthquake survey data obtained in 1977 [2], [13] were at hand.

The activity related to the simplified evaluation of the resistance characteristics of buildings consisted of two main steps.

First step

This step included the collection of the drawings of buildings analyzed, gathering of information of the quality of building materials, determination of the dynamic characteristics and, to the extent to that this was possible, drawings of the damage pattern, as well as non-destructive testing aimed as help in the evaluation of the building material properties.

Second step

The second step included the processing of basic data and the proper evaluation work. As was previously mentioned, the evaluation technique should be different for the different specific evaluation methods being adapted to this kind of building material and structural system.

The second step consisted in fact of following operations :

- evaluation of the built areas for the different stories of a building;

- evaluation of the mass of a building dealt with;

- evaluation of the active area of the horizontal section through bearing walls, for each of the two main directions of a building;

- evaluation of the capable shear stress (the shear stress that could be borne by the masonry in case of horizontal loading);

- calculation of the critical (static) accelerations of a building, for each of the two main horizontal directions;

- estimate of the spectral acceleration believed to have affected a building during the strong earthquake of 1977;

- calculation of the ratios of spectral, to critical, accelerations.

The evaluation of the critical acceleration, denoted, a_{cr}, was thus based on the evaluation of the static strength, S_{cap}, given by the expression :

$$S_{cap} = A_{act.} \tau_{cap} , \tag{4.1.}$$

were A_{act} represents the active area, while τ_{cap} represents the capable shear stress (determined with consideration of the influence of nominal, gravitational stresses).
The critical acceleration is given by the ratio

$$a_{cr} = \frac{S_{cap}}{m} \tag{4.2.}$$

where m represents the mass of the part of the building above the horizontal section for which the verification is performed.

The basic parameter to be used in correlation analyses is the ratio

$$r = \frac{a_{sp}(T)}{a_{cr}} \tag{4.3.}$$

where $a_{sp}(T)$ represents the value of the spectral acceleration response spectrum of absolute acceleration for 0.10 critical damping) determined to have affected the site of the building. Given some data in the literature (the logarithmic relationship between macroseismic intensity and kinematic parameters of ground motion, logarithmic relationship between the damage degree and the acceleration [3], [11]) it appears to be reasonable to consider also the logarithms of the ratio r,

$$s = \log r = \log \frac{a_{sp}(T)}{a_{cr}} \tag{4.4.}$$

4.2. Studies Carried out and Results Obtained

The correlation analysis was carried out in a sub-sample of 80 buildings, pertaining to the categories A.2 (30 buildings) and A.4 (50 buildings). The sub-sample buildings were selected from a 1 km x 1 km square located North of the central area of Bucharest (it was shown in [1] that 62 such squares were investigated during the post-earthquake survey referred to). The statistical data related to this square were relatively rich and characterized by a higher degree of reliability.

Architectural drawings were procured for these buildings and all available information on the building materials, the erection year, etc., was gathered. The basic data were processed and centralized in forms of direct use in obtaining the elements that are necessary to correlate analytical evaluations with observation data. The information considered for a building consisted of :location, number of stories, type of floor (flexible or rigid),mass, active areas of the bearing walls at the first floor, for each of the main horizontal directions;critical accelerations along the two main directions.

For each building an outline drawing was prepared.The degree of damage, as assessed during the post-earthquake survey,was noted too (the damage degree varied, according to the MSK mathodology, as mentioned in section 2, from 0 to 5).

The estimate of critical accelerations on the basis of relation (4.2) was carried out as follows :

- the active areas of the first-story walls were determined for each of the main horizontal directions ;

- the average normal (compressive) stress was determined for the horizontal section corresponding to the first story ;

-the capable shear stress was determined, considering the influence of the compressive stress, on the basis of a linear relationship.

The capable shear stresses were determined considering the type of mortar (lime) that was usual for buildings built of brick masonry before 1940.

The spectral accelerations were evaluated on the basis of response spectra determined for the strong-motion record obtained at INCERC in 1977[1]. Given the similarity of local conditions between the site of INCERC and the square investigated, it was assumed that the acceleration spectra did not differ significantly. The natural periods considered for the buildings were estimated by means of simplified methods. The potential errors due to this approach are limited.

To illustrate the form, the basic data used and the results of calculations, the data referring to one of the buildings are reproduced in fig.4.2.

The results obtianed are represented in fig.4.3. for buildings of category A.2 and in fig.4.4. for buildings of category A 4. These figures plot the ratio r (4.3.) or its logarithm s(4.4) against the degree of damage estimated during the post-earthquake survey, for each of the sub-sample buildings referred to . The average s and the r.m.s. value were also determined for each of the sub-sample for which the same value of the damage degree d was assessed during the post-earthquake survey.

The results plotted in fig.4.3 and 4.4 show a general tendency toward an increase of the damage degree with the increase of the parameter s. The sensitivity of the damage degree is high. The tendency toward increase is affected by randomness represented by the zig-zagging of the average curve s(d). This is due, primarily, to the limited size of the sub-sample investigated. The r.m.s. is high and so is the corresponding coefficient of variation.

The relatively high scatter of results may be due to factors affecting both the abscissa and the ordinate of the figures 4.3. and 4.4.

The factor s represented in the abscissa raises the following main questions and doubts :

- the uncertainties about the estimate of a_{cr}, created primarily by the lack of direct information on τ_{can};

- the uncertainties about the estimate of $a_{sp}(T)$, created by the transfer of the spectrum from the recording point to the site of the building and by the lack of data on the influence of the non-linear behavior on the equivalent natural period to be considered ;

- the fact that important factors and features of the post-elastic behavior (ductility characteristics, influence of ground compliance etc.) were not considered in the analysis;

- the lack of data about the pre-earthquake building history (effects of the strong 1940 earthquake, effects of the war events, effects of the interventions of man etc.).

The assessed degree of damage raises also some important questions and doubts :

- the possible non-homogeneity of survey techniques, due to the participation of different survey teams at different sample buildings and to the limited qualifications of the members of teams

- the level of significance of damage quantification according to the methodology adopted, [1], for the degree of exhaustion of resistance after the earthquake.

5. SOME COMMENTS ON THE-STATE OF THE-ART

The experience of vulnerability analysis carried out to date makes it possible to discuss several methodological aspects, related both to the characterization and quantification of factors used for defining vulnerability characteristics, as to the methodology of deriving such characteristics, itself. The factors for which characterization and quantification should be discussed are, as mentioned in section 4, the object of analysis, the effects of seismic action and the seismic action itself. The methodology of deriving the characteristics referred to will be considered in relation to the sequence of steps of work, with attention for the input data and for the unknown quantities connected with each of the steps.

Regarding the characterization of buildings dealt with, it is desirable to introduce more complete qualitative characterization, along the guidelines of [12] on the evaluation of exis -

ting buildings and to determine the values of various quantitative characteristics, such as natural vibration periods, critical accelerations, ductility characteristics, ultimate deflections, as well as characteristics of the ground conditions and foundation system.

With respect to the quantification of seismic effects, the two main approaches are represented by the use of measures of observable damage, as was actually done according to the methodology described in this paper, and by the use of a monetary measure (the fraction of replacement cost). None of these measures is perfect. First of all, they do not have a direct significance. The damage degree determined according to an MSK-type methodology does not account for the degree of exhaustion of earthquake resistance of a structure. The fraction of replacement cost does no more account for the actual losses, since the indirect losses due to damage occurence may be many times higher than the fraction of replacement cost (consider the value of costly equipment eventually damaged, the losses due to interruption of industrial output or even the cases of injury or loss of life).

The most reasonable way of improving the system of characterizing and quantifing earthquake effects appears to be that of attempting to evaluate the degree of decrease of the ability of structures to resist earthquakes. This task is particulary difficult and very important research efforts are necessary in order to reach positive results in this direction. Indeed achievements along this way should be of particular importance.

With respect to the characteristics of seismic action, it is important to consider not only a unique intensity for a given ground motion, but also a spectral characteristic. Given the current state-of-the-art, the most reasonable solution is to consider the response spectrum (absolute acceleration). It is important to dispose of a picture of the whole spectrum, in order to make possible, in some cases, the evaluation of effects for higher natural modes and also the effects of lengthening of natural periods under regime of post-elastic deformation. The methodological implication is, in this case, to develop appropriate ways of inference of response spectra at sites where proper accelerographic recodrs are not at hand (which is, practically, the rule).

From the viewpoint of methodologies to be used in deriving the vulnerability characteristics, some specific aspects connected with the specific data and uncertainties must be kept in mind. Some quantifications can be considered here, going along the lines of [4]. Assume that a (scalar) quantification of seismic action is adopted and that the corresponding parameter q are attributed only discrete values (e.g. integer and half-integer MSK intensities), denoted q_j. Consider also, that some distribution of statistical or probabilistic nature must be considered due to some reason to be discussed further on. The discrete probabilities of occurrence of values q_j are denoted p_j.

Assume now that a (scalar) quantification of the damage degree is also adopted and that the corresponding parameter d are attributed only discrete values (e.g. integer values from 0 to 5, according to the scale presented in section 3 of the paper), denoted d_k. Consider here too, that some distribution must be considered and that the discrete probabilities of occurrence of values d_k are denoted p_k.

The discretization of the parameters q and d permits expression of the vulnerability characteristics in terms of conditional distributions. Consider in this connection the conditional discrete probabilities $p_{k/j}$.

The symbols and quantifications introduced permit application to this case of the formula of total probabilities,

$$p_k = \sum_j p_{k/j} p_j \qquad (5.1.)$$

This formula may be used with different purposes. When the vulnerability characteristic $p_{k/j}$ is known (e.g. on the basis of VP-analysis) and the probabilistic distribution p_j is given too (e.g. by means of some assessment on the isoseismals of a future earthquake) the distribution of damage, p_k, may be derived/ predicted from the relation (5.1). When the damage distribution p_k is given by post earthquake surveys and some assessment on the distribution p_j of intensities is adopted too, the vulnerability characteristic $p_{k/j}$ may be derived. When the damage distribution p_k is given and the vulnerability characteristic $p_{k/j}$ is given too for some class of buildings (note that the macroseismic scales include a system of statements that represent, basically, descriptions of $p_{k/j}$ type), a macroseismic analysis, leading to estimate of p_j becomes feasible.

Note here that, when one wants to determine one of the factors of the right member of (5.1), it is not possible to start from a linear algebraic system with a square matrix. To overcome the difficulties raised in this respect, it will be necessary to use, on the basis of (5.1), the condition of minimum error.

$$\sum_k u_k \left(p_k - \sum_j p_{k/j}\, p_j\right)^2 = \text{min.}, \qquad (5.2.)$$

where u_k represent some non-negative weights, or utilities, expressing the degree of confidence. More developments in this respect are given in [3].

The real situation in case of VO-analysis in the following:

- the available data are related only to the distribution p_k, provided by post-earthquake surveys ;

- the assessments on the distribution p_j (which are made in a rough form) are based on some broad statements on the values $p_{k/j}$, for wide categories of buildings and other works;

- the assessments on the vulnerability characteristics $p_{k/j}$ are based on the data on p_k and on the previous assessments on p_j.

The VO-analyses include thus, in principle, a two-step approach which could become tautological. It is most desirable to try to eliminate the bias and uncertainties raised in this context, and the main possibility of achieving some progress in this sense is represented by the use of some additional, different sources for the assessments on the distribution p_j. The radical solution in this view is represented by the availability of instrumental data. This possibility is unfortunately not a realistic prospect for the present and for the near future, given the cost of instrumentation and the differences in ground motions even at closely spaced points. In spite of these dificulties, efforts should be devoted primarily to the development of more refined techniques of retrodiction of the characteristics of ground motions.

6. FINAL REMARKS

1. The task of deriving "V" characteristics for various classes of buildings, as formulated in the framework of the paper was novel at a national scale.

2. The "V" characteristics derived in relation to this paper represent results of obvious interest. Even in this imperfect form, they will be used for various engineering purposes.

3. The results obtained suggest some significant differences in the "V" characteristics of different classes of buildings. Influence of the constructive systems, of the age and degree of engineering, are present in these results.

4. The analysis of "V" characteristics may be of direct use for design engineers, since they indicate some unwanted sensitivities of various constructive systems and may suggest, in this way, the adoption of more suitable design solutions.

5. The "VO" characteristics should be used in relation to the derivation of "VP" characteristics and to risk analysis. It is felt that some specific developments are needed in this relation.

6. The evaluation of results obtained evidences the need for future fundamental work in relation to the development of methodologies of post-earthquake surveys and of derivation, on this basis, of more significant assessments on ground motion characteristics and on the vulnerability of various classes of buildings.

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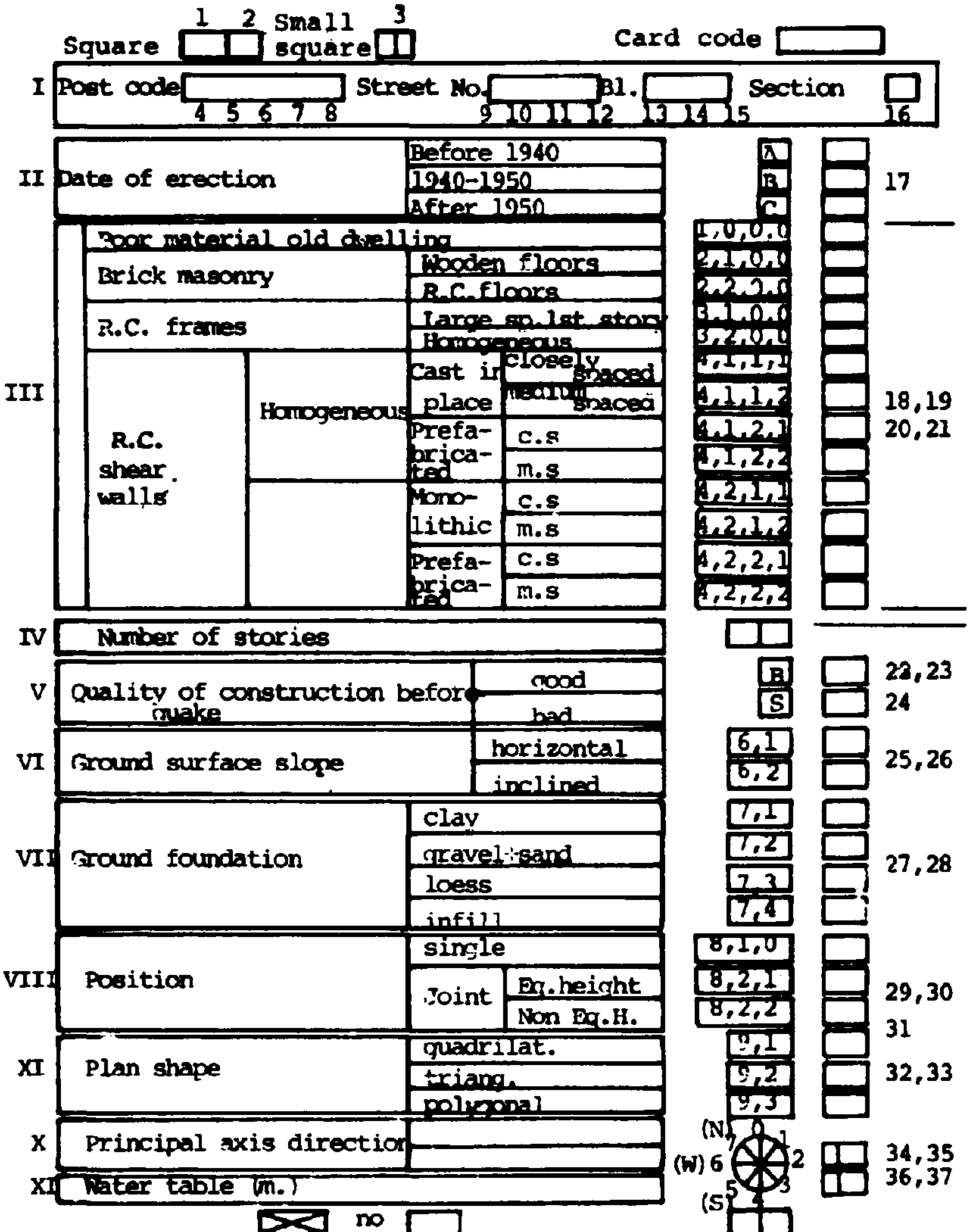


Answer yes
At IV write number of floors
At X write direction (e.g.1.5)

Fig.2.1.a

DESCRIPTION OF DAMAGE

I.38

BRICK MASONRY			01 N.A.
Bearing walls	unaffected		39
	small cracks 1.7		40
	large cracks 2.3		41
	45° cracks 3		42
	got out from v.4		43
	fallen down 5		44
Separation walls	unaffected 0		45
	small cracks 1		46
	large cracks 2		47
	fallen down 3		48
R.C. containing columns	unaffected		49
	small cracks		50
	large cracks		51
	buckled reinf		52
R.C. floors	unaffected		
	small cracks		
	large cracks		
	fallen down		
Timber floors	unaffected		
	affected		
	fallen down		
Attic, Gable	unaffected		53
	cracks		54
	fallen down partially 2		55
	fallen down totally 3		56
Bow-window	unaffecte		57
	small cracks		58
	large cracks		59
	dislocated		60
Chimneys	unaffected	61 62	
	small cracks	63 64	
	large cracks 2	65 66	
	fallen down 3	67 68	

II.38

R.C.FRAMES			
R.C. Columns	unaffected 0.5		39
	small cracks 2		40
	large cracks 4		41
	crush.of.c.4		42
	buckled reinf.		43
R.C. Beams	unaffected 0		44
	small cracks 1.7		45
	large cracks 2.5		46
	failure 3.5		47
R.C. Floors	unaffected		48
	small cracks		49
	large cracks		50
	fallen down		51
Infilling	unaffected 0.5		52
	contour cr.1		53
	small cracks 1.5		54
	large cracks 2		55
	dislocated 2.5		56
	fallen down 3		57
Bow-window	unaffected		58
	small cracks		59
	large cracks		60
	dislocated		61
	fallen down		62
Chimneys	unaffected	63 64	
	small cracks	65 66	
	large cracks	67 68	
	fallen down	69 70	

III.38

R.C.SHEAR WALLS			
Sh. walls	unaffected 0		39
	small cracks 2		40
	large cracks 3		41
	failure 4		42
Lintels	unaffected 0		43
	small cracks 1		44
	large cracks 2		45
	failed 2.5		46
R.C. Columns	unaffected 0		47
	small cracks 2		48
	large cracks 4		49
	crush of c.4		50
R.C. Beams	unaffected 0		51
	small cracks 1.7		52
	large cracks 2.5		53
	failure 3.5		54
Infilling	unaffected 0		55
	contour cr.1		56
	small cracks 1.5		57
	large cracks 2		58
	dislocated 2.5		59
	fallen down 3		60
Elevator case	unaffected		61
	small cracks		62
	large cracks		63
			64

IV.38

	OLD DWELLINGS		01
Walls	unaffected 0		39
	cracks 2		40
	large cracks 3		41
	fallen down 5		42
Chimneys	unaffected	0	43,44
	small cracks	1	45,46
	large cracks	2	47,48
	fallen down	3	49.50

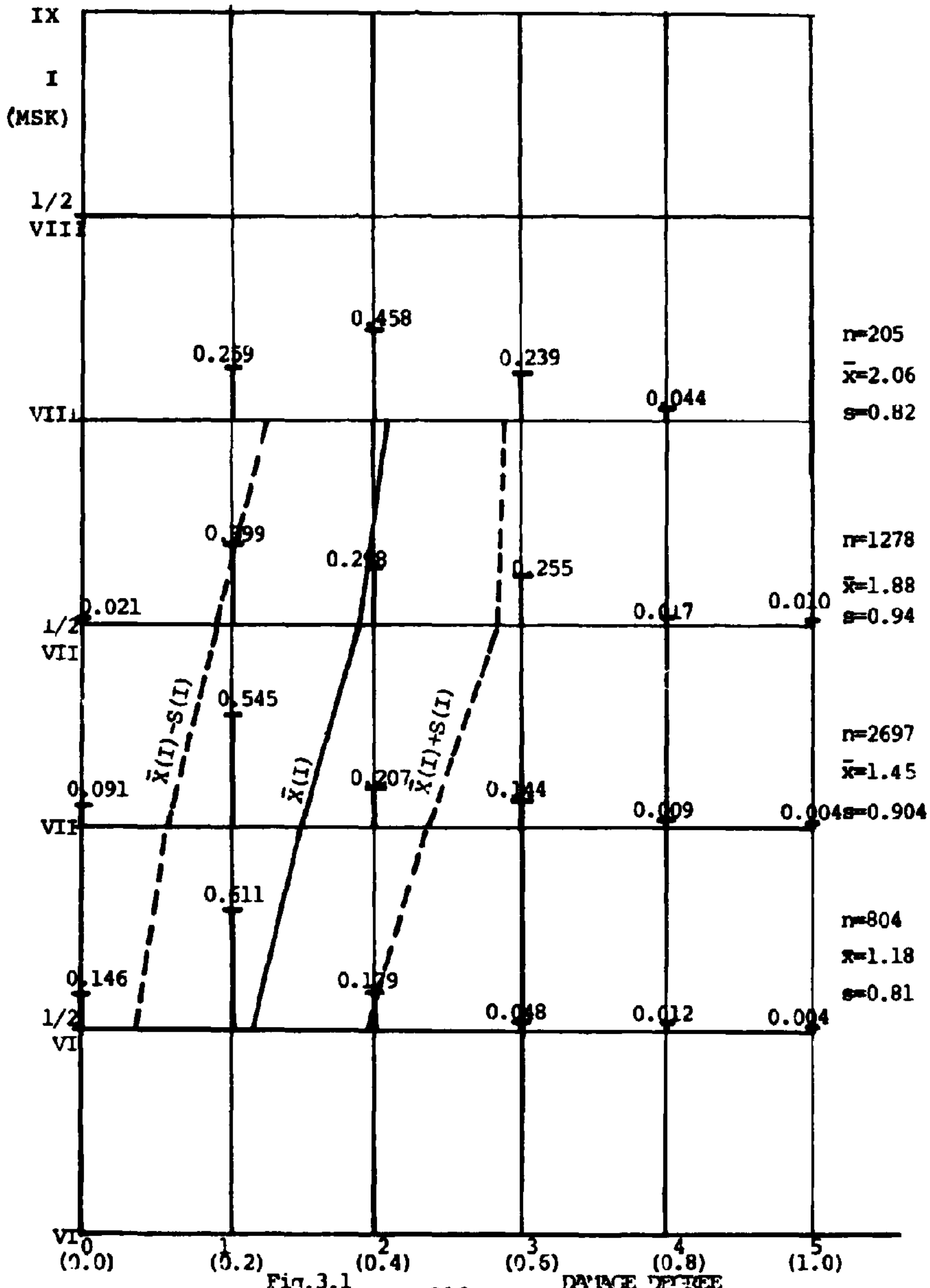
COORDINATES

	71	72	73
x			
y			
	74	75	76

FIELD TEAM MEMBERS 1.------------

2.------------

Fig.2.1.b



IX
I
(MSK)
1/2 VIII
VIII
1/2 VII
VII
1/2 VI
VI
0.259
0.458
0.239
0.044
0.199
0.208
0.255
0.021
0.017
0.010
0.545
0.091
0.207
0.144
0.009
0.004
0.511
0.146
0.179
0.048
0.012
0.004
$\bar{x}(I)-S(I)$
$\bar{x}(I)$
$\bar{x}(I)+S(I)$
n=205
$\bar{x}$=2.06
s=0.82
n=1278
$\bar{x}$=1.88
s=0.94
n=2697
$\bar{x}$=1.45
s=0.904
n=804
$\bar{x}$=1.18
s=0.81
0 (0.0)
1 (0.2)
2 (0.4)
3 (0.6)
4 (0.8)
5 (1.0)
DAMAGE DEGREE

Fig.3.1

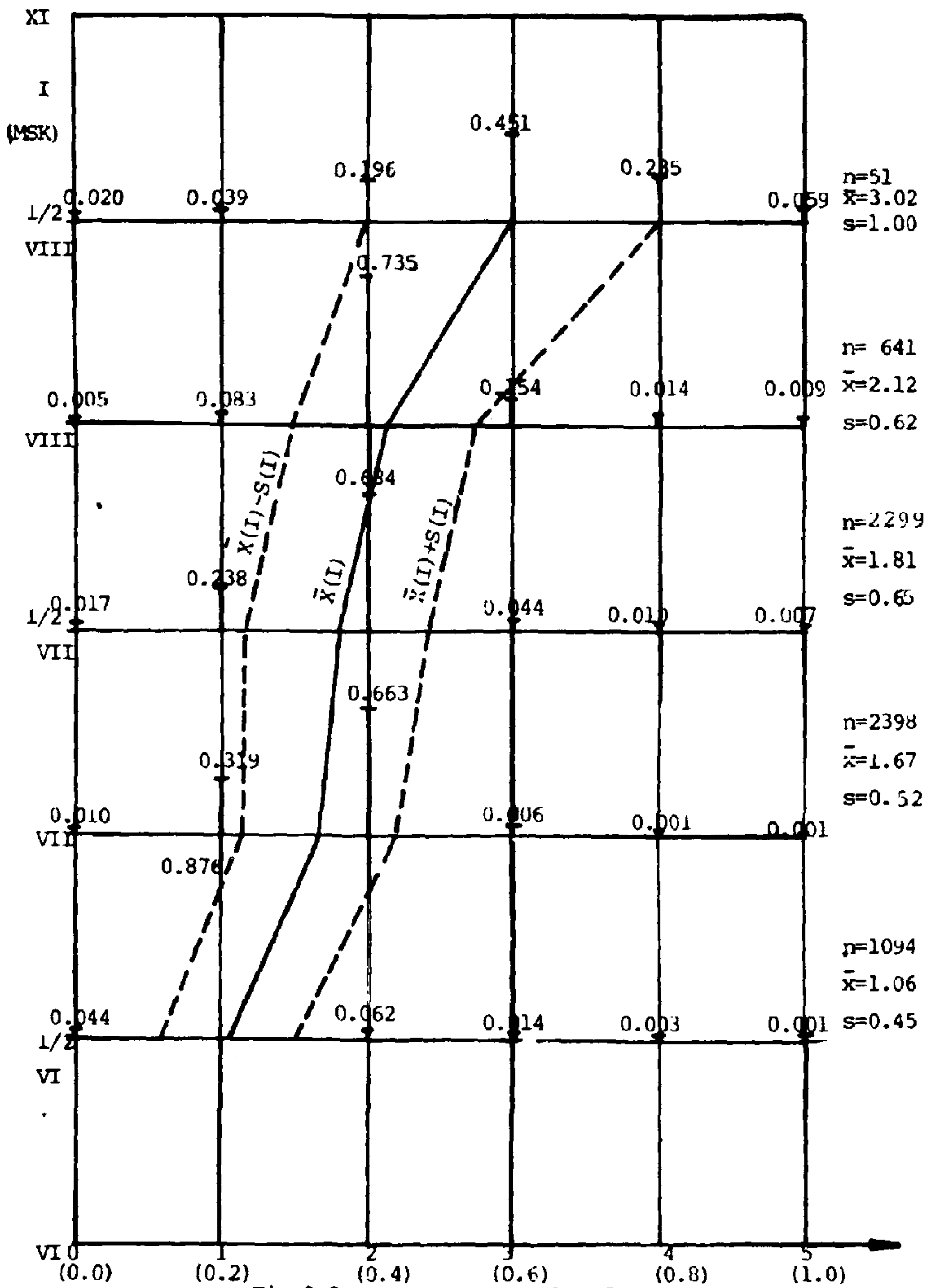

XI
I
(MSK)
1/2
VIII
VIII
1/2
VII
VII
1/2
VI
VI
0.020
0.039
0.196
0.451
0.235
0.059
0.735
0.005
0.083
0.154
0.014
0.009
0.684
0.238
0.017
0.044
0.010
0.007
0.663
0.319
0.010
0.006
0.001
0.001
0.876
0.044
0.062
0.014
0.003
0.001
X(I)-S(I)
X̄(I)
X̄(I)+S(I)
n=51
x̄=3.02
s=1.00
n= 641
x̄=2.12
s=0.62
n=2299
x̄=1.81
s=0.65
n=2398
x̄=1.67
s=0.52
n=1094
x̄=1.06
s=0.45
0
(0.0)
1
(0.2)
2
(0.4)
3
(0.6)
4
(0.8)
5
(1.0)
DAMAGE DEGREE

Fig.3.2

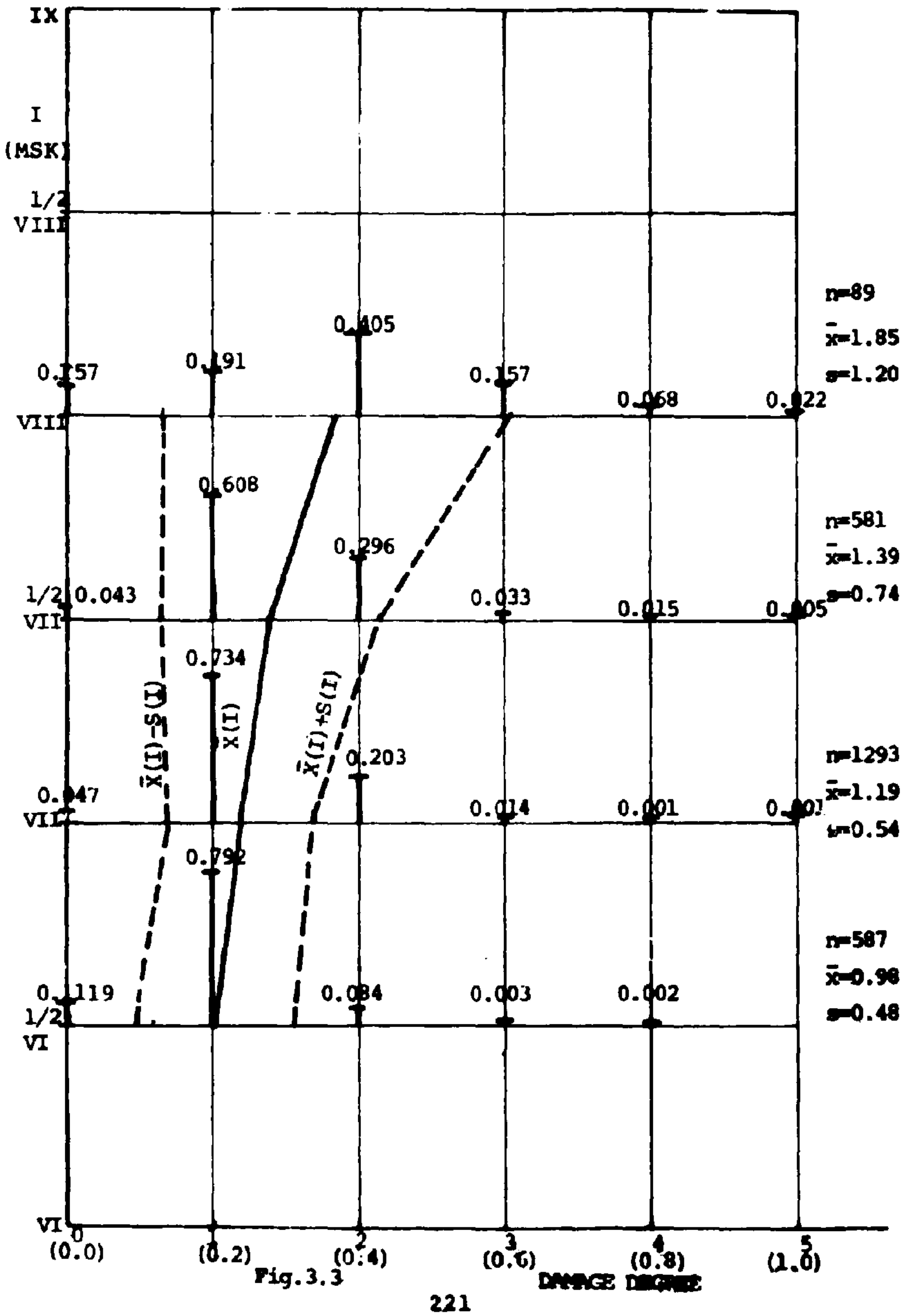
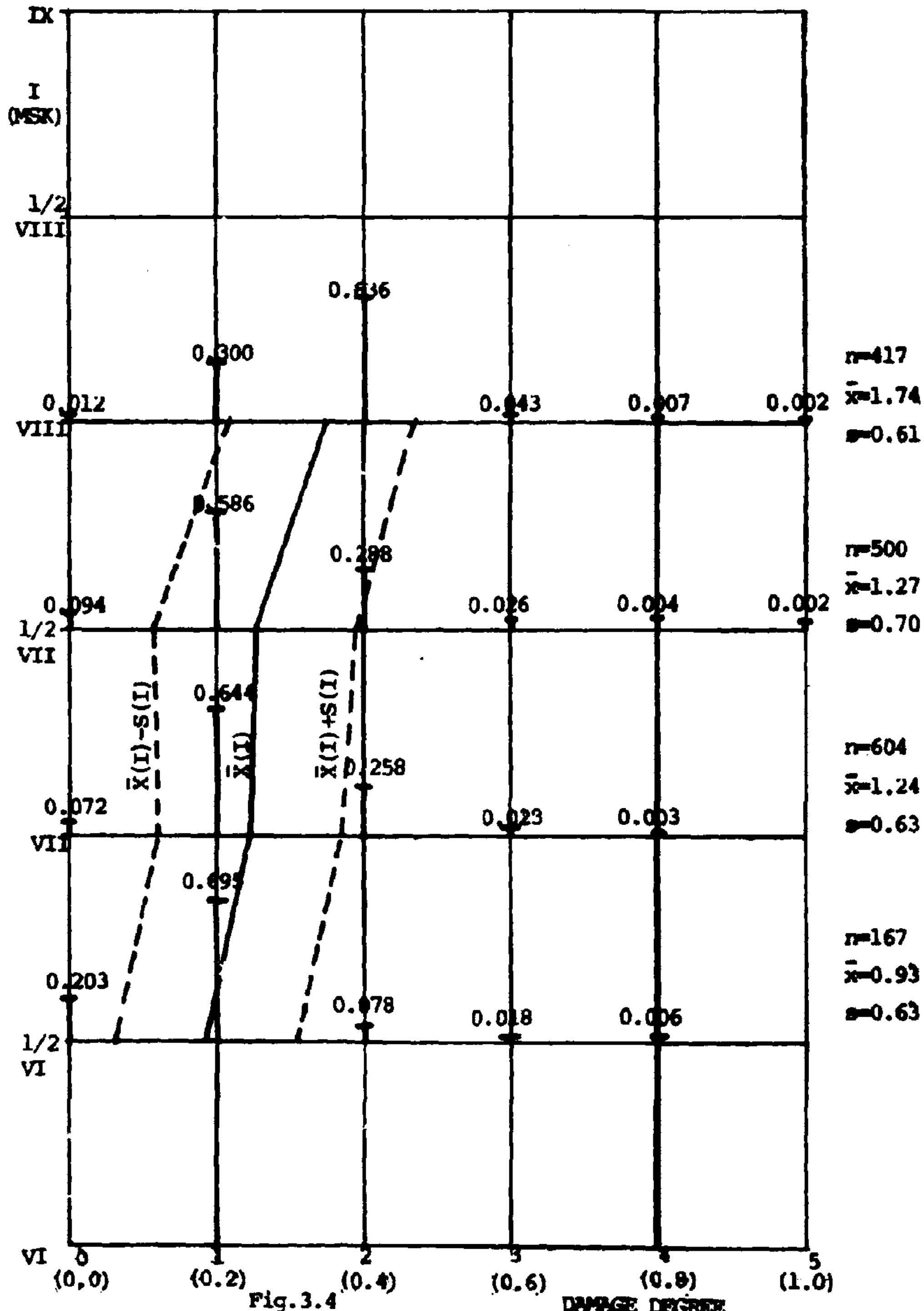
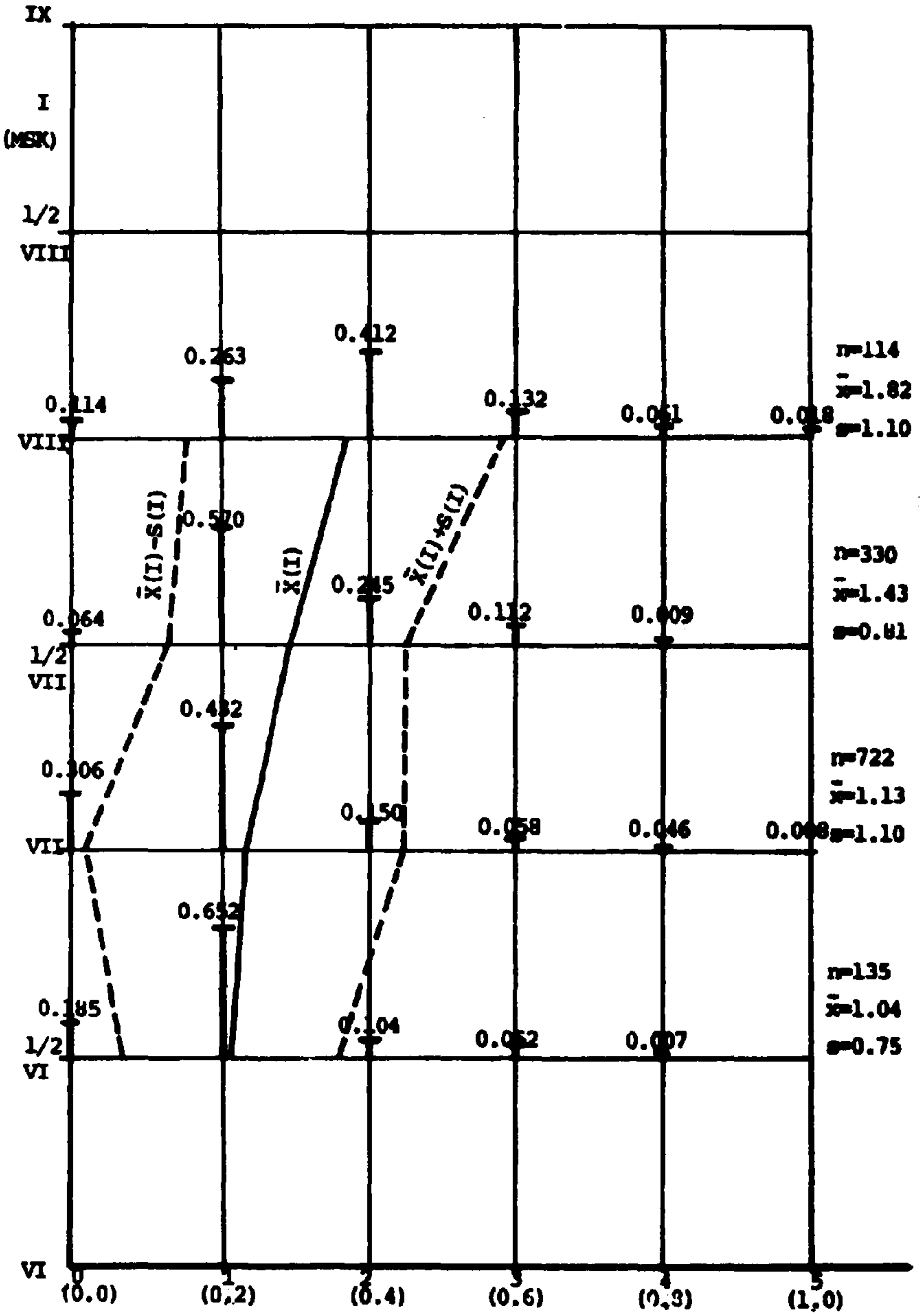

IX
I
(MSK)
1/2
VIII
VIII
1/2
VII
VII
1/2
VI
VI
0.157
0.191
0.405
0.157
0.068
0.022
0.608
0.296
0.043
0.033
0.015
0.005
0.734
0.203
0.047
0.014
0.001
0.001
0.792
0.119
0.084
0.003
0.002
$\bar{X}(I)-S(I)$
$\bar{X}(I)$
$\bar{X}(I)+S(I)$
n=89
$\bar{x}$=1.85
s=1.20
n=581
$\bar{x}$=1.39
s=0.74
n=1293
$\bar{x}$=1.19
s=0.54
n=587
$\bar{x}$=0.98
s=0.48
0
(0.0)
(0.2)
2
(0.4)
3
(0.6)
4
(0.8)
5
(1.0)
DAMAGE DEGREE

Fig.3.3


IX
I
(MSK)
1/2
VIII
0.836
0.300
0.012
0.043
0.007
0.002
VIII
0.586
0.288
0.094
0.026
0.004
0.002
1/2
VII
$\bar{X}(I)-S(I)$
$\bar{X}(I)$
$\bar{X}(I)+S(I)$
0.644
0.258
0.072
0.023
0.003
VII
0.695
0.203
0.078
0.018
0.006
1/2
VI
VI
0
(0,0)
1
(0.2)
2
(0.4)
3
(0.6)
4
(0.8)
5
(1.0)
DAMAGE DEGREE
n=417
$\bar{x}$=1.74
s=0.61
n=500
$\bar{x}$=1.27
s=0.70
n=604
$\bar{x}$=1.24
s=0.63
n=167
$\bar{x}$=0.93
s=0.63
Fig.3.4



IX
I
(MSK)
1/2
VIII
VIII
1/2
VII
VII
1/2
VI
VI
0.412
0.132
0.064
0.306
0.185
n=114
x̄=1.82
s=1.10
n=330
x̄=1.43
s=0.81
n=722
x̄=1.13
s=1.10
n=135
x̄=1.04
s=0.75
X̄(I)-S(I)
X̄(I)
X̄(I)+S(I)
0 (0.0)
1 (0,2)
2 (0.4)
3 (0.6)
4 (0,8)
5 (1,0)

Fig.3.5

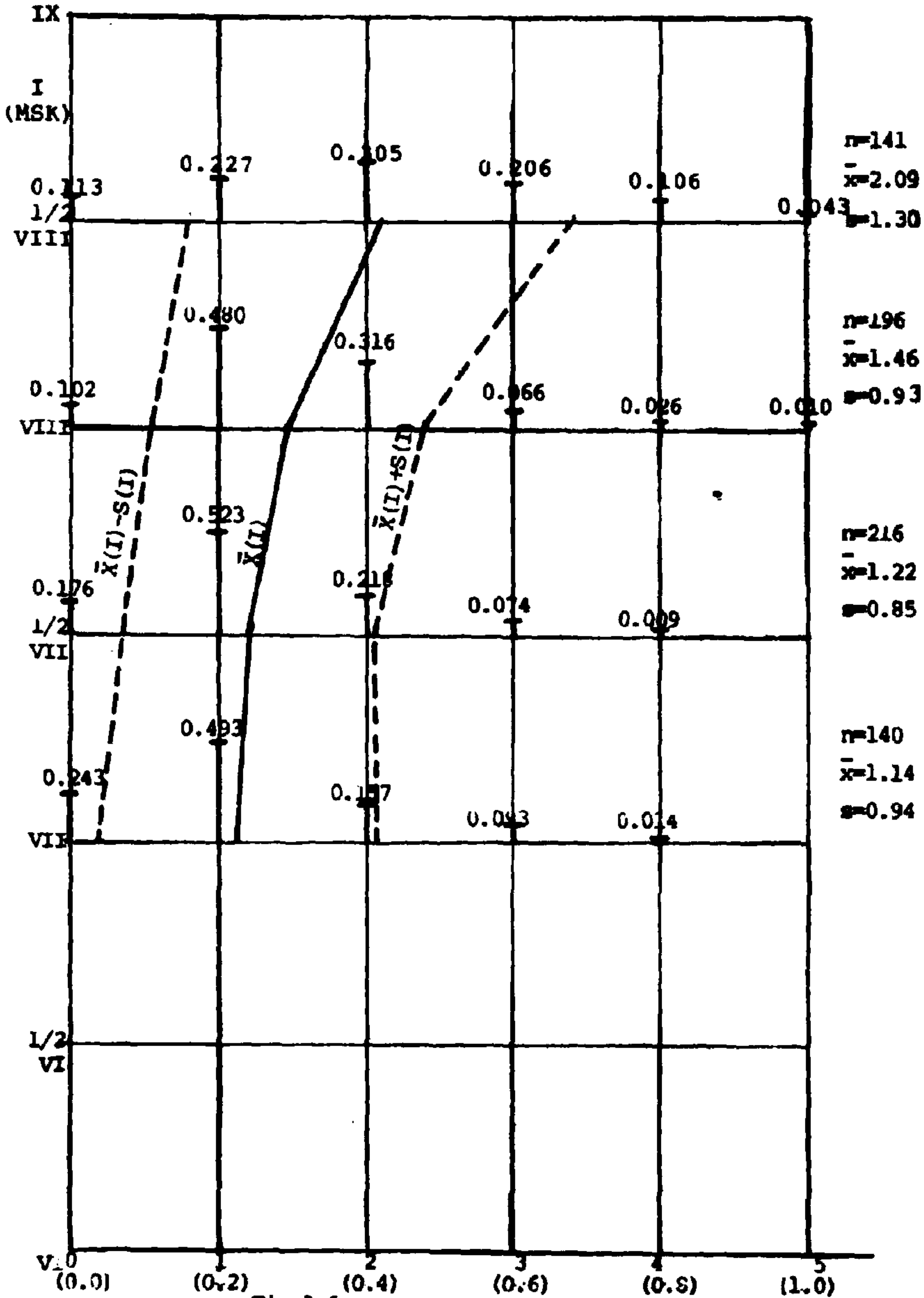

IX
I
(MSK)
1/2
VIII
VIII
1/2
VII
VII
1/2
VI
V
0.113
0.227
0.105
0.206
0.106
0.043
0.480
0.316
0.102
0.066
0.026
0.010
0.523
0.176
0.218
0.074
0.009
0.493
0.243
0.117
0.093
0.014
$\bar{X}(I)-S(I)$
$\bar{X}(I)$
$\bar{X}(I)+S(I)$
n=141
$\bar{x}$=2.09
s=1.30
n=196
$\bar{x}$=1.46
s=0.93
n=216
$\bar{x}$=1.22
s=0.85
n=140
$\bar{x}$=1.14
s=0.94
0
(0.0)
1
(0.2)
2
(0.4)
3
(0.6)
4
(0.8)
5
(1.0)
DAMAGE DEGREE

Fig.3.6

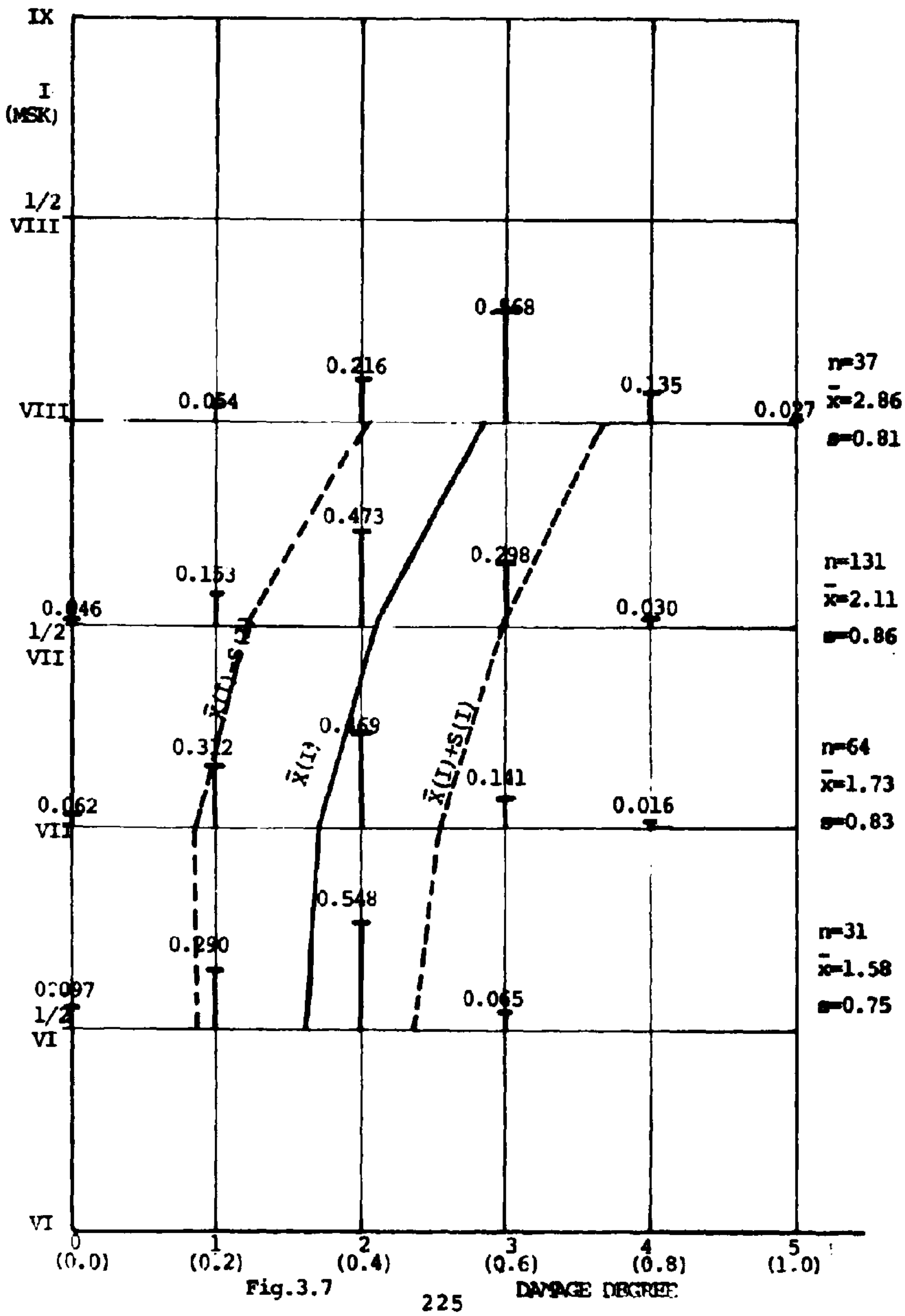
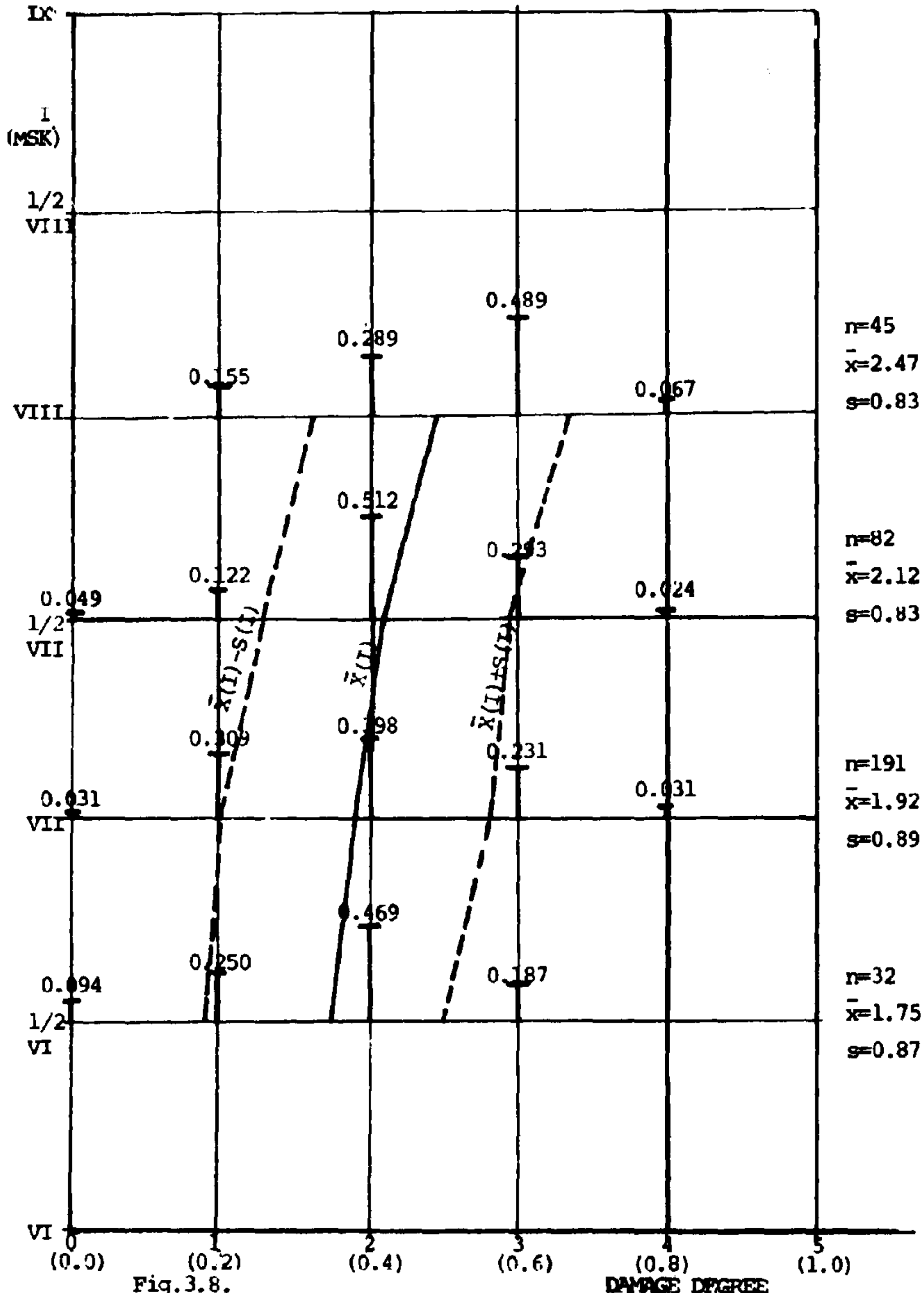

IX
I
(MSK)
1/2
VIII
VIII
1/2
VII
VII
1/2
VI
VI
0.216
0.054
0.135
0.027
0.473
0.298
0.153
0.046
0.030
0.469
0.312
0.141
0.062
0.016
0.548
0.290
0.097
0.065
n=37
x=2.86
s=0.81
n=131
x=2.11
s=0.86
n=64
x=1.73
s=0.83
n=31
x=1.58
s=0.75
0
(0.0)
1
(0.2)
2
(0.4)
3
(0.6)
4
(0.8)
5
(1.0)
DAMAGE DEGREE

Fig.3.7



IX
I
(MSK)
1/2
VIII
VIII
1/2
VII
VII
1/2
VI
VI
0.155
0.289
0.489
0.067
0.512
0.122
0.253
0.049
0.024
0.309
0.198
0.231
0.031
0.031
0.469
0.250
0.094
0.187
$\bar{x}(I)-S(I)$
$\bar{x}(I)$
$\bar{x}(I)+S(I)$
n=45
$\bar{x}$=2.47
s=0.83
n=82
$\bar{x}$=2.12
s=0.83
n=191
$\bar{x}$=1.92
s=0.89
n=32
$\bar{x}$=1.75
s=0.87
0
(0.0)
1
(0.2)
2
(0.4)
3
(0.6)
4
(0.8)
5
(1.0)
DAMAGE DEGREE

Fig.3.8.

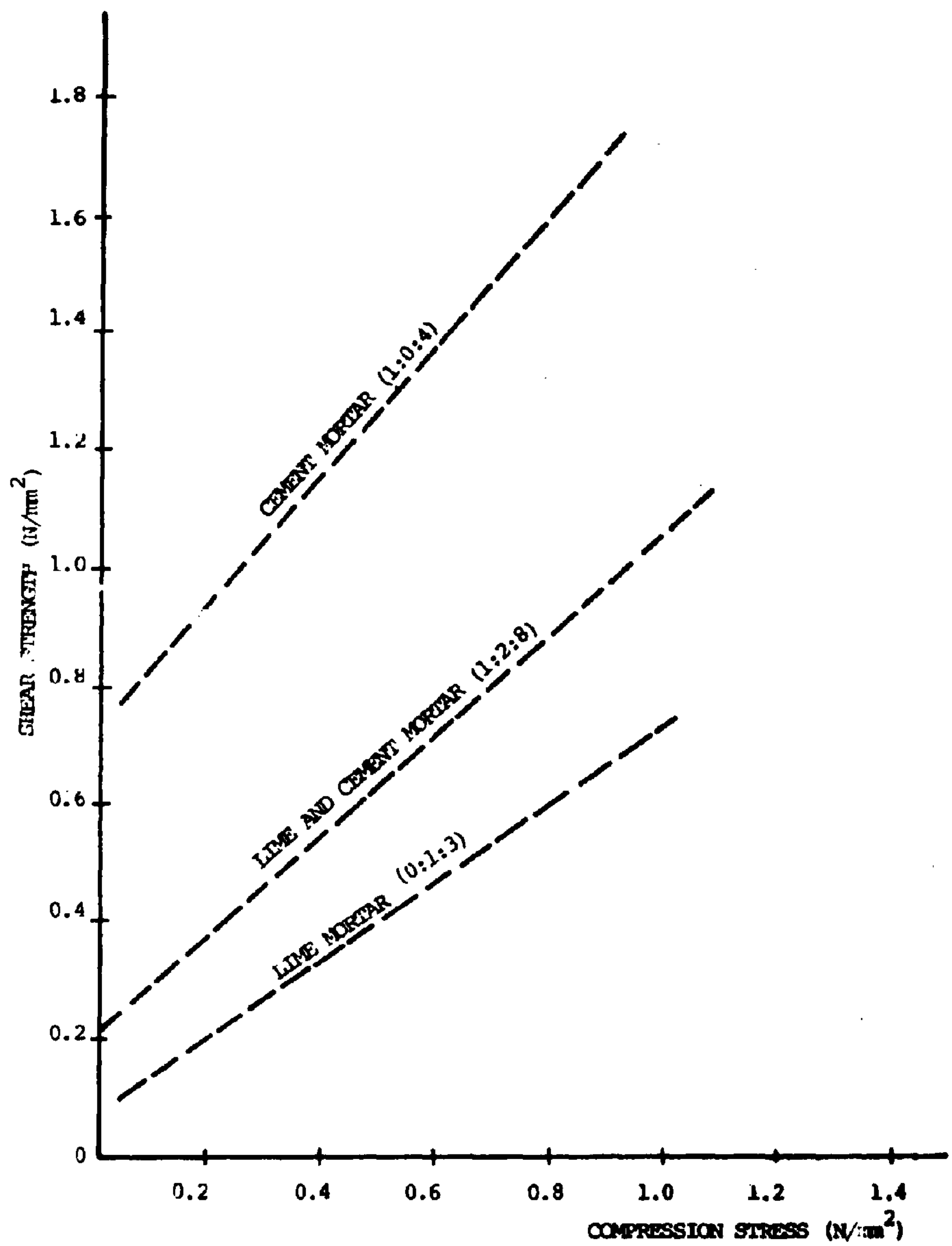


Fig.4.1. SHEAR STRENGTH OF MASONRY VS.COMPRESSION STRESS FOR DIFFERENT QUALITIES OF MORTAR

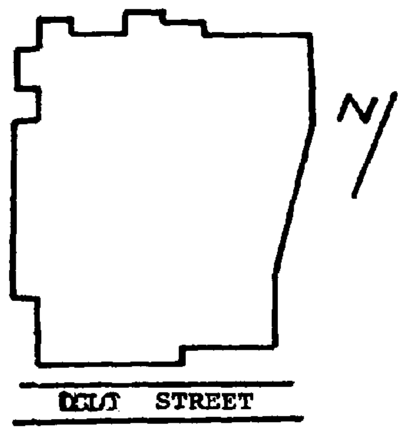
BUILDING ADDRESS	NUMBER OF STOREYS	FLOOR TYPE R:RIGID F:FLEXIBLE	SCHEME OF THE BUILDING	MASS OF THE BUILDING (TONS)	ACTIVE AREA OF THE HORIZONTAL SECTION THROUGH BEARING WALLS (m^2)		CRITICAL ACCELERATION (m/s^2)		OBSERVED DEGREE OF DAMAGE
1	2	3	4	5	6	7	8	9	10
OSLO STREET NR.10	P+4E	R		876,8	6.91	9.33	3.3	4,5	2,3

Fig.4.2.

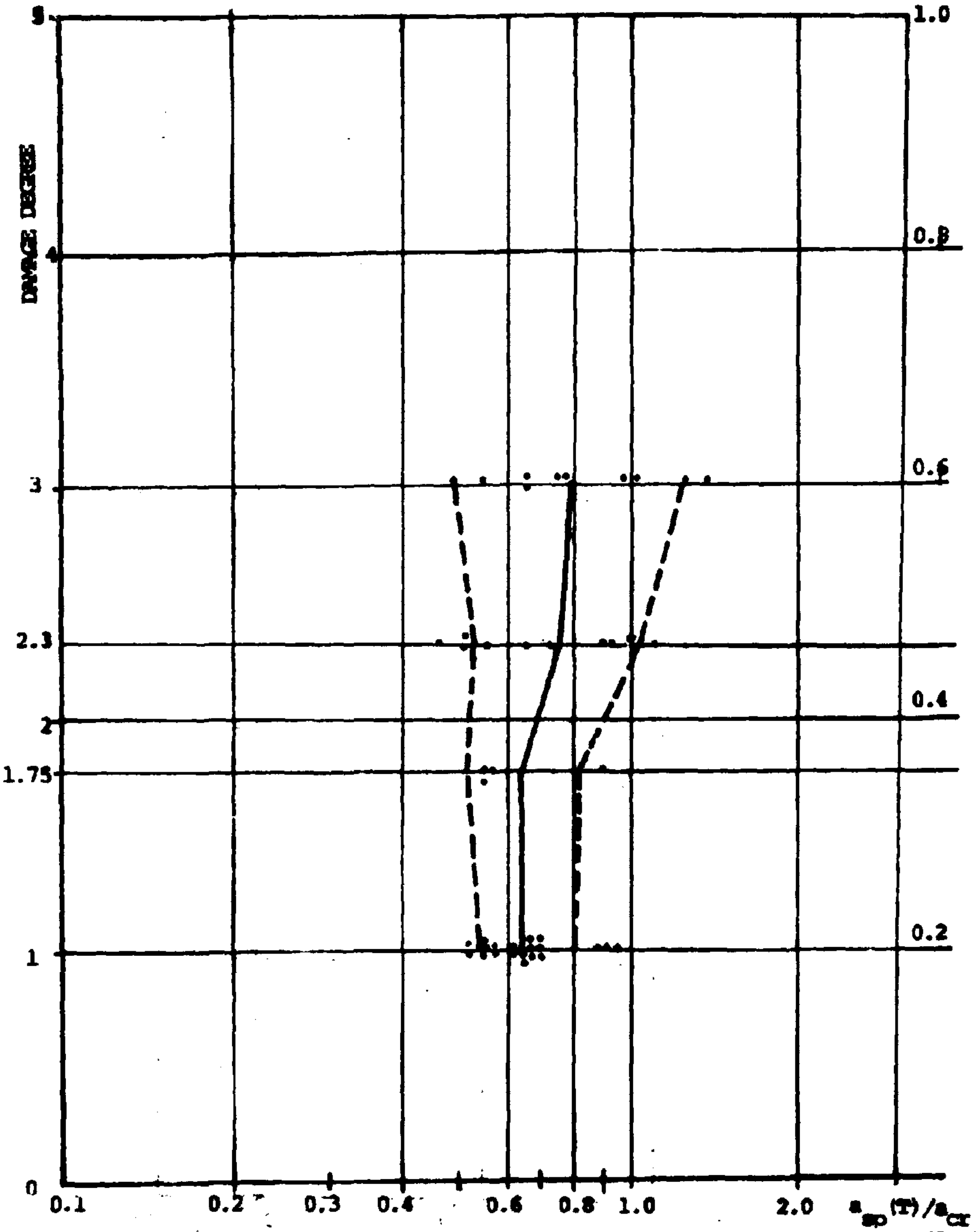


Fig.4.3 CORRELATION BETWEEN THE OBSERVED VULNERABILITY AND THE RATIO a_{sp} (T)/ a_{cr} FOR BEARING MASONRY BUILDINGS (TYPE A_4)

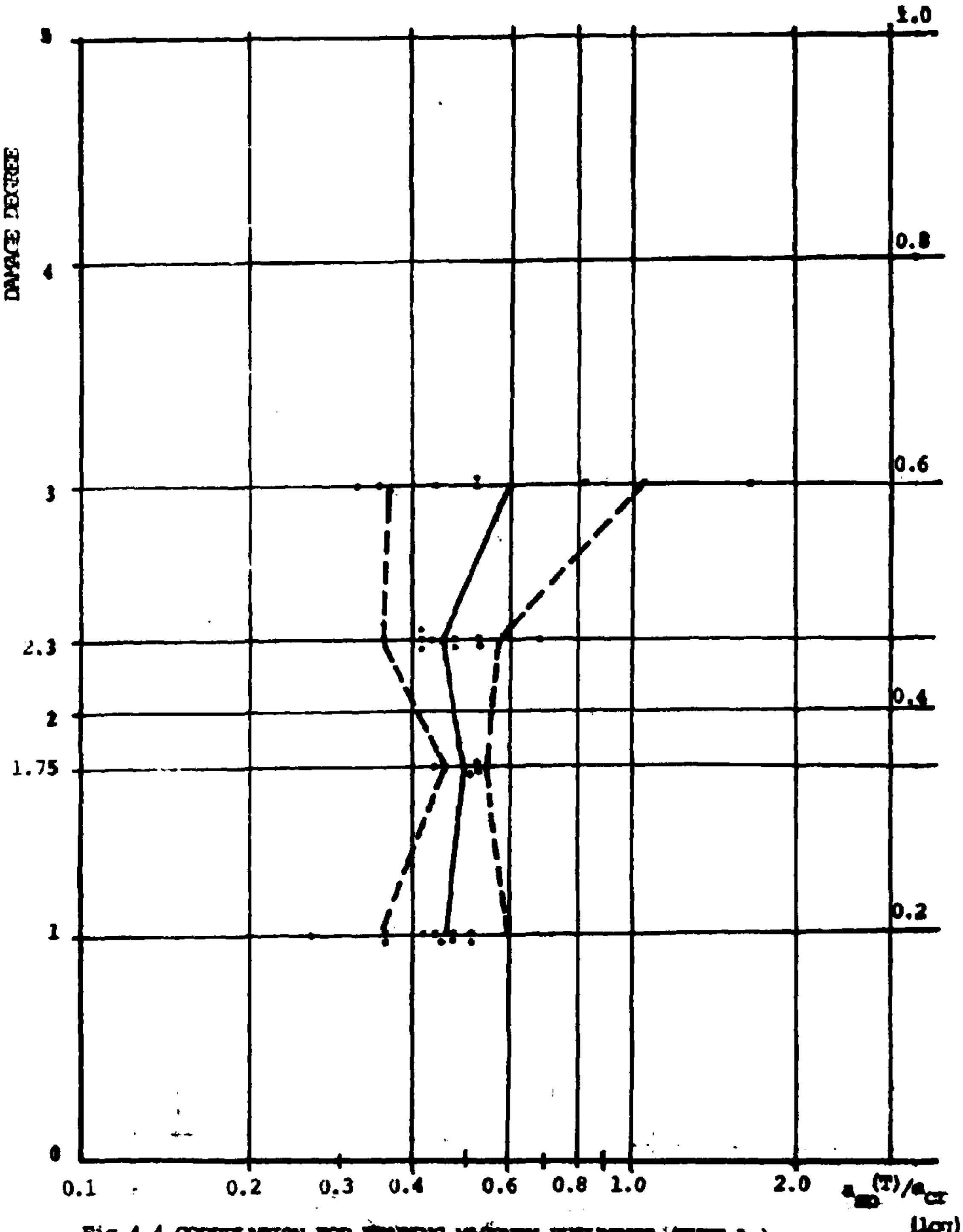


Fig.4.4.CORRELATION FOR BEARING MASONRY BUILDINGS (TYPE A_2)

II.5 STATISTICAL STUDIES FOR THE PREDICTION OF SEISMIC DAMAGE OF REINFORCED CONCRETE STRUCTURES

Cristian Radu Constantinescu x)
Mihail Stancu xx)

The experimental research and studies on the dynamic behavior of existing buildings performed in Romania are more than 20 years old (9).

The first ambient vibration tests of full-scale structures offered an efficient way of studying the linear response of structures and identifying the dynamic properties of new onstructive-system buildings. This research and these studies were of great importance for the permanent improvement of earthquake engineering knowledge and constituted a reference point for the experimental studies carried out after the Vrancea Earthquake of March 1977 (1).

The identification of changes in ambient vibration response between pre- and post-earthquake conditions created the possibility of obtaining data necessary to find a correlation between experimental results and observed damage of buildings.

METHODOLOGY FOR EXPERIMENTAL DATA ANALYSIS AND INTERPRETATION

Giving the importance of adequate behavior of existing buildings, new or old, which were damaged during strong motions, several methods were developed for the evaluation of their seismic resistance. Some are based on analytical procedures, others on ambient vibration testing of full-scale structures excited by wind and microtremor ground motions.

The experimental methods consist of analysis and interpretation of data obtained on existing buildings by means of adequate measuring equipment. These ambient vibration tests assume that the structures can be approximated by damped, linear dynamic systems, and that the excitation is a wide band frequency vibration. The structures amplify those frequencies closed to their natural frequencies. This amplification depends on the

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xx) Senior Scientific Researcher - INCERC, Romania

damping capacity of structures. The natural frequencies of structures can be identified by Fourier analyses of experimental data.

The damping ratio of the soil-structure system, which is of great importance for the dynamic behavior of a building, can be obtained by high level procedures, using the autocorrelation function of structural response (3).

Such experimental studies were performed on 35 beam and column R.C. structures built before 1940.

In order to illustrate the methodology described previously, before the presentation of the statistical study results and the correlation with the observed damage after the earthquake of March 1977, some experimental results obtained on a representative R.C. structure with beams and columns damaged during the earthquake are shown.

Figures 1 and 2 indicate the time response of a building, recorded at the 8^{th} level, on the two main directions of building.

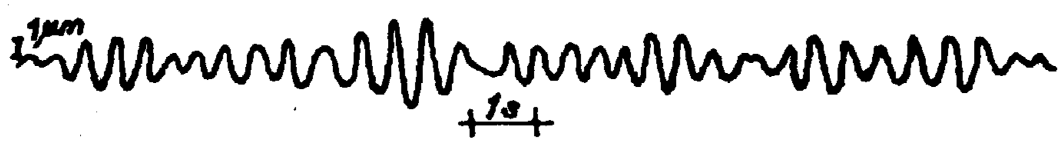

Fig. 1. oscillations of structure on transversal direction

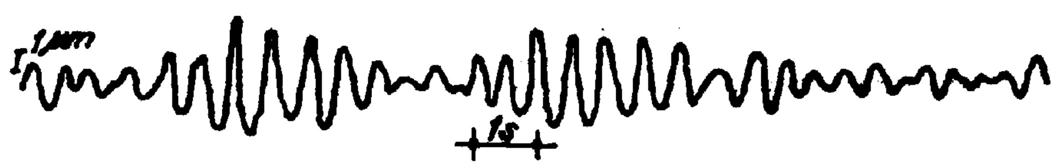

Fig. 2. oscillations of structure on longitudinal direction

Figure 3 indicates the Fourier spectra of oscillations shown in figure 1 and 2 used for identification of translational and torsional natural frequencies.

Figures 4 and 5 indicate the autocorrelation functions of

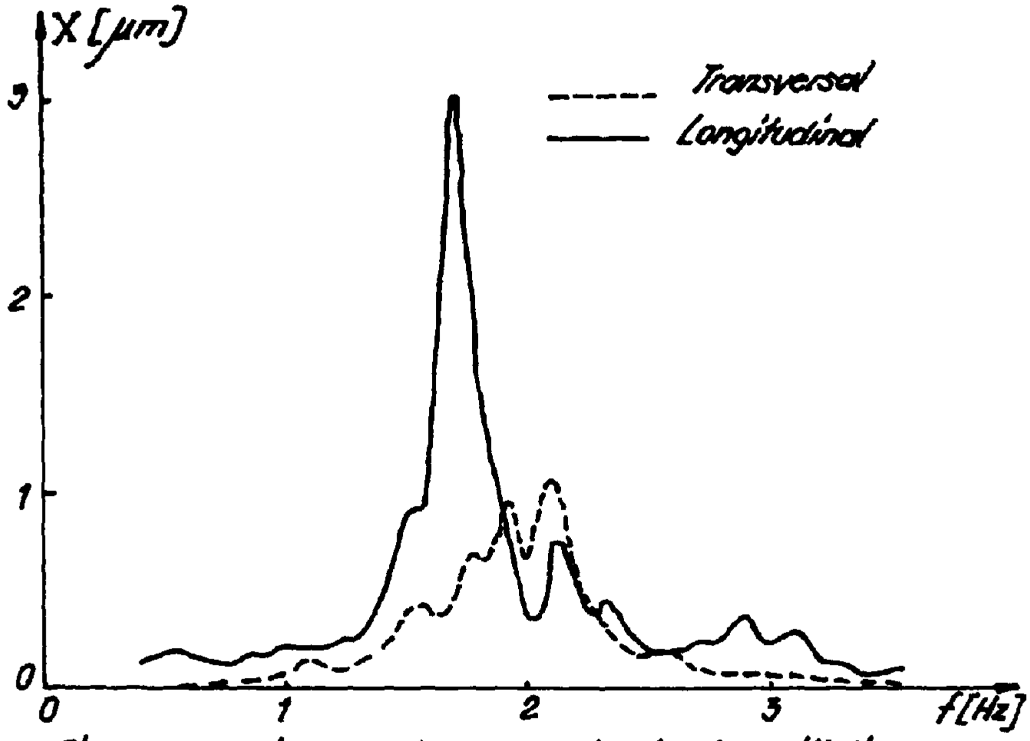

Fig. 3. Fourier spectra of structural oscillations

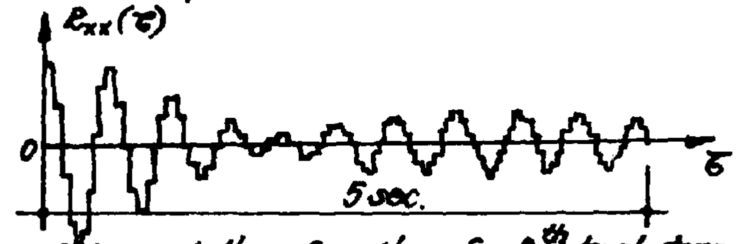

Fig. 4. Autocorrelation function for 8[th] level transversal oscillations

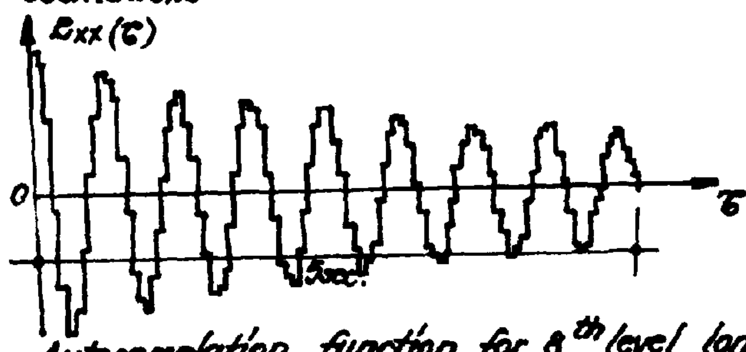

Fig. 5. Autocorrelation function for 8[th] level longitudinal oscillations

oscillations shown in figures 1 and 2 used for identification of damping ratios on the main directions of building.

MATHEMATICAL MODEL USED FOR DATA ANALYSIS AND INTERPRETATION

The experimental studies proved that structural deflection and rocking motion are predominant within the total motion of buildings. The swaying motion is small and can be neglected. In this case, the buildings are better represented by rocking-structural deflection systems as shown in Fig.6.

The equations of motions of the system are:

$$\begin{cases} m\ddot{x} + c_s\dot{x}_2 + k_s x_2 = 0 \\ I\ddot{\theta} + c_f\dot{\theta}_1 + k_f\theta_1 = 0 \end{cases}$$

or

$$\begin{cases} m(h\ddot{\theta}_1 + \ddot{x}_2) + c_s\dot{x}_2 + k_s x_2 = 0 \\ I(\ddot{\theta}_1 + \dfrac{\ddot{x}_2}{h}) + c_f\dot{\theta}_1 + k_f\theta_1 = 0 \end{cases}$$

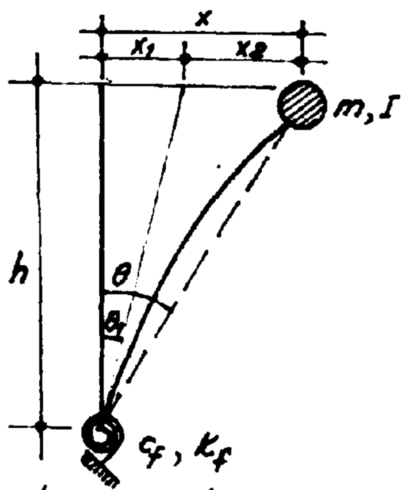

Fig. 6. Rocking- structural deflection system

in which:
- m - mass of structure
- I - moment of inertia of structure
- x - total displacement of the mass
- θ - angular displacement of the mass
- x_1 - displacement due to rocking motion
- x_2 - elastic displacement of structure
- θ_1 - rocking angle at base
- c_s - viscous rigidity of structure
- c_f - viscous rigidity of foundation
- k_s - elastic stiffness of structure
- k_f - elastic stiffness of foundation
- h - height of the mass.

By assuming the forms of solutions as

$$\begin{cases} \theta_1 = Ae^{pt} \\ x_2 = Be^{pt} \end{cases} \quad \text{(where } A, B \text{ constants)}$$

the following frequency equation can be obtained:

$$2(n_s+\gamma n_f)z^3+(4\gamma n_s n_f+\gamma^2+1)z^2+2(\gamma^2 n_s+\gamma n_f)z+\gamma^2=0$$

where $\gamma=\frac{\omega_f}{\omega_s}$

Expressing: $z_{1,2}=\alpha\pm i\beta$

the general solution for the total displacement of structure becomes:

$$x=e^{\alpha\omega_s t}(c_1\cos\beta\omega_s t+c_2\sin\beta\omega_s t)$$

where: c_1, c_2 - constants.

Thus the natural circular frequency of soil-structure system is:

$$\omega=\omega_s\sqrt{\alpha^2+\beta^2}$$

and the damping ratio of soil structure system is:

$$n=-\alpha\frac{\omega_s}{\omega}=-\frac{\alpha}{\sqrt{\alpha^2+\beta^2}}$$

The last two relations show the influence of rocking motion on the dynamic characteristics of structure.

In general, the damping of the structure is very small and the soil-foundation system has large damping. For a combination of n_s = 0.005 and n_f = 0.5 the results are shown in Fig.7.

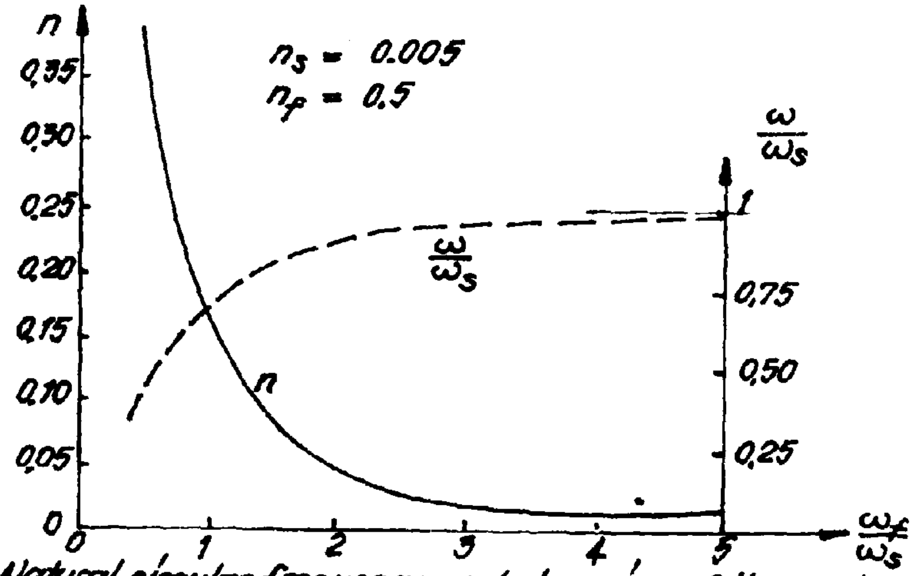

Fig. 7. Natural circular frequency and damping of the system

The mathematical model described offers the possibility of an accurate interpretation of experimental data and a good correlation between experimental data and simplified evaluations of structures.

STATISTICAL STUDIES

Giving the weight of beam and column R.C. structures, built before 1940, in an urban area with buildings, the statistical studies took into consideration all the available data concerning the dynamic characteristics of this type of buildings. The statistical data related to these buildings were relatively rich, homogeneous and characterized by a high degree of reliability. A first category of results is shown in Fig.8.

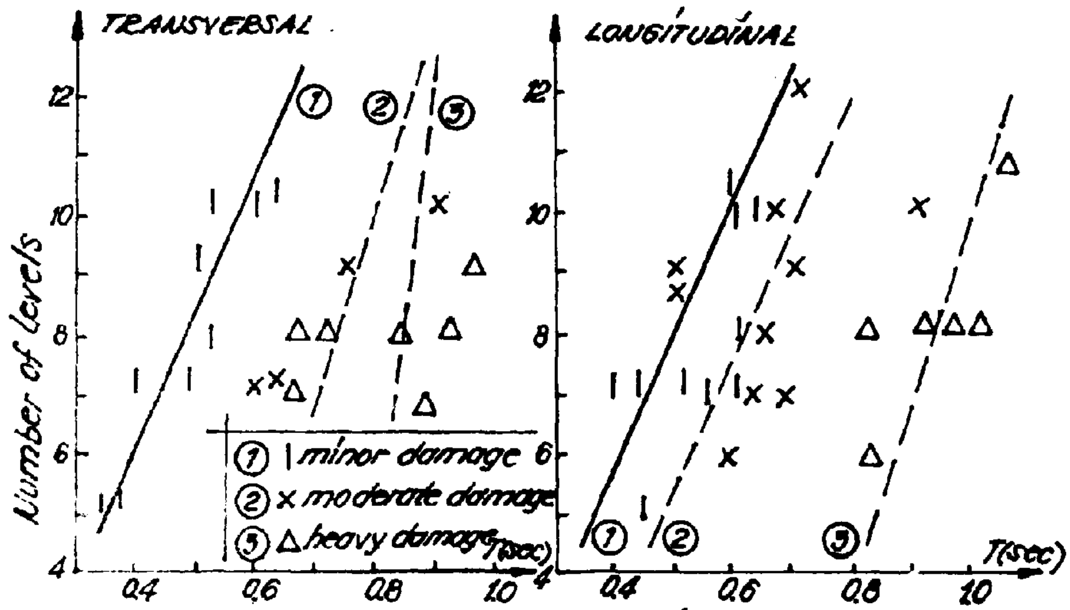

Fig. 8. Correlation between experimental data and observed damage.

The relatively good correlation between the experimental data (fundamental natural periods vs. number of levels) and the degrees of observed damage should be noted. The experimental data were obtained before any strengthening work.

A second category of statistical analysis results is shown in Fig.9. These results are related to experimental studies to crack the efficiency of strengthening works carried out after the 1977 earthquake. The changes in natural periods of buildings on the main directions between pre- and post-strengthening works are presented.

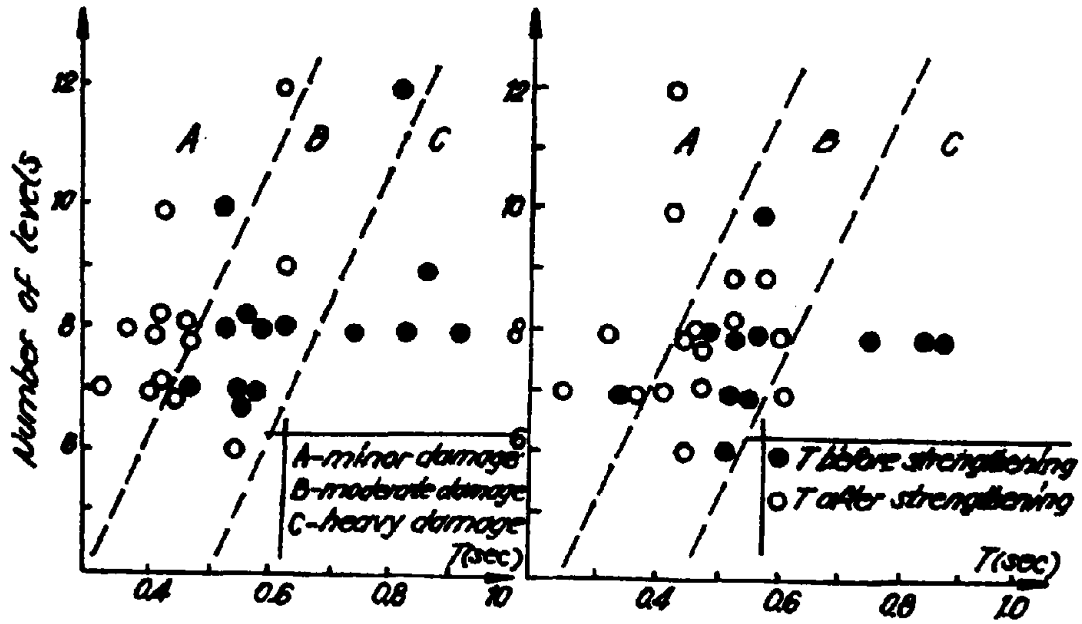

Fig. 9. Experimental data before and after strengthening works

The regression lines for the delimitation of observed damage domains are shown in the same figure. Two important aspects can be observed: the individual effect of change in natural dynamic characteristics due to strengthening works and the possibility of predicting, by means of statistical data, the future seismic behavior of these buildings.

The last aspect is more clearly put into evidence in a third category of statistical analyses on a sample of 42 buildings, others than those analysed previously. For these buildings the experimental studies were performed only after strengthening works. The results are shown in Fig.10. They are in agreement with engineered data concerning the strengthening works.

The analysis of data on damping capacity of structures offers the last category of results presented in this paper.

The damping capacity of the soil-structure system is correlated with the elastic stiffness of the structure (Fig.11). The damping capacity decreases when the elastic stiffness of structure degrades.

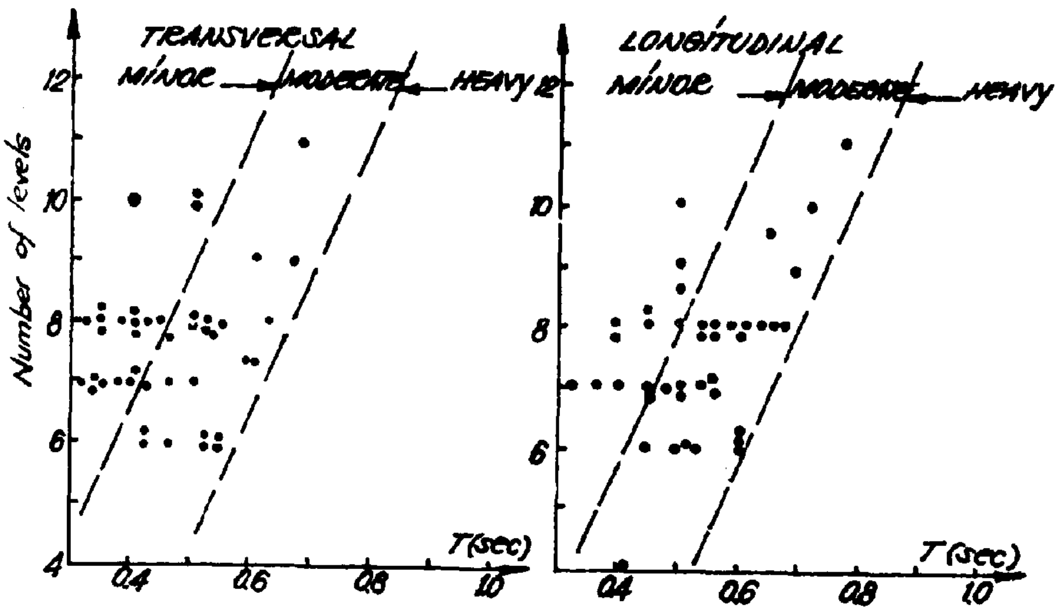

Fig. 10. Predicted degree of damage using experimental data after strengthening

This same aspect was suggested by a theoretical analysis of the simplified model described in Chapter 3 of this paper. It is in contradiction with experimental results obtained on reaction-wall tested structures in which the damping capacity of a structure increases with the decrease of elastic stiffness. This paradox can be explained by the diminution of the soil damping influence due to the decrease of elastic stiffness of structure.

FINAL REMARKS

Experimental methods to predict future seismic behavior of R.C. structures can be efficiently used for buildings with predominantly elastic structural deflection in the total deflection obtained by experimental methods.

This paper described only the main procedures, data analys-

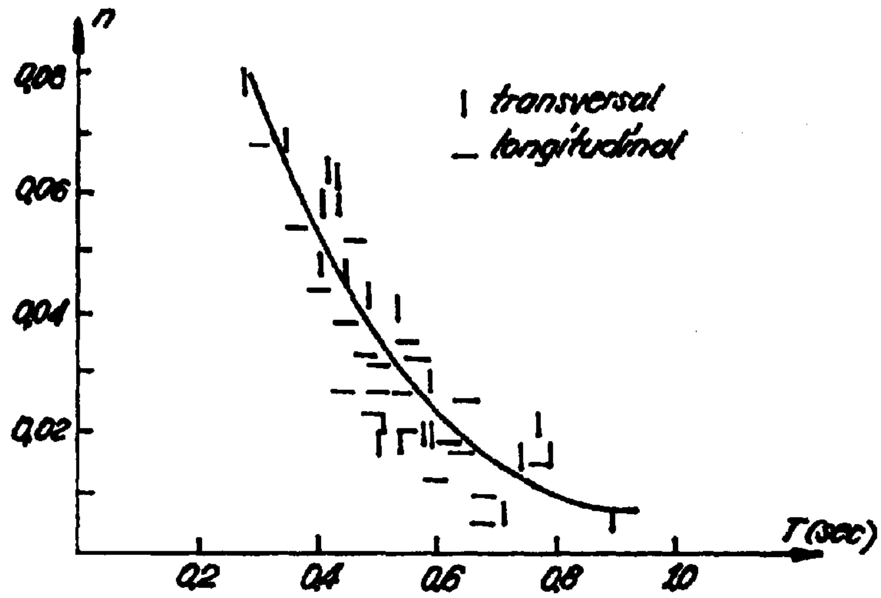

Fig. 11. Damping ratio vs. natural period (experimental data)

is and interpretation. There are also many other procedures for interpretating experimental data (filtering procedures, cross-correlation techniques, etc.) which offer the possibility of identification of some dynamic structural sensibilities (the degradation of floor stiffness, the interaction between two buildings, etc.).

Although it is acceptable only for the structural behavior in elastic range, the method offers important data for the prediction of seismic response of R.C. structures. The information is of great importance especially in cases when experimental data on pre- and post-earthquake dynamic structural characteristics are available.

The comparative evaluation of damping capacities for different types of soil-structure systems offers important data regarding the future seismic behavior of buildings. The damages which occur in structures leads to a decrease of damping capacity of soil-structure systems and, as a result, to an increase of structural stresses.

Another important result of the experimental studies is the good correlation with the mathematical model described in this paper.

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II.6

RESEARCH ON STRENGTHENING METHODS FOR EARTHQUAKE DAMAGED MASONRY

M.Simonici x)

Purpose. A widely used strengthening method for masonry shear walls, simple and relatively moderate in cost, consists of external reinforcement made of welded wire mesh Ø 4 mm/10 cm, bonded to both sides of the wall by means of a 3-4 cm thick layer of hand-applied mortar at 1 : 3 (cement:sand) ratio. The main purpose of the research was to evaluate the effects of this method on strength, stiffness and other characteristics of the strengthened shear walls response. Considering the obtained results, another method was worked out and tested, quite different from the preceding one, in the way reinforcement is distributed and used. Reinforcing is performed on both sides of the shear walls, using reinforced concrete bars with the diameter established by calculation. Vertical reinforcement is composed of structural and repartition reinforcement. The former is discontinous, the bars being concentrated at the ends of the shear walls, or when needed in its span. The structural reinforcement is anchored in the foundation and in the floor slabs. Thus holes are perforated in the floor slabs and structural reinforcement is introduced, being then spliced by welding to the upper floor reinforcement. The holes are then concreted. The vertical repartition reinforcement is evenly distributed and not anchored. Horizontal reinforcement is evenly distributed on the height of the shear wall and is anchored to it. At the basis of the shear wall 2-3 bars close to each other on the vertical should be provided in order to prevent buckling of vertical bars and to confine compressed zones, the distribution pattern of reinforcement is shown in Fig.1. Steel anchor blocks (shown in Fig.1), having the respective bars introduced and fixed, were used to reproduce under laboratory conditions the effects of the reinforcement anchorage in the floor slabs.

Methodology. In the first stage, masonry elements were tested up to ultimate load, then strengthened with welded wire meshes and tested again under the same conditions. The results of unreinforced and strengthened masonry elements tests were then compared. In the second stage, unreinforced masonry elements were again tested up to ultimate load and were then strengthened according to the second method previously described.

The criteria adopted for evaluation of gradual effects of

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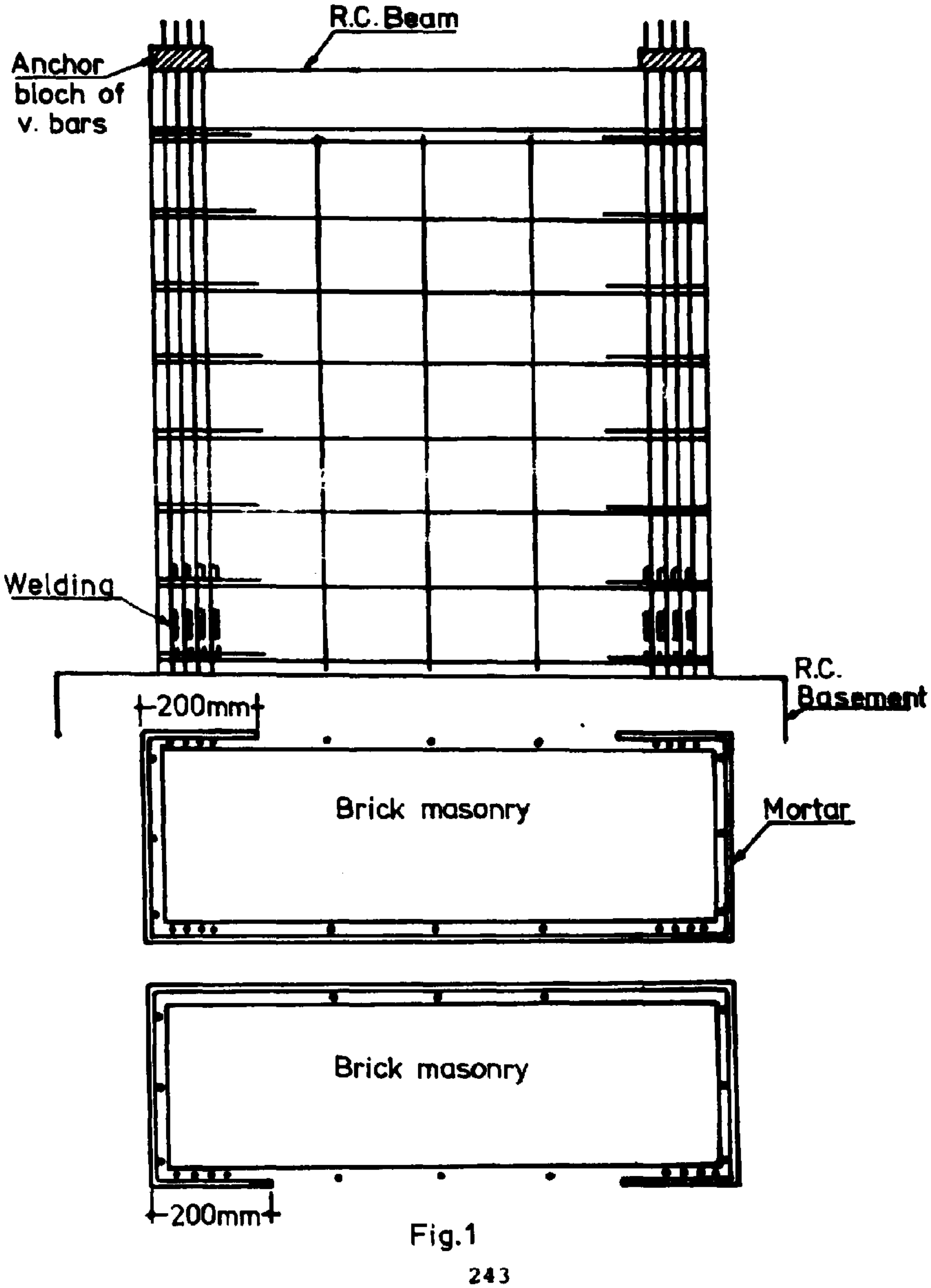


R.C. Beam
Anchor bloch of v. bars
Welding
R.C. Basement
200mm
Brick masonry
Mortar
Brick masonry
200mm

Fig.1

the two strengthening methods were: mode of failure, strength, stiffness and stiffness degradation, ductility, etc.

Experimental results were compared with theoretical ones obtained by applying the calculation method from (20). This allowed the checking of calculation hypotheses, confirming the possibility of using the above mentioned method.

Results of the research

Testing of the welded wire mesh strengthening method

Tests were carried out on 20 masonry specimens, 135cm long; 90 cm high and 29 cm thick (h/l = 0.67). The specimens were erected on a reinforced base beam. At the upper part of the specimens a 15 cm high reinforced concrete beam was cast. Specimens loading was carried out using a 13 t constant vertical load and horizontal displacements applied in alternating cycles with increasing amplitude. Unreinforced masonry specimens were marked with ZS symbol followed by the order number of the specimen. After strengthening, their symbol was changed to ZC. For the evaluation of effects obtained by means of this strengthening method used under site conditions, at certain specimens certain execution specifications were deliberately omitted, such as masonry surface cleaning with wire brushes, joint deepening and cement grout spraying.

The following conclusions can be drawn from the tests performed:

Unreinforced masonry specimens (ZS) failure was produced generally by alternative crushing of compressed ends and, with certain specimens, cracking along the two diagonals followed, as a result of shear force (Fig.2).

Very often, with specimens strengthened with welded wire mesh, mortar layers came off because of poor adhesion to masonry

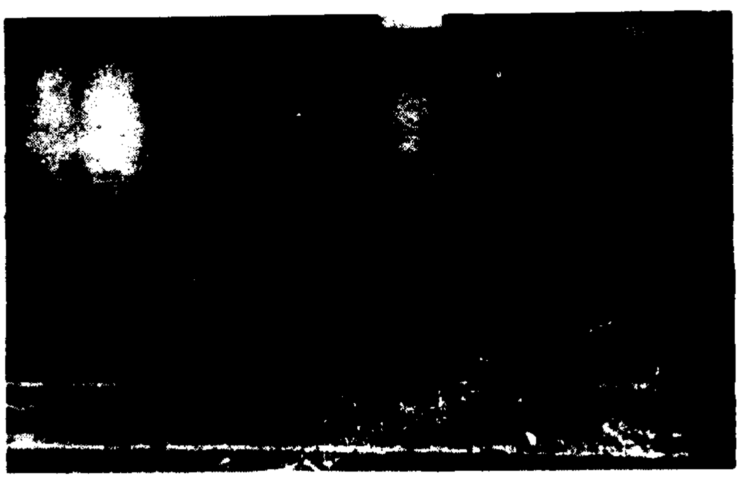
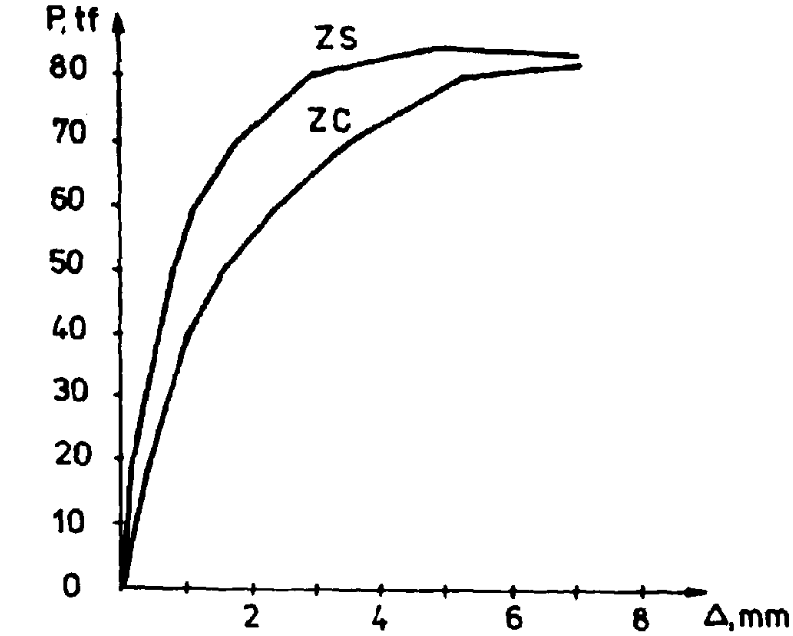
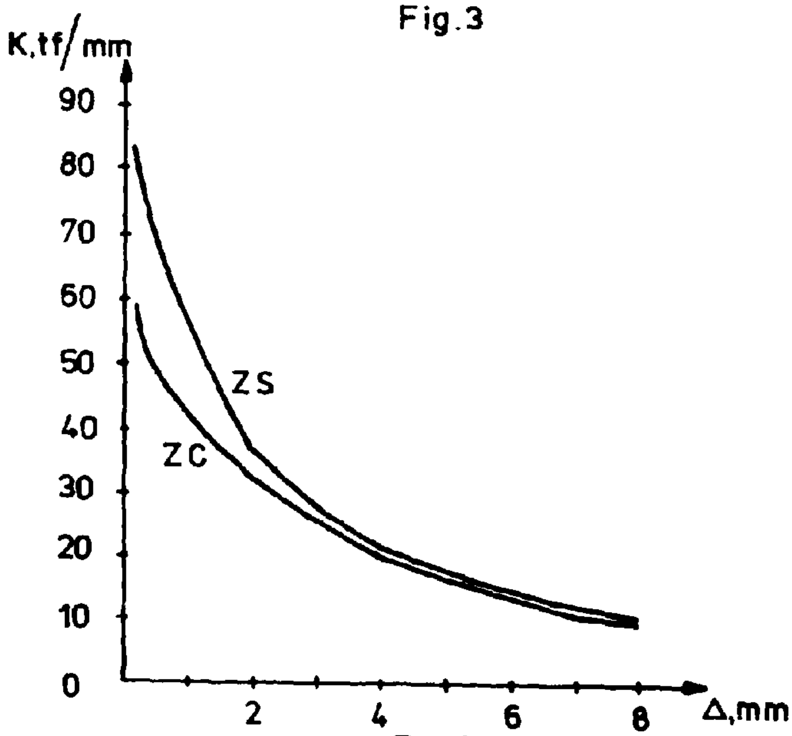


Fig.2


P, tf
80
70
60
50
40
30
20
10
0
ZS
ZC
2
4
6
8
Δ,mm
Fig.3


K, tf/mm
90
80
70
60
50
40
30
20
10
0
ZS
ZC
2
4
6
8
Δ,mm
Fig.4

The strengthening did not influence the failure mechanism. It can then be said that strengthening with welded wire mesh does not increase masonry specimens resistance or stiffness.

Fig.3 shows the average load - deflection hysteretic envelopes for unreinforced masonry specimens (ZS) and for the strengthened ones (ZC), while Fig.4 reproduces stiffness degradation curves for the two groups of specimens. The comparison of the respective curves clearly show that by applying the described technology the damaged masonry can only be repaired partially but not at all strengthened.

Testing of the strengthening method with discountinous reinforcement.

Tests were performed on the same materials, with sizes identical to the previous ones. The specimens were loaded similarly but the vertical load was higher (20 t as compared to 13 t).This explains the difference between P - Δ curves of unreinforced masonry specimens from the two series of testings. Unreinforced masonry specimens were marked MS and the strengthened ones MC.

The conclusions of the tests are the following:

Failure of unreinforced masonry specimens occurred at eccentric compression and was characterized by plastic deformation of the masonry at the lower ends of the specimens alternately compressed (Fig.5).

Fig.5

Fig.6

Fig.7

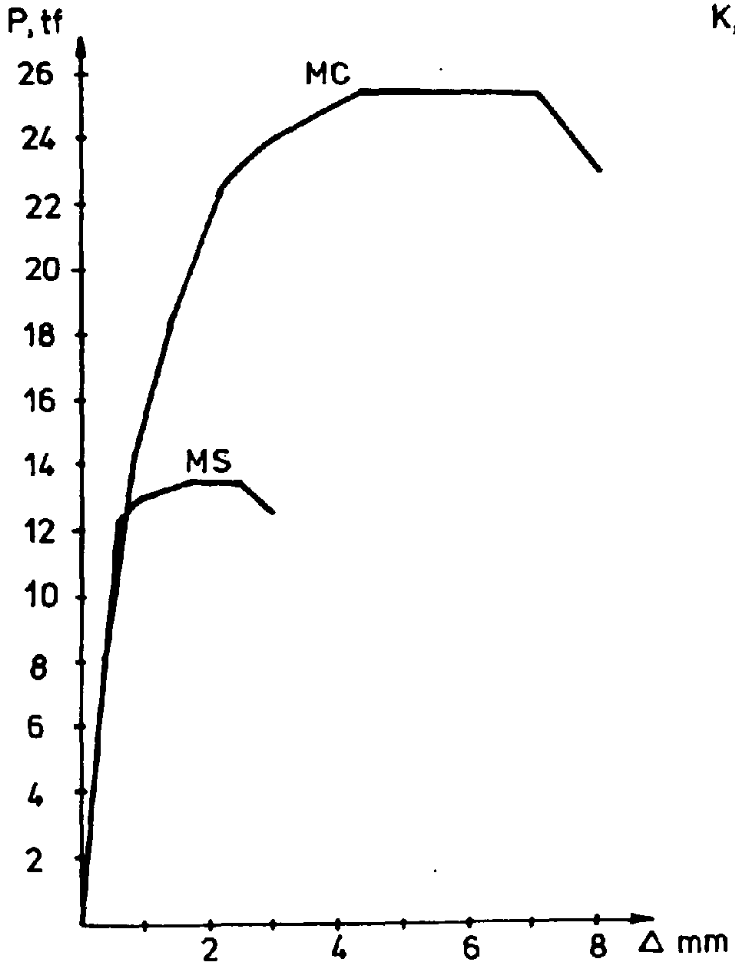

P, tf
26
24
22
20
18
16
14
12
10
8
6
4
2
MC
MS
2
4
6
8
Δ mm

Fig.8

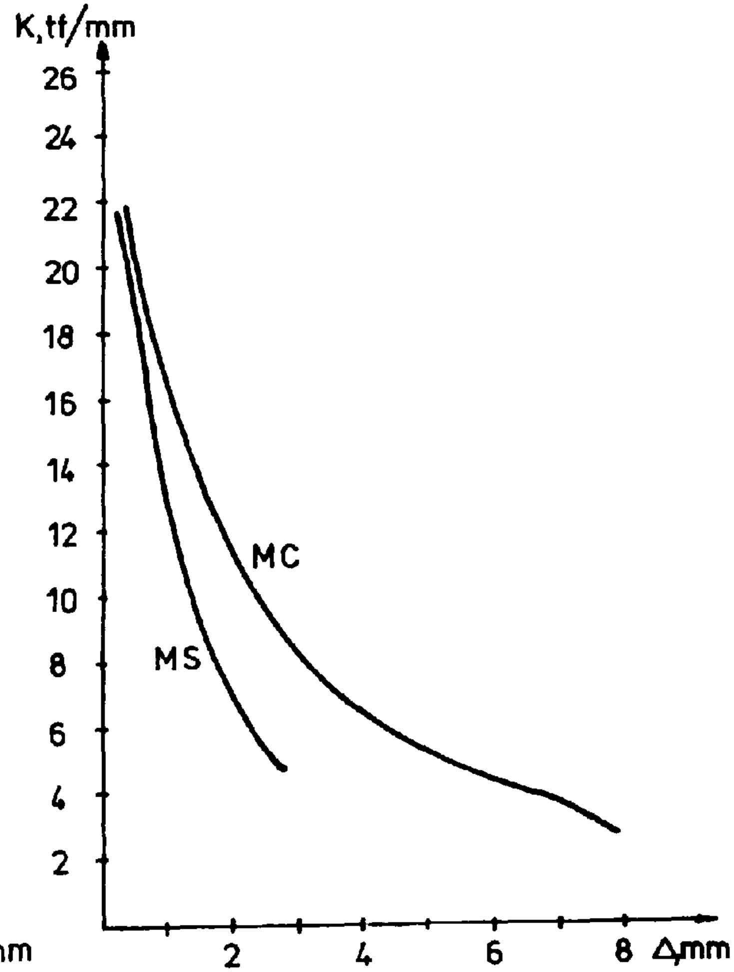

K, tf/mm
26
24
22
20
18
16
14
12
10
8
6
4
2
MC
MS
2
4
6
8
Δ mm

Fig.9

Failure of strengthened specimens occurred due to reinforcement yielding, with plastic deformations lower than for the reinforced masonry (Fig.6,7).

Strengthening effects on masonry resistance, ductility and stiffness are shown in the average hysteretic envelopes and stiffness degradation curves in (Fig.8,9). It is obvious that the tested method doubles the strength of the masonry and increases its stiffness and ductility.

Conclusions

The present paper clearly shows that strengthening with welded wire mesh Ø 4 mm mortar bonded is not a means of strengthening the damaged shear walls but a method of partial restoring the initial characteristics of masonry.

The second method, where the vertical reinforcement is anchored in the foundation and in the floor slabs while the vertical one is mechanically fixed to the masonry, presents the following advantages:

- at low reinforcing ratios and using hand applied mortar, significant increases in strength, stiffness, ductility and energy dissipation capacity is observed;
- by adequate choice of the ratio between vertical and horizontal reinforcement masonry failure occurs at flexure with compression proving thus a ductile behavior;
- comparison experimental results of both unreinforced and strengthened masonry with the analytical calculation obtained by calculation method (20) almost similar values for forces corresponding to cracking, yielding and ultimate stage respectively. Greater differences among displacement values of unreinforced masonry are explained by the conditions under which tests were carried out. Thus the effects of the proposed strengthening method can be controlled and selected by means of calculation.

The mentioned results are conditioned on the adequate application of execution technologies.

It is of utmost importance to provide a good adhesion between mortar and masonry, which can be achieved by adequate masonry cleaning and wetting, joint deepening and cement grount spraying

Research must be continued in order to establish the limits of the reinforcing ratio, elaboration of rational detailing specifications for reinforcement anchoring in the foundation, for strengthening the connections between the shear walls particularly at the corners and cross joints, etc.

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II.7 POSSIBILITIES OF ASSESSING SEISMIC RESISTANCE OF OLD BUILDINGS

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Daniel Diaconu[x]

ABSTRACT

The assessment of resistance capacity of existing old buildings designed and constructed prior to the use of adequate aseismic provisions is a necessary and important step for reducing the global seismic risk in a given area. The problems of identification, assessment and elimination or mitigation of high seismic risk have received increasing attention in recent years.

This paper examinations various possibilities in the earthquake resistance of old buildings consisting of masonry walls, reinforced concrete frames, or a combination of the two and describes a relatively simple approach of estimating the capacity of a structure to withstand earthquake loading. The approach is based on available documentation or field collected data concerning the features of a structural system the quality of materials used, strength and ductility characteristics and other factors influencing the behavior of buildings during earthquake ground motions.

INTRODUCTION

In recent years developments in earthquake engineering have been reflected in substantial changes in seismic design provisions, usually requiring increases in lateral design loads or other corrective actions /1/. As a result a these changes it has become necessary to re-evaluate the existing structures located in seismically active areas, especially when there are doubts their ability to resist the recommended minimum lateral forces. Earthquake codes are legally enforceable rules for the design and construction of new buildings but scarce or rather

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no explicit references are made to the existing stock of old buildings. A most essential requirement of these codes is that they are applicable to all structures, so that the whole population-is protected. A special case, treated in this paper, is that of old buildings constructed before adequate seismic design provisions were developed. In recent damaging earthquake s most material and life losses have been recorded in old buildings made of brick or stone bearing walls inadequately interconnected to floor or roof slabs /2/. Generally, in a given town the seismic risk related to old buildings is much higher than that related to new buildings designed in accordance with adequate aseismic provisions. On the other hand, the seismic risk should be very low for essential facilities and can be relatively high for a purely functional structure such as warehouse Although it is practical nor economically feasible to provide the same level of protection against earthquake for all buildings (existing or planed) in a given town it becomes however, necessary to identify the unsafe structures and to re-evaluate them with respect to their ability to resist seismic forces. These buildings may require some modification or strengthening to minimize the risk of injury or loss of life, although full compliance with the current loading code may be both expensive and technically demanding.

The investigation of factors and basic relations to be considered in a rational approach to the decision on intervention on existing structures was presented by Sandi /3/ . Even if the acceptable level of damage of existing old buildings is greater than that for new buildings the minimum level of protection must be assured and structural collapse prevented in all buildings. The assessment of the earthquake of a structure is a problem of decision - making under uncertainty. The decision refers to the required levels of strength and ductility which must be assigned to the building. The major uncertainties arise from the unknown nature of the earthquake that should be anticipated at a particular site, the interaction of the structure with the ground, and random nature of the parameters modelling the structure. In predictions the earthquake resistance of a n existing building one would like to be able to calculate or at least estimate the probability that the structure will survive the earthquakes it may experience during its lifetime. Perhaps survival with a specified degree of damage is of greater interest than survival alone. On the other hand the investigation of the residual resistance of an existing building measured against current design criteria is of great importance.

Generally in the assessment of the building structures, the accepted level of safety is taken to be that of elements designed exactly in accord with the seismic code in force and the examined building and its individual components are assesed in terms of a γ value where:

γ=(actual ultimate strength)/(required ultimate strength)

For an old building as a whole, γ values of 0.25 or even less are frequently encountered although for individual members values as low as 0.1 are not uncommon. A large variety of methods for evaluating the structural adequacy of existing building may be used. Ideally, appropriate three-dimensional dynamic response analyses for different types and intensities of ground motion would provide the most reliable results. These analyses must account for soil-structure interaction and for the nonlinear behavior of structural elements. Such methods are likely to be considered for application to structures whose survival and integrity during an earthquake are sufficiently important to warrant extra care and accuracy in the evaluation. Therefore, the most desirable and practical method for evaluating earthquake resistance would be one combining simplicity of computation with an accepted level of reliability. In addition the application of engineering judgement is paramount so that the appropriate weight may be given to factors that cannot readily be quantified but which, nevertheless, have a significant effect on the performance of the structure as a whole.

The main purpose of this paper is to present a relatively simple procedure of assessing the aseismic resistance of old buildings constructed of masonry walls or reinforced concrete frames processing some inherent weakness in their main structure and other similar systems. The procedure includes a rapid evaluation technique to analize the structural adequacy of a large number of buildings in a seismically active area and it is not intended for detailed structural analysis of individual buildings.

The basic phylosophy of the paper is similar to that presented by Aoyama /4/ for reinforced concrete structures. The main difference consists of a more explicit consideration of the features of old low-rise buildings of unreinforced masonry walls or reinforced with less steel that is considered adequate to quantify the structure for the term "reinforced". The results of an application of the procedure in evaluating the lateral resistance of 43 buildings affected by the 1977, March 4, Romanian earthquake are also included in this paper.

THE BASIC PRINCIPLE OF THE ASSESSMENT PROCEDURE

Generally the existing old buildings located in seismic areas exhibit deficiencies including: (1) Lack of strength in relation to level of lateral forces; (2) Lack of ductility; (3) Presence of major life safety hazards particularly during moderate earthquakes due to nonstructural components; (4) Irregular plan shape or other conceptual inadequacies; (5) Lack of inte -

grity in detailing; (6) Construction defects.

Within the proposed procedure, the earthquake resistance of an existing structure is evaluated through several indices taking into account the strength and ductility characteris t ics of the used materials, types of structural components and other parameters influencing the behavior of a structure during earthquake motion such as soil-structure interaction, overall seismic adequacy, deterioration in time of initial characteristics. The procedure was developed to allow for a rapid evaluation of a large number of buildings and to provide a basis of compa- rison of relative levels of safety between buildings.

The capacity of earthquake resistance of an individual building may be evaluated by a sequence of procedures which are repeated in successive steps using more refined idealizations of behavior from one step to the next. The first step repre - sents a relatively simple estimate based on limited data re- garding the present condition of the structure. The second and third speps require a more complete description of the struc- ture and a classification of its structural system according to available deformation capacity and ductility at the damage threshold.

The aseismic resistance index I_s of a structure con- sists of the product of four factors related to the strength capacity and ductility under lateral loading, site conditions, structural adequacy and time deterioration of the elements:

$$I_s = R.\ S.\ P.\ D \qquad (1)$$

Vertical members such as load carrying walls, pillars, columns and lift or stair cores are frequently maintained at the same dimensions through the height of the building for architectural, planning or errection reasons, and the level of safety is larger at the upper levels.

Generally the index I_s is evaluated at each story and in each principal plan direction. For those buildings with uni- form stories the evaluation may be limited to the ground floor level. The terms in relation (1) are defined as follows:

R-basic factor of lateral resistance capacity, depending on ultimate horizontal strength, failure mode and ductility of vertical loadbearing elements.

S-site factor representing local geological conditions, soil characteristics and relationship between the dynamic characteristics of the structure and the kind of soil.

P-structural aseismic adequacy factor to consider the overall configuration, the distribution of stiffness and strength and plan or elevation irregularities.

D- time degradation factor to consider modification of original properties due to cracking, chemical or corrosive attack and other agents.

The most substantial part of the computational effort consists of determining the value of factor R, as will be shown below. Some considerations regarding the criteria for evaluating the factors S, P and D have been given in a previous work /2/It may be observed that for unity values assigned to these factors, the index I_s = R. In other words, the last three terms in relation (1) may be considered as corrections of R due to the influence of some structural and site characteristics.In this respect the factor S was introduced, as mentioned, to account for local soil conditions that may attenuate or amplify the structure response under a given seismic excitation, depending on the dynamic characteristics of the structure and underlying ground. A single value of S is assigned for the entire building in all the assessment steps.

The factor P is usually evaluated for each story and each principal direction, excepting the first step evaluation when a single value is adopted for the structure under consideration. Apart from the influence of discontinuities in stiffness distribution and irregularities in structural configuration, this factor may also include the influence of plan dimension ratio of the building, presence of basements of variation of interstory height. The more refined procedures may also include the contribution of eccentricities between rigidity center and mass center at a story as well as the variation of the total weight from one level to another /2/. In a general case, P may be written as a product of terms:

$$P = P_1 \; P_2 \ldots P_i \ldots P_r \tag{2}$$

in which: r is the member of structural parameters considered within the assessment procedure: P_i is the influence degree of i-th parameter and may be written as :

$$P_i = 1 - (1 - g_i)h_i \tag{3}$$

The terms gi and h_i are grading and adjusting factors respectively used in accordance with the situation of each parameter at the level considered. In certain situations P_i (i=1,2...r) may be assigned dirrectly, provided that a unitary criterion is used for all buildings under consideration.

The time degradation factor, D, was introduced to account for the effects of the modification of the original structure characteristics during the life of the building.The following types of damage are considered: (a) cracks due to shrinkage or accidental overloadings; (b) remanent deformation produced by differentiated foundation settlements; (c) deterioration of material quality (d) damages due fires, vibrations or earthquakes; (e) corrosion of reinforcement and other chemical attack. An overage degree of damage is assigned to each story ranging from o (no damage) to 1 (total failure) as may result from a field survey o f the building. The conversion of overage degree of damage to value of D may be performing according to Table 1.

Table 1

AVERAGE DAMAGE DEGREE	0	0,1	0,2	0,3	0,4	0,5	0,6	0,7	0,8	0,9	1,0
D	1	0,98	0,96	0,93	0,90	0,87	0,82	0,76	0,68	0,58	-

ASSESSMENT OF THE BASIC FACTOR OF LATERAL RESISTANCE CAPACITY

Since S, P and D in relation (1) are modification factors of R as described above, some considerations are made here regarding the assessment of the R values. The general form of R is:

$$R = R\,(\beta\ ,\ c,\ \ d) \tag{4}$$

in which β = story coefficient; c ; d = ductility coefficient, For simplicity all vertical structural elements are classified according to their assigned ductility coefficients into three or fewer groups. The minimum value within a group is assumed as the coefficient d of the group and the d value of group 1 should be the smallest ($d_1 < d_2 < d_3$). The ductily coefficient d may vary from 0.8 to 3.0 depending on the type and characteristics of the elements. Since low ductilities are expected in the presence of high shear stress, the ductility factor for unreinforced and inadequately reinforced masonry walls as well as extremely brittle columns is taken to be 0.8. The larger value of 3.0 is assigned to ductile members like a column governed by flexure.

When a structure attains its ultimate strength at the point of failure of the most brittle elements, the value of R is evaluated with the following relation:

$$R = \beta\cdot\left(\alpha_1\cdot c_1 + \alpha_2\cdot c_2 + \alpha_3\cdot c_3\right) \tag{5}$$

in which α_1, α_2 and α_3 are reduction factors allowing for deformation compatibility of vertical structural elements at a given story.

The basic factor of resistance for a structure attaining its ultimate strength at the failure point of the ductile members is:

(6)

The strength coefficient of the i-th group of elements is calculated by dividing the sum of the ultimate shear strength of elements by the total weight of the portion above the floor under inspection:

$$c_j = \sum_{k=1}^{m} T_{jk} \Big/ \sum_{k=1}^{n} G_k = T_j / G^* \; ; \quad (j = 1,2,3) \tag{7}$$

in which n = total number of stories; m= total number of elements included in the j-th group; i= the level under consideration.

In evaluating the strength capacity of an existing masonry building it is necessary to account for the possible modes of failure under combined stresses. The resulting failures may be by shear along critical planes or tension failure incorporating the interaction of the block, mortar and grout. The expected performance of a brick bearing wall construction is usually determined by comparing the maximum stresses calculated in the lower level shear walls, due to the design earthquake forces with the critical strength. On the other hands, ultimatestrength for shear, flexure and compression should be based on the construction technique used and the extent of supervision rather than the strength of the masonry constituents (masonry unit,mortar, grout and reinforcement). It could be argued that sound detailing is even more important to satisfactory seismic performance than the provision of adequate shear and flexural strength.

The terms primary structural system and secondary structural system are used to distinguish between members whose lateral resistance is included in the assessment process and those whose resistance is considered in supporting gravity loads only. Walls smaller in length than 60 cm are not considered in the evaluation of coefficient c_i in relation (7). When the structure is made of brick bearing walls in combination with reinforced concrete pillars and horizontal bands forming an integral part of the bearing walls at roof and intermediate floor levels, the lateral shear force in relation (7) may be evaluated with an approximate relation of the form:

$$T_j = \tau_m (A_w + A_c) \tag{8}$$

in which $\tau_m = \tau_w + 20 \; ; \; (N/cm^2)$

In the case of old buildings made of reinforced concrete frames and masonry infill panels the participation of masonry in resisting seismic forces should also be considered. For the purpose of this study, a simplified failure criterion was used to evaluate the expected performance of the structures. After masonry cracking occurs, the post cracking response of the frame wall system is modelled conservatively using the braced frame mechanism shown in figs. 1 and 2. It is assumed that the cracked

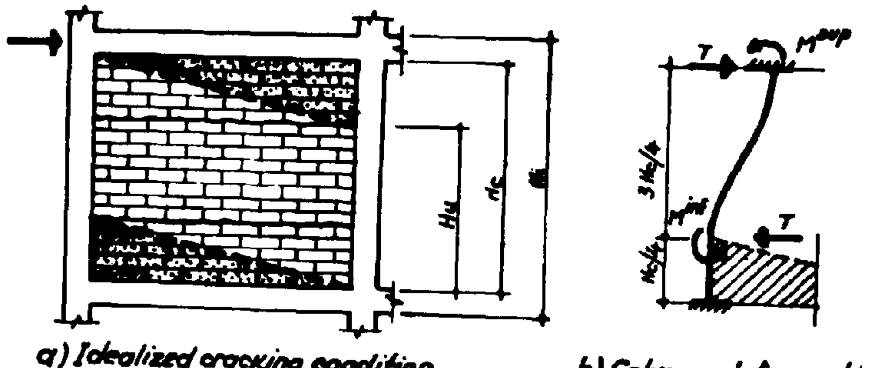

Fig. 1- Simplified model of behaviour for frame-wall system.

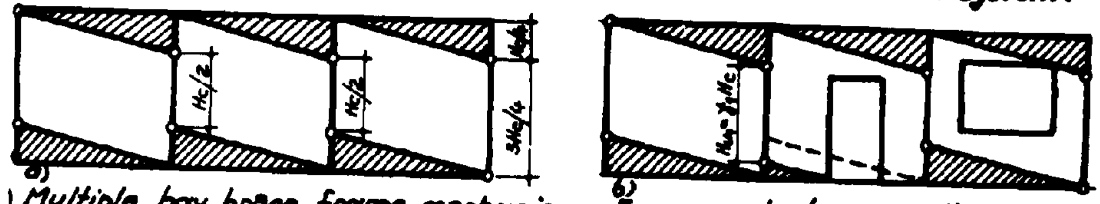

Fig. 2 - Simplified failure mechanism.

masonry forms wedges that brace the lower portion of the tension column and the upper portion of the compression column. Plastic hinges are assumed to develop at the extreme points of the wedges and the unsupported height of the columns. H_u, is less than the column height. The shear capacity of the column is given by:

$$T_u^M = \left(M_u^{upp} + M_u^{bott} \right) / H_u \tag{9}$$

where $H_u = \gamma H_c$; $(\gamma \leq 1)$ is the unsupported height.

The coefficients c_i of reinforced concrete columns are calculated by introducing the minimum value of shear force capacity into relation (7), namely:

$$T = \min \left(T_u^M , T_u^S \right) \tag{10}$$

in which T^S is the shear force at the ultimate shear failure of the column.

When reinforced concrete shear walls are present as resisting members within the structure, the term T corresponding to these members takes the following form:

$$T = \min \left(\phi M_u^w / H^w, T_u^w \right) \tag{11}$$

in which M_u^w is the ultimate bending moment of the wall at the level under consideration, T_u^w is the shear capacity of the wall and ϕ = 2 for a current level, excepting the top level where ϕ = 1, H^w is the total weight of the reinforced concrete wall from the considered level to the top. To calculate flexural and shear strength of the structure component members various relations may be used, similar to these used in the conventional design procedure, or more simple relations based on experimental data. This question, however, is not examined in this paper.

The story coefficient β is calculated as:

$$\beta = \frac{2}{3} \cdot \frac{2n+1}{n+i} \tag{12}$$

This expression may be used for structures exhibiting fundamental mode shope close to a straight line both in the elastic and inelastic range, A more conservative expression is used for types of structures that do not meet the assumption relevant to the linear mode shape:

$$\beta = (1 + n)/(i + n) \tag{13}$$

in which n = total member of stories and i = the examined story.

SOME CONSIDERATIONS REGARDING THE PRACTICAL APPLICATION OF THE PROCEDURE

Usually, a preliminary evaluation may be performed by calculating certain parameters pertinent to structural adequacy and comparing them with reference values resulting from laboratory tests or field observation of earthquake effects upon building structures. For example, a Ω value may be calculated:

$$\Omega = \Sigma L_w / A_f \tag{14}$$

in which ΣL_w = effective corss-sections (cm^2) of the walls at the level under inspection and A_f = floor area (m^2). The following values are considered as satisfactory for the structure adequacy: $\Omega > 2n$ $(cm/m)^2$ for buildings of axis stories or less or $\Omega > n + 6 (cm/m)^2$ for buildings having more than six stories (n is the total member of stories).

A more detailed discussion of those aspects was presented in a previous paper /2/.

In general, the first level procedure should be used when a large number of buildings are to be evaluated by a unitary approach based on an empirical, judgemental procedure considering relevant factors such as possible consequence of collapse for grading the existing buildings. When insufficient data on the structure are available at the time of evaluation, additional field survey may be performed and the values listed in Table 2 may be used together with one of the relations

Table 2

Group	Structural element	d	τ_m (N/cm^2)
1	Brick masonry walls	0.8	30*
	Short columns ($H_c/h < 2.5$)	0.8	150
2	Reinforced concrete shear walls	1.0	100
	Walls with two boundary columns	1.0	300
	Walls with one boundary column	1.0	200
3	Reinforced concrete columns $2.5 \leq H_c/h \leq 6$	1.0	100
	Slender columns $H_c/h > 6$	1.2	60

* Or acording to relation (8)

Table 3

case	R_o	α_1	α_2	α_3
$c_1 = c_2 = 0;\ c_3 \neq 0$	$\beta \cdot \alpha_3 \cdot c_3 \cdot d_3$	–	–	1
$c_1 = 0;\ c_2 \neq 0;\ c_3 \neq 0$	$\beta(\alpha_2 c_2 + \alpha_3 \cdot c_3)\, d_2$	–	1	0.7
$c_1 \neq 0;\ c_2 \neq 0;\ c_3 \neq 0$	$\beta(\alpha_1 c_1 + \alpha_2 c_2 + \alpha_3 c_3)\, d_1$	1	0.7	0.5

in Table 3 for assessing the value of R. The relation (7) takes the form:

$$c_j = \left(\tau_j \sum_{\ell=1}^{m} A_{j\ell}\right) \Big/ \sum_{k=1}^{n} G_k \tag{15}$$

As mentioned earlier, within more refined procedures the resistance capacities of the component members are calculated with appropriate relations to determine the c_j values. The term R are then determined by the relation (5) or (6) depending on the ductility characteristics of predominant structural elements.

The procedure was applied with some particularities described below to the analysis of a total number of 43 old buildings in the city of Jassy, affected by the March 4, 1977, earthquake. The correlation between values of the aseismic resistance index, I_s, and the overage degree of damage was studies and the main results are shown in figs. 3 and 4. Since the data on damages produced by the earthquake were collected during a survey pursuing mainly the evaluation of material losses and not the evaluation of residual strength of the buildings, it was necessary to re-evaluate the damage degree on the basis of the existing data. In some case ad hoc field investigations were carried out together additional data on the structural system and materials used. It should be pointed out that in lieu of the data regarding the structural damage existed in those buildings before earthquake occurrence, the factor D was considered 1 for all buildings analized. In this application of the

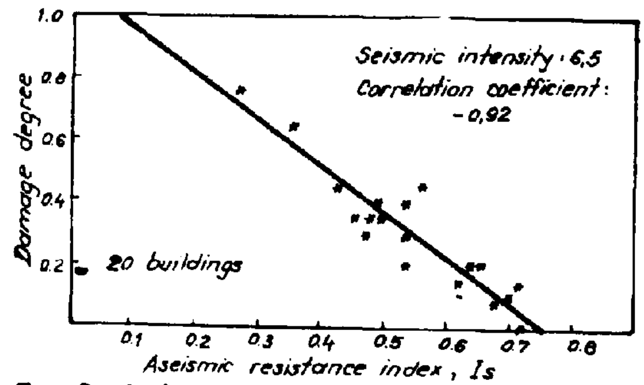

Fig. 3 - Lateral resistance - damage degree relation for buildings at a site of intensity 6.6 on MSK Scale

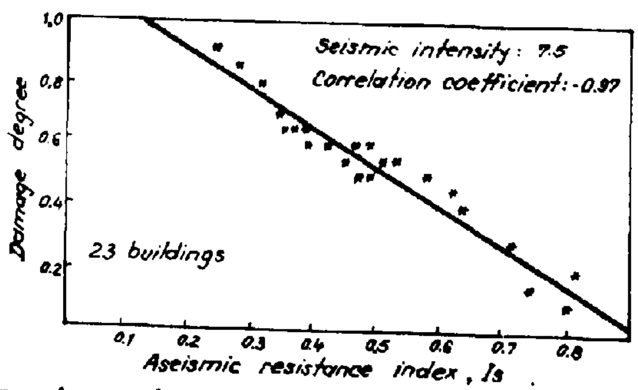

Fig. 4 - Lateral resistance - damage degree relation for buildings at a site of intensity 7.5 on MSK Scale

procedure, the value of T for a particular building was considered as an absolute measure of its earthquake resistance. Therefore, to compare value of I with overage degree of damage it was considered appropriate to clasify the buildings in distinctive groups according the seismic intensity of the site on the MSK scale as resulted from a post-earthquake study. Of the 43 buildings analysed, 20 were located in a zone of intensity 6.5 and the other 23 in a zone of intensity 7.5. While these two groups represented small statistical samples in view of large number of variables involved, the correlation is nevertheless significant, This may be partly due to the fact that this analysis included building for which sufficient data were available or where those data were easy to get. The parameters I_s and R as defined in relations (1) and (4), respectively, may also be used in various engineering investigations on the seismic safety of existing stock of buildings in a given region of high seismicity. Such investigations requires additional data as the importance of a building, its degree of occupancy, vulberability of nonstructural elements and other factors. An empirical procedure was proposed /2/ to assess the earthquake safety level by means of an index A given by the following relation:

$$A = \frac{1}{1+\gamma} \frac{\gamma I_s + I_n}{F_c \cdot F_s} \qquad (16)$$

in which I_n = aseismic resistance index related to nonstructural elements. Past experience has shown that a major life risk particularly during moderate earthquake is due to falling objects from parapets, ornamentations, chimneys and roof mounted equipment. Repair of replacement to eliminate these risks with emphasis on the major occupancy areas, could be carried out with a relatively short period of time and a reasonable cost. F_c is a correction factor that takes into account the structure category and the fact that failure of some structures will have greater consequence than those of other ones. F_c is also a correction factor to consider the influence of site seismic intensity according to zoning map. To take into account the share of structural elements versus nonstructural ones in the overall risk, a weighting factor γ larger than unity was introduced. In a recent study on the vulnerability of building located in the city of Jassy a value of 3 was adopted.

The buildings analyzed by the above methodology are graded on their safety level according to values of A listed in Table 4. The intent of the methodology was to avoid as much as pos -

Table 4

SAFETY INDEX	EARTHQUAKE SAFETY LEVEL				
	INSUFFICIENT	LOW	MODERATE	GOOD	EXCELLENT
A	≤ 0.20	0.20 - 0.35	0.36 - 0.60	0.61 - 1.0	≥ 1.0

sible the variation in grading due to individual judgement /2/ . While the level of safety is not known in absolute terms, the approach provides a basis of comparison of relative levels of safety between buildings of structural or nonstructural characteristics and different locations.

CONCLUSIONS

Of the major problems encountered in reliable assessments of earthquake resistance of existing constructions, the most stringent are the reasonable prediction of the aseismic index I and the specification of the ground motion at particular sites in more accurate terms than thoce prescribed for the region as a whole. This paper describes some possibilities of empirical judgement for assessing the earthquake resistance of old buildings and analyzing their structural adequacy. Although the results of a practical application showed good correlation between avaluated levels of strength and the amount of damage observed during an earthquake, further study is needed to examine in more detail the effects of variation of mainparameters of the assessed levels of strength. In addition some detailed analyses of representative buildings are necessary to calibrate the results and to compare them with the results of the dedign engineering practice.

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II.8 OBSERVATIONS CONCERNING VULNERABILITY AND SEISMIC RISK OF RESIDENTIAL BUILDINGS OF MEDIUM HEIGHT

Constantin Mihai x)

The 1977 Vrancea earthquake affected virtually all old buildings in the city of Jassy. The damage survey concluded that 145 buildings totalling 25,000 sq. m. of area were damaged. Some of them had to be demolished. More than 90% of the affected buildings were constructed before 1940, when seismic design codes were non-existent. These buildings were made of brick (72 %) adobe, trellis work and other unburnt materials. The new buildings erected after 1960 generally behaved satisfactorily during the 1977 earthquake, although important differences regarding the manner and degree of damage of different categories of structures were observed. The recorded degradations did not affect stability of those structures.

More significant degradations were observed in buildings resting upon soils formed from contractile clays having shallow water table, rather than in those buildings resting upon water sensitive soils. Structural systems utilised for those buildings were as follows:

- reinforced concrete shear walls, 36 %;
- large panels, 36 %;
- masonry bearing walls, 15 % and,
- current and lamellar frames 13 %.

Sites of new buildings generally were located on the two levels of the city:

= an upper level having a macroporous water sensitive soil. In that region the foundations were realized upon a compressed ballast bed;

- a lower level where the foundation soil is composed of contractible clay, the underground water table having variable level, up to soil surface.

In both these regions foundations of increasing rigidity were adapted.

BUILDINGS CONSTRUCTED IN SPECIAL SITE CONDITIONS

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In the following, references are made to some buildings having medium height erected in the period 1958 - 1965 in the city of Jassy. During the 1977 Vrancea earthquake these buildings underwent some degradations. Three types of structures erected on the lower level of the city have been selected for this investigation:

- an apartment block located on the left bank of Bahlui river designated as "B" (fig.1) consisting of reinforced concrete shear walls.
- an apartment block consisting of reinforced concrete frames, placed in front of Nicolina rail station of Jassy, designated as "N" (fig.2);
- an apartment block consisting of shear walls and columns placed near "Tesătura" Factory, designated as "H" (fig.3).

Some fundamental structural elements of these buildings are presented in the above mentioned figures.

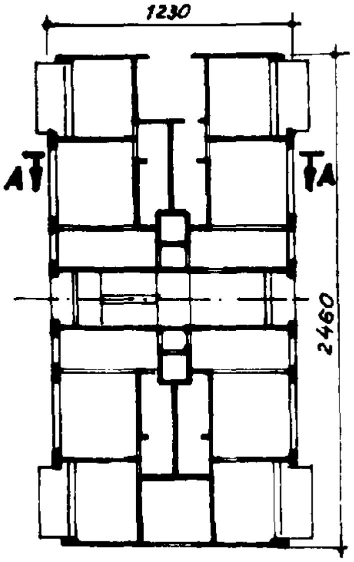
The foundation soil, exhibiting an accentuated non homogenous stratification, presents the following geotechnical characteristics:

- index of consistency 0,55 - 0,85;
- porosity 0,45 - 0,57 %;
- modulus of elasticity 36 - 70 kg/sq.cm;
- cohesion, 0,20 - 0,30 kg/sq.cm;
- angle of internal friction 2^{o} - 7^{o}.

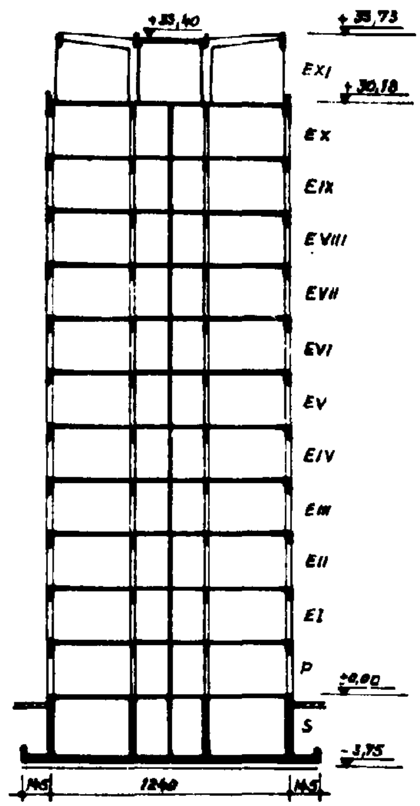
The foundations were of spatial box type with and without cantilevers at block "B", respectively "N" or foundations of a network type with cantinuous footings at block "H". The design of these structure was performed by considering the soil-foundation - structure interaction depending on the ensemble rigidity in the two directions. Due to the difficult foundation soil conditions, it was necessary to survey in time the behavior of the block by measuring their settlements in terms of fixed reference points. It was found that within approximately 2 years, soil deformations were stabilized, the settlement produced during the execution of the building being 65 - 70 % of the total settlement, which have had on average value of 11 cm. The total settlement represented 43 % of the calculated one. The foundations suffered maximum inclination of 3,5 %, the settlement of mobile reference points ranging between 5,8 and 14,3 cm.

SOME ASPECTS OF OBSERVED VULNERABILITY AND SEISMIC RISK

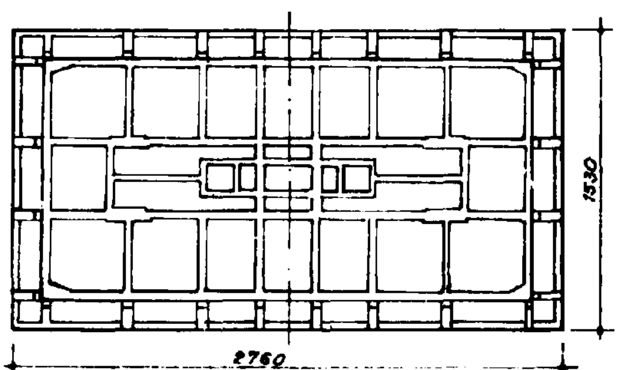
The most frequent damage, which occurred in the blocks of "B" and "H" type, were vertical or slow inclined cracks of lintels, the vertical cracks of the nonpierced shear walls located generally at upper levels as well as some perpendicular cracks appeared an the floor perimeter. The majority of these cracks



Structure plan

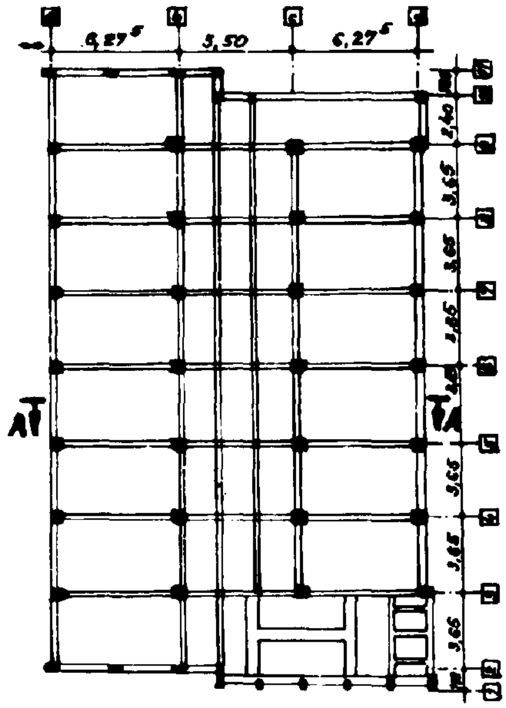


Section A - A

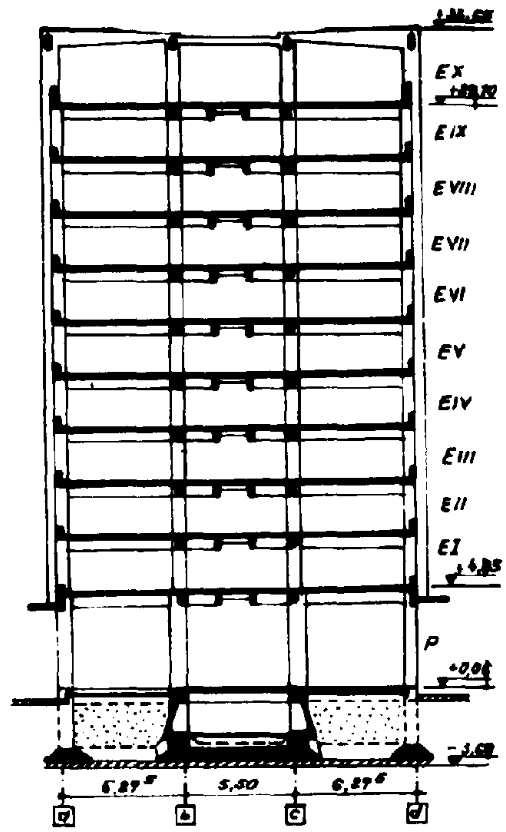


Foundation plan

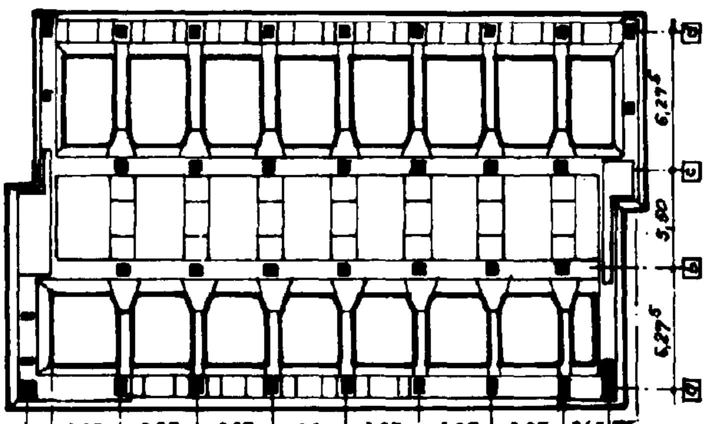
Fig. 1 - BLOCK „B"



Structure plan

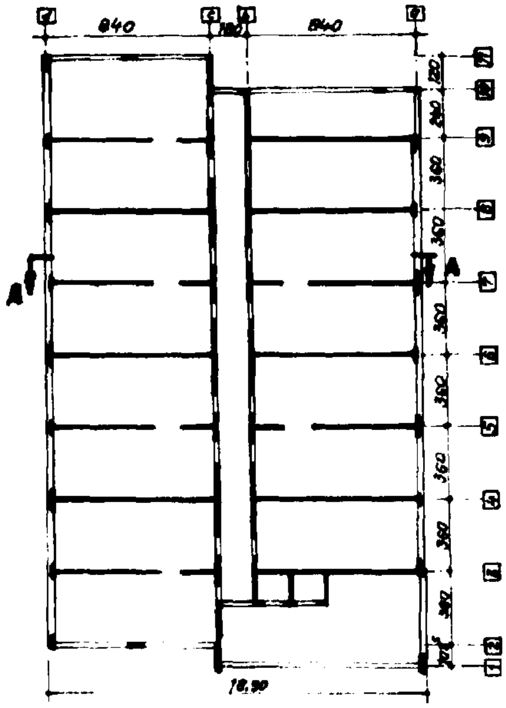


Section A - A

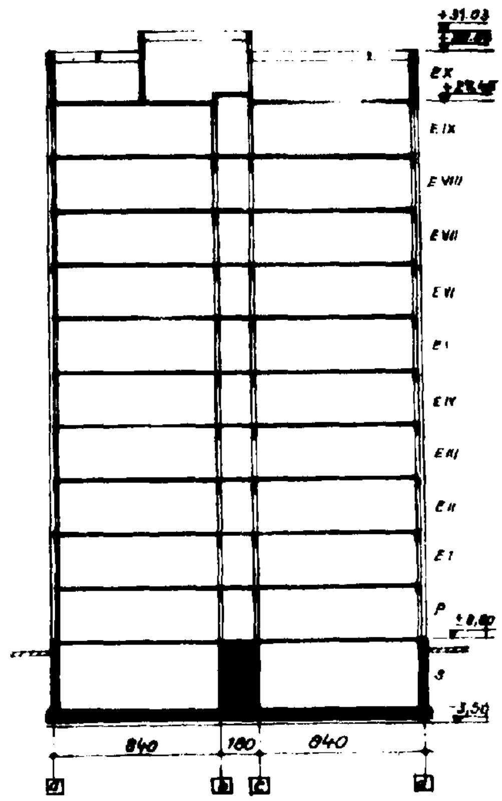


Foundation plan

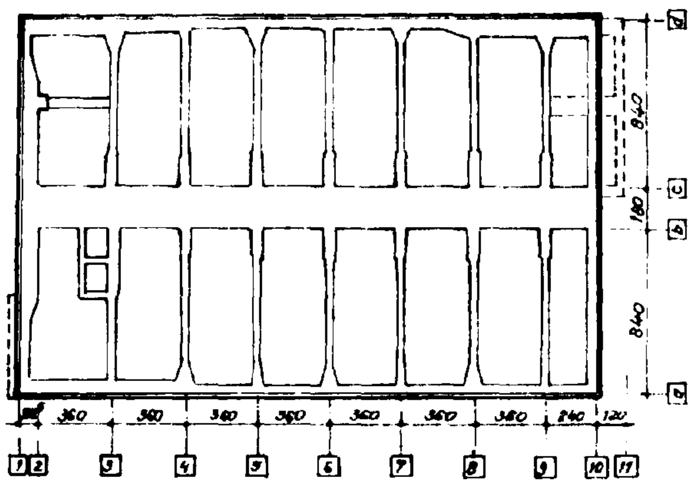
Fig. 2 - BLOCK „N"



Structure plan



Section A - A



Foundation plan

Fig. 3 - BLOCK „H"

were initially due to concrete shrinkage as well as temperature effect. During the 1977 Vrancea earthquake these cracks extended and become wider. Some degradations occurred in the type "H"block during the earthquake action, and were located in the region connection of the central rigid portion (on the shear walls) with the elastic regions from the columns, situated at the two ends of the block, in longitudinal direction.

The structural resisting system of block type "N" was not damaged during the earthquake. Some degradations appeared through infill walls as well as through the partitioning ones; these elements detached out of the structure and presented some cracks, particularly at levels 3, 4 and 5. The foundations of all three types of blocks did not present any degradation. Due to significant displacements and rotations, side pavements detached out of the socles.

A statistical analysis revealed that the examined blocks presented a medium damage degree of 2 and a standard deviations ranging between 1.05 and 1.20 /1 , 4 /.

Significant variations were observed in the histograms of damage degrees from one structural type to another / 4 /.

It is well known that the problem of the dimensioning the structures under earthquake forces is a problem of accepted seismic risk for every country. An important fact is to establish seismic risk for a certain type at building and to select acceptable risk in terms of economic and social conditions. To do this it is necessary to know the quality of materials utilized, the existing degradations, life duration of the building, initial architectural and structural details, subsequent modifications, geological conditions, etc, etc. /2, 3, 5, 7 /.

Akkaş and Shale / 8 /proposed the following important factors to be considered for analyzing the risk: overall resistance of the structural system, story resisting structural system the plan symmetry, elevation regularity, building quality as well as level of applied codes.

By means of a weight coefficient the expected risk may be estimated by determining the damage of the whole building.

The observations performed in the present paper intended to decrease the risk of the existing stock of buildings and to provide a rational long-term development of the city of Jassy.

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II.9 CHOICE OF STRENGTHENING SOLUTION BY THE EVALUATION OF POST-SHAKING STRENGTH CAPACITY OF A MULTISTORY BUILDING HAVING A STEEL SKELETON STRUCTURE AND BRICK MASONRY INFILLING

Virgil Fierbinţeanu [x]
Mircea Balcu [xx]
Dumitru Perovici [xxx]
Mircea Dima [xxxx]

ABSTRACT

1. GENERALITIES

1.1. The building to be examined here benefits from special consideration within the environment and architecture at its disposal; moreover, from historical point of view, it was the first Telephone Exchange Office for public use installed in Romania /1/ (fig.1 and fig.2).

Fig.1

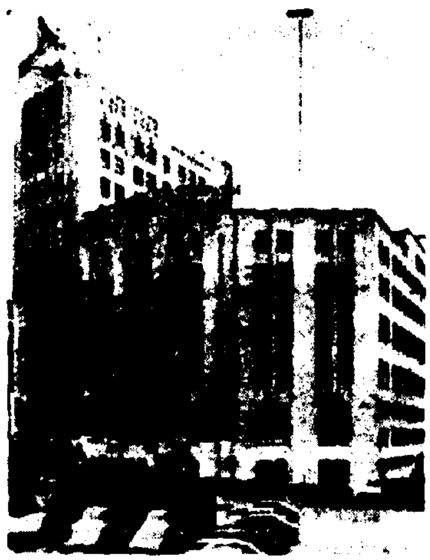

Fig.2

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xx Lecturer, idem
xxx Dipl.Eng., The Railroad Design Institute of Bucharest
xxxx Dipl.Eng., idem

The Victoria Telephone Exchange Office building has a three-side plane arrangement (fig.3). and a six-level height, except in the left corner area, where an extra seven-story tower was built. (fig.4).

During 50 years of use, the Victoria building proved to be an ideal shelter and home for the assembly of electrical in - stallations

The Bucharest Telephone Exchange Office located at 37 Victoria street was erected between 1931-1933, conforming to a project of L.S. Weeks company - New York, USA, later between 1938-1939 and 1945-1946, the initial structure (12 levels) was enlarged with two adjacent buildings (6 levels each).

The ground of the building required a heavy foundation in view of laying it on a gravel-sand layer, water imbued settled to a depth of 8m, under the sidewalk level.

The main effects of March 4, 1977, Vrancea earthquake considerably reduced the bearing capacity of the structure through the compromise between the metallic frame and non-metallic ele - ments as well as other local damages.

The computation model obtained on the ground of these on - servations was subjected to a dynamic analysis of the building got through the direct integration of Vrancea earthquake accele-rogramms.

The results were used in the consolidation measures that details the method of embedding the brick pannels in the metallic structure.

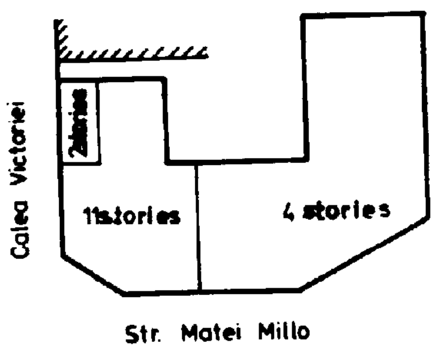


Fig.3

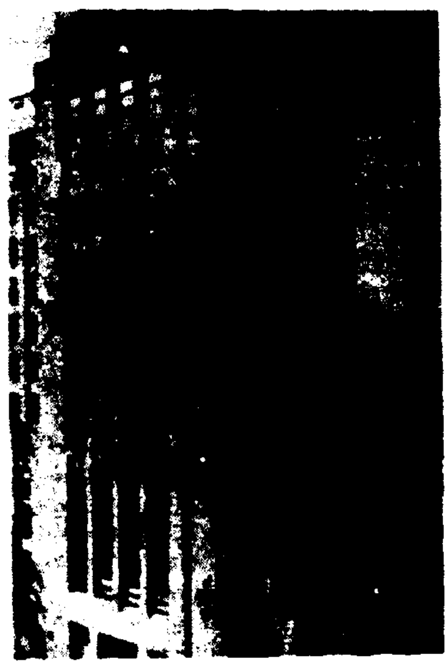
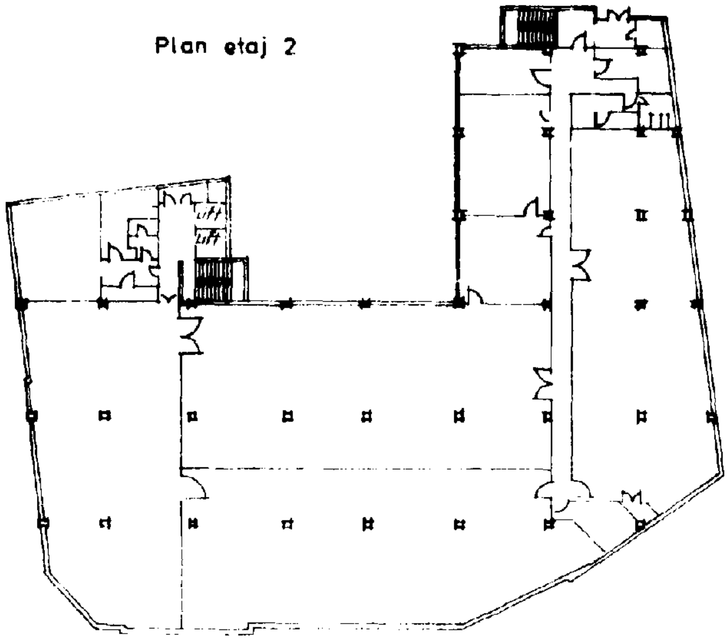


Fig.4

Fig.5



1.2. The structure building assembly is three-dimensional steel frame (i.e. columns, beams and girders) whose cross sections are riveted with I-beams and structural angles, channels and strips.

The individual column clamped to its foundation base is made using two or three-level I beam lattices packed into the concrete foundation block. Altogether these blocks are inter - connected by reinforced concrete beams to form a plane grid foundation structure. The reinforced concrete floor diaphragms are designed in a way that the girder steel beams are included into the floor structure. The grid floor arrangement is shown in fig. 3 with the third floor plane view (i.e. the first overground story) and the typical floor plane view of tower extra stories (i.e. 5^{th} to 11^{th}).

The floor design was made using hanged timber forms and the common deformation of concrete and steel skeleton has been accomplished by the use of steel wire backing that covers the I beams grid structure.

The brick external walls facing the streets are plystoned with Rusciuk stone pieces; those facing the inside courtyard are covered with limestoned mortar. The plystone of the front walls is 10 cm thick, giving it a 40 cm thickness together with the brick thickness.

The anchorage of stone pieces on wall support has been

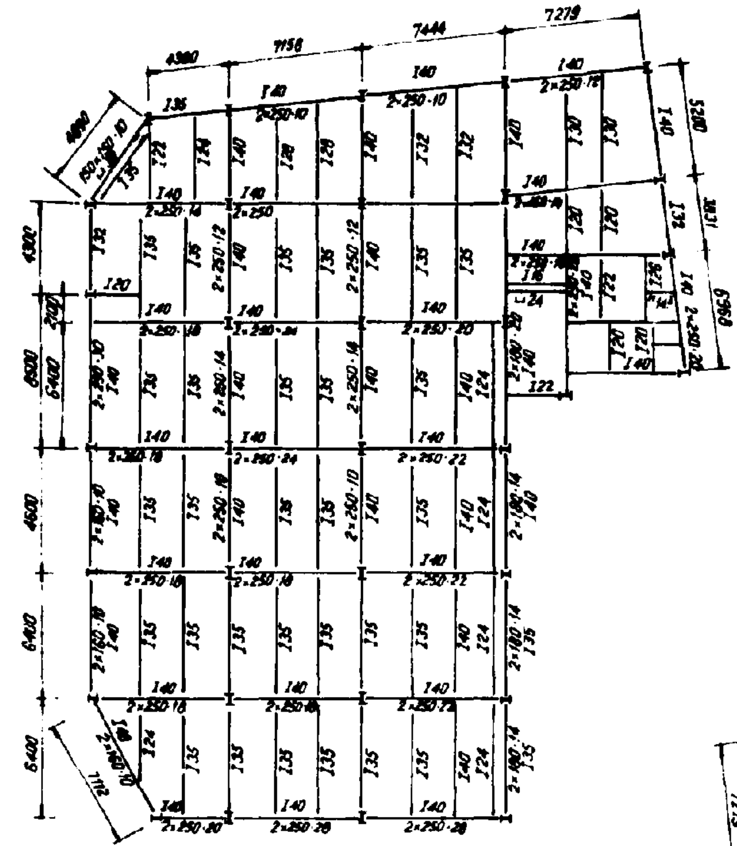


Fig.6

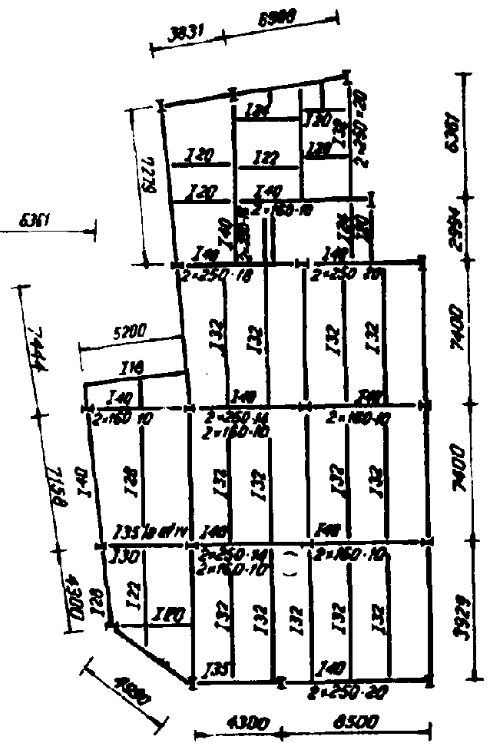


Fig.7

accomplished using steel wires and white mortar of Fieni. The walls facing the courtyard are built up using cemented mortar and the bricks are plotted according to a specific "American brick arrangement technique". The interior partition walls are made from hollow bricks, 7 cm thick each, and cemented mortar. The layout of the interior walls was determined according to functional criteria only, regardless of "embedded" grid disposal.

The structure columns have also been "embedded" using special sized, hollow bricks; they are built up on the narrow size around the steel column and the remaining space is filled with pieces, limestoned and cement mortar.

1.3. The foundation ground of the "Victoria" building is not suitable for the placement of such a monumental building. Soil tests in the area have revealed the following ground layer structure /2/:

- an unstuffed, unconsolidated layer of 6.20 m thick beneath the sidewalk level of Calea Victoriei street;
- a thin clay layer, having sand inclusions;
- a gravel-sand layer, water imbued, at a depth of 8 m under the sidewalk level.

The structure's foundations are clamped within this last layer; it is the level where base blocks of each columns are grid-connected by reinforced concrete girders.

2. STUDY MOTIVATION

2.1. After the earthquake of March 4, 1977, some damage was observed during the examination of "Victoria" Telephone Exchange Office in Bucharest; the most important ones are presented and discussed in the spirit of strengthening works to be undertaken.

The structure's columns underwent local damage (i.e. cracks in cemented mortar volume) and overall disturbances, such as some observed lateral displacements from the vertical axis, as large as 4 cm to the North above the ground story level. The lateral displacements extend up to the third floor, in the front wall facing Calea Victoriei street; from these one might conclude that the permanent, inelastic displacements registered up to the third floor have an average value of 1/150 H (where H denotes the story height). A column at low-story levels has remanent bending curvatures with the maximum in-plane deflections up to 3 cm ; this observation renders the hypothesis according to which columns were bent in two principal planes of inertia and the bending moments are augmented by the axial force and the initial geometry imperfections of the steel structure.

There were, of course, large number of columns which underwent the deformations of the type described above, but the "embedding" into a less damaged protection material prevented the evaluation of their permanent deflections and curvatures.

2.2. Among other structural elements (walls, floors and the foundation grid) some damage has been observed up to the third story (i.e. parallel cracks on the facade to Matei Millo street and some splits, up to 2 cm big, on the facade tower module of the building).

In the exterior walls some small, isolated dislocations between plystone pieces occurred; the damage of interior walls reveals that they had undergone "constrained" shear panel effect during the earthquake motion.

2.3. These observations lead to the following important conclusions regarding the total strength of the building after the earthquake of March 4, 1977:

(i) the metallic strength structure presents at low - story levels, joints, beams and columns which gathered significant plastic deformations;

(ii) in the brick panels and in the floors placed at low-story levels of the building, the post-elastic deformations caused fissures and cracks characteristic for the shear panels, i.e.for transversally loaded plates;

(iii) the post-shaking common deformation of the basic components of structure is a compromise between the deformations of the metallic frame and those of the non-metallic structural members; the compatibility of the deformations is achieved at the expense of maintaining some residual elastic tensions in the metallic structure, compelled to observe a deformed configuration imposed by floors and walls.

2.4. The measures of immediate strengthening suggested by the experts commission /3/ aimed at:

(i) consolidating with double diagonal metallic ground floor panels on the front side facing Calea Victoriei

(ii) unloading the building (water in the air-conditioning tank should be maintained at minimum level; storage batteries should be moved in the basement; certain equipment and aerials should be located in other places etc.); those provisions were mostly observed so that the telephone exchange office functions normal through the building is not strengthened.

Final strengthening measures impose the removal of facilities so as to perform surveys, by uncovering the column coating near the joints of the metallic structure in view of knowing their state of deformations and of establishing practical possibilities of stiffening.

The final strenghtening operations should accomplish the main objective mainly to bring the structure to its initial bearing capacity (by jacketing the columns, stiffening the joints, rewedging the brick walls, recontinuing the floors etc /4/) at an (energetic) level as close as possible to the situation before the earthquake of March 4, 1977.

Taking into account that the strengthening operations require the removal of facilities, the effort of "X-raying" the present condition of the building, under normal functioning regime is thoroughly justified by means of automatic computational model, "identified" (in the sense of the theory of systems) by measuring the dynamic characteristics after the earthquake /5/.

3. COMPUTATIONAL MODEL

The complexity and the characteristic features of the structure of Telephone Exchange Office building may be summarized as follows:

(i) one may identify 32 distinct plane frames, with a variable number of levels and arranged both in directions x and y in plane, and in arbitrary directions;

(ii) the number of distinct cross-sectional areas of the columns exceeds 20 (an explanation of this variety of shapes and dimensions is that the building was erected in three phases over 17 years). The mean geometric characteristics of the column cross-sectional areas were grouped in four different types and are listed in table 1.

Table 1

LEVEL	SECTION NUMBER	AXIAL AREA	INERTIA I_2	INERTIA I_3
VIII, IX, X	1	127.00	19518,00	8125.00
V, VI, VII	2	204,00	39850,00	13580,00
II, III, IV	3	250,00	49241,00	15610.00
S, P, I	4	318,00	61710,00	20553.00

(iii) the brick walls are distinguished by thickness, composition and covering (by coating or plastering) 12 frames from a total of 32 are "wedged" with shear panels arranged at all levels or only at certain storis; it should be noticed that the walls increase the rigidity only in the plane of the frame in which they are arranged, their rigidity in the direction normal to this plane being neglected;

(iv) the floors are considered as rigid washers in a horizontal plane and as plates under the action of vertical forces, norm al to their plane. The hypothesis regarding the behavior of the floors as rigid washers essentially simplifies the spatial computation of the multi-story structures and is widely used in literature (/7/, /8/). The validity of this hypothesis is examined for the case of the building tower (the corner facing the crossroad Calea Victoriei - Matei Millo street) by performing the computation and the plot of the proper forms of vibration obtained without using the stated hypothesis.

Fig.8 shows the components of the first form of vibration at two of the tower levels; it can be seen that the floors suffer motions of rigid, solid type in horizontal plane (roto translations), hence, the floors may be considered as washers in horizontal plan, i.e. correlators of the displacements of the spatial frame joints.

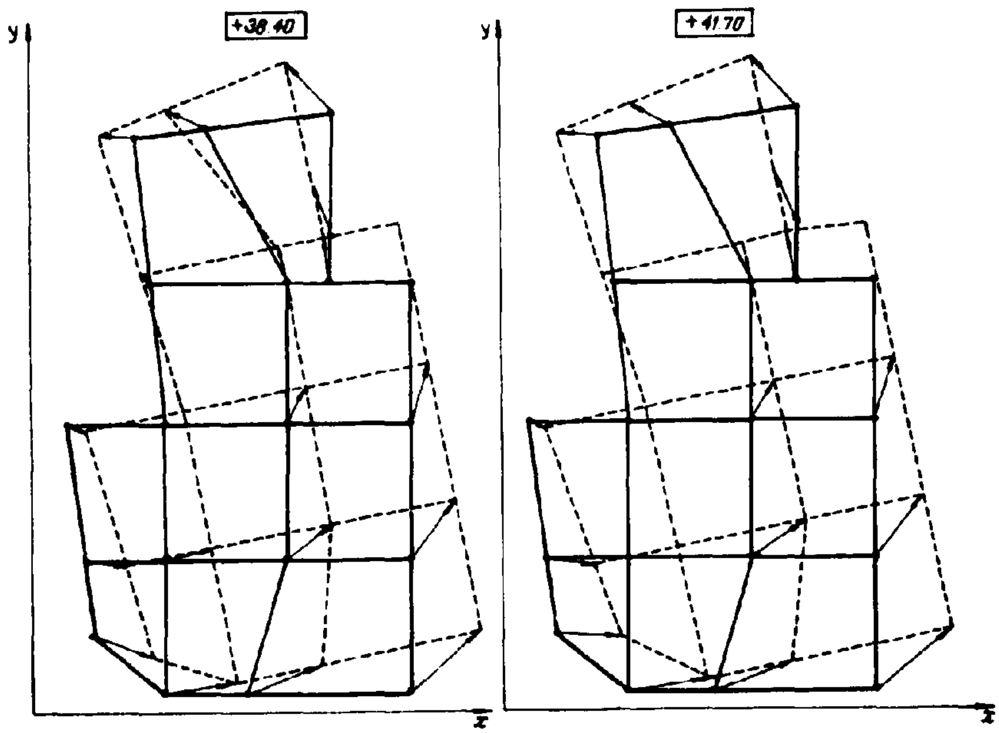


Fig.8

(v) The computational model of the strength structure is supplemented, taking into account the dynamic computation, by a system of elastic springs arranged in horizontal plane of every floor, meant to "stiffen" the spatial frame until identifying its dynamic characteristics with the measured ones (see next section). At the same time, the co-operation between structure, foundation and ground is taken into the computation by intro - ducing a "fictition story", under the level of the foundation base; the total rigidity of the "fictitious story" introduced was obtained from the condition that the maximum side displa - cement computed by directly integrating the 1977 earthquake of March 4, represent, at the upper level of the building, a fraction equal to 1/100 of its total height (about 5 cm).

4. PROBLEMS OF DYNAMIC IDENTIFICATION

4.1. When examining the dynamic behavior of the building to dynamic loads, the fundamental parameters are taken as being the first 10 nature vibration modes of the elastic linear model of strength structure (proper periods T_i and proper vec - tors $\emptyset_i$, i=1,2...10). Among these dynamic characteristics the fundamental natural period T_1 represents the basic parameter used in this work for "identifying" the computational model with the actual structure.

Thus, the computational model of the structure with post-earthquake damage was finished according to the criterion of the identity of the natural period T_1^c1, computed with the fundamental period of the building measured after the earth - quake T_1^m1 = 1.25 sec in direction N-S /5/; the results T_1^c1 = 1.226 sec in table 2 meet the requirements of the stated cri - terion.

Table 2

Natural periods of vibration of post-shaking computational model										
Mode of vibration	1	2	3	4	5	6	7	8	9	10
Period T_i^c1	1.226	1.070	0.666	0.544	0.490	0.340	0.258	0.246	0.206	0.189

The procedure of "identification" is certainly an incomplete one; it is however estimated to achieve the best modelling of the real situation, for which knowledge is so limited (see par. 2 "Study Motivation").

4.2. Unfortunately, one does not know the fundamental natural period $T_i^m o$, which the building in question had before the earthquake; such a value certainly would have represented "the object-function" (in the sense of theory of optimal systems) for the computational model of the building brought to the initial bearing capacity by cobsolidating it. When this parameter is absent, the strengthening solution suggested in the work /5/, drafted according to the results of the numerical computation /4/ appears like a spectrum of the natural periods $T_i^c 2$, in which the strengthened structure increases its rigidity by about 25 % and comes closer to values $T_i^c o$ computed for the building before the earthquake, as resulted from table 3.

Table 3

Natural periods of vibration of computational models
for structure before earthquake, $T_i^c o$
for structure after strengthening $T_i^c 2$

Vibration mode	1	2	3	4	5	6	7	8	9	10
Period $T_i^c o$	0.938	0.693	0.651	0.531	0.504	0.441	0.410	0.363	0.350	0.322
Period $T_i^c 2$	0.962	0.863	0.718	0.635	[illegible]	0.518	0.437	0.410	0.354	0.327

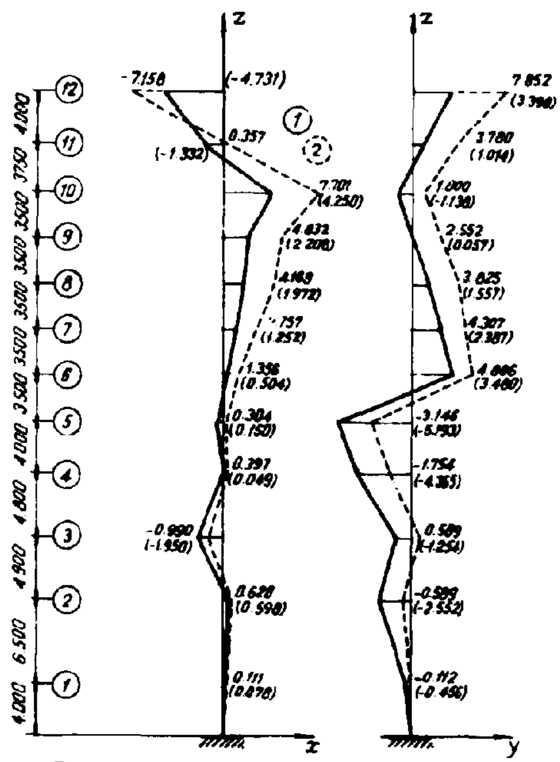
The eigen vector (the eigen form) related to the eigen period of vibration by values $T_i^c o$ and $T_i^c 1$ are shown in fig.9 ; this comparison is used to describe the performances of the strengthening solution.

5. CONCLUSIONS

Taking into account that before the earthquake the building had a fundamental (computed) period $T_1^c o = 0.938$ sec., one may generally estimate that the earthquake caused a reduction of building reliability by 35-40 %; this reliability is to be re-established to a larger extent by the building. For the time being, one has proceeded to reduce and re-distribute loads, in view of bringing the strengthened building to dynamic parameters as close as possible to those of the building before the earthquake.

From the examination of the eigen modes of vibration, the "whip effect" introduced by the upper modes of vibration on levels 10-12 is rendered evident. This effect leads to an increased absorbtion of energy in the upper area of the building, more sensitive to the upper modes of vibration.

Fig.9



NOTE : (1) Stucture before earthquake

(2) Strengthened structure

Taking into consideration that certain elements of the building which contributed to the taking over of the horizontal loads were damaged, their operation being partially or totally stopped, the suggested strengthening solution is conceived so as to achieve a clear and safe structure regarding its behavior to seismic stresses.

Bringing the building to its capacities from the view-points of strength, rigidity and dynamic stability represents the main objective of the strengthening solution.

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II.10 THE STRENGTHENING STRATEGY FOR EXISTING BUILDINGS,DAMAGED BY EARTHQUAKES

Mircea Neicu[x]
Zefir Apostol[xx]

ABSTRACT

After we had studied and strengthened about 2500 tall buildings damaged by the 1977 earthquake we established the main causes which led to the collapse of thirty buildings and the serious damage of some hundreds of buildings. As a rule the following design concepts have been used depending on the degree of damage and the architectural value:

1. Reconstruction of the damaged buildings, preserving their original destination and the outer aspect;

2. Restoration of damaged buildings by complete demolition of the collapsed section, strengthening of the remaining part and joining the two sections as a single whole from a structural and architectural point of view.

3. The reconstruction of some buildings by changing the architectural style and in some cases by the demolition of 1- 3 floors.

4. The complete demolition of the damaged buildings and the construction of a new one, on the site.

1. CHARACTERISTICS OF THE MARCH 4, 1977 EARTHQUAKE

The strong earthquakes that have occurred during the history of Romania were mainly foccussed in the southeast zone of the Carpathian Mountains. The hypocenter depth was between 100 and 160 km, for the last two earthquakes, and the magnitude on the Richter Scale for the earthquake of 1977 was estimated at 7.2.

The seismic intensity of the 1977 earthquake,which esta -

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blished the basis for study of thousands of buildings in Bucharest varied between 7 1/2 and 9. We must specify that the determination of seismic intensity, on the basis of the MSK scale, doesn't allow an objective evaluation of the damage. The behavior of reinforced concrete buildings ranged from slight damage to general collapse.

Generally, the degree of damaged buildings erected during the same period of time (for example: 1930-1940,1940-1950; 1960-1983; and 1983-1977) depended on the following factors: the structural design conception, the execution and quality fo concrete in the structure, the ground-floor uses, the building's height, and layout, alternations and transformations in the building since its construction, and so on.

Of main importance for the buildings'behavior was the fact that, while some buildings were subjected to their first strong earthquake, several other buildings experienced their second earthquake and, due to the Second World War, those structures were insufficiently strengthened, or not strengthened at all, after the 1940 earthquake.

The 1977 earthquake caused partial or total collapse in about thirty important buildings erected between 1930 and 1940, as well as serious damage in about 300 other buildings. In the case of high-rise buildings erected after 1941, however,only two partial collapses were recorded.

The behavior of these high-rise buildings has to be underlined, since as a result of the length of the seismic motion, the degree of damage was much higher for flexible high rise buildings than for rigid ones.

The acceleration spectra recorded in Bucharest have shown, for the predominant direction N-S, values between 1.3-1.6 sec., while the most well-known and frequent spectra of the seismic acceleration on earth have been characterized by short periods, with maximum amplitude T=0.4 sec.

II. STRENGTHENING BUILDINGS

The importance of strengthening buildings to withstand admissible conditions without loss of lives and great material damage under a new strong ground motion, was brought out in strong relief by the disaster of March 4, 1977. If the buildings damaged in 1940. The method for drawing up a strengthening project is well-known from the Romanian and foreign technical literature and refers to the achievement, step by step, of all the following stages:

- the control of the building's state-of-the-art;
- determination of damage causes;

- choice of the strengthening solutions;
- calculation of the new resulted structure;
- drawing up of all the strengthening details, for each building member;
- technical assistance for the repairings.

The mere mention of the working stages might give the wrong impression about the complexity of the problem.

Starting with his experience, inventiveness, technical common sense and professional love, the structural engineer has been assigned the task of solving the extremely difficult problems raised by the strengthening of a building, since every damaged building is a unique case for which, to a certain extent, there are no general prescriptions that may be used without proper judgement.

Further on, we deal with the strengthening strategy used in Bucharest for 2500 high-rise buildings.

According to the type of structure, the degree of damage the town planning requirements of Bucharest, the historical and architectural importance, different structural design concepts have been used, as follows:

1. Strengthening of the damaged building to preserve its exterior style and initial functionality.

Many residential buildings, the Law Court, Medicine Institute are included in this category (see photo 1,2,3).

The damaged structural elements of most of these buildings entirely rebuilt. Additional stiffening walls were used as well as jacketed beams and columns.

Generally, the purpose was to provide the strength capacity of the vertical elements(beams and columns) at shearing force, reduce the eccentricities by bringing the gravity center nearer to the structure's rigidity center, provide the correct transmission of the horizontal forces to the vertical elements through the horizontal diaphragms (floors) etc.

The experience and the lab tests proved that the results obtained by jacketing the columns and beams are better than those obtained by strengthening with steel profiles as the technical guide books often suggest .

At reinforced concrete shear wall structures, the cracks were injected with epoxy-resins, restoring the initial resistance of the members. This method has been used on a large scale.

In the future, the behavior of these buildings depends

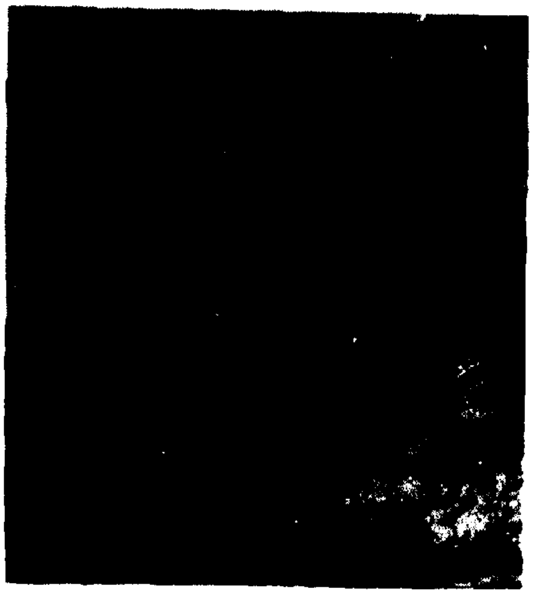

Photo 1 - Institute of Medicine - damaged

Photo 2 - Institute of Medicine - strengthened on the correct execution of the injection works.

In many cases the reinforced concrete framed structures have also been strengthened using stiffening reinforced brick walls on the stories, especially on buildings with high ground floors.

In many countries with high seismicity zones, a great deal of research has been performed on the stiffening of frame structures with masonry, monolith reinforced concrete or reinforced concrete precast panels walls tec. Nevertheless, there are many other problems of interest in this field that are worth study.

Photo 3- The Law Courts.

Photo 4-"Lido"building damaged

Photo 5-"Lido"building strengthened.

Photo 6-7 Academiei building damaged.

Photo 7-7 Academiei building strengthened.

Photo 8-33 Galaţi Street building - damaged

Photo 9-33 Galaţi Street building - strengthened

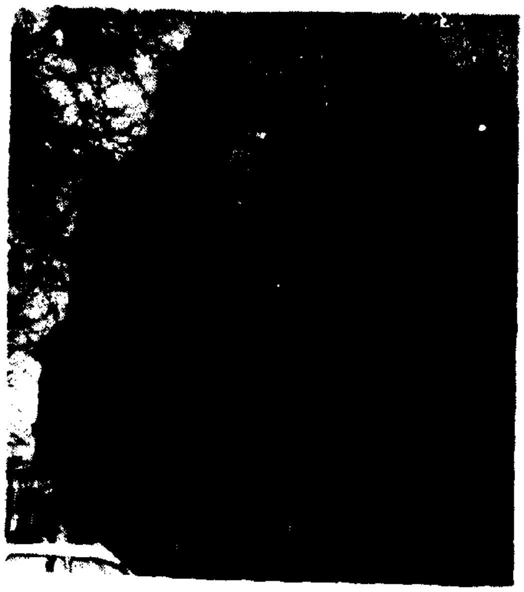

Photo 10 - 102 Lipscani street damaged building

Photo 11-102 Lipscani street strengthened building

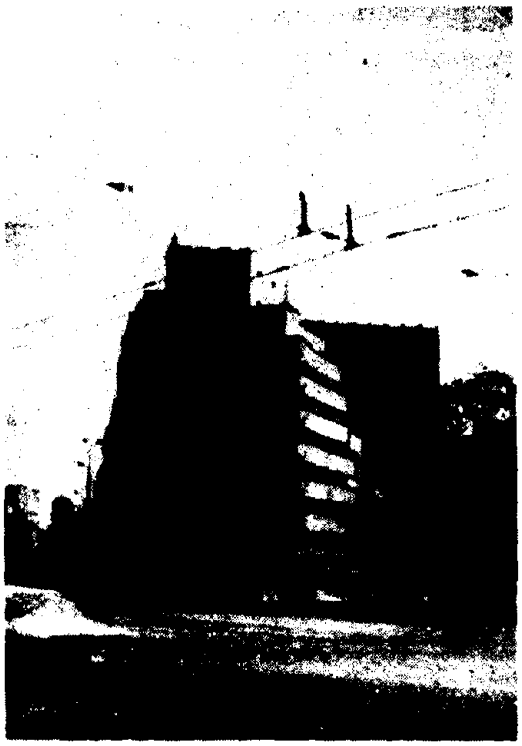

Photo 12 - "Turist" building - strengthened

Photo 13-"Dunarea" building damaged.

Photo 14-"Dunarea" building reconstructed.

2. The strengthening of partially collapsed buildings.

In a relatively small number of buildings, no more than one third of the whole structure collapsed. The causes of this partial collapse are numerous and are not dealt with in this paper. The economic efficiency of the various strengthening methods was studied for each case separately and in certain cases (such as the Lido building, a building on 7 Academiei St. and a building on Galaţi St.) the collapsed sections were demolished and reconstructed and the remaining part of the building was strengthened. Finally, the two sections became a single whole structure with the same architecture. The new structure has been designed to withstand a severe earthquake in good conditions.

3. The strengthening of a building and the changing of its architectural style.

For architectural or urban planning reasons or because of great technical and economical difficulties, other buildings had to have their facade changed (in a few buildings the num - berof stories was reduced, as in the case of Turist and Lipscani buildings.).

4. Demolition

For almost 20 buildings, the collapsed areas exceeded 60% of the whole structure. Their reconstruction and strengthening were not necessary since they had no architectural value and the cost would have been higher than of a new structure.

In such cases the demolition of those buildings was ad - visable, followed by the erection of new structures (for example Dunărea building).

II.11 SIMPLIFIED AND MEDIUM COMPLEXITY METHODS TO ESTIMATE THE RESISTANCE AND DEFORMATION CAPACITY OF EXISTING BUILDINGS CASE AND STATISTICAL STUDIES USING THESE METHODS

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Andrei Bortnowschi[x)]
Teodor Brotea[x)]
Adrian Stănescu[x)]

On the occasion of strengthening works that the authors had to do for buildings damaged by the earthquake of March 4, 1977, simplified and medium complexity analysis methods were devised concerning the resistance and deformation capacity of existing buildings.

In this paper the respective methods are shown with case studies of buildings with reinforced concrete shear walls, with unreinforced clay brick masonry structure and with reinforced concrete frame structure and infill-masonry panels. All have reinforced concrete slabs.

A statistical study is also presented, including a lot of existing buildings investigated by the Design Institute Carpaţi after the earthquake of March 4, 1977.

2. DESCRIPTION OF ANALYSIS METHODS

Both analysis methods are based on a model with a single degree of freedom, through which the entire construction is investigated, especially its bottom part, which is most vulnerable for general or local collapse.

2.1. DESCRIPTION OF THE SIMPLIFIED METHOD

2.1.1. The mean compression stress is determined in the vertical elements of the structure at the ground floor level for its whole area. This affords very useful information concerning the post-elastic resistance and deformation capacity of the structure.

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2.1.2. The actual ultimate shear force capacity of the vertical elements of the structure is determined for each principal direction of it, Pact.ult. = A x τ_a, where A is only the area of the element web and τ_o = 0,09 R_b for reinforced concrete elements, R_b being the mean compression resistance on 20x20x20cube, τ = 0,5 daN/cm^2; 1,0 daN/cm^2; 2,0 daN/cm^2 and 3,0 daN/cm^2 for light concrete or clay brickwork of any mark, with M_4; M_{10}; M_{25} and M_{50} or M_{100} mortar, representing the prismatic resistance of mortar.

2.1.3. The fundamental transverse vibration period $T_1 = 0{,}075.H/\sqrt{B}$ is determined. Based on the elastic response spectra of accelerations, taking a damping factor of 0,05 for reinforced concrete, respectively 0,20 for masonry, the maximum acceleration response is obtained. Then $P_{max.elast.} = Q \times \varepsilon_1 \times a_{max}/g$ is calculated, where Q is the entire weight of the construction in KN, ε_1 = 0,8, a_{max} in m/sec^2, g = 9,81 m/sec^2. The comparison of $P_{max.el.}$ $P_{act.ult}$ helps to explain or to determine the damage degree for a given earthquake.

2.2. DESCRIPTION OF THE METHOD OF MEDIUM COMPLEXITY

2.2.1. The relative level stiffnesses of the structure or stiffness matrices are determined.

2.2.2. A model analysis is then performed which provides ω_1, T_1, ε_1, the vibration characteristics for the fundamental mode, respectively, and z_1 - the position of the resultant of forces for this mode.

2.2.3. The position of the masses, center and of the stiffness center for each level are determined. Later on, the fundamental frequency for uncoupled torsion vibrations is found, ω_1^{θ}, which is compared with the uncoupled lateral vibration frequencies ω_1^x and ω_1^y determined to 2.2.2.

If $0{,}8 < \omega_1^{\theta}/\omega_1^x$ or $\omega_1^y < 1{,}2$ a more sophisticated method must be taken into consideration.

2.2.4. The diagrams P - δ are determined for all the vertical subsystems of the structure. By their summation, the P - δ diagram for the whole structure or construction is obtained.

The relations P - δ are obtained by taking into consideration the cracking, yielding and ultimate stages for the sections of the bottom part of the elements at flexure with compression and also the failure by principal stresses (shear failure).

With the help of the envelope O, A, B a direct dynamic analysis could be made for the model with a single degree of freedom, out of which the maximum displacement for a given earthquake could be obtained, δ_u.

This value, compared with the relations P $-\delta$ of each element or building as a whole, helps to explain or interpret their behavior at a given earthquake.

2.2.5. The resistance and deformation capacities of the ground under the foundation in the elastic and post-elastic range are determined and then compared with the ultimate resistance capacity of the structure at the ground floor level. The ground deformations are taken into consideration to amplify the structure deformations, and consequently the total energy balance.

2.2.6. If the building has been previously subject to seismic or other actions leading to stresses beyond the elastic range, or it was repaired or strengthened, these could be taken into account by assessing the P $-\delta$ relations.

2.2.7. Fig.1 to 6 help to explain the concepts and assumptions at the basis of the two analysis methods.

2.2.8. Further on, case studies are presented for three of the constructions investigated and strengthened after the earthquake of March 4, 1977 by the Design Institute Carpaţi.

The Student's Hostel of the Stefan Gheorghiu Academy is a building with a reinforced concrete shear walls structure, the Colentina Hospital is a building with a unreinforced clay masonry structure, and the Athénée Palace Hotel a building with a reinforced concrete frames structure with infill-panels of unreinforced clay masonry.

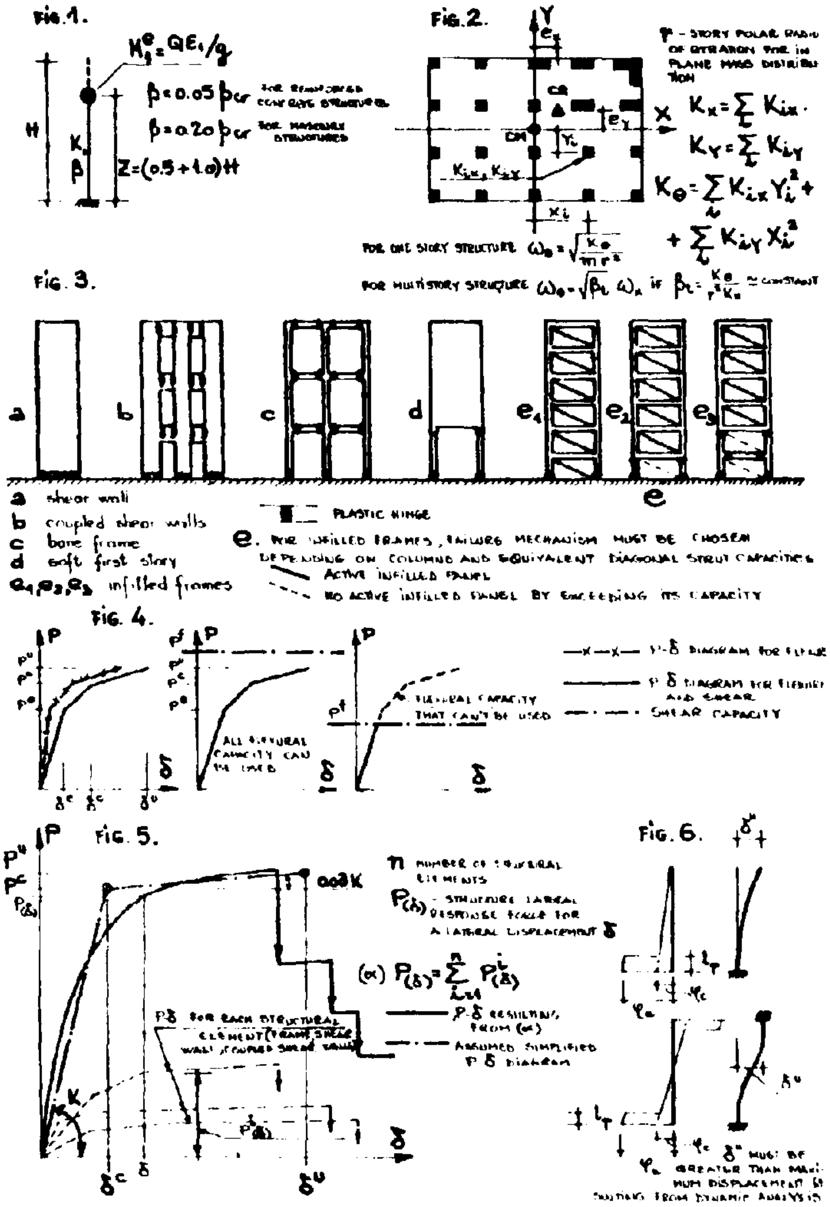
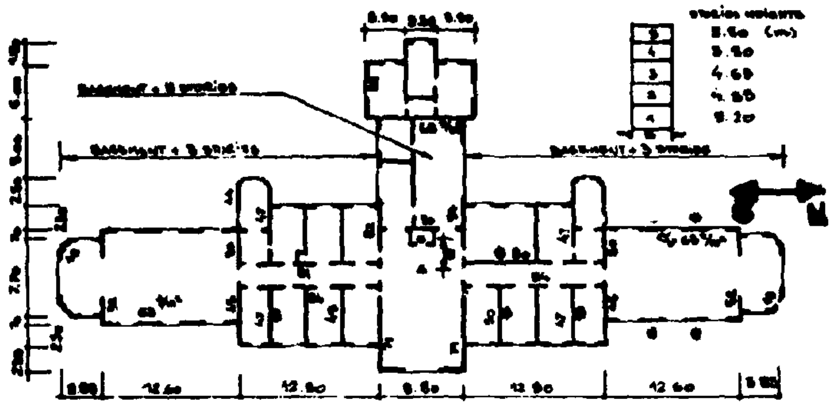


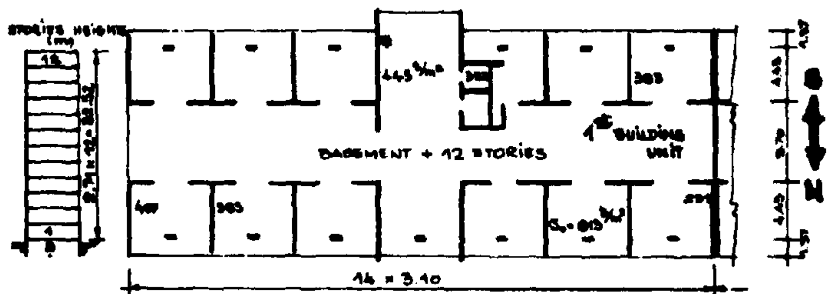
Fig. 1.
Fig. 2.
Fig. 3.
Fig. 4.
Fig. 5.
Fig. 6.
a sheet wall
b coupled shear walls
c bare frame
d soft first story
e1, e2, e3 infilled frames
PLASTIC HINGE

COLENTINA HOSPITAL - GROUND FLOOR PLAN



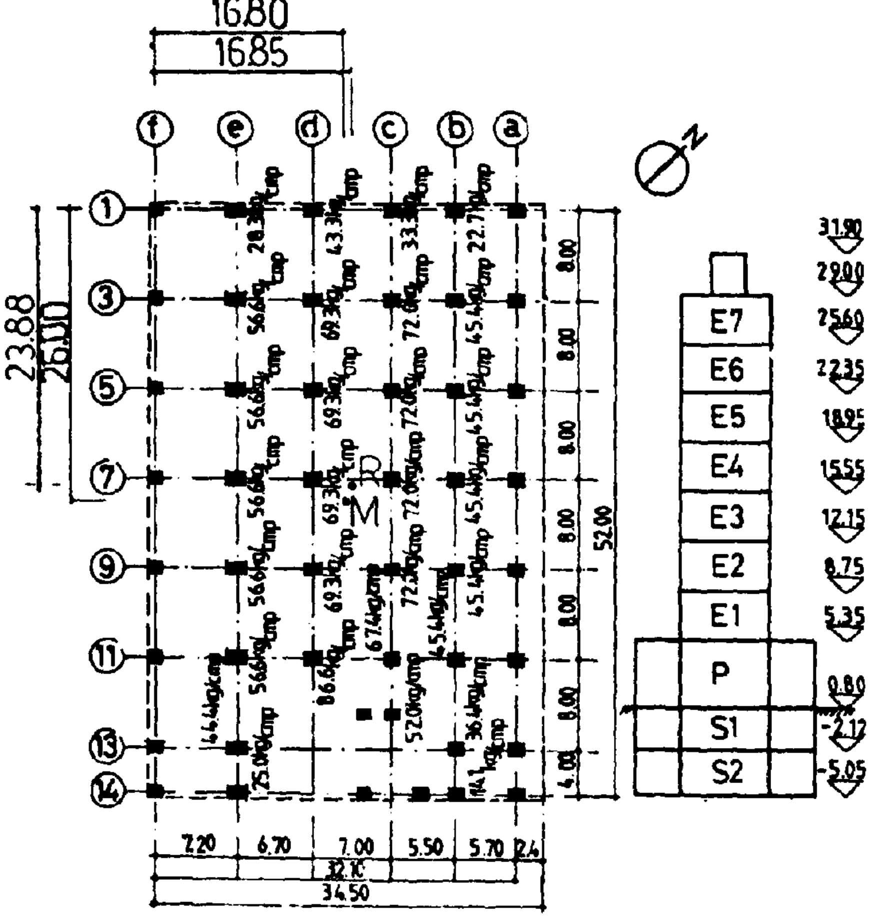
ERECTION YEAR; SITE - 1913÷1937 - BUCHAREST / STRUCTURE - UNREINFORCED CLAY BRICK MASONRY WALLS AND COUPLED SHEAR WALLS; REINFORCED CONCRETE SLABS / DATA FOR P-Δ DIAGRAMS: MEDIUM COMPRESSION STRENGTH - 25 kg/cm²; ULTIMATE COMPRESSION STRAIN - 0.0035; ALL ELEMENTS WERE CONSIDERED HAVING RECTANGULAR CROSS SECTION / MAIN OBSERVED DAMAGES AFTER 4 III 1977 EARTHQUAKE - FLEXURAL AND SHEAR TYPE OF CRACKS IN AL COUPLING BEAMS AND LOCAL IN WALLS (FOR MAIN DAMAGED WALLS SEE ✱ - ON FLOOR PLAN AND ON P-Δ DIAGRAMS) / ⊙ MASS CENTER / ▲ RIGIDITY CENTER / e = 4.9 m / FIGURES NEXT TO STRUCTURAL WALLS - SHOW COMPRESSION STRESS DUE TO GRAVITY LOAD / FUNDAMENTAL PERIODS OF VIBRATION - TRANSVERSAL - 0.449 SEC; - LONGITUDINAL - 0.685 SEC - TORSION - 0.432 SEC - ELASTIC RANGE, UNCRACKED SECTIONS.

STUDENT HOSTEL - TYPICAL FLOOR PLAN

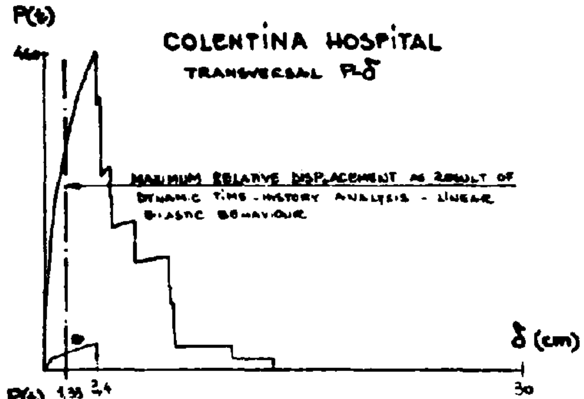


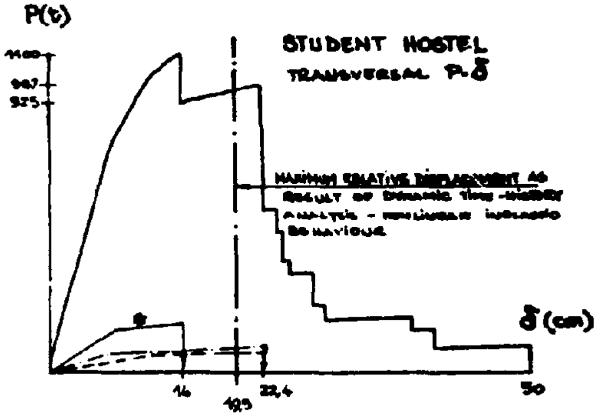
ERECTION YEAR; SITE - 1972 - BUCHAREST / STRUCTURE - REINFORCED CONCRETE SHEAR WALLS, COUPLED SHEAR WALLS, FRAMES AND SLABS / DATA FOR P-Δ DIAGRAMS - MEDIUM COMPRESSION STRENGTH - 200 kg/cm² (AT 28 DAYS ON 20×20×20 CUBE SPECIMEN); ULTIMATE COMPRESSION STRAIN - 0.0035; RECTANGULAR AND FLANGED CROSS SECTIONS WERE CONSIDERED / MAIN OBSERVED DAMAGES AFTER 4 III 1977 EARTHQUAKE - FAILURE IN COMPRESSION ZONE FOR ✱ WALLS ON FLOOR PLAN; SOME LOCAL AND SMALL SHEAR CRACKS FOR ALMOST ALL SHEAR WALLS FROM 1st TO 5th STORY / FIGURES NEXT TO STRUCTURAL ELEMENTS - SHOW COMPRESSION STRESS DUE TO GRAVITY LOAD / FUNDAMENTAL PERIODS OF VIBRATION - TRANSVERSAL - 0.80 SEC; LONGITUDINAL - 0.87 SEC - ELASTIC RANGE, UNCRACKED SECTIONS.

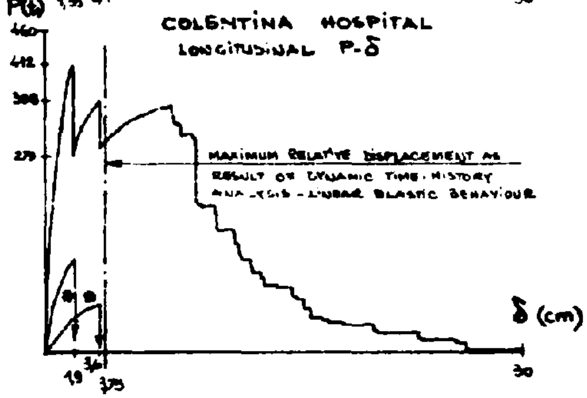
ATHENE PALACE HOTEL-THE NEW UNIT



ERECTION YEAR-1965; SITE-BUCURESTI; STRUCTURE-REINFORCED CONCRETE FRAME WITH INFILLED BRICK MASONRY PANELS; DATA FOR P-Δ DIAGRAMS-MEDIUM COMPRESSION STRENGT 54.6 kg/cm², ULTIMATE COMPRESSION STRAIN 0.0035; MAIN OBSERVED DAMAGES AFTER 04.MARCH 1977 EARTHQUAKE-CRACKS IN MASONRY PANELS FROM 2th AND 3th STORY, DIAGONAL CRACKS IN INFILLED BRICK MASONRY PANELS FROM N-V SIDE, e=2.12m; FUNDAMENTAL PERIODS OF VIBRATION-TRANSVERSAL 0.87s, LONGITUDINAL 0.99s.

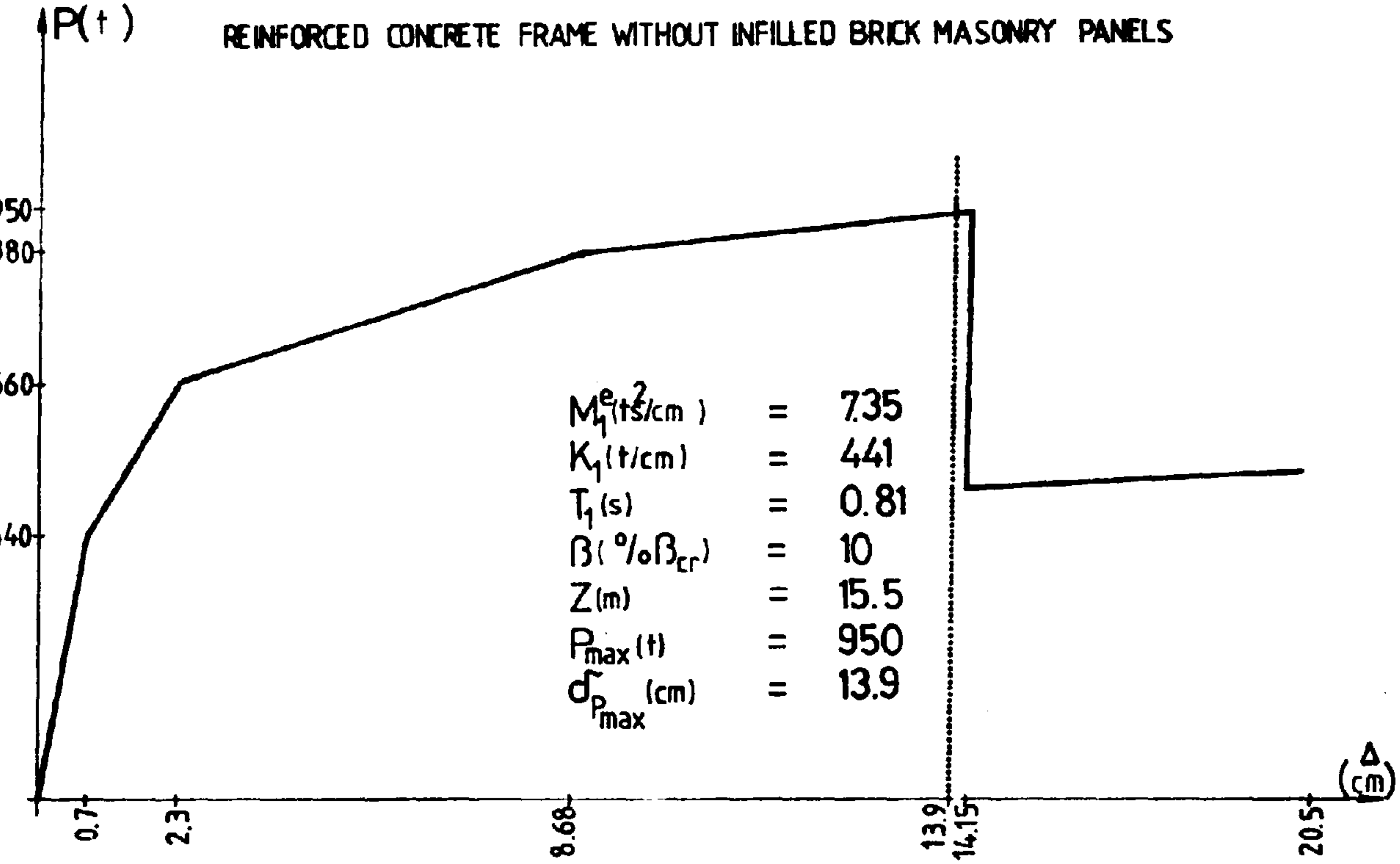






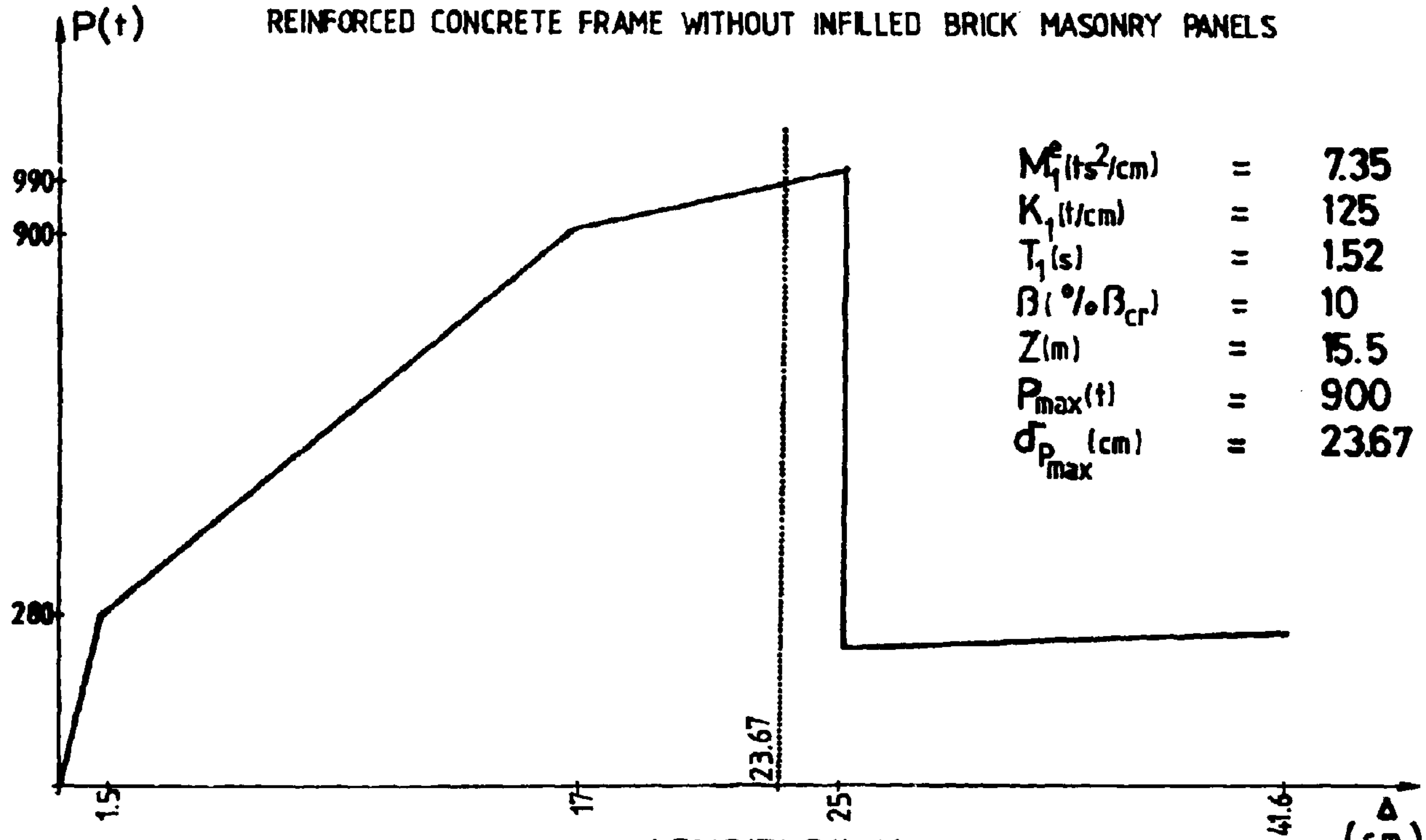
	COLENTINA TR	COLENTINA LG	STUDENT HOSTEL TR
M_1^* ($t s^2/cm$)	5.31	3.28	6.63
K_1 (t/cm)	1193	276	263
T_1 (s)	0.419	0.685	1.0
p (% P_{cr})	20	20	5
Z (m)	11.7	11.7	24.0
P_{max} (t)	460	412	1100
$\delta_{P_{max}}$ (cm)	3.4	1.9	14

* SIGNIFICANT P-δ DIAGRAMS FOR STRUCTURAL ELEMENTS WITH THE SMALLEST LATERAL FAILURE DISPLACEMENTS AT Z POINT.



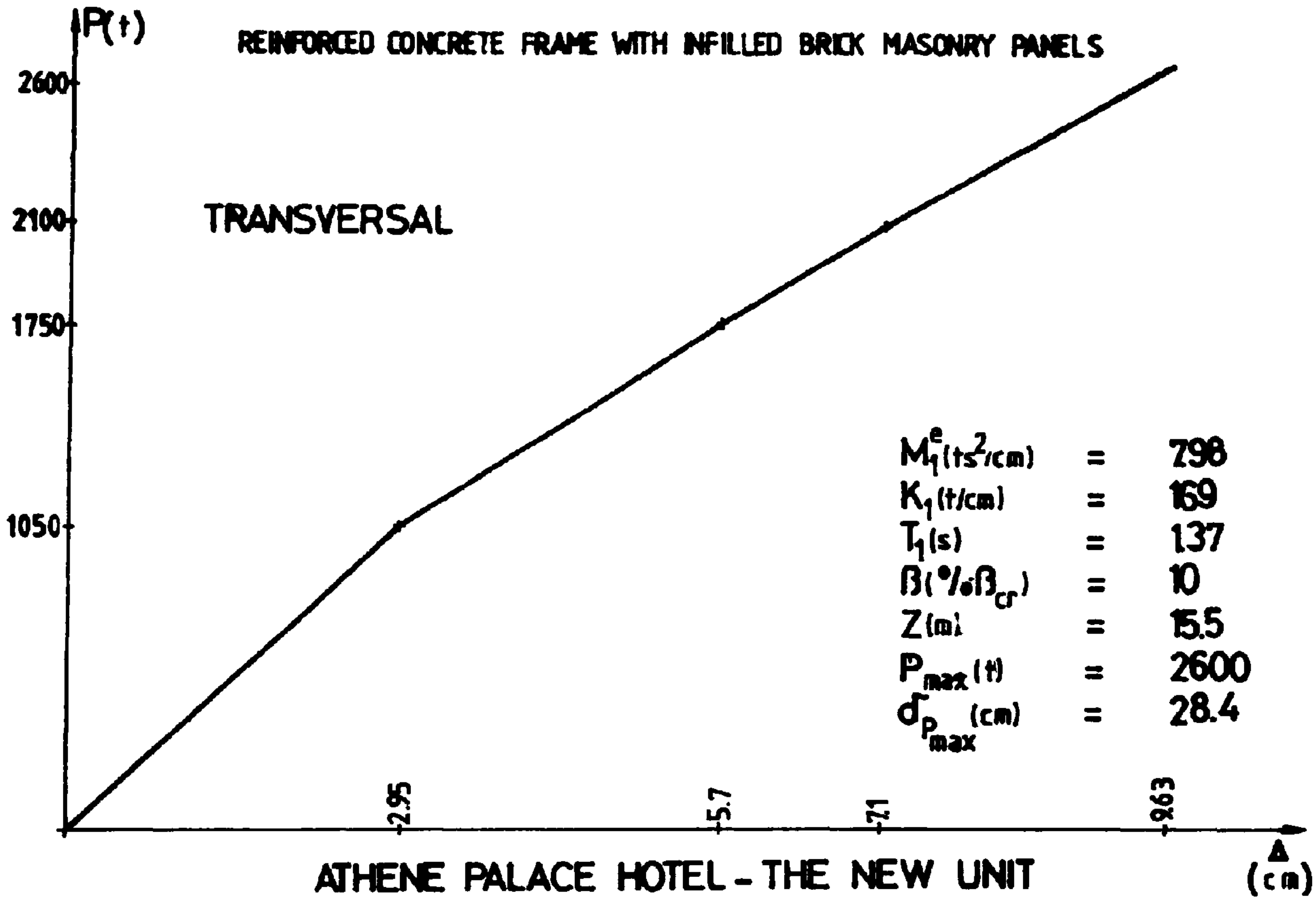
TRANSVERSAL

ATHENE PALACE HOTEL THE NEW UNIT



LONGITUDINAL

ATHENE PALACE HOTEL - THE NEW UNIT



P(t)
REINFORCED CONCRETE FRAME WITH INFILLED BRICK MASONRY PANELS
2600
2100
1750
1050
TRANSVERSAL
$M_1^e (ts^2/cm)$ = 798
$K_1 (t/cm)$ = 169
$T_1 (s)$ = 1.37
$\beta (\%\beta_{cr})$ = 10
$Z (m)$ = 15.5
$P_{max} (t)$ = 2600
$\delta_{P_{max}} (cm)$ = 28.4
2.95
5.7
7.1
9.63
Δ (cm)
ATHENE PALACE HOTEL - THE NEW UNIT

$M_1^e\,(ts^2/cm) = 8.5$

$K_1\,(t/cm) = 640$

$T_1\,(s) = 0.73$

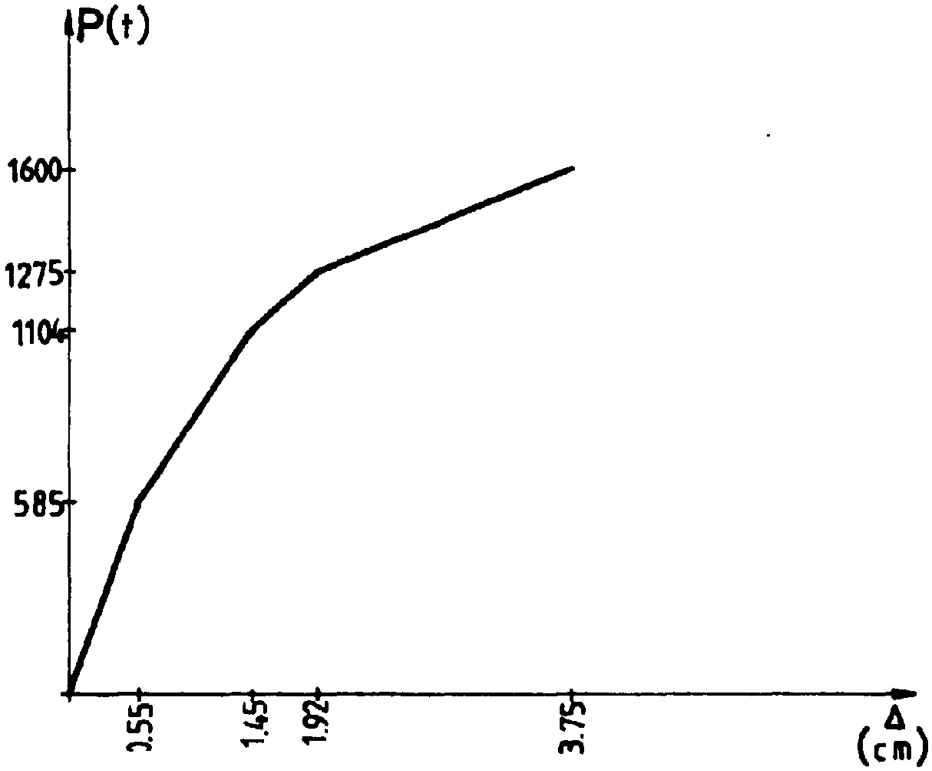
$\beta\,(\%\,\beta_{cr}) = 10$

$Z\,(m) = 15.5$

$P_{max}\,(t) = 1600$

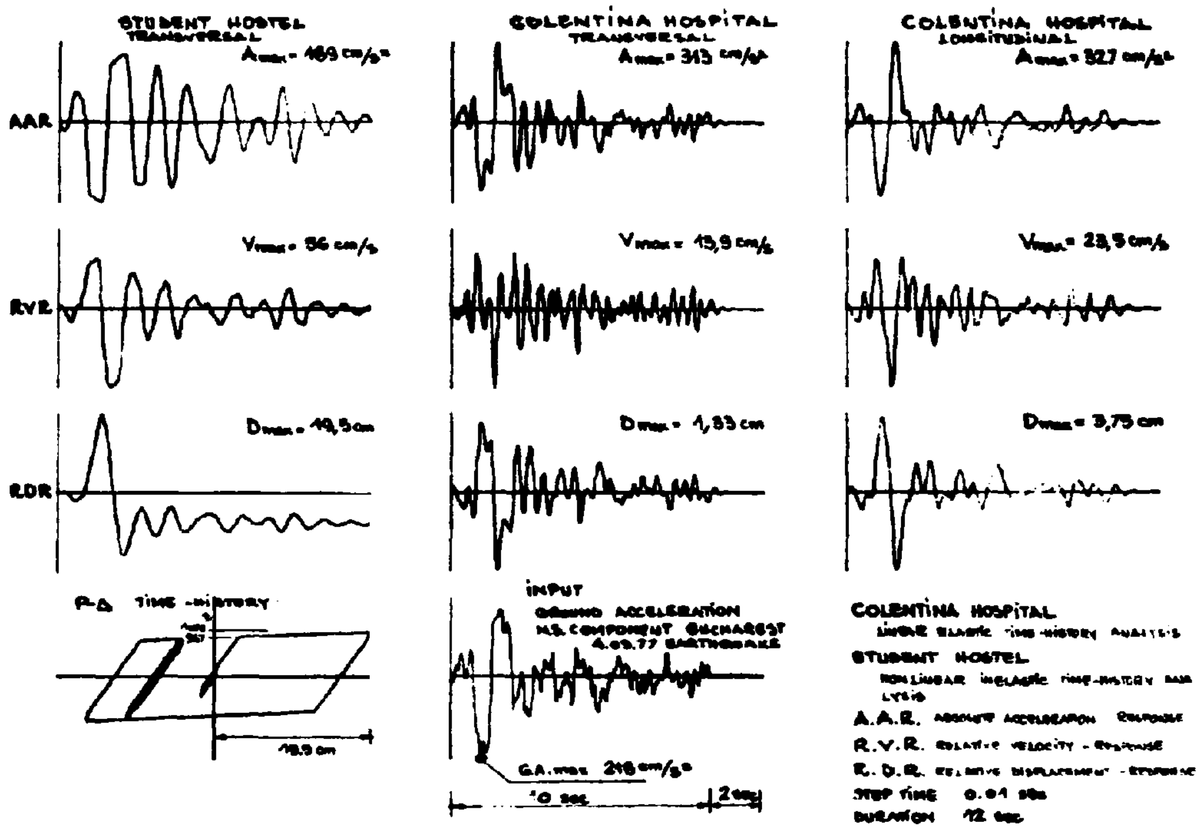
$\sigma_{P_{max}}\,(cm) = 9.27$

REINFORCED CONCRETE FRAME WITH INFILLED BRICK MASONRY PANELS



LONGITUDINAL

ATHENE PALACE HOTEL - THE NEW UNIT


STUDENT HOSTEL
TRANSVERSAL
COLENTINA HOSPITAL
TRANSVERSAL
COLENTINA HOSPITAL
LONGITUDINAL
Amax = 489 cm/s²
Amax = 313 cm/s²
Amax = 327 cm/s²
AAR
Vmax = 56 cm/s
Vmax = 13,9 cm/s
Vmax = 23,5 cm/s
RVR
Dmax = 49,5 cm
Dmax = 1,83 cm
Dmax = 3,75 cm
RDR
P-Δ TIME-HISTORY
49,9 cm
INPUT
GROUND ACCELERATION
NS COMPONENT BUCHAREST
4.03.77 EARTHQUAKE
G.A.max 218 cm/s²
10 sec.
2 sec.
COLENTINA HOSPITAL
LINEAR ELASTIC TIME-HISTORY ANALYSIS
STUDENT HOSTEL
NONLINEAR INELASTIC TIME-HISTORY ANALYSIS
A.A.R. ABSOLUTE ACCELERATION RESPONSE
R.V.R. RELATIVE VELOCITY - RESPONSE
R.D.R. RELATIVE DISPLACEMENT - RESPONSE
STEP TIME 0.01 sec
DURATION 12 sec.

STATISTICAL STUDY

Within the Design Institute Carpaţi a methodology was used on 69 buildings among the works investigated by the Institute or for which the Institute prepared designs for repair or strengthening after the earthquake of March 4 1977.

The buildings have been divided in two classes, those with a clay brickwork structure and those with a reinforced concrete structure.

34 bearing masonry buildings and 35 reinforced concrete buildings were investigated where the structure was either of frames constructed before or after 1950, or of shear walls, or of a dual, system (frames and shear walls) carried out after the 50s.

Among them are 66 buildings situated in Bucharest, 2 in Sinaia and 1 in Craiova.

To define the classes of damage we have chosen (large . moderate, small), the following criteria have been chosen:

We understand to be damage of a building the following a straight or inclined remanent crack which can be observed after an earthquake in building elements and which proves that the element in that zone surpassed the range of elastic behavior during the seismic action.

Damage also means crushing the compressed zone or the inclined crack along the whole width of the element, which we consider at the same time as unusable.

A "large" degree of damage is understood to be damage of the non-structural elements, the horizontal structural elements (beams, coupling beams) and one or more vertical structural elements.

A "moderate" degree of damage is characterized by damage of the horizontal elements (beams, coupling beams) and the vertical secondary elements.

A "small" degree of damage is marked by damage only to the vertical secondary elements (partitions, façade etc).

All 69 constructions were examined with the first method and 9 with the second one also.

By processing the data, means and mean square deviations were calculated for many different characteristics: we present the most interesting and significant ones which in our opinion is the ratio of maximum elastic shear force P to actual ultimate shear force P capacity.

Type of construction	Degree of damage					
	large		moderate		small	
	means	mean square deviation	means	mean square deviation	means	mean square deviation
Constructions with clay masonry structure	2,70	0,93	1,34	0,44	0,84	0,32
Constructions with reinforced concrete structure	1,99	0,37	0,96	0,27	0,49	0,19

CONCLUSIONS

The analysis of existing buildings presents several aspects, among which the most significant and important from the point of view of the structural engineer is the theoretical analysis.

The paper is concerned with two analysis methods, three case studies and a statistical study using such methods.

There are numerous difficult problems in applying such methods and these appear from the beginning, from the identification of the structural elements, the features of the constituent materials etc.

All these difficulties could be overcome, but we should not get discouraged because the structural engineer, like any technician, needs quantification to analyze and solve a problem such that of the behavior of a construction during seismic actions.

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II.12 STRENGTHENING SOLUTIONS FOR SOME LARGE SPAN FRAMED STRUCTURES

Neagoe L. x)
Bălţăţeanu L. x)
Mihăilescu St. xx)

In this report, we describe a strengthening solution for some reinforced concrete framed structures damaged by the March 1977 earthquake. This solution was applied to some apartment buildings within the "Bucur Obor" complex of dwellings and stores (ALMO) in Bucharest.

The ALMO-Complex consists of five apartment buildings placed in enclosure, closing inside a space for the stores. All the buildings have a basement, a ground floor, a second floor, and then 10 stories above that - except for building II, which has 14 stories above the first two. The first two levels are taken by the stores.

The resistance structure of buildings I, II and III consists of cast-in-place reinforced concrete frames laid out in two directions. Buildings IV and V have a combined structure, comprising frames together with shields - their behavior and their degree of damage as a result of the 1977 earthquake not being the subject of this report.

Buildings I and III are made up of three units each, all similar in layout and number of frames, spans, and size of elements.

The basement is 6 m high and designed as a rigid box, protecting the structure at the floor level above the basement.

The ground floor is 4,00 m high, the second floor 3,50 m high and the regular floors are 2,75 m high.

The foundations for all the transoms are built of plain concrete blocks and reinforced concrete bearings for the contour pillars, and reinforced concrete elastic foundations for the central pillars.

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x) Engineers
xx) Dr.Eng. Design Institute of Bucharest Municipality
"Proiect Bucureşti", Bucharest, Romania

The lift wells are made of 15 cm thick reinforced concrete separated from the structure by 2 cm wide joints.

The external and internal masonry was entirely made of autoclaved cell concrete, in blocks or bands.

The whole complex was designed by the "Bucharest Project Institute" in 1973-1974.

The seismic design of the structures was carried out for degree 7 of seismic intensity, according to the prescriptions of the Romanian Norms P 13-70, in force at the time when the project was designed.

For one transom of the type used in buildings I and III, the result was a vibration period of 1,50 - 1,60 k2 an an overall seismic coefficient of 1.3 - 1.4 %. The dimensioning of the structures elements was carried out for earthquake strains corresponding to a prescribed seismic coefficient of 2.5% from the gravity loading value.

The damages caused to these buildings by the March 1977 earthquake are characterized not by their seriousness, but by a high frequency - especially at the first 5-6 stores and with a similar evolution at all the units - which led to a series of solutions or these damages, applied, without closing the stores and vacating the flats.

The damage centered at the lower levels and consisted of the following:

A. The exterior walls made of autoclaved cell concrete blocks cracked in X - shape, with joints displacements - some of walls being completely ejected (in the internal courtyard).

The inner walls, located between the reinforced concrete frames, were destroyed, dislodged or cracked, particularly at the lower levels.

B. At the structural elements, only the following damages occurred:

- 45° and X - shaped cracks at most of the beams, frames and bearings, at the first 6 - 7 levels with spans of 0,3 - 1,5 mm and even 2.0 mm.

Larger cracks were noticed near the central joints of the frames. At the marginal joints at the lower side of the beams, vertical cracks occurred as well.

- The secondary beams in the staircase area had vertical cracks spread almost uniformly along the whole span.

- All the casting joints performed incorrectly along some beams, were evidently made by cracking.

- At the columns, crushings of the coating concrete of the steel bars reinforcement were noticed in the joints area, resulting in the stripping of the reinforcement.

All the areas with concreting defects - segregation, pouring joints in incorrect positions and treated inadequately etc. were also rendered evident by marked cracks and concrete expulsion. A column from the raised ground floor in building III collapsed as a result of a concreting fault (an inner hollow).

- The floors had numerous small, irregular cracks with widths of 0.3 - 0,4 mm.

- The flights crushed in the areas crossing the stair landings.

- The lift wells, made of monolith reinforced concrete and separated from the rest of the structure by 2 mm wide joints, worked together with the whole unit, so that all the walls cracked in X-shape on the first 6-7 levels, at all transoms.

The fact that the damage recorded at all transoms of buildings I and III - to be found in building II as well - occured to a great extent to both structural and nonstructural elements proves that the main reason was the great seismic force which acted upon the structure, in comparison with the force for which the structure was designed, according to the P 13- 70 normative.

Thus, the designed horizontal seismic force was 2.5% of the gravitational loading, although the P 13-70 normative admitted 2.0% too, and the results of the estimates had been 1.3 - 1.4%.

The processing of the accelerograms recorded by INCERC in Bucharest on the occasion of the 1977 earthquake, led to spectral response curves for accelerations completly different from the acceleration spectral curve prescribed by the P 13-70 normative, drawn up according to the Soviet and American standards.

According to these acceleration curves, the dynamic coefficient value should have increased together with the increase of the fundamental vibration period of the structure, reaching maximal values for the period T: 1.0 - 1.6 sec., just for periods specific to the usual buildings with framed structures of reinforced concrete, with 8-10 levels.

Thus, the main reason for the damage was the actual loading of the structure beyond the value for which it had been designed.

The damage was aggravated also by a series of factors specific to these buildings, such as the following:

- The plan - shaped transoms, with the breaking-up of the horizontal washers of the floors in the direction of the court - yards and stairs, in the central span.

- The secondary beams being subject to tensile forces resulting from the breaking tendency of the floor.

- The lack of walls of a resistant material (brick),wedged up in the frames contour, which should have worked together in taking over a part of the seismic energy.

- The walls of autoclaved cell concrete - a brittle material - were damaged without putting up a great resistance.

- Many performance deficiencies such as: casting of the plastic concrete in beams with breaking-ups and resuming the concreting without respecting the measures prescribed by the norms;too far ties, especially of those supposed to be in close proximity to the beam - column intersection etc.

- The transverse reinforcement of the beams according to STAS 8000, considering that concrete takes too high a proportion of the total shearing force, led to the provision of a reduced number of ties and inclined bars.

In establishing a strengthening method which could bring the structure back to the initial bearing capacity. both the high frequency of the damage and the necessity of a rapid performance without interrupting the activity in the stores and without vacating the flats, were taken into account.

With a view to the increase of the structure's rigidity in general, (rigidity that had been reduced by the extended cracking of the beams and floors), and in order to ensure at least the stability and resistance for which the whole building had been designed, an additional stiffening system was suggested. This system consists of introducing a number of shields of reinforced brick masonry in place of some damaged walls of autoclaved reinforced concrete into the contour of some reinforced concrete frames and cast in place in connection with these frames.

In both directions, the positions of this shield were determined so as not to modify the flats' functionality and disturb the technological flow of the stores on the ground floor and raised ground floor.

We also had in view an uniform distribution of these shields in the plan, on the two directions, and a gradual reduction of their rigidity in height; in this way, the shields are

37.5 cm wide on the ground floor and raised ground floor, 25 cm wide on the first and second floor, 12.5 cm wide on the third and fourth floor; such shields were not laid out on higher levels.

These shield of reinforced masonry were made of solid brick, longitudinally reinforced at each four courses. Fabrics STNB Ø 4/200 were laid on the side faces of the masonry, being connected to this by its cross reinforcement.

The reinforced masonry was edged on four sides in concrete frames, piercing the floors for the vertical continuity of the reinforcements. The solidarization of the vertical frames with the frame columns was made by steel bolts PC. 52 Ø 25 (8 pieces on the ground floor and 6 on the regular levels).

By edging the masonry shields in concrete frames on the four sides, first of all we had in view a perfect wedging between frames and masonry, and then the ensuring that the sliding forces of the in filling masonry between the frames were transmitted to the columns, without loading the beams, which did not have the necessary bearing capacity.

The purpose of these masonry shields is to increase the whole assembly capacity of taking over the shearing forces.

In principle, a structure made up from reinforced concrete shear walls has a much higher rigidity against the shearing forces than the framed structure.

In this case, the brick masonry shields, being made of a material having its own rigidity much lower than that of the reinforced concrete, but with great possibilities of post-elastic distortions, we can estimate that, in case of a possible seismic loading, the absorption of energy higher than that of a shieldless structure can be ensured, by the action of the masonry shear walls before the action of the reinforced concrete structure and subsequently by a post-elastic co-operation with the structure.

At the same time, during a possible earthquake, the lateral displacements of the frame assembly with masonry shields will be smaller than those of a simply framed structure, which will provide a better condition for the common separating walls.

Through the framing system and the solidarization with the columns, the introduced shields of reinforced masonry ensure a total shearing force transmission of minimum 2.5% of the building weight, the limits of a shearing stress in the reinforced masonry which do not exceed 3-4 kg/cm^2.

The proper strengthening works performed at the ALMO buildings I and III consisted of the following:

- Restoring all locally damaged structural elements (beams pillars), by current means.

- Strengthening the reinforced concrete beams, in order to increase the bearing capacity, especially when taking over the stresses from the shearing force by plating their side faces with glass fibre texture incorporated in epoxide resin. We must emphasize that the usual method of strengthening the beams by sheeting them, with reinforced concrete would have been very difficult in this case, under the conditions of working areas and without having the possibility of increasing the beams height in the flats, these having the role of a lintel.

- Marking the reinforced masonry shields, including the contour frames.

- Remaking the plastering and finishing operations.

A conclusion drawn from designing and performing the strengthening operation described, is the fact that the use of the reinforced masonry shields is an efficient easy-do procedure which does not impair the functionality of the building.

Bibliography

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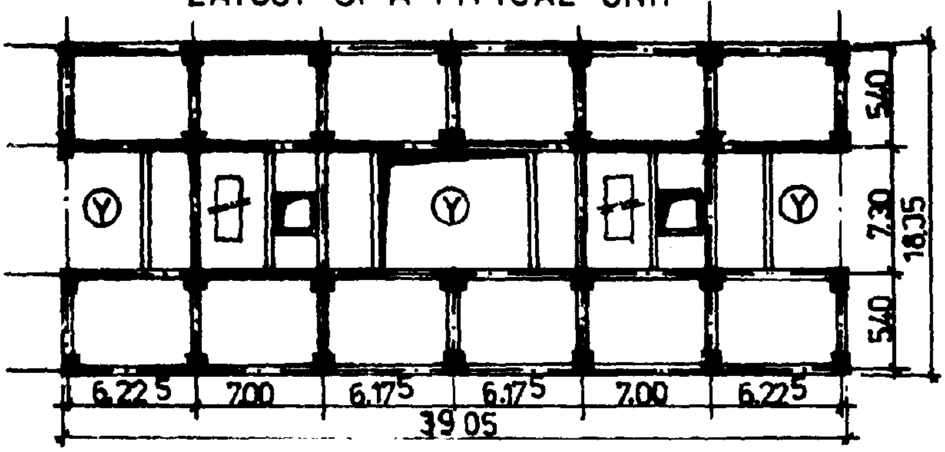
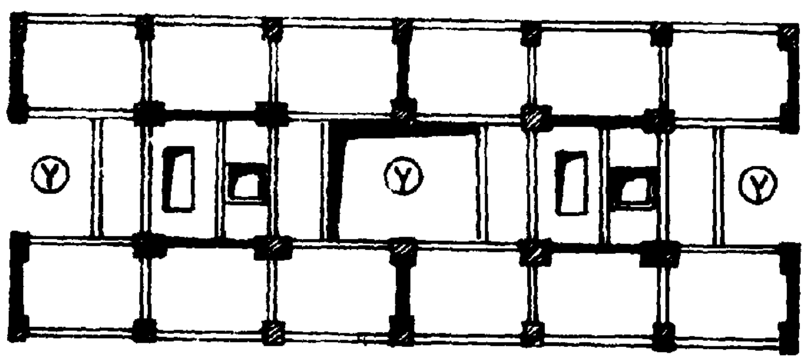
BUCUR OBOR BLOCK OF FLATS AND DEPARTMENT STORES

LAYOUT OF A TYPICAL UNIT



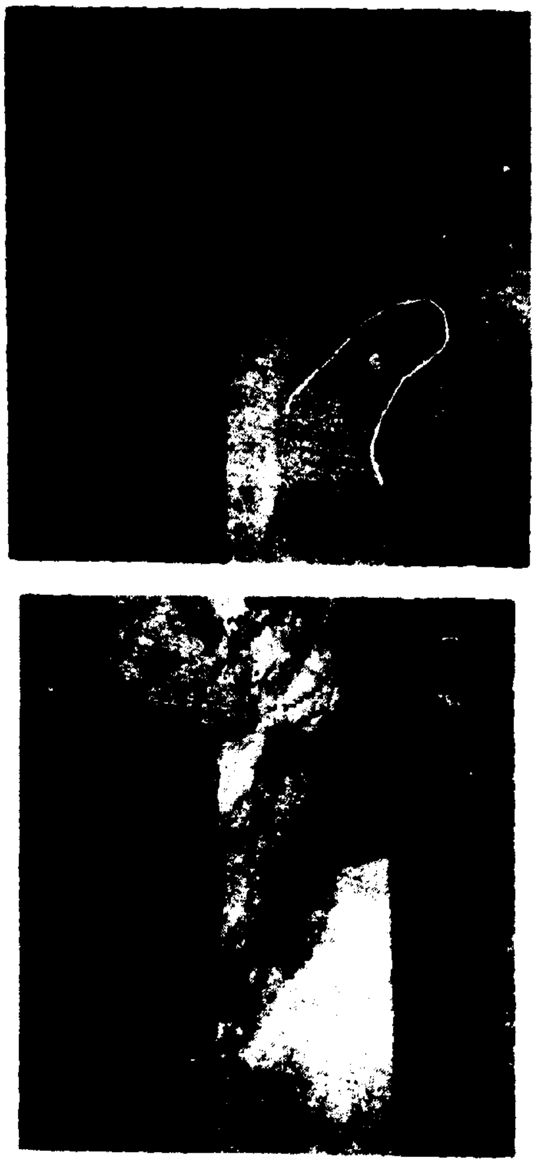
LAYOUT OF REINFORCED MASONRY WALLS



Ⓨ = interior courtyard

INCLINED CRACK IN THE BEAM NEAR THE COLUMN JOINT.

CRACK ALONG THE BEAM



BEAM-COLUMN JOINT FAILURE

SESSION III:EARTHQUAKE PREPAREDNESS. CRITICAL FACILITIES. URBAN AND SOCIOLOGICAL ASPECTS.

III.1

SEISMIC PERFORMANCE OF CRITICAL, EMERGENCY SERVICE FACILITIES

Henry J. Lagorio*

ABSTRACT

The seismic performance of critical, emergency service structures such as major hospitals, ambulance support systems, police and fire stations, and communication centers is of strategic importance to post-earthquake recovery efforts. These facilities, which must be immediately available after a damaging earthquake, should continue to be fully operable following the disaster and not become an additional liability during subsequent search and rescue activities. The 1980 Irpinia earthquake in southern Italy, which resulted in the severe impairment and evacuation of 12 hospitals, and the 1971 San Fernando earthquake in California where four major medical facilities were lost and evacuated, clearly signaled that hospitals and other emergency service structures located in areas of high seismic activity are strategic components in response to the social consequences of major disasters.

In addition, the condition of utility delivery facilities, particularly water supply, electric power and sanitation services, following a major disaster is also of strategic importance. In two recent cases in California, the 1971 San Fernando Earthquake and the 1983 Coalinga earthquake, it was necessary to provide potable water and temporary sanitation facilities for periods of four to six days following the seismic event. Fire and police stations must also continue to function after a damaging earthquake to facilitate search and rescue efforts. Several examples exist wherein severe damage to fire stations, or fire-fighting equipment, in urban centers precluded the use of critically needed components and supplies during the immediate post-earthquake recovery period.

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Recent developments in the performance evaluation of these key, critical systems indicate that the planning, design and construction of emergency service facilities must focus on their ability to remain operable and functional following a damaging earthquake. Such essential structures, which must remain standing after a major seismic event, should continue to serve as responsive elements, and not as deterrents, to a community's recovery efforts. This paper presents current trends for the seismic performance of critical, emergency service buildings in California which address this issue as a social concern in terms of public health and safety.

INTRODUCTION

In the assessment of earthquake hazards mitigation programs, it is equally appropriate to maintain an emphasis on the general, overall aspects of anticipated seismic safety problems from an urban scale point of view, and their perceived interrelationships, as well as a focus on the technical performance of individual structures, buildings, and facilities. In several cases, it has proven to be questionable to have concentrated on one part of the seismic hazards problem without having given consideration to others. Clearly, to avoid unanticipated complications which might arise from having ignored important relationships regarding the overall urban infrastructure, it is essential that hazard reduction objectives be addressed as a shared responsibility between four elements of note:

Geophysical Aspects of Land Use Planning
Technical Performance of Critical Facilities and Services
Interrelationships of Urban Scale Infrastructures
Social/Economic Concerns of Public Health and Safety

A balanced approach between these four elements is required to address the earthquake problem which is filled with many complexities and an infinite number of variables. Consistently, unexpected surprises seem to be the name of the game as new lessons are learned, or old ones relearned, after each new major, damaging seismic event. To better prepare a community for future earthquakes, it is important that public policy emphasize these four aspects of the problem equally in developing earthquake hazard mitigation programs rather than treating it solely as an isolated technical issue of simply improving building design and construction methods.

One significant goal in earthquake hazards mitigation programs, among others, which automatically touches on all four considerations indicated above deals with the design and performance of critical, emergency service facilities. These facilities are of strategic importance to three principal phases of seismic hazards reduction: (1) preparedness efforts prior to an event, (2) fire, search and rescue efforts after an event, and (3) post-earthquake recovery and reconstruction following the event. Special attention, therefore, must be given to those emergency services and facilities which are considered crucial to successful recovery activities in terms of public health and safety when acting on operational efforts related to these three phases. Accordingly, this paper will be purposely limited in scope to cover only the essential aspects of this problem regarding earthquake hazards reduction programs.

CRITICAL, EMERGENCY SERVICE FACILITIES

The seismic performance of critical, emergency service structures such as major hospitals, ambulance support systems, police and fire stations, and communication centers is of strategic importance to post-earthquake recovery efforts. These facilities, which must be immediately available after a damaging earthquake, should be in a position to continue services and be fully operable following the disaster and not become an additional liability during subsequent fire, search and rescue activities.

In addition, the condition of essential utility delivery facilities, particularly water supply, electric power, and sanitation services, following a major disaster is also of critical importance. Failure of these facilities during a damaging earthquake can severely impact the capacity of a community to recover quickly after a major seismic event, and lead to its isolation from the surrounding support area. It is not uncommon to witness situations where it was necessary to provide temporary water supply and sanitation facilities for periods of four to eight days, or longer, following a great disaster. In such cases, the very survival of the community may be at stake.

Among all the service facilities essential to the successful viability of a community, there are several which are considered most critical to continued operations during and after a major, damaging earthquake. Table 1, below, presents a representative listing of several facilities and services critical to the continued function and operation of urban communities. When in perfect working order, they are taken for granted, but when they misfunction, or are heavily damaged or impaired, normal activities can come to a standstill amidst great frustration and consternation to all.

TABLE 1: CRITICAL, EMERGENCY SERVICE FACILITIES AND STRUCTURES

Major Hospitals	Water Supply Plants
Ambulance Services	Sanitation Plants
Fire Stations	Electric Power Plants
Police Stations	Communication Centers
Emergency Service Offices	Transportation Routes
Dams and Reservoirs	

Source: Extrapolated from California Seismic Safety Commission Data, July, 1985.

As a sample preliminary example, it is clear that structures such as fire and police stations must continue to function during and after a damaging seismic event to fight fires following the earthquake and to facilitate search and rescue efforts. Several examples exist wherein severe damage to fire stations precluded the use of crucially needed equipment during the immediate post-earthquake recovery period. Even assuming that police and fire stations have not been damaged and continue to be operational, after the injured have been rescued, they must be taken to medical facilities for treatment and recovery. When medical facilities themselves have been heavily damaged, or rendered nonfunctional due to the earthquake, and can no longer receive the injured, they become an additional liability to the entire immediate post-earthquake recovery period.

Accordingly, current developments in the performance evaluation of these key, critical systems following recent earthquakes indicate that the planning, design and construction of certain emergency service facilities must focus on their ability to remain operable and functional following a major disaster. Such essential structures, which must remain standing after a major earthquake, should also continue to serve as responsive elements, and not as deterrents, to public health and safety in any recovery efforts.

HISTORIC DATA

The vulnerability of critical, emergency service facilities to major seismic events is well documented in California, and elsewhere, by data records of past earthquakes. Specific references clearly reveal that the impairment of these facilities can severely cripple recovery efforts as medical facilities, police and fire stations, government emergency service offices, water supply, sanitation and electric power plants, sanitation services, and important transportation routes are disrupted and/or rendered useless. Disruption of any of these facilities need not be only limited by major structural damage, as in several cases less severe nonstructural architectural damage has also been known to cause complete breakdown of operation. In Italy, after the 1980 Irpinia earthquake, many hilltowns remained without adequate water supply, sewerage facilities, and water treatment plants for weeks, or even months in some cases, after the main shock. Other examples in California, and other areas, of this impairment to critical emergency service facilities are indicated by the citations of historic data which follow.

San Francisco, California, 1906

An accounting of the 1906 San Francisco earthquake offers an excellent example of how a combination of failures to fire stations and fire-fighting equipment exacerbated damage patterns caused by a major damaging seismic event:

"The state-of-the-art citywide fire alarm system was knocked out of action by the first shock wave. The writhing of the San Andreas Fault not only broke telegraph lines and twisted streetcar tracks to stop all transit, it ruptured gas lines and water pipes. The gas fed flames from damaged fireplaces, flues and stovepipes, while the broken water mains rendered fire hydrants pressureless and firemen helpless." (Dillon, 1985)

It is quite evident from this accounting that impairment and disruption of fire stations, fire-fighting equipment, communication systems, and water supply as critical, emergency service facilities of a community increased the damaging incidents behind the total devastation of San Francisco in 1906 which ended in a severe urban conflagration. Even if the fire stations and fire-fighting equipment had survived the initial earthquake wave motions, lack of an adequate water supply would have still resulted in the incapacity of the respective firemen and emergency military troops to stop the ensuing conflagration.

Managua, Nicaragua, 1972

To use a more recent example as an illustration of the importance of fire stations, fire-fighting equipment, communication systems and an adequate water supply to earthquake hazards reduction, one only needs to refer to the 1972 Managua, Nicaragua, earthquake.

"The Managua central fire station, built in 1964 to withstand earthquake damage, was occupied by 20 firemen, 8 fire trucks, and 4 rescue ambulances. The main shock collapsed the second floor, crushing fire apparatus, killing 2 firemen and injuring others. The communication radio was destroyed and no emergency electric power was available. Fires soon began to break out in the city, where hose lines were laid from the lake and pumps put into place because the local water system failed. Some fires resulted from the earthquake, some from arson in order to collect insurance, and some from looting. All fire-fighting equipment and personnel continued on a 24-hour basis for seven days." (Bolt, 1977)

With water lines totally destroyed, fire stations collapsed or knocked out, and most fire-fighting equipment and apparatus damaged beyond use, it was not surprising to note that many fires spread throughout the entire downtown area. The fires developed into a major urban conflagration which raged unchecked for three to seven days. In the end, 750 schoolrooms, 4 major hospitals, and 53,000 housing units were destroyed as block after block of the urban center was consumed by fire. (Cornell, 1979)

San Fernando, California, 1971

After the 1971 San Fernando earthquake, hospital construction generated considerable public concern in California regarding their potential collapse in seismic regions for a justifiable reason: the 1971 earthquake was particularly destructive to health care facilities located in the area, and, in turn, their impairment increased the inability of hospitals to offer emergency care services to treat the injured let alone take care of their own patients who had to be evacuated from the most severely damaged facilities. In the U.S. Department of Commerce publication, "The San Fernando, California, Earthquake of February 9, 1971," Volume 1, Part A, Page 175, Kesler writes as follows:

> "Most of the major structures in the heavily shaken area were medical facilities. Four major hospitals (Olive View, Veterans Administration, Holy Cross, and Pacoima Memorial Lutheran) were located within the radius of 9 miles of the epicenter. At the Veterans Administration Hospital, some of the buildings that were built prior to 1933 collapsed. The other three hospitals, which were built within the last 12 years with earthquake resistance features, all suffered significant damage resulting in evacuation. There were in addition, three medical office buildings (Foothill Medical Center, Pacoima Lutheran Center, and Indian Hills Medical Center), two psychiatric units (Golden State and Olive View), and one mechanical equipment building (Olive View). All, except Golden State, were damaged significantly."
> (Kesler, 1971)

In the aftermath of the San Fernando earthquake it was most clear that health care facilities are a type of building especially important to the immediate post-earthquake recovery efforts of community environments. In this case, their damage and impairment was particularly disruptive to the treatment of injured, and, instead, the four major hospitals in the immediate area became liabilities to recovery activities. In the same publication, Kesler continues:

> "Not only are patients incapacitated in many cases and unable to take, perhaps, even simple precautions to protect themselves, let alone safely endure an interruption in care, but also medical facilities are urgently needed in the hours following widespread destruction and injury due to an earthquake. At the time of disaster, these installations must be functional, rather than being among the casualties . . . "
>
> "Serious consideration should be given to providing increased levels of safety for these important and expensive facilities."
> (Kesler, 1971)

Coalinga, California, 1983

The 1983 Coalinga earthquake was most notable for the collapse and damage to over 37 of unreinforced masonry buildings in and adjacent to its commercial center. However, for the purposes of this paper, it is significant to indicate that there were also several leaks and breaks in the water distribution system (one survey reported a total of eleven) and damage at the water filtration plant which caused problems with water treatment. The filtration plant was inoperable until May 6, four days after the earthquake, by which time all repairs were made and the city water system placed back into operation. A field survey of the situation reported the following conditions on the second day after the earthquake:

> "While most of the major water main repairs had also been made, the water treatment plant had not been put back into operation. People were still cautioned not to use their water. Chemical toilets were brought into the community and set up on some street corners. Potable water was provided by the National Guard and by two breweries that had suspended their normal activities to provide free canned drinking water."
> (Nigg and Mushkatel, 1984)

CURRENT DEVELOPMENTS IN PERFORMANCE STANDARDS

With historic earthquake records clearly indicating the importance of critical, emergency facilities to seismic hazard reduction programs, it is not surprising to see that higher performance standards have been developed for some, and are in the process of being proposed for the planning and design of others. This has already occurred in California where the construction of major hospital facilities is under the jurisdiction of State control in order to ascertain higher performance levels through the California Hospital Act of 1972. Precedence for this Act was found in California under the Field Act and Riley Act which were enacted after the 1933 Long Beach earthquake and mandated by the State to govern improved earthquake resistant design in public schools and other public buildings.

Another emergency facility that has received recent attention as a critical resource during the immediate post-earthquake recovery period is the fire station with its fire-fighting apparatus and equipment. This is particularly true for California where a significant part of the existing building stock is light wood frame construction which represents a significant addition to the fire load found in its urban centers. Facilities and operations provided by fire stations must be available to communities during an earthquake to provide critical public functions such as rescue, fire suppression, ambulance services, and medical assistance. In fact, during and immediately following the 1971 San Fernando earthquake, the need for such services provided by fire and police services increased by 300 to 700 percent depending on the criticality of the adjacent area.

In seeking improved performance standards for such critical, emergency facilities, the key is found in the attempt to keep the facility not only "standing" but also functional and operational during and after a major earthquake. In September 1975, the City of Los Angeles stated in its "Seismic Safety Plan" that:

> "It is important for post-earthquake recovery that critical facilities such as police and fire stations, hospitals, dams and reservoirs, power facilities, and emergency communication systems remain operative after an earthquake."

In order to limit the scope of this subject, the remaining sections of this paper will focus on the planning and design of major hospital buildings and fire stations. Although it is agreed that all of the critical, emergency service facilities listed in Table 1 are equally significant to post-earthquake recovery, for purposes of objectivity and brevity this paper limits emphasis on earthquake safety to two building types: major hospitals and fire stations.

CALIFORNIA HOSPITAL ACT OF 1972 AND 1983 REVISIONS

The 1971 San Fernando earthquake gave a clear signal that major acute care hospitals located in urban areas of high seismic risk represent a particularly critical resource in response to a major, damaging earthquake. Public reaction to the questionable performance of the four major hospitals, in which one collapsed and the other three were extensively damaged and evacuated, resulted in legislation which developed and passed the "Hospital Safety Act of 1972," as a direct product of Senate Bill 519. The revised Act became effective in March 1983.

In passing the Hospital Act, it was the specific intention of the California Legislature that any new hospitals constructed in California "which must be completely functional to perform all necessary services to the public after a disaster, shall be designed and constructed to resist, insofar as practicable, the forces generated by earthquakes, gravity, and winds." For the first time in the history of California, performance standards for the design and construction of a building required that a specific facility remain functional and operable after an earthquake in contrast to earlier provisions, such as the Field Act for public schools, which included the limitation only that the building survive an earthquake without injury to occupants. The end result was a new concept in damage control in that not only the structural system, but also the architectural, mechanical, electrical, and life support systems, as well as all critical medical service systems, were expected to maintain their integrity and remain operational after a major earthquake. Damage control in new hospital buildings became mandatory when the State pre-empted hospital construction from local control. The intent of the legislation is not that the hospital must remain "undamaged," but that it must remain "functional" and "operational" in order to perform all necessary services after a major earthquake.

After 10 years' experience the law was revised to clarify definition of terms and to make allowance for the relative safety of specific single-story, wood frame construction systems used for small scale medical facilities which house patients requiring skilled nursing. Other additions over a period of time included considerations for disaster preparedness programs for the medical services community to improve disaster medical response. These also contained, among other concerns, guidelines for standby agreements for physicians, casualty identification and registration techniques, inventorying medical supplies for possible aerial shipment, and test and exercises to evaluate readiness.

Today, 12 years after the enactment of the Hospital Act, a general survey limited to the 3 East Bay Counties which comprise part of the Greater San Francisco Bay Area in northern California, indicates that over 25 new hospital facilities have been completed in compliance with the Act's performance standards at a total cost of over $245 million (Office of Statewide Health Planning and Development, 1984).

Title 17 - Safety of Construction of Hospitals

The intent of the 1972 Hospital Act, and amended in 1983, is translated into actions for implementation through Title 17, Public Health and Safety Code, of the California Administrative Code which includes provisions for application of the rules and regulations prescribed. Requirements are included for damage control in which critical building elements, systems, equipment, and apparatus necessary for the complete function and operation of the hospital are to be designed, detailed and constructed to withstand the maximum acceleration and deflections of the basic structure without excessive displacement, or damage, which would disrupt essential operations and services to be performed. Deflection under lateral forces are to be established by a dynamic analysis, or assumed to be two times the static deflection computed for the prescribed seismic or wind forces.

The regulations established for hospital construction under Title 17 are the basis design, plan checking, and approval of working drawings and specifications for all construction, and alterations, of hospital facilities in California. It is made clear that the application of the regulations and standards is not intended to limit the creativity of the designer, nor to prevent designing to a higher standing. Supervision of construction must be under the direct responsibility of the architect or engineer who prepared and signed the document for that particular work. Independent State review of the design and construction documents prior to the start of work is completed by the Office of Architecture and Construction, in addition to the State Fire Marshall. Submission of the design and construction documents for State approval are made in three stages: (1) Site Data and Site Plan, (2) Preliminary Floor Plans and Outline Specifications, and (3) Final Working Drawings and Specifications.

Site data information must include a "geological and Earthquake Engineering Report" presenting all scientific data derived from an assessment of the geophysical aspects of the site and the potential of earthquake damage based on geological, foundation, and earthquake engineering investigations.

General Design Requirements

The Hospital Act also coincided with a change in general design philosophies and overall planning concepts of hospital operations wherein vertical circulation was de-emphasized in favor of more economical and efficient horizontal circulation in location of work stations, recovery wards, and other medical services. These changes in hospital planning and design occurred simultaneously with implementation of new administrative procedures related to cost containment goals.

Damage control standards, limited by design requirements related to the continued function and operation of a hospital building during and after an earthquake, give particular attention to the deflection of wall assemblies. The horizontal deflection of vertical structural systems, in the plane of the wall, due to lateral forces is not to exceed 1/16 of an inch per foot of height of any story. Deflection from head to sill of glazed openings, in the plane of the wall, is not to exceed 1/32 of an inch per foot of height of the opening unless the glass therein is prevented from taking shear or distortion, or where tempered, safety or wired glass is used. Design solutions are also required to take into account: (1) deformation compatibility of structural and nonstructural elements, and (2) inelastic deformation in any portion of a connection between elements which could create unsafe conditions.

Architectural, Mechanical, and Electrical Engineering Design Implications

Title 17, Safety of Construction of Hospitals, has particular requirements which have specific impact on the design of architectural, nonstructural, mechanical and electrical elements when their performance, and potential failure, has a direct bearing on the continued function of the basic building. Each element, so identified, must be examined in detail to determine its role in maintaining the continued function and operation of the medical facility.

Anchorage details of all fixed items and components, including major equipment and critical movable apparatus such as autoclaves, sterilizers, kitchen fixtures and appliances, laboratory material, x-ray equipment and cubicle enclosures, must be detailed in consultation with the engineer, or technical consultant, of record. In the architectural set of drawings, the manner in which all nonstructural partitions, window-wall assemblies, and wall openings are attached, or connected, to other components and systems of the structure must be completely accounted for and detailed.

The general end result is that in the overall architectural planning and design of hospitals, lightweight panels and wall assemblies are now being used on building exteriors to reduce the total weight of the facility's envelope. Using lesser building masses has made it easier to control building deflection and story to story drift. The basic structural system of a multi-story hospital now tends to be shear wall construction, or braced frame, rather than moment frame. In terms of mass and volume, hospitals in California now tend to be of lesser height compared to the high-rise, slab type building employed for health care facilities in the past. The typical medical facility is currently being planned and designed as a medium-rise, squat, three to four story structure rather than the tall, slender, glass wall, towers of six to eight stories (or more) which were common twelve years ago. There are exceptions, of course, but generally speaking it is simpler to design for damage control, at the moment, in medium-rise building types in contrast to the older higher, slender, and more flexible ones.

Economic Implications

Based on studies by the California Building Safety Board, it has been estimated that the average cost for compliance with the new seismic performance standards has resulted in about a 25 percent increase in the structural components of a hospital project with the understanding that the structural portions of a new project account for about 12 to 15 percent of total project costs. Therefore the total construction cost increase attributable to structural items ranges from 3 to 5 percent.

The estimated increase in costs for the mechanical and electrical portions brought on by the regulations is approximately 15 percent with the understanding that the mechanical and electrical items constitute about 35 percent of the entire project costs. This results in a total project cost increase of about 5 percent for this portion of the new project.

Thus, the total project cost increase brought on by the new regulations is approximately 8 to 10 percent for the design of new hospital facilities in California. These cost increases are in addition to any costs imposed by the 0.7 percent fee assessed by the State Office of Architecture and Construction for plan review and checking, other field construction inspection costs, required geologic study costs, and any unanticipated project delays.

PLANNING AND DESIGN OF FIRE STATIONS

During a major earthquake disaster, it is well known that local fire departments will find themselves exceedingly taxed to perform the critical services expected of them. A former section of this paper dealing with the historic record, which described damage to critical emergency services, expected of them. A former section of this paper dealing with the historic record, which described damage to critical emergency services, presented an accounting on the damage and impairments to fire stations during the 1972 Managua, Nicaragua, earthquake. An accounting of the 1971 San Fernando, California, earthquake indicates similar patterns:

> "The County Fire Department had some $400,000 damage to structures. Two fire stations in the City of San Fernando were severely damaged. Apparatus in the two fire stations in the Sylmar area were also rendered temporarily inoperative due to the obstruction of apparatus room doors by damaged equipment and racking. Electrical power failure in other locations necessitated the manual operation of many apparatus doors normally activated by motorized units, causing delays. In two or three cases fire department automotive apparatus had either been permanently or temporarily shifted by earth movement (ground shaking) damage or impingement upon the apparatus floor doors. In one instance, apparatus moved laterally as much as five feet, as well as longitudinally, with damage to both itself and the fire station apparatus room. One county fire station was so severely damaged that it took nearly 30 minutes to extract the department pumper truck from the building following removal of obstructing debris, including the fire alarm system control panel, and freeing the apparatus floor doors.
>
> Three fire stations were damaged to the extent that demolition of the structures was necessary. Apparatus room doors experienced binding of rollers, failure of a retraction spring, and general binding, in addition to damage by shifting automotive apparatus. In one station all on-duty firemen were thrown from their beds, struck by articles and falling plaster, and sustained cuts, scratches, and minor bruises." (Kennett, 1977)

Prior studies had already indicated that a high potential existed for extensive earthquake damage and disruption to fire stations in California, even with the enforcement of current seismic code requirements (Algermissen, 1973). At the time, there weren't any considerations or guidelines available for professionals to use in designing fire stations to remain functional or operational during a major earthquake by going beyond mere code provisions. For example, a risk analysis study of the Los Angeles, California area in 1973 indicated that a major earthquake on the Newport-Inglewood fault would leave 35% of the fire stations nonfunctional, while a similar study of the Wasatch Fault in the Salt Lake City, Utah, area projected that 50% of the 85 city and county fire stations surveyed would be critically impaired (Algermissen, 1976).

In 1978, as a result of risk analysis studies and the historic record indicating the vulnerability of fire stations as a critical, emergency service facility to a major earthquake, the National Science Foundation awarded a grant to the AIA Research Corporation to review the problem, assess design alternatives, and recommend appropriate guidelines to mitigate the situation. The problem lies in the potential for severe damage and disruption of emergency services exactly at the time in which the public demand for critical resources and assistance will be at the maximum. Fire stations, as well as police stations, are consistently called upon to provide emergency search and rescue services during a disaster as well as being relied upon for their fire-fighting capabilities. Without their services and resources available, it has been shown that the potential of fire damage, life loss, and injuries will increase dramatically under the complex conditions which occur during a severe earthquake.

A comprehensive survey completed as part of the AIA study on fire stations indicated that many of their elements are most vulnerable to earthquake loads. Table 2 gives an indication of the range of elements and components which were identified as being particularly vulnerable, and frequently cited, by the study in addition to any structural damage which might be sustained including collapse. Free interior circulation inside the fire station is also very important.

TABLE 2: EXAMPLES OF FIRE STATION ELEMENTS AND COMPONENTS POTENTIALLY VULNERABLE TO TYPES OF EARTHQUAKE DAMAGE

Jambing/Binding of Doors	Electric Power Failure
Communication System Failure	Nonstructural Damage
Fire Truck Impairment	Apparatus Impairment
Personnel Injury & Loss	Water Supply Failure
Site Access/Egress	Interior Circulation

Source: Seismic Design for Police and Fire Stations, AIA Research Corporation, Washington, D.C., 1978.

Considerations for Mitigation Options and Strategies

Two approaches exist under which the seismic vulnerability of fire stations may be reduced: (1) Promulgation of special performance standards for new facilities, and (2) Guidelines for the retrofitting/upgrading of existing facilities. To date, no local or regional public policy has been issued to develop special performance standards, or criteria, for new facilities similar in vein to the Hospital Act. However, many communities now realize the importance of fire stations in providing critical, emergency services during a seismic disaster, and are giving special attention to the planning and design of new public facilities. Others have started programs to strengthen and upgrade existing facilities under the impetus of ordinances enacted for retrofit programs dealing with privately owned, existing hazardous buildings.

Hazardous conditions in fire stations seem to fall essentially into three problem areas: (1) Access/egress, (2) Personnel safety, and (3) Operational continuity (Kennett, 1977). For any mitigation Solutions to be effective, equal consideration must be given to all three since they are all closely interrelated. Accordingly, the immediate recommendation made is to develop guidelines for addressing each on an equal basis.

Access/egress problems relate to both personnel circulation and vehicular traffic within the building, external to the facility, and to and from the site itself. If the approach route to the facility is impaired or blocked, as happened to the main fire station in Managua, the fire station may be rendered useless even when no damage is sustained.

Consideration to be given to requirements for personnel safety is self-evident. Loss of technically trained and highly experienced persons can severely reduce the effectiveness of any special services teams. If personnel are not available to operate fire-fighting equipment and apparatus, the fire station is of no value whatsoever no matter how well equipped the facility may be with the latest and most expensive materials. Special attention, however, must be given to the unique personnel characteristics to preserve the specific functions of a fire station: 24 hour occupancy, instant mobility, emergency communication commands, intense circulation patterns, and complex technical tasks. These must be maintained and executed without interference, distractions, and encumbrances from falling debris, inoperable apparatus, or other hazards caused by building movement.

Operational safety depends on: (1) Insuring the reliability of utility services, including back-up emergency generators, (2) stabilizing and protecting necessary equipr , (3) safeguarding circulation spaces, and (4) taking care of the integrity of all other critical areas so that the facility can continue to function during the emergency period (Kennett, 1973).

In addressing the issues posed by this part of the problem, all potential hazards must be evaluated for their implications to planning and design concerns. Elements which pose the greatest risk to the operations which are crucial to the continued function of a facility must be given top priority for they are the ones which can easily shutdown the station and render it useless. Maintaining electric power, water supply, operational apparatus, personnel safety, clear circulation patterns, and open access/egress routes requires the highest priority and performance standards available.

SITE PLANNING AND FACILITIES LOCATION

Finally, some issues regarding site location and site planning merit attention relative to critical, emergency service facilities, whether they be hospitals or fire stations. The best of planning and design approaches, even those using the highest seismic performance standards available, are useless unless attention has first been paid to the location of the facility and its site planning. Many examples exist wherein outstanding buildings, well-constructed and adequately designed to resist seismic forces, have been rendered inoperable and useless for their intended function during an emergency when planned without consideration to site location and conditions.

Of all of the prior issues and concerns for the planning and design of emergency, service facilities indicated in this paper, perhaps the most important is the necessity to give appropriate attention to potential hazards relative to geological and geophysical characteristics of the site and adjacent areas. Landslides, soil failure, such as liquefaction or subsidence, flooding, seiches, and tsunamis induced by earthquake ground motions can be a significant threat to the continued operation of any facility. All, or any of these, in any combination of circumstances have the potential of closing down a facility unless properly addressed. This in itself probably carries the highest priority in the initial planning and design of critical, emergency service facilities. In California, this is where the enactment of the Alquist-Priolo Special Study Zone Act, which encompasses fault line zoning of active, known faults, stands out. No major facility can be planned for, or located on, a site in the Special Study Zone without first completing a thorough geological investigation, including trenching if necessary, of the site. Such site investigations must include evaluations of known and potentially active faults, both local and regional, as well as assessments of slope stability, subsidence, and liquefaction potential of the site and its surroundings.

This Act was State mandated after the 1971 San Fernando, California, earthquake, to supersede and pre-empt local jurisdiction and local building code requirements. By law, provisions of the Act must be met before any building permits can be issued for construction on any site within the Special Studies Zones. The only exception made is for simple, single family, detached residences, or duplexes, designed for private use. In particular, the location and siting of all critical, emergency service facilities must conform to the provisions of this State mandated legislation.

SUMMARY

This paper, in general, has presented some of the concerns regarding the seismic performance of critical, emergency service facilities located in urban centers. In particular it focused on characteristic problems to be approached in the planning and design of major hospitals and fire stations, and identified high priority issues to be addressed in developing alternative options and strategies for the reduction of their exposure to earthquake hazards.

Two case studies in California, the Alquist-Priolo Special Study Zone Act and the Hospital Act of 1972, were emphasized as examples of model legislation developed for consideration in earthquake hazard mitigation programs on a regional level. Both have been in effect, and enforced, in the State for the last thirteen years as successful programs. While new major structures planned, designed, and constructed under the requirements of both Acts have yet to be tested by a major damaging earthquake, it is anticipated that results will be most favorable in performance relative to public health and safety.

It is recommended that other areas of high seismic risk consider establishing similar programs, modified for local conditions and regional characteristics, as an effective consideration in reducing the exposure of critical, emergency facilities to earthquake hazards. Although, in terms of economics, such measures will increase design and construction costs, the anticipated payoff in terms of damage control, public safety, community aspirations, and social goals will be substantial after the next major, damaging earthquake.

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III.2

PREMISES OF A ROMANIAN EARTHQUAKE PREPAREDNESS PROGRAM FOR BUCHAREST

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A first attempt to analyse the physical and functional resistivity of the Bucharest urban area, the capital of Romania in the case of a future earthquake (probable around the year 2000) is presented below. Several factors as population (2.2 million) and its structure on occupation, sex, age, economic and social activities within the actual geological, seismic geographic and urban context of the city have been listed in this paper. Special attention is paid to old building stock as a probable source of life losses, injuries and damage. Life lines are also analysed. Alternatives taking into consideration the motion of the active population during the day and night, the season, day or time of the earthquake occurrence, type of principal or secondary effects were included.

THE NECESSITY AND THE SCOPE OF AN EARTHQUAKE PREPAREDNESS PROGRAM FOR BUCHAREST

The Capital of Romania has been strongly affected by all Vrancea earthquakes the most recent being those of November 10, 1940 and of March 4, 1977. The periodic activity of Vrancea focus indicates return probabilities of a similar event around the year 2000 /2/ . After the late earthquake 1.391 loss of life and 7.596 injuries (90% of the country losses) as well as a great part of the collapses, damages of old buildings and other losses reaching over 2 billion dollars for all Romania, were recorded only in Bucharest. Bucharest has important social and economic, cultural and political functions related to the whole country (10% of the industrial production, a.s.o.). The aim of this paper is to analyse elements at risk and the possibilities to produce losses and disruptions of activities in Bucharest, as a first step towards a disaster scenario. Consequently, one can develop further on a complex earthquake preparedness program for Bucharest.

SPECIFIC ELEMENTS FOR THE DISASTER SCENARIO

Natural causal elements

Location. Bucharest is located in the South of Romania, in the Romanian Plain, at 60 km of Danube and 100 km of Carpathian

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Mountains. The mean altitude is 80 m. The geological superficial structure is represented by sands and gravel varying from 3 to 6 m, covered by flood mud, sometimes with loessoid aspect the and clays. Cuaternary deposits vary from 500 m thickness in South to 1500 m in the North. The seismicity of the zone is dominated by the Vrancea focus (160-200 km N-E of Bucharest). From the record of the 4 March, 1977 earthquake, one can appreciate that long period motions could affect the territory (1.5 - 1.8 s). The Fratesti gravel layers containing pressurized water (under 100-120 m depth) are considered the bedrock for the Bucharest zone. The superficial multilayer over this gravel layer is the main reason of the length of this periods. The climate is continental temperate, slightly excessive, with 4 seasons.
Conclusion. For the initial disaster scenario one can admit as possible a strong motion of magnitude M=7.2 with long period of oscillation and intensity in Bucharest, I_{BUCH} = VIII - IX MSK or MM, similar to the 1977 one.

ELEMENTS AT RISK

The town as an urban community. Bucharest is an old town, for the first time mentioned in 1459 in a royal muniment of the famous Romanian King Vlad the Impaler, surnamed Dracula. At present, the Capital of Romania, organized as a municipality including the agricultural sector Ilfov, has 1,521 km^2 from which the town itself reaches maximum dimensions of 20 x 20 km^2 with 159 km^2 built area. The town is surrounded by two major circulation rings and has two main axes oriented N-S and E-W, with neoclassical and high-rise R.C.,pre-1940 buildings. In the interior of the first circulation ring,, along the mentioned axes, the locative density is higher, buildings are often adjacent, with a height of 6 - 12 stories. In the new districts along the main roads and at the crossings, structures with 8-18 levels were erected while in the interior structures with 5-11 levels were built. Quarters with one story buildings coexist near new quarters.

Building stock. A rough estimation gives for Bucharest a stock of 760,000 aparttments./5.6/. From these, 164,000 apt.were built before 1948, 123,000 apt. between 1948-1963, 147,000 apt. between 1964 - 1970, 146,000 apt. between 1971 - 1977, 180,000 apt. between 1978 - 1984. The Aseismic Design Codes were put in force in 1942 (provisional rules), 1963, 1970, 1978 and 1981) /3,4,5,6/. The main critical aspect is considered therefore the bulk of cca. 300 high-rise r.c. buildings built before 1940 similar to the 28 buildings collapsed at 4th of March 1977. On the other hand, the frame buildings erected up to 1977 were designed to the code forces more reduced as compared with those of rigid buildings.

Conclusion. Bucharest has a building stock and an urban model characterized by the existence of many high-rise old structures, as well as by the large number of new, typified and precast buildings, aseismically designed. Due to all these urban characteristics, the elements presenting the highest seismic risk exist in the central zone. The adjacence phenomena could make difficult the intervention of the rescue teams or of the firemen. The hypothetical collapse of several old high-rise buildings in one of the crossings along the main axes of the City would lead to difficult traffic problems during all recovery period. Also very old neoclassical buildings with rich ornaments, stucco and balconies, as well as chimneys could endanger the life and limb of pedestrians. The city has 20 green parks (312 ha) and 800 ha lakes, nominally able to camp the evacuees. Using only 1/4 from this area, 375,000 persons (2 m^2 per person), 50% from the inhabitants of these 164,000 apt. built before 1948 could be camped if necessary. Concerning the problem of temporary housing, one can mention too that in Bucharest, the state enterprises have built in the last years 20,000 - 25,000 apt. per year, from the state or private funds. In the emergency case, a sufficient number of flats will be always available for homeless people as well as in 1977.

Life lines. The different life lines of Bucharest present the following general state /3/:

- the water supply network (2046 km pipes) composed of main prestressed concrete pipes and distribution pipes in steal or cast-iron, local pumping stations;
- the sewage network (1600 km) composed of concrete pipes and sewage works;
- liquefied natural gas network (906 km) consisting of steel pipes;
- central heating network (437 km) including main underground ducts for high pressure steam local heating stations;
- power supply network consisting of underground electric cables and transformers and also aerial power network;
- the phone network including 535,000 sets.

The F.M. - radio network. In the emergency period, communications and information will rely very much on independent F.M. radio sets having their own power supply. Beside the Ambulance Service, in Bucharest, these apparata exist on cars, cabs and trucks of the Enterprise of Public Transport-ITB at several building enterprises, railroad personnel, traffic control teams of police, municipal and sectors' administration;

- the transportation network includes railroads, public , roads tramways, buslines, trolleybus lines, cabs, subway, domestic and international airports.

Conclusion. Lifelines are well developed in Bucharest; following the experience of the 1977 earthquake when effects on life-lines were not disastrous and analysing the state of art the structure of life-lines in Bucharest, one can nevertheless remark several potential critical points:

- damage potentials in the power transformers stations , the temporary cut off in the water and gas supply as well as in the phone service, due to the power cut off or following the overturning of the accumulators and racks;

- the general positive situation of access roads towards the city can go worse due to the obstruction of streets and tram-lines as a result of the collapse or damage of buildings in the central zone of the town few narrow streets;

- the blocking of some vital road crossings due to rescue, demolition or strengthening works during the emergency period. Bypass branches in these lifelines are required.

Population. In Bucharest live 2,227,568 inhabitants (2,011,927 people in the town and 215,641 in the agricultural sector Ilfov).(1983).

The overall municipality density is of 1464.5 inhabitants/km^2 and of 12,500 inhabitants/km^2 of built area in the town (1983), /3,5,7/.

The age structure of the population (1977) indicates that the groups of age between 20 and 59 years represent 59.2% of population, the group 0-19 years represents 30.20%, while those over 60 years 10.6%.

Women represent in the Bucharest Municipality 51.8% of population. In the town the active population represents (1984) 53% of population:1,063,400 inhabitants. From this amount 772,500 are workers (72,5% of the active population) 22.6% are working in commerce, transport, public services, 10% in education, science, culture, a.s.o. In Bucharest a lot of commuters comming from localities situated at 50 - 100 km out of the town are working daily.

The daily motion of population.Several significant variants for the correlation of the earthquake occurrence time with the daily motion of the population will be proposed for Bucharest disaster scenarios:

- for seismic event during the day, one can propose intervals as: 23,00 - 6,00, 6.00 - 9.00, 9.00 - 14.00, 14.00 - 17.00 - 19.00, 19.00 - 23.00.

For the case of a seismic event during a working day one can analyse the professional and age categories exposed predominantly at risk at home (children, old people, housekeepers, persons at home before or after job), exposed at risk while working or during travelling in town, a.s.o. For Bucharest, the preliminary estimation concerning the numbers of people at work gives for the interval 9.00-14.00, 800,000 persons (maximum) and 130,000 persons between 23.00-6.00 (minimum).

The earthquake during weekends or holidays will catch a great part of the population at home, except people working in special industries and tourism. A lot of people will walk or have entertainment in crowded public places, at sport games or shows. The cold season will keep 40% of Bucharest population (children and old people) at home. Mass secondary illness could occur because of an unexpected evacuation. The warm season implies a reduced number of people present at home because of vacation or activities in the open (especially in summer the reduction could reach 30%)-

Conclusion. Bucharest is a dense populated town, where the earthquake could produce major effects. However, the active and the 20-50 years old population, being in majority, the physical and psychological reaction ensuring the evacuation and rescue operations, will be positive. The daily motion of the population can considerably facilitate their reaction if earthquake preparedness is correlated in this respect. The education and informat ion of population about the pre-seismic measures could reduce panics, rummors, disorganization and pshychlogical stresses.

Industrial functions. Bucharest got in 1983 13.8% of the country industrial production, due to 214 enterprises. Machine industry and metal processing (42.4%), chemistry (12.7%), food industry (11.4%) are the main branches. Chemistry, medicine industry, oil industry, gas storage and supply that could spill dangerous substances following earthquakes are located in suburbs. However, new developed quarters are now very close to some of these new facilities.

Conclusion. A detailed analysis on damage potentials in facilities of each factory is necessary in order to avoid interruptions in the industrial activity. Special industry working with chemical substances must be included in particular programs of protection.

The commercial network includes (1983) 6,727 units out of which 1731 are public foodstuff units./3,6,7/. Warehouses are situated near railroad rings or on main highways. In the central zone of the town there are shops, usually at the ground floor of apartment buildings. In some quarters shopping centers were built also as independent units.

Conclusion. The commercial network plays an important role during or after the emergency period. Therefore, aseismic protection should be realized by:

- protection of buildings, especially the pre-1940 type;
- protection of goods in the warehouses and shops;
- protection of access roads to warehouses; storage of enough quantities of goods for a first necessity.

Politic and administrative functions. Science, education, culture, information, sport and tourism functions. The governmental, politic, administrative central leading institutions of the country and of the municipality are located in Bucharest. Buildings of different ages and condition are used by these institutions. A new politic and administrative civic center is under construction.

Science. There are 193 institutes of research and design with 96,618 people as personnel.

The education system includes 666 scholar institutions(1984) /3,5,6,7/ with 460,000 students.

Culture. There are 27 institutions such as theatre, opera,

1,297 libraries, 93 movies, 5 concert halls having over 1000 places each of them./3,5,6,7/.
The mass-media system includes radio and television, newspapers journals, a.s.o. Each family has radio and TV sets (465,388 radio sets and 590,846 TV-sets registered in 1983). Out of all the main daily newspapers seven are edited in Bucharest./3,5,6,7/.

Sport has a large network of endowments including 3 stadia (total of 130,000 places), 3 halls with over 20,000 places /3,5,6,7/.

Tourism benefits of 41 hotels with 10,000 beds. In this respect over 1.3 million tourists per year are registered.

Conclusion. The presence of the main central institutions in Bucharest, will obviously facilitate the entire fulfilment of measures for the earthquake preparedness. Starting with the experience of 1977, one can estimate that the existing administrative structures could include new tasks from the earthquake preparedness programs , too. The actual laws concerning the intervention in the emergency situations (floods, storms, earthquakes, a.s.o.) as well as the law in force concerning the safety of constructions are a useful juridic background.

The present education system should be protected against earthquake, used as a tool of antidisaster education and used even in the emergency period for temporary housing. Halls with large concentration of people should be paid special attention.

The mass media system in Bucharest has a modern structure but it must be prepared for transmission of exact information in a manner adequate to the earthquake phase. It is obviously necessary to protect their power supply. In the same range the institutes of research and design should be protected taking into account their role in the programs for the pre-earthquake protection, in taking emergency measures, in the rehabilitation works the survey and collection of data after earthquakes.

Anti-disaster functions. A seismic event could become a disaster in function of the mass loss of life, fires following earthquakes, epidemics, shortage of food, water, housing, etc. Functions able to facilitate the antidisaster operations in Bucharest present the following state of art:

Medical centers. There are 562 medical facilities with 6,811 physicians, 17,000 assistants and 7,5000 auxiliary personnel in Bucharest. In this framework 55 hospitals with 27,552 beds are in function. As main emergency facilities one can mention:

- the Ambulance station, established in 1906, endowed at present with 400 ambulances with FM radio communication; it has over 500,000 patients per year;
- the Clinical Emergency Hospital, established in 1934, endowed at present with 635 beds;
- the Municipality Clinical Hospital,with 1,540 beds;

- the Hospital " 23 August " with 700 beds;
- the Hospital of Neurosurgery;
- the Central Military Hospital.

Firemen, military units, traffic police teams, erection companies, the Red Cross Society, the Civil Defense have a good endowement and should cooperate in the earthquake preparedness programs.

Conclusion. The relative concentration of medical personnel in Bucharest (327 persons per physician) is beneficial but it cannot avoid some critical aspects. For instance, if from the mentioned number of 300 old high-rise buildings we accept a hypothesis of 45 collapsed buildings (that means an increase of only 50% as related to 1977) one can expect 3-4,000 potential victims. Let's suppose proper rescue operations done by the mentioned anti-disaster institutions.

Taking into account a ratio of injured per dead persons equal to 5 (in Bucharest, in 1977, this ratio was 5.45 /1,4/ , a huge mass of 15-20,000 injured persons in the emergency phase results. Therefore, the simultaneous treatment will be not possible. In several hospitals (only for 7,596 injured persons of the whole city) this problem occurred in 1977. It is obvious that the transportation of those persons (among which 5-7,000 could easily require an emergency treatment) will rise problems to the ambulance service and hospitals. For the dispatch and redistribution of injured persons the Ambulance Station of Bucharest and the main emergency hospitals use at present new FM radio-sets and phones interconnected for a joint intervention.

THE PRIMARY DISASTER SCENARIO FOR THE BUCHAREST MUNICIPALITY

The primary disaster scenario should provide to specialists planners, administrators and inhabitants an image about the level of damage and disruptions produced by the mentioned hypothetical earthquake on the town, using rough methods. Data on vulnerability of structures, functions and people in 1977 should be used for calibration of results. Because the building stock and the industry are certainly in evolution the scenario will be not a simple repetition of the 1977 event. Population and managers are changing too, therefore the experience of 1977 could not entirely be adequate to the present position of each person. Thus, this initial scenario, even rough, will represent a starting point for measures to be included in the earthquake preparedness program.

CONCLUSIONS CONCERNING THE BUCHAREST EARTHQUAKE PREPAREDNESS PROGRAM

The Program will consist of a set of documents including duties for different state and voluntary institutions. As Romania utilizes for all its social and economic development 5 years and 1 year plans, this program could be easily integrated in the general system. Long term measures (10-15 years) will include the general identification of the problems, studies and research, need of new codes, scheduling of activities, etc. Mid-term measures (5 years) will include actions for the reduction of the most evident disaster potentials. The disaster scenario will include cause-effect hypothesis adequate for the real conditions of the city. Priority should be given to lifelines. The education, information and antidisaster drills, adequate to different categories of population, should be implemented. The emergency phase measures (during and immediately after the earthquake, 10-60sec.) will depend on the degree of endowment and preparedness provided by previous phases, on the population experience concerning other past earthquakes, on the efficiency of the communication- information system. The response measures, specific in the following 24-72 hours after the shock are dependent to the degree of the social and economic planning and subordination, to the mass media activities, a.s.o. According to the rules in force in our country, in this period the necessity state is decreed and special funding aloted, rescue operation continue, the buildings are inspected in order to assess their degree of damage and safety. This operation implies that regulations concerning rescue,strengthening repair works should be in force before the seismic event. The Romanian experience, especially the experience of Bucharest has shown that the notion of recovery period must be understood in very large temporal limits, but the duration should be not increased by the shortage of means of intervention. It is well known the fact that in 1977 in Bucharest a lot of persons have been rescued from the collapsed buildings after incredible time intervals. For instance, a little girl (6 years old) after 62 hours, a woman of 22 years after 128 hours, an old woman after 188 hours. One of the world wonders of all times was the rescue of a young man (19 years old) after 251 hours under the debris, in a basement of a collapsed building. Therefore, measures in this recovery period will be started immediately and should refer to all the affected fields of activity.

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Fig.1.Right side of "Magheru Ave." in Bucharest, having mainly pre-1940 buildings. Building "A"is a reconstruction after 1977 of the collapsed building "Casata".

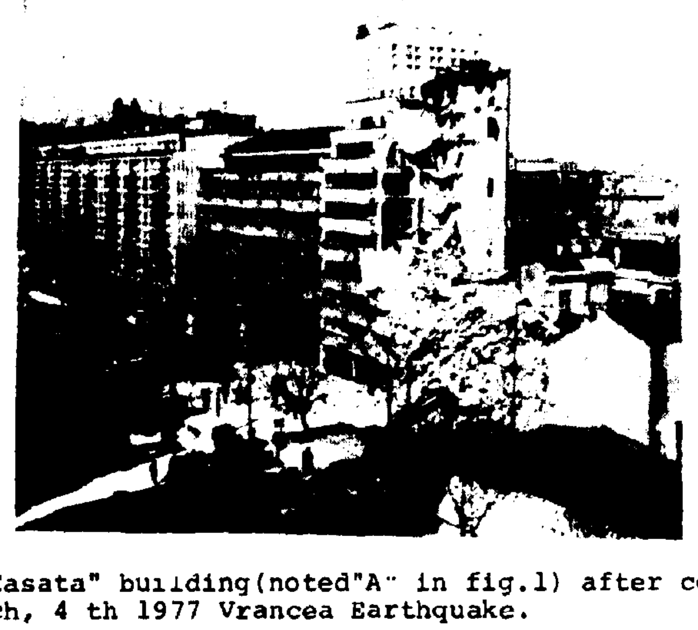

Fig.2 "Casata" building(noted"A" in fig.1) after collapse at the March, 4 th 1977 Vrancea Earthquake.

Fig.3 The New Concert Palace Hall in Bucharest built in 1960, ca.5000 places.

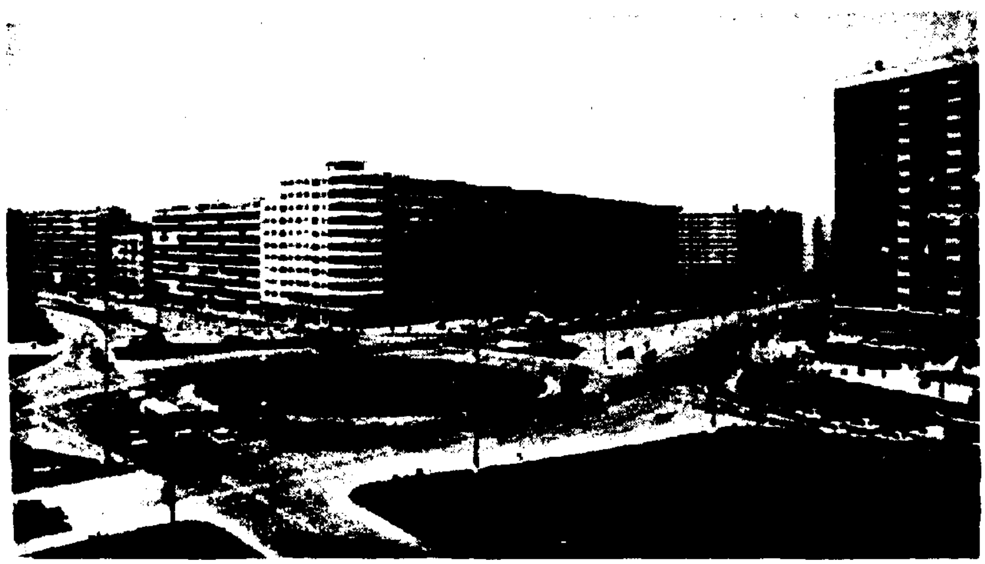

Fig.4 The new street crossing Bucur Obor with residential high rise buildings of the 70's Surface public transport and underground passage for trams, cars and buses traffic.

III.3 ARCHITECTURAL, ENGINEERING AND INDIVIDUAL REACTION ELEMENTS CONCERNING THE OPPORTUNITY TO EVACUATE APARTMENTS DURING THE EARTHQUAKE

Sever Georgescu[x]
Diana Măndruţă [xx]

1. INTRODUCTION

The specialists usually recommend to the occupants not to leave the apartment during the earthquake, advice that is not always observed or understood.

In this paper the problem consists of determining if the seismic elements as well as the structural, architectural and individual reaction elements allow or not a safe evacuation of structures with modern architectural layouts during the phases of a strong earthquake.

2. SEISMOLOGICAL CAUSAL ELEMENTS

From the analysis of the accelerograms regarding the crustal intermediate earthquake, results the following important aspects concerning the opportunity to evacuate apartments during the earthquake /1,2/;

- <u>initial phase</u> with oscillations in the range of 0.001 - 0.02 - (0.05)g on a duration of 2-10 s;
- <u>main phase</u> with major oscillations of above 0.05g but <u>especially</u> with values of 0.1 - 0.2 g, on a duration of 10-50 g (for magnitudes of 5.5 - 3);
- <u>final phase</u> of gradual damping of the oscillations <u>under perception</u> limit, on a duration of 17-30 s.

3. ELEMENTS OF DYNAMIC STRUCTURAL RESPONSE

Depending on the ratio between the natural dynamic characteristics of the structure and spectral dominant ones and the values of the seismic oscillations, the structure responds to the excitation transmitting certain effects to the lodger.

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The way the structure amplifies or attenuates the excitation also depends on the soil-structure damping system as well as on the story the apartment is located. Amplification values 1.5 - 2.9 were obtained at the earthquake in San Fernando, USA, 1971 on instrumented reinforced concrete buildings. Some corrections are necessary because of real location of the accelerographs.

During the Miyagi-Ken-Oki earthquake in Japan on June 12, 1978 accelerations of 0.264g at ground level, and of 1.06 g at the 9th floor of a building at Tohoku University were recorded, the amplification factor being 4 /4/.

Nowadays, before the earthquake, the dynamic analysis offer both theoretically and instrumentally the calculus data for each building type regarding the amplification assessment.

Concerning Vrancea earthquake with a predominant long period (1.5 - 1.8s), tall buildings, especially framed ones will significantly amplify the main seismic motion.

4. INTERACTIVE ARCHITECTURAL ELEMENTS

4.1. The layout type

For the uniform analysis of the relation between the apartments partition scheme type and the evacuation the defining of an evacuation vector ($\overline{E}$) (a displacement vector) is suggested that quantifies the necessary way crossed by the accupant from one room to another, up to the leaving of the apartment.

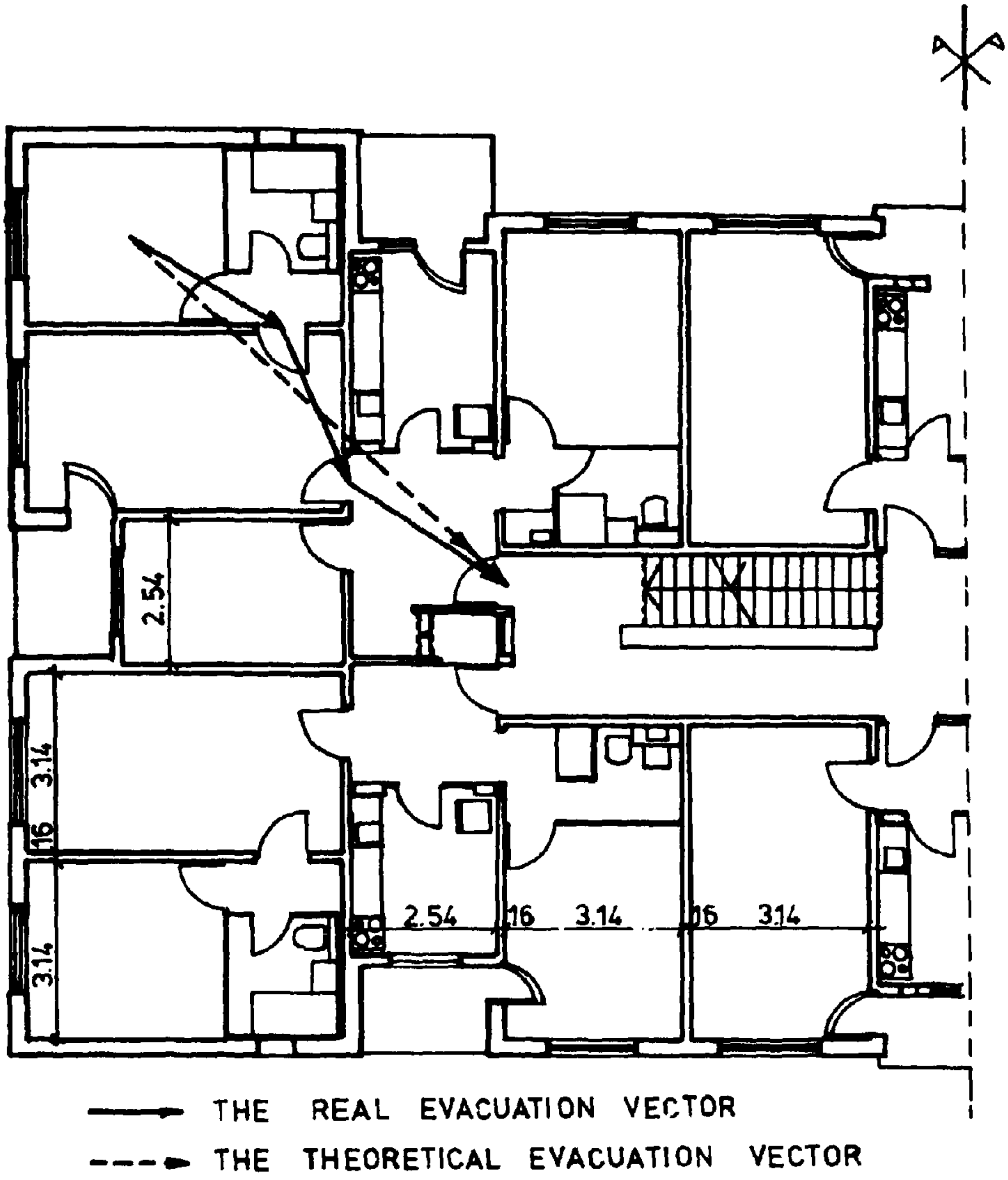
The ability to evacuate each room is different depending on the ratio "e" between the real possible evacuation vector ($\overline{E}_R$) and the position vector of the door ($\overline{E}_T$) as well as on the value ($\overline{E}_R$).

Generally speaking we will have:

$$e = \frac{(\overline{E}_R)}{(\overline{E}_T)} = \frac{\sum (\overline{E}i)}{(\overline{E}_T)} \geqslant 1$$

The values ($\overline{E}_R$) and "e" for 3 types of apartments from 2 Romanian standard building types are presented in table 1. These values allow for the following conclusions:

- the total value of the vector E increases (as expected



THE EVACUATION VECTOR FOR A ROOM IN THE CASE OF AN APARTMENT IN A STANDARDISED BUILDING

The real evacuation vector modulus and evacuation factor for apartaments in 5-9 storied precast buildings

Table 1

Parameter / Room	$\sum \lvert E_i \rvert$ (m)	e	$\sum \lvert E_i \rvert$ (m)	e	$\sum \lvert E_i \rvert$ (m)	e
5 Story Project type 770	Section P.b.2		Section P.a.4.mm			
	2 rooms		3 rooms		4 rooms	
Bedroom	5.75	1.095	7.1	1.07	7.1	1.07
Bedroom	–	–	6.6	1,4	6.6	1,40
Bedroom	–	–	–	–	8.7	1.87
Living room	3.8	1,04	5.6	1.07	5.6	1.07
Kitchen			3.6	1.14	3.6	1,14
Bathroom	5.1	1.0	6.35	1.18	6.4	1.18
Shower	–	–	5.1	2.5	7.8	2.8
9 Story Project type 772	Section B. 12.					
	2 rooms		3 rooms		4 rooms	
Bedroom	–	–	8.5	1.04	9.25	1,03
Bedroom	6.10	1.09	6.00	1.0	4.85	1.04
Bedroom	–	–	–	–	4.65	1.03
Living room	3.80	1.03	4.75	1.02	6.32	1.00
Kitchen	3.15	1.00	3.80	1,05	4.75	1.57
Bathroom	4.85	1.02	7.60	1.16	8.00	1.07
Shower	–	–	3.60	1.24	3.80	1.57

with the increment of the rooms number in the apartment;

- a bedroom and an afferent bathroom in an apartment exit; long corridors can cause crowding effects during evacuation;
- the living rooms have both the advantage of their location close to the exit and their purpose that presume a prompt reaction, unlike in the case of bedrooms where wakening and dressing delay the reaction;
- evacuation factors "e" have moderate values 1 - 1.1for 2 rooms apartments up to values of 1.16 - 1.57 for bathrooms and kitchens at 3-4 rooms apartments. By their purpose these room s have some other disadvantages as the dressing operations, water and fire turning off etc.

4.2. Staircase type

Staircase type is different function of the flight type, lighting system, number of apartments at each landing (the use of the elevator during earthquake is not considered).

Taking into account the typification of the stairs, the value of the initial vector that can characterize the displacement on the landing and on the flight between two stories is defined as follows:

$$(\widehat{E}_A) = \quad (H \text{ storey} \,.\, r + |\bar{E} \text{ landing}|)$$

where r = 1.0 for direct flights
1.1 for flights with intermediate landing base.
1.2 for flights with more landing bases or flights.

The index "r" introduces the slowing effect of the people leaving the building because of the change in the descending direction by turning.

The lighting system is included in the calculus by an incremental coefficient of the displacement vector as follows:

- natural lighting to exterior through windows:
  i = 1 (day); i = 1.1 (night).

- lighting to a small courtyard through a trap door or other sources: i = 1 (day); i = 1.2 (night)

- artificial lighting: i = 1.1 (day); i = 1.3 (night).

The number of the apartments on the floor is included in the calculus as follows:

1 - 4 apartments	n = 1.0
5 - 8 apartments	n = 1.1
more than 8 apartments	n = 1.2

Thus we will obtain:

$$/\overline{E}_S/ = /\overline{E}_R/ \cdot i \cdot n = (H_{story} \cdot r + /\overline{E}/_{landing}) \cdot i \cdot n$$

The values $/\overline{E}_R/$, $/\overline{E}_P/$, $/\overline{E}_S/$ for typified buildings in Romania are presented in table 2.

Table 2

Building	Parameter	$\lvert\overline{E}\rvert_{flight}$		$\lvert\overline{E}\rvert_{landing}$ (m)		i	n	$\lvert\overline{E}_S\rvert$ (m)	
		H_{story} (m)	r	min.	max.			min.	max.
GF + 4E non-lighted staircase		2.70	1.0	1.43	5.63	1.2	1.0	5	10
GF + 4E lighted staircase		2.70	1.1	1.9	1.9	1.1	1.0	5.4	5.4
GF + 4E non-lighted staircase		2.70	1.0	1.2	9.7	1.3	1.0	5.1	1.6

Enough cases where the staircase proved dangerous for evacuation either by its collapse or by the falling of some non-structural elements, plasters etc (San Fernando - 1971, Vrancea - 1977) are known.

4.3. Furniture type

Furniture delays the evacuation by:
- prevents the occupants to advance;
- overturning and hitting of the occupants.

By analysing the furniture types used in Romania, the minimal overturning accelerations according to building types are shown in table 3

Table 3

Furniture type	H/B or H'/B	$A_H (cm/s^2)$	I MSK
Modulated book-cases	4.6-5.00 (7.00)	(140)-180-210	VIII
Glass cases or high supports	2.9 - 4.6	210 - 340	IX
Sideboards	1.90 - 2.66	370 - 445	IX - X
Wardrobes	3 - 3.35	325 -300(0.3g)	IX
Cupboards	4.28	230	IX
Refrigerators	2.50	400	IX
Beds	0.4 - 0.25	2.5g- 4g	-
Tables, desks	1.75 - 2	490 - 560	X

Beside the probabilistic aspect of the oscillation and overturning under seismic effect some conclusions can be drawn

It is noticed that the greatest number of possibilities of colliding with an oscillating object occur in the offices with high book cases, glass cases, and so on, at story accelerations of VIII degree MSK.

In bedrooms, wardrobes present a uniform medium hazard at almost all types but starting with story accelerations of IX degree MSK.

Consequently, the furnishing of a dwelling must be done as to avoid agglomeration of rooms and corridors with objects or high furniture pieces. Fixing or furniture on walls or other members starting from a certain story could be taken into account in flexible structures.

4.4. Finishing type and non-structural elements

The degradation to oscillations of thin coatings of new prefabricated, industrialized structures is not dangerous. However, the degradation of framed structural systems with masonry infilling that often require plastering, veneering, etc.

can influence people phychology or endanger occupants life in a building.

5. ELEMENTS OF PHYSIOLOGICAL AND PSYCHOLOGICAL REACTION

Regarding the frequency range 0.1 - 10 Hz (T = 10 -0.1s) in ISO norms and other studies /5.6/) the following levels for the human vibrations are considered:

- the perception of the vibrations 0.001g - 0.01g.
- annoyance sensation 0.015g - 0.02g.
- tolerance limit 0.10g - 0.25g - (0.5g).

It can be noticed that the first two levels are characteristic for the seismic phase I, period when the occupant has not yet decided how to act.

The characteristic frequencies of the human body experimentally exposed to some vibrations on shaking tables are estimated as follows:

- f = 3 - 3.5 Hz; (T = 0.33 - 0.29s) for lying body;
- f = 3 - 6 Hz; (T = 0; 25 - 0.16s) for person sitting;
- f = 5 - 12 Hz; (T = 0; 2 - 0.08s) for person standing;
- f = 2 Hz (0.5s) for shoulders
- f = 3 Hz (0.33s) for head

at vibrations transverse to the body

These frequencies are mainly present in the IInd phase of the earthquakes, when the body or certain parts of it suffer oscillations of high amplifications at resonance. At the same time the components of low frequency that cause sensations similar to sea sickness can add to the unpleasant sensation, especially for Vrancea earthquakes with spectral content of this type.

Feeling his body exposed to oscillations and knowing the possible effects of the earthquakes the occupant will instinctively try to leave the apartment.

Thus the IInd phase is the phase of decision and the begining of evacuation.

It was experimentally concluded that under the effect of the oscillations the most instable position is the standing one. It is admitted that over 0.2g and sometimes even starting from 0.1g it is difficult to stand up or walk without leaning to somethihg.

The capacity to carry out action can be maintained up to 0.5 - 0.6g, only if the person can act sitting /3/.

The acceleration of 0.2g is important, because for an amplification factor 1.5 - 2, upper stories of some high buildings reach the acceleration of 0.2g when the ground has 0.1g (VII degree), the situation being of great interest for almost half of Romania

Under this strong oscillation a middle age person can walk by 0.3 - 0.5 m/s and kids and old men up to 0.3 m/s/7/.

For the descent (according to the authors determinations) the following speed under the effect of strong oscillations can be considered:

- 0.3 m/s (2 stairs/s) for teens.
- 0.2 m/s (1.5 stairs/s) for adults.
- 0.1 m/s (0.75 stairs/s) for children, aged men etc.

Season and hour of earthquake occurrence influence the evacuation by the increasing of the irresolution duration. Health, age and even sex differentiate the reaction during the earthquake.

An important psychological element is to know in advance the behavior of the structural elements existing inside each occupant's apartment under the earthquake action.

Thus the occupant in a framed building with infillin g masonry must not be surprised by stronger oscillations as well as by cracking of some walls right under his eyes.

The occupant of a large panel structure must know the fact that the rigidity of hte building is greater as compared to other types of structures.

The occupant of an old structure, with chimney, attics, ornaments, balconies and so on, must not leave the building during seismic shocks and even immediately after the earthquake in order to avoid the accident.

6. THE EVACUATION DURATION

According to the elements mentioned above the evacuation duration can be divided into the following partial durations:

- irresolution dutation that mainly depends on the functional aspect of the room dwelt by the occupant. Its maximum value is generally equal to the duration of the initial seismic phase; it is greater in the bedroom, bathroom, kitchen than in the livingroom;
- evacuation duration up to exit from the apartment

$$T_{ev.\ ap.} = \frac{|\overline{E}_R|}{V} \cdot e \qquad (5)$$

where the evacuation factor "e" introduces the increasing ef - fect caused by turnings, changes of direction, stops, doors opening and shutting, corridors as well as other factors that depend only on the apartment type.

- evacuation duration on the staircase

$$T_{ev.\ staircase} = \frac{|\overline{E}_R|}{V_r} + \frac{|\overline{E}_p|}{V} \cdot b$$

where V_r represents the descent speed on the flight.

V represents the speed of displacement on the landing and b is the number of the stories.

In tables 4 and 5 the partial and total evacuation duration, calculated for typified buildings with GF + 4 and GF + 8E are given.

The calculus was carried out regarding an earthquake occuring at night at 22.00 o'clock, when the members of the family can be simultaneously present in various rooms.

Aggravating and favouring elements were included in the factor "e".

CONCLUSIONS

1. The types of apartments and the staircase in the analysed multistoried buildings do not allow evacuation before the main phase of the earthquake.

Regarding most cases, the evacuation duration will occur with the main phase and in other cases the descent will not be ended before the last phase of the earthquake. Thus,it can be concluded that other more complicated architectural layouts are more difficult to be evacuated.

2. At tall buildings the evacuation duration on the staircase takes the longest duration and at the same time it is the most dangerous because of some specific structural and non-structural elements existing on this way.

3. The analysed elements can allow a better architectural

Partial and total duration of the evacuation for 2, 3 and 4 room apartments in nine storied large panel building typified project 772 - IPCT

Table 5

Duration of evacuation / Room	Duration of indecision (s)	Duration of evacuation for the apartment (s)		Duration of evacuation for the staircases (s)		Total duration of the evacuation	
				Electric light	Natural light		
		min	max	min	max	min	max
Bedroom	10	8.8 - 9,6	29 - 32	10s (g.f) - 120 s (9th story)	25s (g.f) - 615 s (9th story)	28.8 - 140	64 - 657
Bedroom	10	6 - 6.7	20 - 22			26 - 137	55 - 647
Bedroom	10	4.8	16			25 - 146	51 - 641
Living room	5	3,9 - 6,3	13 - 21			19 - 131	43 - 641
Kitchen	10	3,2 - 7,5	11 - 25			23 - 137	36 - 650
Bathroom	15	5 - 8,9	17 - 29			30 - 144	57 - 659
Shower	15	4,7 - 6	16 - 20			30 - 143	56 - 650

Partial and total duration of the evacuation for 2, 3 and 4 room apartments in five storied large panel buildings typified project 770-IPCT

Table 4

Duration of evacuation / Room	Duration of indecision (s)	Duration of evacuation for the apartment (s)		Duration of evacuation for the staircases (s): Electric light		Duration of evacuation for the staircases (s): Natural light		Total duration of the evacuation (s)	
		min	max	min	max	min	max	min	max
Bedroom	10	6,3 – 7,6	21 – 25	9 s (g.f) –58 s (5th story)	24 s (g.f) –220 s (5th story)	8 s (g.f) –57 s (5th story)	22 s (g.f) –148 s (5th story)	25 – 76	55 – 255
Bedroom	10	8,4	28					28 – 77	62 – 258
Bedroom	10	16	54					35 – 84	88 – 284
Living room	5	4 – 6	13 – 20					18 – 69	42 – 244
Kitchen	10	4	13					23 – 72	47 – 243
Bathroom	15	5 – 7,5	17 – 25					29 – 82	56 – 260
Shower	15	12,8 – 22	43 – 73					37 – 95	82 – 308

design and furnishing correlated with the seismicity.The people reaction during earthquakes to avoid endangering of life and limb integrity could also be improved by earthquake preparedness programs.

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III.4 CONTRIBUTION OF TOWN PLANNING TO THE MITIGATION OF URBAN VULNERABILITY

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GENERAL CONSIDERATIONS

Town planning may influence in certain circumstances economic and social processes, becoming an useful instrument for the developing strategies that decrease the disastrous effects of the earthquakes.

It is well known that Romanian territory is exposed to the dangerous action of the earthquakes.

According to the records from 15^{th} century until now(fig.1) the recurrence time of earthquakes of intensity VII or more in the Vrancea Region is about 30 years.

An earthquake of intensity equal or more than VII degrees is expected to occur in the next 30 years.

This closed probability must determine the development process in near future.

THE 1977 EARTHQUAKE

The 1977 earthquake caused damage to a large area including some important towns. Bucharest, the capital of Romania, was affected too, a great number of buildings being damaged and some of them being totally destroyed.

The town is developed within two major traffic rings, with Unirii Square and Universității Square as centers.

- it must be noted that 30 years is only significant as a probability. Generally it is considered that a devastating earthquake occurs two or three times in every 100 years.

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The North-South axis is composed from Road Victoriei and -Ana Ipătescu - Gh.Magheru - N.Bălcescu - 1848 - boulevards which cross the town from North to South (fig.2). On this axis the traffic was obstructed for about 10 days by the wreckages of the collapsed buildings or by the rescue actions (fig. 3,4,5,7,9,11,13).

This situation led to the disfunction of the transport systemnaffecting the whole urban system too.

The principal causes of the damage were:

- structural design without precautions for seismic forces
- reduction of strength due to the 1940 earthqauek;
- deterioration of structural strength due to the passages of time;
- major alterations after construction involving the removal of structural elements.

Now, we can also say that the loss of coherent actions aimed at ensuring a preparation before the disaster, such as vulnerability studies, also contributed to the recorded losses. These vulnerability studies would undoubtedly have demonstrated the unfavourable situation of the damaged or collapsed buildings. Studies on urban planning problems would undoubtedly have demonstrated the critical points of the urban system and would indicate directions for intervention.

RECONSTRUCTION

The reconstruction process was very fast, a great number of the collapsed buildings being rebuilt and the great majority of the damaged buildings being repaired.

Unfortunately not all the lessons of the recent earthquake were kept in mind.

Analyzing the zone around Romana Square, Universităţii Square (fig.3), from the urban planning point of view, we can favorably appreciate the large spaces existing on one or another part of the traffic axis. This solution wans't used at the other important traffic axes built after the 1977 earthquake - for exemple Moşilor Way.

The reconstruction using interbuildings in the original form or respecting only their original volume (fig.6,8,10,12,14) with framed structures with autoclaved cell concrete masonry leads to vulnerability situations such as these:

- joining buildings of different stiffness and heights may generate breakings and damages;
- this also may affects transmission phenomena at adjoining buildings (by the eventual yielding of the old buildings) towards new buildings calculated as individual ones;
- a potential danger for passers-by outside and for those

inside the building may result from using framed constructions whose finishing is achieved by thick plastering on autoclaved cell concrete that do not resist to seismic loads.

CONCLUSIONS

The probability of occurence of a great intensity earthquake in the next 30 years must be considered for development process.

The fact that all the new buildings are aseismically designed is not enough.

A complex aseismical program is necessary including vulnerability analysis and scenarios, which must consider a great number of problems in order to ensure the strength of the urban system against seismic shocks, encouraging not only local changes of the individual components, but also a systematic approach of the whole urban structure.

Town planning may offer the complex image of the urban structure, including important data concerning the critical points of the urban system.

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	INTENSITY	NUMBER
1941	VIII	1
1516	IX	1
1543-1545	VIII	2
1569	VIII	1
1590	VIII-IX	1
1604-1606	VIII	2
	VII-VIII	1
1620	VII-VIII	1
1637	VII-VIII	1
1679-1681	VIII	2
1701	VII-VIII	1
1738	VIII-IX	1
1778-1893	VIII	1
	VII	3
1802	IX	1
1892	VIII-IX	1
1868	VII-VIII	1
1893-1896	VII	4
1908	IX	1
1940-1945	IX	1
	VII-VIII	1

Fig.1

Earthquake of Intensity VII or more in the Vrancea Region since 1491.

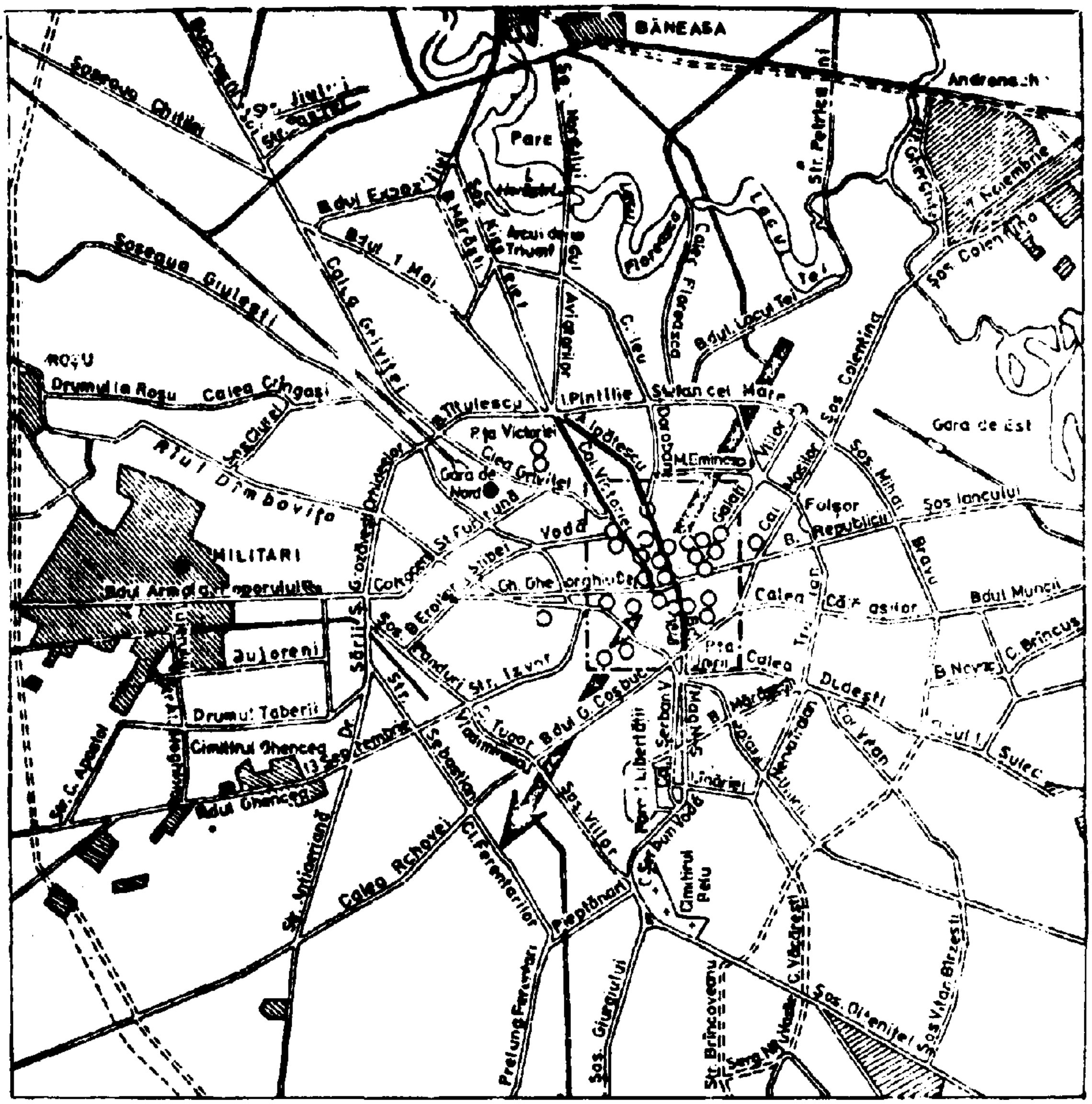

Fig.2

Colapsed buildings in the 1977 earthquake:

○ buildings built before 1940
● new buildings

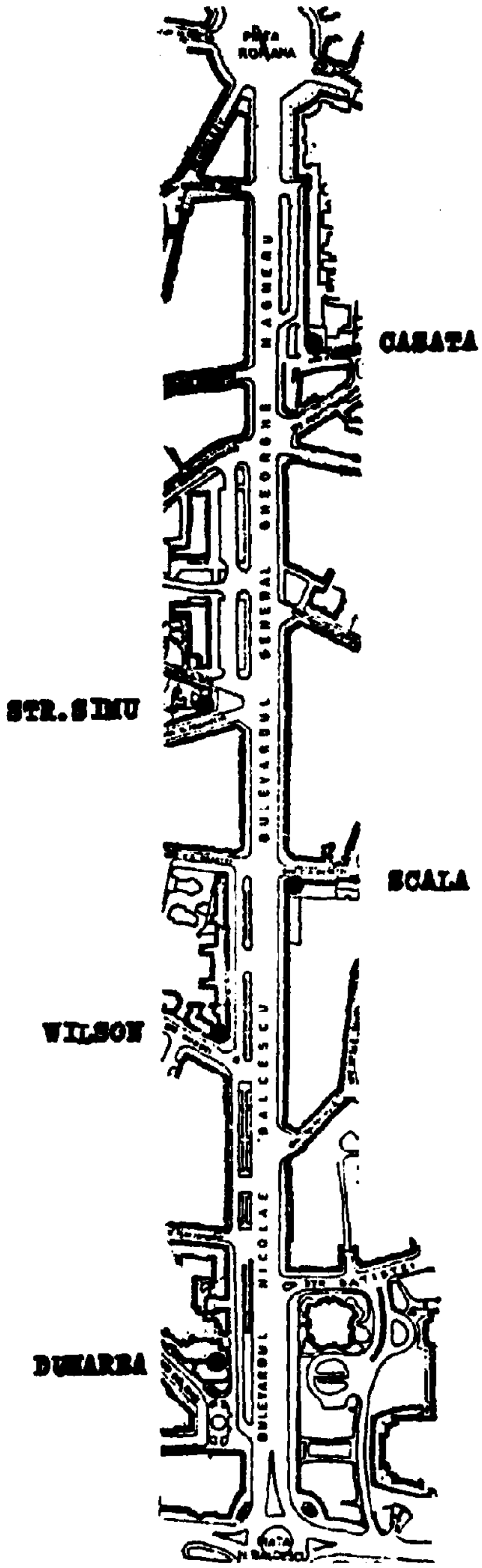
CASATA
STR.SIMU
SCALA
WILSON
DUNAREA
BULEVARDUL GENERAL GHEORGHE MAGHERU
BULEVARDUL NICOLAE BALCESCU

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Fig. 7

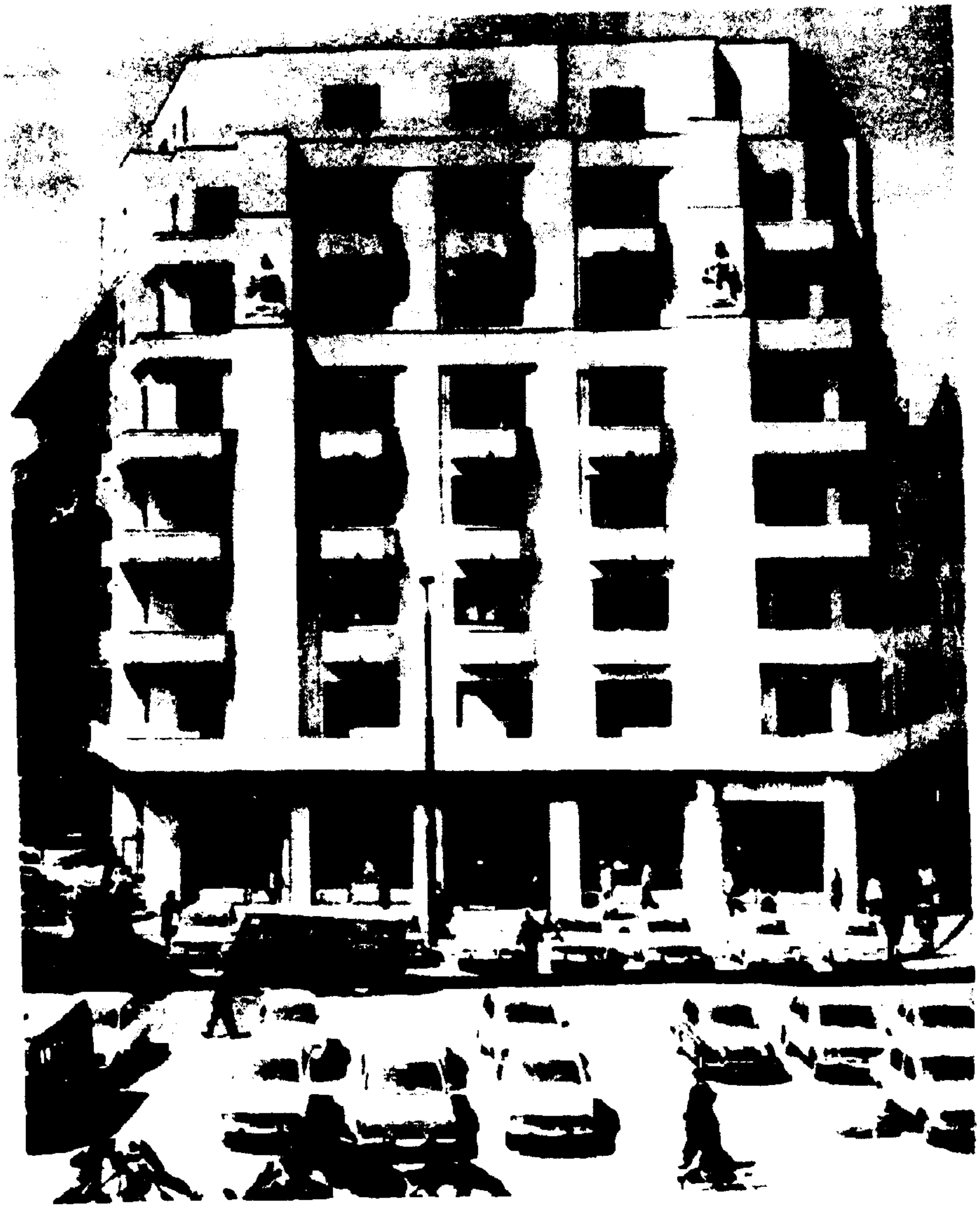

Fig. 8

Fig. 9

Fig. 10

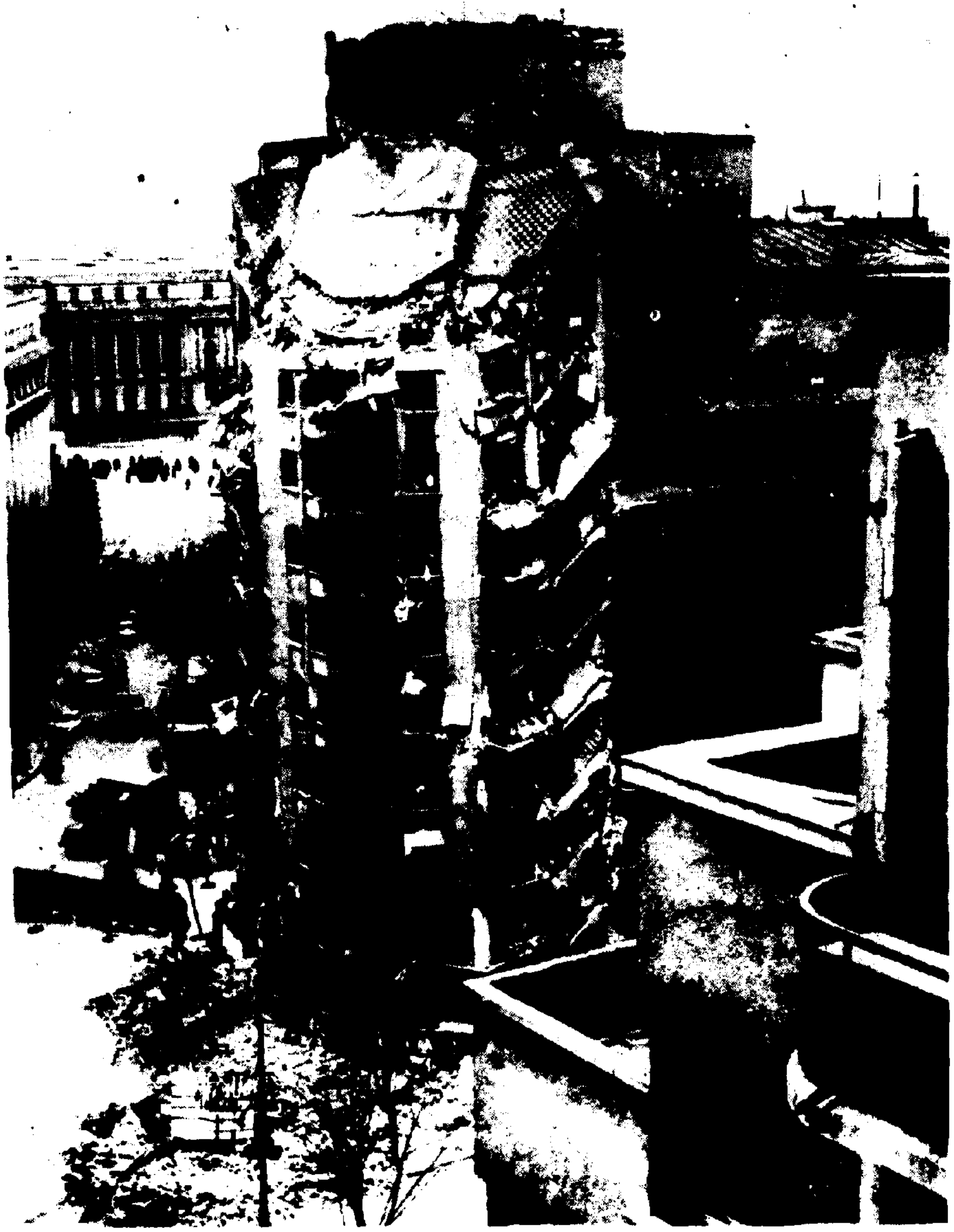

Fig. 11

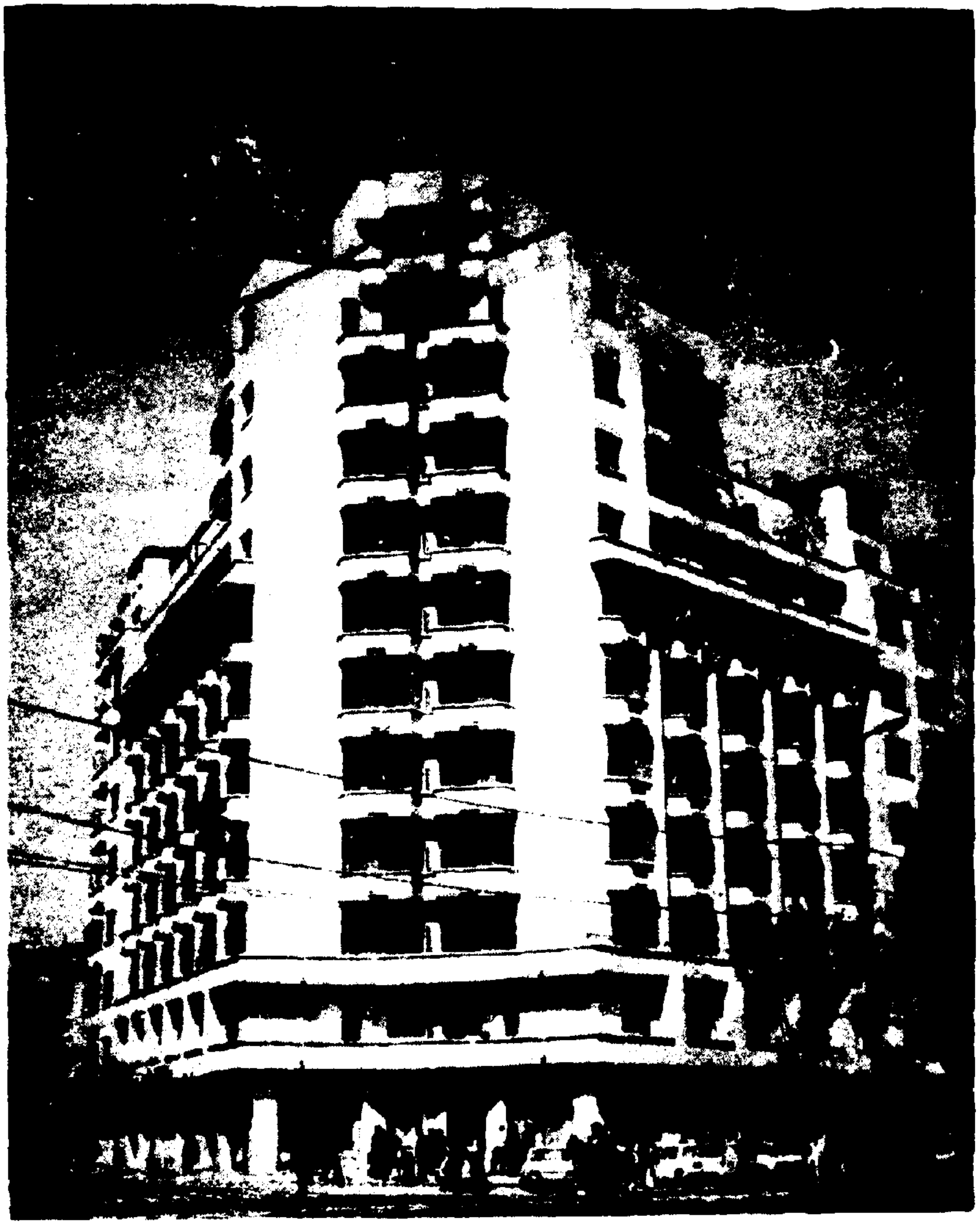

Fig. 12

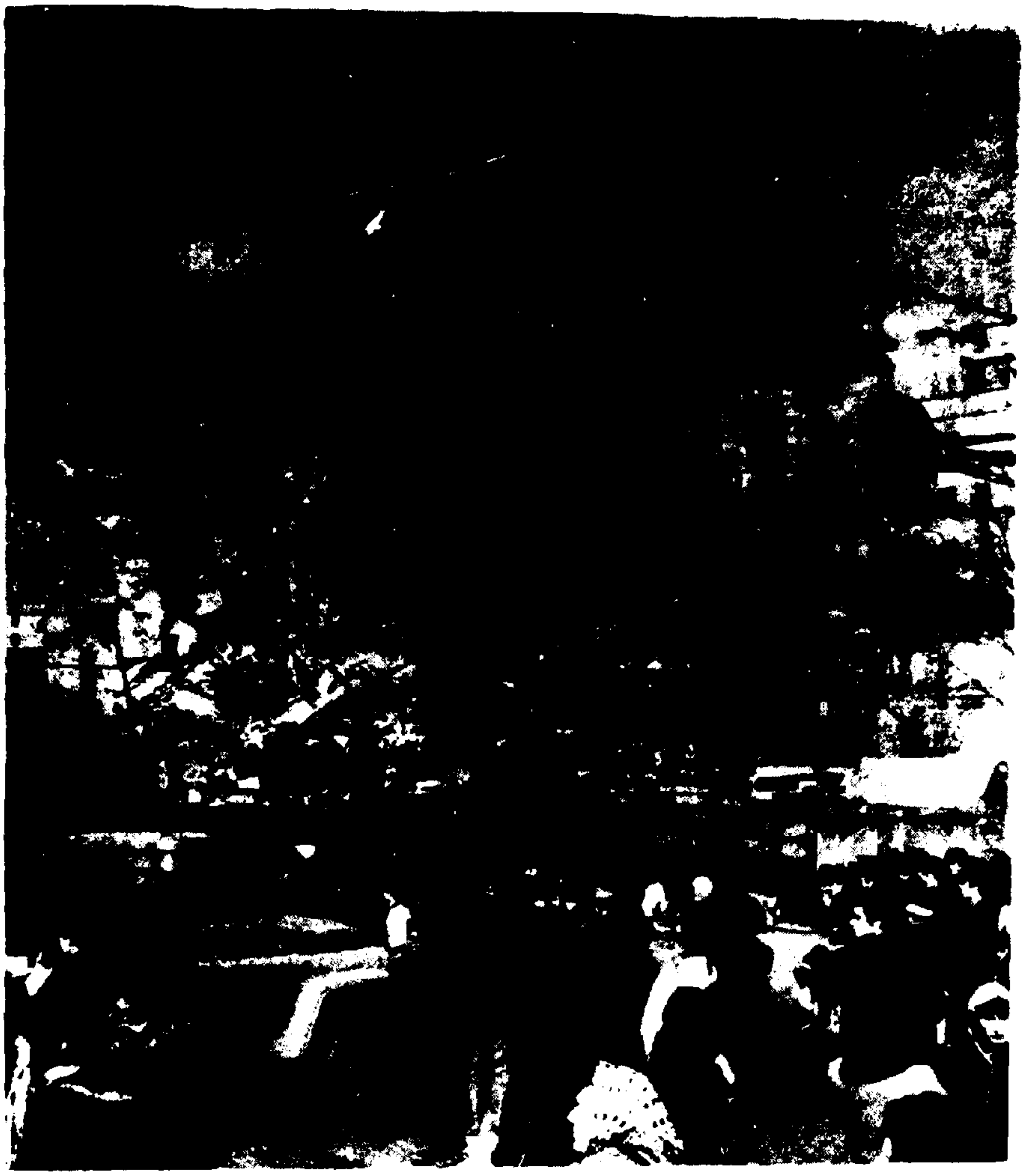

Fig. 13

Fig. 14

III.5 THE SOCIAL RESPONSE TO EARTHQUAKE

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Nicolae Gheorghe [xxx]
Mircea Kivu [xx]

One of the basic ideas of our studies is the acceptance of the earthquake protection, widely speaking, as overall action representing societal response to seismic situation, including, besides the technical-engineering measures, a large set of social ones.

From the study of the population behavior during the March 4, 1977 Earthquake (Romania) it resulted that the main elements of these particular type of social response are:

a) the preparedness of the population for the emergency situation, namely the understanding and learning of the reactions to earthquake, both from the point of view of the information they must posses and of their effective behavior they must develop during the seismic situation;

b) rescue and emergency assistance actions for the affected population;

c) means for the population's return to normal life,focused on the solving of problems related to "provisional shelter".

The specialized literature stresses that an earthquake becomes a disaster only by reference to a social context.An earthquake is a catastrophe when it disturbes some essential social functions, respectively it bringsabout the disruption of the social structure, endangering the life and survival of people, the social order and the cultural values. By its content and its effects (at different social and temporal levels),the earthquake may be considered a developmental matter.

The social response to earthquake (as program for the reduction of earthquake social effects) is efficient only integrated in development politics.

The human, economic, social and psychological costs

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caused by an earthquake are strong arguments for the complex social programs of social preparedness in the aim of prevertion and the reduction of destructive earthquake effects.

The experience of the countries that accept the idea of the preparedness for earthquake - including the people information, the behavior scenarios and the specifical measures for earthquake - proves that people are able to accept the earthquake as a normal phenomenon where the learning of the response is fundamental.

Moreover, the preceding exercises for disaster situation allow to avoid social disorders specific to limit situations.

From the view point of the response motivation,the preparedness for earthquake may be assimilated with other preventive actions for crises situations, like the prophylactic measures of avoiding the epidemics, already accepted by the population like normal ones.

Also, the earthquake action contributes, indirectly, at the stimulation of ideas, promoting the social change(and the interest in this), correlated with the creation of a more favourable context for the reconstruction and the modernization of the building stock.

Consequently, such preparedness programs (already existing in different countries at seismic risk: Greece, Turkey, Japan, USA, etc.) include many factors that "co-operate" from the seismicity of the soil until the education and the attitudes and reactions of the population.

In the specialized literature, it is often emphasized the necessity of analysing the direct earthquake effects in correlation with the effects of the intervention measures for reducing the damaging consequences of the earthquake, as natural phenomenon.

At the same time, there is a consensus that the affected individuals and social groups are not only the subjects of some formal measures of emergency assistance, they have their own patterns of acting in such situations, in other cases differently (or even oppositely) from the official ones, they have their "personal" rhythm and time representations for taking decisions, according to tradition and private and group experience.

Basically the social response to earthquake may be studied at four levels: society, community, organizations and individuals.

If the social level is not very analysed in the literature of the field, other aspects of social response to earthquake are largerly approached, with respect to different spatial and temporal limits.

At the community level, the social response to earthquake is elaborated by and in the process of changing the normal functions and priorities system of the community.

These transformations consist of changes and adjustements of current activities and social processes, for the purpose of elaborating an appropriate answer at disaster situations.

In case of earthquake the normal system of priorities (informal and formal ones) is endangered. The period of the disaster action (the direct effects and the correlated social ones) produces change in the social relations by generating new adaptative patterns, "the social system of the emergency period".

A major disaster that menaces the fulfilment of some valued activities determines a drastic demand of certain services or activities, simultaneously with the reduction of the capacity of providing them.

The answer at the emergency state include, first of all, a new priorities and values system.

These changes in the "normal" order of the priorities system, generally noticed in the communities affected by a major disaster and "tested" in our country, are found on some particular criteria:

- the population survival is the first priority
- rescue operations, to save people, to provide medical assistance, become priority actions.

In the case of the March 4, 1977 Earthquake, Romania, this problem acquires a nati nal importance, the program of intervention being co-ordinated by the central government.

The survival and rescue operations are correlated with the activities of providing bare necessities as food, the"provisional shelter" a.s.o.

- re-establishing and maintaining of essential servicies and resources for the normal development of social life.

The public needs get top priority with respect to private activities, sometimes coming into conflict with personal and formal interests and duties of the individuals.

The most importance activities in this period are the rehabilitation of the public system of communication and transportation, of the institutions directly involved in the activities for social life preservation (hospitals,police, army etc.).

- maintaining of the public order, protection of the social and private property, control of circulation and distribution of the rescue resources;

- maintaining of people's morale, and keeping up psychical condition are very important in the emergency period.

In present societies, mass media play an important role in the "description" and the interpretation of disaster effects, in the process of presenting proper information for the intervention activities, for the maintaining of the public quiescence and discipline.

A very important fact to keep up the community morale is the rejoining of the separated families and to gather information on the condition of these families.

Concomitantly with the priority accorded to these functions in the emergency period, some traditional functions suffer a diminution of their social importance.

Therefore, some of the functions of goods production,distribution and consumption are partially re-oriented, others change importance of the factors which contribute to their achievement (see the function of socialization). Also, the functions of social control and mutual helping acquire new dimensions and values.

- the organizational level of the social response to earthquake is sustained by organization and institutions (in the classical sociological meaning).

The analysis of the social answer at disaster situations at the organizational level becomes especially needful. Generally, it is considerated (both at popular level and, sometimes at academic one) that the principal effect of the disaster is the disorganization of the social structures and of the daily life. But the systematic research concerning disasters prove that these phenomena have, in the respective period, some "integrating effects" with a therapeutic role for the affected community", both the "disorganization" and the " reintegration" being - the dual effects of the complex reactions of a community confronted with a disaster.

A disorganized community generally develops an organi -

zational structure able to satisfy the new requirements by a disaster.

The analysis of the social answer at organizational level convers the identification of different organizations of the community, before the earthquake; the assessment of the probable consequence of the disaster actions on the specific functions of the organizations; the knowledge of the way in which the available resources are mobilized and of the new systems created by the community and the particular social groups for the solving of the newly created problems. Some organizations contain in their statute the responsability to be involved in urgent problems and situations (or in "emergency period specific situations"), and in the actions of disaster pre- and post-impact period.

Other organizations, woth no responsabilities and structure for the immediate intervention, become necessarly involved in the managing of the particular problems generated by an earthquake, suffering the costs of the new adaptative efforts.

Hence, the capacity and the efficiency of the existing organizations constitute a decisive factors in the solving of these particular crises; their structure and their capacity of intervention have to be estimated beforehand.

The preceding classification is valid particularly for the organizations that are usually confronted with urgent events; the police, the firemen, the hospitals, the transportation system etc. The involvement and the co-participation of a lot of organizations, associations, social groups(sometimes newly created by this specific social context), non- organized individuals draw up a complex problem of coordination between all those social factors: this problem represents a particular social characteristic of the emergency period.

The characteristics of the coordination and communication between these social agents and of the process of "raising" a legitimated authority structure, have specific cultural and political variations in different countries and areas(in respect with the cultural, social and political structures of the region).

The interorganizational coordination and cooperation is easier to be achieved and more efficient in the central planned societies. In such countries, the social response for earthquake is considered to be, firstly, a national responsability that implies the government control of all the process of distributing and using material resources and people.

In the case of March 4, 1977 Earthquake, Romania (as in other periods of disruption produced by calamities in our country, like the 1970 flood), the coordination and the adjustement of the organizations at the national, regional and local levels was achieved by teams especially created for acting in such situations.

The teams'experience proves that one of the principal means in eliminating the earthquake effects is the mobilization of the local resources and their integration in new patterns of utilization.

A very important aspect of this process of mobilization is the increase of human solidarity (the emergency consensus):

- the experience of the "headquarter for reconstruction" of Zimnicea town, locality reconstructed after the earthquake, demonstrates the capacity to mobilize economic resources and labour force from all over the country;
- the individual level of the social response to earthquake covers the reactions of the people in disaster situations.

The Romanian experience of the 1977 Earthquake "advocates" a major conclusion of the specialized literature: the population develope adaptive answers for the different stages of the disaster action, the panic and the disorder of the immediate post-impact period being stereotypes and lacking the understanding of the human behavior, considered only by some superficial and transient aspects.

Generally, people are innovating "new" behaviors, apparently chaotic,, ineffective or alarmist, but in fact testing creative solutions for unexpected situations or/and"bringing up-to-date" traditional representations and experiences concerning the response to earthquake.

With respect to the particular increase in human solidarity, the therapeutic role of the communities and social groups affected by calamities is emphasized.

The people's behavior in such situations is modelled by the particular and social experiences during periods disasters actions, by tradition, by some group leaders influence etc.

People act for surviving, for saving and helping their family members, neighbours, friends, and develop an open and active attitude towards the community problems.

The people preparedness for earthquake is an important factor of rapidity and efficiency of the individual and social response.

The preparedness programs have to train people for adequate actions at different seismic stages and for the ability to coordinate (psychological and actional) the private and the social acts and interests.

Mass media and other communicational agents have an important role on people behavior providing accurate information and even guidance in acting.

The preparedness programs for earthquake have to be developed in advance, using all the available formal and informal means, including the schools, the different organizations and institutions, mass media, the civil defence actions, in the purpose of a complex learning in advance of specific behaviors.

III.6 SOCIAL AND URBAN ASPECTS OF SEISMIC PROTECTION OF TOWNS

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Before the earthquake of March 4, 1977 the seismic protection of the built environment was approached from a design technical and engineering viewpoint, emphasizing the building protection as an individual subject. The interaction between the seismic energy and the structure of the building represents the object of this approach. The technical and engineering approach on the relation between great intensity earthquakes and buildings was largely developed after the November, 1940 earthquake and it offered constructive protection solutions that lead to a real diminishing of the building seismic vulnerability. The elaboration of the seismic protection prescriptions raised great interest, as the lack of a specific norms collection made the research very difficult.

The scientific studies settle the following immediate town-planning objectives, in order to grant seismic protection:

- risks mitigation;
- diminishing of the negative consequences of the damaging events;
- diminishing or preventing of the secondary and tertiary effects;
- taking under control of the calamity area and the effects circumscription;
- facilitation of the emergency operations;
- ensuring of an optimal functionality of the urban area immediately after the earthquake;
- people organizing and return to normal life;
- reconstruction process facility.

Theoretically towns with a total seismic protection may be built, but experience proved that, if this solution is possible for certain important buildings, it is not economically achievable for all the existent urban settlements. The analyses

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of world wide achievements based on seismic protection programs proved that the town planning activities were the least covered by indicators. For the elaboration of the general town-planning measures and principles that must be taken into consideration at national or regional levels in order to prevent or to diminish the calamity consequences, the following aspects were considered: the selection of the localities site; population and activities distribution; ways of using land functions disposing; road network; general conception of the urban under structure; detailed conception on different elements of urban agglomeration (buildings shape, streets lay-out, landscape arrangement).

The town planning problems and the urban evaluation are also strictly connected to seismic risk analyses at a system level, the definition of intervention alternatives and the adoption of the adequate decision.

Thus, dealing with the built systems that may be affected by seismic action - the built assemblies geographically concentrated or the infra-structural networks - the events "chains" that may produce great losses are studied: fire rise and spreading; impediments in evacuation caused by wreckage blocking the streets; intervention impossibility in order to save people or to struggle against fire. It is thus necessary to adopt an indices system of the critical components. These indices must be included a general plan for protection and decrease of the seismic risk.

This plan should consist of a code base for "aseismic design" at the system level, the adoption of intervention decision, the designing of the repair and consolidation works.

For a better understanding of all aspects concerning the aseismic protection of the built environment (considered as a whole of buildings and functions, in a unitary and structural conception), we consider the integration of different elements that will make up specific components of the aseismic town-planning very important. In this way we were interested in defining the main ways of including the aseismic protection in the town-planning activity and the urban development in Romania.

The estimation of the aseismic town-planning component elements - that are very important for the working out of some prescriptions and recommendations concerning the town-planning activity - is the result of the research studies developed by a Delphy-type investigation among 80 experts (engineers and architects) from designing, research and higher education level, in two stages.

The necessity of the second stage that included the investigation of a small number of experts (29 engineers and architects) was required to clarify and to establish an agreement about some disputed aseismic urban problems and about the actions in the case of an earthquake, resulted from the first round of the investigation. The cooperation with the experts was achieved on the account of some questionnaires mainly interested into the affirmation/infirmation of the agreement concerning the possible elements of aseismic town-planning that had were not agreed on during the first stage. From the comparison and the synthesis of the answers received during the two phases, a number of characteristics, definite elements and propositions that are of help in seismic protection were determined, to become a component of urban planning in Romania.

The result was that for the aseismic town-planning the following elements and the following priorities should be taken into account:

- (i) local seismic conditions and geologic structure ;
- (ii) strength capacity of buildings;
- (iii) social-economic importance of the building;
- (iv) buildings density;
- (v) supply network functionnality;
- (vi) widening of the thoroughfares;
- (vii) distance of the dwellings frontage from the road;
- (viii) green spot area inhabitant;
- (ix) free spaces;
- (x) site of the industrial areas in respect with the dwellings areas.

The agreement on these items allowsus to estimate that in the planning studies for certain areas of the country the elements mentioned above should be considered. As many the previous elements are in fact correlated, the hierarchy above is orientative and should not become a diminishing criterion for any element and in no case the favourable solution for one to the detriment of the other.

Another aspect underlined in the research study is the influence of the seismic degree of the areas on the main functional and constructive solutions for apartments. We considered the possible link between the seismic macro-area of the country and the selection of the partition solutions for dwellings, as these partition solutions directly influence living comfort.

Of a special interest are the architects answers, as they are first confronted with the possible limitations implied by the building structure over the functional spaces into the apartments. Among the problems raised by dwelling design in highly by seismic areas are:

- the limitation of the apartments flexibility becomes an important indicator for modern dwelling;
- the stiffness of the structural strength leads to

limiting the openings of the construction elements and imposing certain forms on the plan.

Another problem connected to the town-planning and urban development actions is also the renovation strategy of the localities damaged by a strong earthquake. The space reorganizing, the town functions and activities normalizing are conditioned during the first reconstruction stage by the house stock stability and strength. These aspects represent the most of the activity types that are developing in the community. Thus, reconstruction is parallel to demolition or to the strengthening of the buildings that were subjected to important damages. In order to establish the urban development ways for the localities subjected to considerable damages, the experts consider the following criteria as most relevant:

- (i) the historical-architectural value of the building;
- (ii) the technical-economic aspects;
- (iii) town-planning requirements;
- (iv) inhabitants' expectations.

It is interesting to notice the common opinion of the experts on the historical-architectural value of the damaged buildings as being one of the most important criterion. This fact expresses the engineers' and architects' interest to offer stability and permanence to some buildings that, except their daily functionality, are also representing historical, cultural or aesthetic value. Considering the planning requirements in terms of the above mentioned hierarchy leads to the conclusion that urban space organization and development are less influenced by the seismic degree of the considered area.

The strengthening of the damaged buildings should also be considered as an aspect of housing comfort improvement. The majority of the questioned experts appreciated as very convenient the strategy of combining the strengthening works with the modernizing of dwelling functionality. In spite of the additional costs required by modernization, about half of the experts that carried out strengthening designs for old buildings also used a large set of elements meant an increase living comfort.

The seismic protection modern conception has a larger meaning that covers the structural strength part, the nonstructural elements and the town-planning criteria. Many of the opinions concerning the necessity of some new design systems for the appartments configuration mention the necessity of additional prescriptions for the nonstructural elements, the access ways or the installation types. These objectives influence the expenses for dwellings building that are to be analysed and then a priorities set must be settled. It was noticed that the preoccupations concerning the buildings stability under seismic action have to consider studies on strengthening, to provide seismic stability to the nonstructural elements and to the height of building systems.

The lack of agreement on the importance of the other elements (spaces and access ways, buildings density, energy and heat installations) does not mean that they are not important but it proves that their influence is indirect and that the secondary and tertiary effects are more difficult to be foreseen.

In our country the seismic protection principles in urban planning may be introduced into the systematization, restoration and modernization strategies of each town. The relation between danger and vulnerability - beyond its technical character -should also consider the economic,political and cultural value that is different in each country. Among the important elements of the seismic protection that may influence urban planning and development that resulted from our researches we mention: the density and the distance between the buildings; the distance between the buildings frontage and the street, depending on the buildings height; spatial disposal of the buildings compared to the main direction of the seismic shock waves propagation; the location of different types of activities into the urban area considering micro-areas maps; access ways corresponding to the buildings of social-economic importance and to the buildings with special functions, to be used when an earthquake occur ; the setting up of the streets network and of the thoroughfare widthness with regard to the localities dimensions and the seismic vulnerability of the area; the avoiding of the stranglings in the area exposed to the seismic danger; the site of the green spots and the ratio between these surfaces and the number of inhabitants,the ratio between the built surface and the surface of the free spaces; site and foresight of emergency hygienic and sanitary endowments on the free spaces; types of materials that are seismic resistant for the town pipe lines; flexible catching up systems , automatic disconnecting systems, division of the network by sectors a.s.o.; the location of the energy supplies providing functional independence; technical-economic and social options concerning the possibility of a temporary dwelling. The data and the values of such indicators or criteria may be the result of studies and interdisciplinary researches in which architects, engineers, economists, sociologists and physicians should cooperate.

Another research chapter is devoted to the reconstruction process as it was developed in Romanian towns that were greatly affected by the earthquake of March 4, 1977: Bucharest, Craiova, Ploieşti, Vălenii de Munte, Zimnicea. Interviews were taken with the specialists of the regional designing institutes who directly participated at the elaboration of the reconstruction designs, governmental and districtual papers of that period were studied and the most important data were extracted from the speciality studies.

An approach by stages of the process, was agreed on according to literature; the following post desaster action stages were settled:

I. The emergency period is the one when most of the normal social-economic activities are interrupted, the community being confronted with the problem raised by the damage extent, by the number of victims and of those who lost their homes. The indicators of the final part of this stage are: the ending of the emergency actions, the great decrease of the emergency dwelling and feeding, the release of wreckage along the main streets.

II. The second stage, the restoration period is marked by the reconstruction of public utilities and the recovery of normal social-economic activities. The end of this stage is proved by the normalizing of public services (education and administrative activities), the restoration of the complete functionality of the dwellings whose repairing did not require qualified intervention, and the complete removal of the debris from the streets.

III. During the replacing-reconstruction period, the housing-stock of the locality is rebuilt at the level existent before the earthquake.

IV. The improving and developing reconstruction period is characterized by large designs that must achieve "the safer and better town".

Certainly this periodicity is conventional, the limits between stages overlapping as elements of a pariod persist in the following one. As concerns the earthquake of March 4, 1977, the first period lasted 11 days (4-14 March), at the end of it the wreckage from Bucharest being cleared away, and at March 15 a presidential decree stated that the state of necessity ceased. The second period lasted about until the end of May, when in Bucharest the construction of some buildings that replaced the damaged ones started and at Zimnicea the first inhabitants were moving into new blocks, leaving the barracks. The replacing-reconstruction period ended on 23.08.78 when in Bucharest almost all the blocks built around the demolished ones were finished. Concerning the improving and developing reconstruction, it is still continuing in Craiova, the modernizing of the central area being in course the new industrial investments in Zimnicea will soon reach the planned capacity and at Vălenii de Munte the town-planning is partially achieved. We consider that this period will come to an end by the end of this year. We mention also that in comparison with similar cases presented in literature, the main economic activities were not stopped during the above mentioned periods in the greatly affected localities.

From the application of a similar periodicity to the periods that followed the desasters in other countries, the following regularity resulted:

$$lgt_I = lgt_{II} - lgt_I = lgt_{III} - lgt_{II} = lgt_{IV} - lgt_{III}$$

where t_I, t_{II}, t_{III}, t_{IV} represent the number of days from the moment of the disaster to the end of the I^{st}, II^{nd}, III^{rd} and IV^{th} period.

For the March 4, 1977 earthquake we have $lgt_I = 1.04$; $lgt_{II} = 1.95$; $lgt_{III} = 2.72$; $lgt_{IV} = 3.51$; thus, generally speaking, the regularity is confirmed. On the other hand, useally, each period is 10 times longer than the former one. For our country the respective ratio oscillates around the figure 6 therefore the reconstruction rate was more rapid than the one noticed in similar situations.

During the emergency period, the main dwelling problem is that of the temporary accomodation for people whose houses were damaged by the earthquake. The temporary character is however relative as the period for the use of temporary dwellings is usually prolonged up to the end of the reconstruction period, namely several seasons.

In the large towns - Bucharest, Craiova, Ploieşti-although the number of damaged dwellings was great, their rate into the total housing stock was lower. Usually in such towns there is a free housing stock namely apartments in the blocks that are to be ready and houses uninhabited due to different causes which are to be distributed.

To all these are added the possible places in schools,hostels a.s.o. cleared out during the emergency period. Consequently, for these towns it was not necessary to built temporary houses.

The situation is quite different in the small towns where the housing stock is for the most part damaged (in 1977, at Zimnicea and Vălenii de Munte more than 75% from the housing stock was affected). The arrangement of temporary dwellings (barracks) or some spaces for public services (barrack-type buildings for schools, caravan-shops) was necessary.

Besides the many difficulties raised by the technical achievement of the temporary dwellings, the living in this type of accomodation pointed out some social aspects that have to be taken into consideration.

A first phenomenon is the separation of the pre-existent social groups. It has smaller implication in larger towns where the spatial aspect of the social structure is less determinant. But in smaller communities the groups superposed on the neighbourhood structures have a very important part. There, the relatives relationship, the economic mutual help, the friendship re lations have a leading part especially in the household. The dislocation of the neighbour groups by distributing them into houses placed in different areas generates a function perturbation of the informal groups; disfunctionality of the whole ensemble of the community results.

On the other hand, the dwelling in a spatial agglomeration, greater than the one usually met in small towns, as well as the forced closeness with members of other social groups, will raise tension which, extended in time, may generate conflicting situations.

These two complementary facts lead to the solution of some small ensembles of temporary dwelling that should preserve as much as possible of the old neighbourhood.

During the reconstruction period, the main activity from the point of view of reconstruction is the estimation of the buildings state and the adoption of the demolishing decisions (total or partial) or of strengthening an activity with a decisive incidence on the towr future configuration.

This decision does not exclusively depend on the damage degree but also on the way in which the building corresponds to the future town planning. This was obvious at Zimnicea where the idea of abandoning some buildings that could have been strengthened was accepted for achieving a unitary and qualitatively superior layout of the city.

At Craiova, the decision was postponed (for the buildings placed into the central perimeter, subsequently modernized) until the final town-planning design was worked out.

For a town as Craiova, where many modernizing studies for the central part were drawn up and whose achievement was postponed (both because of lack of necessary funds and because it was considered that the buildings to be replaced could still be used) we may speak in a certain way of a positive effect of the earthquake, namely it stimulated the projects achievement.

During the replacing-reconstruction period, the new aspect of the town is shaped. It consists in strengthening the less damaged buildings and in the construction of those which have to replace the damaged ones.

The problems of this stage are different in terms of the town dimensions, of the damages amplitude and characteristics.

According to these criteria we may elaborate the following typology:

Damages Characteristics / Town Dimension	Quasi-total	Concentrated on a limited area	Spread-out
Small (100,000 inhabitants)	Zimnicea Vălenii de Munte	Cîmpina	Plopeni
Large (100,000 - 500,000)	-	Craiova	Ploieşti
Very large (500,000 inhabitants)	-	-	Bucharest

Before the earthquake, both Zimnicea and Vălenii de Munte were under replanning. The systematization plans were firstly considering the building of some urban civic centers, the next step being the improvement of the architectural style of the town type from the central area to the outskirts together with the simultaneous achievement of some blocks assemblies nearby industrial units that are to be achieved (or developed).

With Zimnicea, the reconstruction did not mean the plain recovery to the situation before the earthquake, but a strong acceleration of the social-economic development. The main objectives were: the restriction of the built area from 350 ha to 80 ha, the building, until 1980, of about 6500 apartments, the building of a civic center and of some commercial, social-cultural endowments, the building of some powerful economic units, so that the workers staff increased with 5,000 till 1980. The supplementary dwelling capacity was settled in accordance with the working staff "flow" generated by the building of some new industrial units. The national economy requirements imposed that the dimensions of this objective should be subsequently reduced. At the same time, it was proved that the transition from the country-side way of living to the urban one means a slower adaptation process than changing the physical work environment.

The initial situation being almost the same for Vălenii de Munte, a different solution was selected. The central area was here too almost completely damaged, but the reconstruction had a stage conception.During the first 4-5 years a reduced number of blocks, were built maintaining large open spaces between them. Simultaneously, the construction of the electric and electronic and electroning measuring devices plant started. During the following stages, as the working staff in this objectives (on account of a migratory "flow" from the neighbour localities) will increase, the free spaces are to be occupied by other buildings consequently.

These two localities represent two alternatives of the reconstruction action. In favour of the second pleads the possibility to regulate "on the fly" the ratio between the dwellings need and disponibility, the achievement of a gradual acceptance of the new way of living by the community and an economic effort easier to be achieved, covering longer period. A common element is the fact than in both cases there exists a continuity between the existent development tendences before the earthquake and the present lay-out of the localities.

At Craiova, the area with the greatest number of damaged buildings was the old business center. The district designing institute carried out modernizing studies for this area, considering the preservation of the most important buildings and the construction of some new ones observing the existent stylistic layout (quite heterogeneous). These studies were not perfect because of the difficulties in achieving a unitary style of the existent buildings as also for the narrowness of the main grid surface of the streets.

The disaster brought the solution to overpass the main obstacles raised against the systematization. That is why, although after the earthquake many buildings were still erect and they could have been strengthened, these buildings (with one exception) were all completely demolished and the achievement of a unitary design including the reconstruction and the modernization process started.

In Ploieşti, the buildings damaged by earthquake were spread out in the whole old housing stock. The already existant town-planning was modified including the replacement of the damaged buildings by new ones as well as changes of priorities. There, where a strongly damaged building was placed outside of the area that was to be systematized, it still was completely demolished letting the place temporarily unoccupied, or only the façade was repaired despite the fact that the building was disaffected, in order to keep the aesthetical qualities of the architectural environment.

In Bucharest also large compact groups of buildings were affected. As the architectural environment was quite unitary, it was not necessary to modify it with new constructions. The main principle of the reconstruction was that of including the new buildings in the already existent environment, some times even by reproducing the architectural characteristics of the lost buildings. "Dunărea" was the only building assembly achieved during the reconstruction period, and it was created in a rather different manner as compared to the neighbour buildings, being connected to the tendency of modifying the architectural conception already visible in other new buildings in that area.

The above mentioned examples prove that the reconstruction process had a great variety of tendencies according to the particularities of localities and to the effects of the earthquake. In all the situations, the localities developing tendencies, existing before the earthquake, were continued, and the opportunities generated by the new situation are leading to the acceleration or the amplification of those tendencies.

The main designing objective for reconstruction is the maximum revaluation of those opportunities, objective that may be achieved only if the community's latent developing tendencies are observed. We must also consider the fact the achievement of the social-economic transformations generated by the reconstruction process also means a process of adaptation to the new conditions.

SESSION IV : STRUCTURAL PERFORMANCE UNDER EARTHQUAKE LOADINGS. STRUCTURAL DESIGN.

IV.1 **COMPUTATION AND TESTING OF DYNAMIC RESPONSE FOR REINFORCED CONCRETE STRUCTURES**

Daniel P. Abrams*

ABSTRACT

Earthquake resistant design of concrete structures relies on numerical models that are based on static and linear structural behavior. Use of these models, though appropriate for proportioning strength in new construction, may not be safe or cost effective when used for assessing vulnerability or prescribing strengthening procedures for existing structures. This is because the inelastic energy dissipation is neglected as well as the interdependence of the lateral interial forces and the progressive softening of the structure.

The present era provides the engineer with the computational capability to study nonlinear dynamic response much more simply than in the past. Given descriptions of the resistance of a nonlinear oscillator from laboratory experiments, and of the accelerations of a base motion from the ever increasing data base, response may be calculated and viewed within seconds. Because there is now a need to improve accuracy and simplicity of numerical models for response computation, a better understanding of the fundamentals of nonlinear response is required.

This paper presents research of the author and other researchers at the University of Illinois on behavior of both physical and numerical models. A summary of experimental work is presented which includes earthquake-simulation tests of reduced-scale ten-story concrete buildings, and force-reversal tests of structural components. Numerical representations of hysteretic behavior will be presented as well as simple ways to incorporate these models with models of building response.

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EARTHQUAKE-SIMULATION TESTS OF 10-STORY STRUCTURES

It is a well established fact that the energy dissipated by a structure is enhanced once the elastic limit is exceeded. Lateral deflections may actually be less for a nonlinear structure than for a structure responding within the linear range as a result of an earthquake motion of lighter intensity. There is a need to have a quantitative description of nonlinear response for design of new construction, but more importantly, for assessing the vulnerability of existing construction and prescribing strengthening schemes. Unfortunately, nonlinear dynamic analysis of multi-degree-of-freedom (MDOF) systems is not well understood, and is an art much like analysis of continuous structures subjected to transverse loadings was in the earlier part of the century. For even simple structures, present calculation procedures for nonlinear response require a substantial amount of computational effort, and do not necessarily reflect physical response.

To help understand dynamic response fo nonlinear MDOF reinforced concrete building systems, several small-scale nine or ten story test structures have been subjected to strong base motions on the University of Illinois earthquake simulator. Test structures (Fig. 2.1) have been configured as idealized models of lateral-force resistance rather than true replicas of typical construction. In general, structures have been planar walls, frames or some combination of each. Strengths have been proportioned such that nonlinear behavior would occur at specified regions such as at the ends of beams or columns or at the base of a wall. Lateral degrees of freedom have been slaved for all points at a particular level with massive floor slabs that were rigid within their own plane.

Recorded ground motions were scaled with respect to time and intensity so that test structures would be excited, and incur damage with progressive increases in base-motion intensity similarly to that of actual buildings. This was done using linear spectral-response curves for the model motions as the basis (Fig. 2.2). Measurements consisted of absolute accelerations, and displacements relative to the base at each level (Fig. 2.3). In addition, for structures with walls, lateral forces resisted by these components were measured as well

In a study of frame-wall interaction (1), larger lateral forces were attracted to test structures with walls that were designed to remain elastic (Fig. 2.4) than to structures that could dissipate energy through nonlinear effects. Lateral deflections of each structure were similar (Fig. 2.5) indicating that serviceability was not influenced significantly by inelastic action. Tests showed that arbitrary softening of particular elements of a structure could result in a much more economical design with no loss of function.

The second conclusion deduced from this study was that nonlinear response could be calculated on the basis of generalized coordinates (a concept commonly used for linear modal analysis). The conclusion was based on the finding that shapes of displacement response histories at each of ten levels were essentially the same. In other words, deflected shapes were nearly invariant for all amplitudes of motion for the nonlinearly behaving structures. Response of the test structures could be calculated on the basis of a single nonlinear force-deflection relation for the entire structure. The inference of this conclusion is that the computational effort required for a nonlinear analysis may be reduced greatly, even to that of today's microcomputers.

The third conclusion was that a linear analytical model with reduced stiffnesses could be used to determine response maxima. This conclusion was based on the finding that maxima of base moment and lateral deflection were related linearly for the design-basis earthquake. The slope of this relation could be used to estimate maxima with a linear modal analysis. This conclusion infers that the concept of using reduced member stiffness with a linear model to represent nonlinear behavior (such as the Substitute Structure Method, Ref. 2) is a viable approach.

INELASTIC BEHAVIOR OF REINFORCED CONCRETE COMPONENTS

Experimental tests have shown that behavior of reinforced concrete members and connections under load reversals are not governed solely by constitutive properties of materials. Substantial deflections may be a result of opening and closing of flexural or shear cracks, and slippage of reinforcing bars relative to concrete. Most tests of concrete components have shown that after a few large-amplitude cycles, specimens respond with a marked reduction in resistance upon reversal of the load (Fig. 3.1). After deflections are reversed an amount "a" or "b", specimens stiffen as cracks close. Because of bar slippage and crack closure in the load-reversal range, and reductions in loading stiffnesses, "k_1" and "k_2", strengths are reached at deflections which are much larger than would occur under static acyclic forces. Most specimens tested deformed very large amounts without suffering a significant loss of strength, however, energy dissipation characteristics were poor for those specimens that had incurred substantial slippage in the load-reversal region.

Tests have shown that inelastic behavior is dependent on the number of large-amplitude cycles. Tensile strains in the reinforcement are seldom balanced with equal compressive strains for opposite directions of loading because of the added resistance of concrete to aid steel when in compression. If the reinforcement yields while in tension, strains will accumulate with each large-amplitude cycle of deformation. After a sufficient number of cycles, the width of flexural cracks will enlarge which will result in marked differences in stiffness and strength characteristics.

In addition to unequal tension-compression straining of reinforcement, sections and members are subject to unequal inelastic curvatures and rotations for each direction of loading. Like the strains, these deformations accumulate with each large-amplitude cycle. A simple example helps to illustrate this phenomena. Design of negative reinforcement entails an assumption regarding the amount of maximum each of ten levels were essentially the same. In other words, deflected shapes were nearly invariant for all amplitudes of motion for the nonlinearly behaving structures. Response of the test structures could be calculated on the basis of a single nonlinear force-deflection relation for the entire structure. The inference of this conclusion is that the computational effort required for a nonlinear analysis may be reduced greatly, even to that of today's microcomputers.

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In addition to unequal tension-compression straining of reinforcement, sections and members are subject to unequal inelastic curvatures and rotations for each direction of loading. Like the strains, these deformations accumulate with each large-amplitude cycle. A simple example helps to illustrate this phenomena. Design of negative reinforcement entails an assumption regarding the amount of maximum gravity loading which will probably be present during an earthquake. In actuality, it is conceivable that a lesser amount of gravity loading may be present than assumed for design. In this case, the top steel will not strain as much as the bottom steel, and perhaps may not yield at all (Fig. 3.2). This results in a possible inelastic rotation which accumulates for each large-amplitude cycle, resulting in large crack widths and a possible reduction in shear capacity.

Similar illustrations can be made for other typical cases where the proportioning of resistance is not in absolute accordance with actual phenomena. Conservative design assumptions with respect to the effective flange width of T-beams (Fig. 3.3) may result in asymmetrical straining of top and bottom reinforcement, and thus, an accumulation of tensile plastic strains in the bottom reinforcement. This phenomena is augmented by asymmetrical elastic stiffnesses which are a result of differences in flange effectiveness when in tension or compression. For equal sways of the structure in each direction, the bottom reinforcement may yield whereas the top would not.

Tests of beam-column joints (3) have shown that inelastic behavior is sensitive to bond mechanisms under repeated and reversed loadings. Free-body diagrams (Fig. 3.4) illustrate the difference in bond demands for beam reinforcement in exterior and interior-joint specimens. For interior-joint specimens (Fig. 3.5a), bond strength for beam reinforcement was lost across the width of the column member which eliminated the effectiveness of the bars to resist compression. Upon reversal of the load, specimen stiffness reduced to zero because reinforcement was not effective to resist closure of the previously opened flexural crack. After the crack closed, the specimen stiffened until the tensile reinforcement reached its proportional limit. However, because of the large amount of slippage within the load-reversal region, strengths were reached at very large deflections. Specimens withstood very large inelastic deflections (in excess of 4% of the story height), however, they did not resist a substantial amount of energy. Demand for bond strength was less for bars in the exterior-joint specimens (Fig. 3.5b). The severe stiffness reduction upon reversal of the load did not occur, and the specimen was able to resist more energy.

NUMERICAL MODELING OF HYSTERETIC BEHAVIOR

Former path-dependent models for hysteretic resistance functions have been developed to represent one, or a few, aspects of nonlinear behavior for a particular form of component. Some of these formulations have included simple bilinear, or elasto-plastic models such as the stiffness degrading model (Fig. 4.1b) proposed by Clough and Johnston (4), the modified Takeda (5) softening model (Fig. 4.1c) which includes a reduction in stiffness for unloading, and a slip-softening model (Fig. 4.1d) which has been used by Abrams and Tangkijngamvong (6).

Each of these models has been based on a set of linear segments joined at points with an abrupt angle change. Whereas through proper choice of slopes and connecting points, these models may be suitable for dynamic response calculations, the numerical integration process is cumbersome because care must be taken to properly define the time step so that over or under shoot may not be significant at concentrated points of curvature. A new formulation has been developed which uses smooth curves to represent the resistance function, and is sufficiently general to encompass all, or any combination, of the previous hysteresis formulations.

A new path is generated once a change in the sign of the velocity is detected. The path is composed of both cubic and linear segments (Fig. 4.2). Control points which define the shape of the path are selected based on rules established from past or new formulations. Four segments are used to describe (a) linear unloading, (b) softening upon reversal of force, (c) gradual softening upon closure of cracks followed by softening at large forces, and (d) strength after yield of reinforcement. The cubic-spline model is most useful for representing portion (c) of the path. The rounded nature of the curve tends to become more pronounced as the separation between points B and C becomes larger. This mathematical property is closely related to what happens physically in a reinforced concrete member or connection as a result of crack closure at low amount of force, and the Bauschinger effect in the reinforcement at larger amounts of force.

The example path shown in Fig. 4.2 depicts that of a structure influenced by a slip mechanism such as for an interior beam-column joint. When force is reversed in direction, cracks tend to close and reinforcing bars tend to slip back to original positions, thus resulting in a substantial decrease in stiffness. When the cracks are fully closed, and the reinforcing bars develop anchorages for the new direction of force, the structure is observed to stiffen appreciably. Initially, the stiffness between points A and B is that for a section comprised of solely reinforcement, k_{bo}. As the member is cycled, bond is weakened which reduces this stiffness. The deterioration in stiffness which is represented with the term, β, is related to the amplitude of cycling and the number of cycles. As a simple approximation, the deterioration is expressed in terms of the previous deflection maxima for the same sense of forcing, DM (Fig. 4.3). At a prescribed value of deflection, DBL, all bond is assumed to be lost, and the stiffness upon reversal of the force is taken as zero. The deflection at stiffening which is represented with the factor "α", can also be related to this deflection ratio. A member will stiffen at a zero rotation if cracks do not open before the old ones close. If there has been a substantial amount of bond deterioration in a interior beam-column joint as discussed in the previous section, then the tensile bars may slip from the joint as the compressive bars are pushed into the joint.

If the structure does not contain significant slip mechanisms such as for a wall responding in flexure with well anchored vertical steel, the stiffness from points A and B should be represented without the idealization just described. For this case, the member would respond with the stiffness of the previous unloading slope, k_a, or a value slightly less to model some slight crack closure. If a stiffness, k_b, is prescribed by the user that is greater than the average stiffness between point A and C, then the linear segment AB is eliminated from the path.

Because response to an earthquake motion may include several changes in velocity for a single nonlinear cycle, the algorithm must also account for reversals that are localized in one region of the curve. Linear behavior has been assumed if the member is unloaded and them reloaded before a change in the sign of the force has been reached. If the member is reloaded after a change in the sign of the force occurs, but not a change in sign of the deflection, then the member reloads with a single change in stiffness without slip.

The cubic-spline hysteresis formulation has been incorporated in the numerical solution described in the next section for determining response of SDOF systems to earthquake motions.

SIMPLE COMPUTATION OF NONLINEAR RESPONSE FOR BUILDINGS

The rigorous procedure for determining the response of an oscillator subjected to an earthquake, or any dynamic loading, is to integrate the equation of motion numerically for several instants in time using the Newmark Beta Method (7). For a MDOF system composed of numerous nonlinearly behaving elements such as a concrete frame, this procedure requires substantial computational time because the stiffness matrix must be updated and inverted every time a particular element changes stiffness. This demand usually limits the use of this procedure to a main frame computer.

For building structures with uniform mass and stiffness distributions, it is feasible to express response in terms of a single generalized coordinate even though substantial nonlinear deformations have occurred. Tests of one-twelfth scale models (1) showed that distributions of displacement along the height were quite similar for all ranges of response. Modal participation factors calculated from measured deflected shapes varied within 5% for large and small-amplitudes of motion. Because displacement response was governed by the fundamental mode, lateral displacements at any particular level could be represented by a single dynamic degree of freedom and a single distribution function.

Simplified Method

A simple method of computation is presented which characterizes the hysteretic resistance of the overall structure with a single generalized coordinate. The procedure is similar to the one developed by Saiidi (8). Given a description of the relation between base shear and top-level deflection, and the base motion, response histories of lateral story drift and acceleration are computed. From this information, the designer may judge the worthiness of a structural scheme in terms of the maximum amount of nonlinear deflection, and the number of cycles at a particular range of nonlinear deformation. Estimates of equivalent static forces for which the structure should be designed can also be determined. The approximate method is sufficiently simple to be implemented on a microcomputer. Several analyses of a structure may be done interactively in a short period of time to identify all possible bounds of response for several different strengths, hysteresis types, and expected ground motions.

Resistance of the structure is expressed in terms of a smooth curve rather than a combination of piece-wise linear segments. Unless a drastic change in slope occurs such as at unloading, there is no need to change the time step. If the numerical algorithm is based on convergence of an assumed acceleration, then a resisting force may be expressed directly as a function of deflection. Although this procedure involves one or two iterations per time step, it eliminates the need to rely on estimating resistance with a tangent stiffness which may result in overshoot problems. The attractiveness of the procedure lies in the improved accuracy for systems with ever changing stiffness.

A library of recorded earthquake motions is compiled on diskette from the USGS data base. The user can select particular earthquake motions from a menu shown on the screen. He or she has the options of selecting one portion of the motion, compressing the duration, and altering the maximum acceleration.

Although higher modes can be represented in the same way, the simplified approach is not applicable for systems with a large participation of higher modes because superposition is not valid for nonlinear systems. The approach to be used, therefore, applied only to buildings that would vibrate in the fundamental mode: usually low rise structures not exceeding ten stories in height.

Nondimensionalized Equation of Motion

To use the nonlinear analysis procedure described previously for a single-degree-of-freedom system, physical properties of the structure must be translated to properties associated with a given generalized coordinate. For purposes of simplicity, this shall be considered as the lateral displacement at the top level, Z_n. Knowing the amounts and distribution of mass along the height of structure, and a specified displacement shape, an equivalent mass can be determined which if placed at the tenth level would result in the same inertial forces. This operation is based on conventional modal decoupling procedures which are summarized below.

The equation of motion for a MDOF system subjected to a base acceleration is:

$$[M]\{\ddot{v}\} + [C]\{\dot{v}\} + \{R(v)\} = -[M]\{r\}\ddot{v}_g(t) \tag{5.1}$$

where $\{\ddot{v}_g\}$ is motion of the ground, and $\{\ddot{v}\}$ is relative motion of the structure to the ground.

Components of the {r} vector are displacements of a rigid structure due to unit motions of the ground. This vector is equal to {1} for a system of lumped masses along a vertical line which is subjected to translation at the base.

The motions of each floor level may be expressed in terms of the summation of products of distribution functions and generalized coordinates:

$$\{v(x,t)\} = [\Phi(x)]\{Z(t)\} \tag{5.2}$$

where $[\Phi(x)]$ is a composite of individual modal shapes, $\{\phi_n(x)\}$, and $\{Z(t)\}$ is a series of modal amplitudes, $Z_n(t)$. If modal shapes have been normalized with respect to the top level, $Z_n(t)$ is both the modal and top-level displacement.

After substitution, and invoking orthogonality of different modal shapes, the MDOF equation reduces to the following scalar equation for mode "n".

$$M_n\ddot{Z}_n + C_n\dot{Z}_n + \{\phi_n\}^t\{R(v)\} = -\{\phi_n\}^t[M]\{1\}\ddot{v}_g(t) \tag{5.3}$$

M_n is expressed as:

$$M_n = \{\phi_n\}^t[M]\{\phi_n\} \tag{5.4}$$

Because only the fundamental mode is of interest, the generalized mass, M_1 may be determined from a distribution of lateral deflection for the first mode, $\{\phi_1\}$. For structural systems with uniform mass and stiffness distributions, a triangular or parabolic shape may be sufficiently precise for determination of M_1. For systems with irregular distributions, the first mode shape should be obtained from an eigenvalue solution. The shape may also be derived from a linear static analysis which is based on an assumed lateral force distribution. Subsequent analyses can be done using the derived deflected shape as the lateral force distribution until the exact modal shape is obtained.

The modal damping, C_n, may be expressed in terms of the percentage of critical damping, ξ_n as follows:

$$C_n = 2M_n\omega_n\xi_n \tag{5.5}$$

This relation is based on elastic behavior, however, viscous effects are significant at small displacements which are usually elastic.

The modal resisting force, $\{\phi_n\}^t\{R(v)\}$, can be deduced from the base shear at a particular amplitude of top-level deflection. When an undamped MDOF system is in free vibration, the resisting forces within the structure are in equilibrium with the inertial forces according to the following relation.

$$\{R(v)\} = [M]\{\ddot{v}\} \tag{5.6}$$

Or, for a particular mode, n,

$$\{R_n(v)\} = [M]\{\phi_n\}\ddot{Z}_n \tag{5.7}$$

The base shear for a particular mode, V_{bn}, is the summation of these forces. In matrix notation:

$$V_{bn}(z) = \{1\}^t\{R_n(v)\} = \{1\}^t[M]\{\phi_n\}\ddot{Z}_n \tag{5.8}$$

Solving this equation for $\ddot{Z}_n$, and substituting in Eq. 5.7:

$$\{R_n(v)\} = \frac{[M]\{\phi_n\}}{\{1\}^t[M]\{\phi_n\}} V_{bn}(z) \tag{5.9}$$

Premultiplying by $\{\phi_n\}^t$ to obtain the modal quantity in Eq. 5.3, and noting the definition of modal mass, M_n, from Eq. 5.4:

$$\{\phi_n\}^t\{R_n(v)\} = \frac{M_n}{\{1\}^t[M]\{\phi_n\}} V_{bn}(z) \tag{5.10}$$

Substitution of all expressions in Eq. 5.3 and dividing by M_n:

$$\ddot{Z}_n + 2\omega_n\xi_n\dot{Z}_n + \frac{V_{bn}(z)}{\{1\}^t[M]\{\phi_n\}} = -\frac{\{\phi_n{}^t\}[M]\{1\}}{M_n}\ \ddot{v}_g \tag{5.11}$$

Eq. 5.11 may be nondimensionalized by introducing [illegible], which is the ratio of the base shear to the weight of the structure. If the weight of the structure is expressed in matrix form as:

$$W = \{1\}^t[M]\{1\}g$$

then the third term in Eq. 5.11 may be expressed as:

$$\beta_n \psi_n(z) g$$

where β_n is equal to $\{1\}^t[M]\{1\}/\{1\}^t[M]\{\phi_n\}$. If α_n is used to represent the coefficient of $\ddot{v}_g$ in Eq. 5.11, which is more commonly known as the modal participation factor, then Eq. 5.11 simplifies to:

$$\ddot{Z}_n + 2\omega_n \xi_n \dot{Z}_n + \beta_n \psi_n(z) g = -\alpha_n \ddot{v}_g \qquad (5.12)$$

Solution of Equation

To solve the above equation, the following parameters need to be defined:

(a) the mass distribution
(b) an assumed deflected shape
(c) percentage of critical viscous damping
(d) an estimate of the fundamental period
(e) ratio of base shear to total weight
(f) type of hysteresis formulation.

Note that knowledge of the total amount of mass is not required because the base shear is normalized with respect to this quantity in the ψ_n term.

Items (c) and (d) are used to determine the viscous damping force. Uncertainty is related to the product of these two values, and not to their separate values. For this calculation, the accuracy of the frequency should only be as good as the estimate of damping percentage. During large inelastic displacements, the velocity is usually small, and the effect of this term on overall response is not significant.

The resisting force is represented with item (e). The relation between base shear and top-level deflection needs to be obtained from a static analysis, or from a rough estimate of the fundamental period of vibration. Behavior under monotonically increasing forces may be assumed to represent the envelope for cyclic loadings. The remainder of the force-deflection relation is based on this "spinal curve" using the hysteresis formulation specified in item (f).

VERIFICATION OF PROCEDURE

The procedure is verified by comparing its results with that of a reduced-scale shaking-table model. The sample structure is a 10-story reinforced concrete frame-wall structure with an even distribution of mass at each level, and equal heights at each story. Further details of the test structure may be found in Ref. 1.

The measured deflected shape of the test specimen when subjected to simulated earthquake motions was mostly similar to the parabolic flexure beam idealization incorporated with the computer program. The maximum base shear was approximately 40% of the total weight. The lateral deflection at which the structure formed a mechanism can be estimated at 1.0% of the height. Stiffnesses at unloading were approximated with a nondimensionalized value of 60 which was slightly greater than that for loading within the elastic range. Because the structure was fabricated with model materials, slippage of reinforcement should have been dominant on the cyclic behavior of the frames. For this reason, a low reversal stiffness of 5.0 which reduced to zero when a maxima deflection equal to six times the yield deflection was reached.

The motion input to the base of the test structures was a modeled version of the motion measured at El Centro, California during the 1940 Imperial Valley Earthquake. Duration of the record has been compressed by a factor of 2.5, and the maximum base acceleration was scaled to 0.48g and 0.92g for Runs 1 and 2. Because input information for the program is in a nondimensionalized from, only the time step was reduced by the 2.5 factor.

Comparison of drift and acceleration maxima are summarized in Table 1.

Table 1: Comparison of Measured and Calculated Response

	Run 1		Run 2	
Parameter	Measured	Calc.	Measured	Calc.
Max. Drift	2.30%	2.38%	3.35%	4.25%
Max. Accel.	0.91g	1.28g	1.47g	1.44g

The correspondence between measured and calculated values is within the intended range of accuracy for the simplified procedure. The large difference in drifts for Run 2 may be attributable to the fact that significant hinging occurred at the base. If a triangular deflected shape were assumed to reflect this, the calculated value of 4.25% would reduce to near the measured value.

Computed response histories are shown in Fig. 6.1a and 6.1b for each of two intensities of base motion. Rather than maximum story drift, measured waveforms shown in Fig. 6.2a and 6.2b are deflections at the tenth level. However, direct comparison can be made with the shape of response histories since calculated drifts are a fixed percentage of the top-level deflection.

The shapes of measured and calculated response histories are not in exact agreement, however, when viewed in terms of the stiffness assumptions made, the correlation is acceptable. The general pattern is replicated reasonably well in terms of the response maxima, and the number of cycles at a particular level of deflection.

SAMPLE RESULTS OF PROCEDURE

Response has been calculated for a five-story, large-scale frame structure. Response of the same structure to four different base motions is presented in Figs. 7.1 and 7.2. For these test cases, the strength-to-weight ratio used for the structure was 0.2. The reversal slope was a value of 5.0 to represent a "slip" type of hysteresis.

The input motions for response shown in Fig. 7.1 consisted of the first 8.0 seconds of the motions recorded at El Centro, California. The maximum acceleration of the motion was taken equal to the recorded 0.35g (Fig. 7.1a) as well as 0.60g (Fig. 7.2b). It is interesting to note that because of the increased amount of nonlinearity with the more intense motion, accelerations were about the same. Drifts did increase in like proportion to the maximum base accelerations. The sequence of the response and the number of cycles at large deflections, however, was much different for each of the structures.

Response shown in Fig. 7.2 is a result of ground motions recorded at Tokachi-Oki and Miyagi, Japan. The frequency content of these motions differs substantially from that of the El Centro motions. Much more energy was released from these motions as can be inferred from the relatively large areas under the accelerograms. As a result, deflections and amounts of nonlinear behavior were quite large for the smaller base accelerations. As for the El Centro structures, amplification of base acceleration was small because of the hysteretic energy dissipation and progressive softening of each structure.

Response shown in Fig. 7.3 represents that for a structure without "slip" mechanisms, or a typical wall type building. The strength ratio has been changed from 0.2 to 0.4 to represent conceptually the case of strengthening a building system. Base motions for each case are those measured at El Centro, California. Care must be taken in comparing waveforms because deflections have been scaled in accordance with values at yield which differ by a factor of two.

The important feature of the comparison is that the strengthened structure deflects, and accelerates <u>more</u> than the unstrengthened one. The implication is that a nonlinear analysis, though simple and approximate, can eliminate the need to strengthen a building.

CONCLUDING REMARKS

A summary of investigations has been presented that have examined methods for modeling dynamic response of reinforced concrete structures subjected to earthquakes. Indications from the work suggest that development of numerical models still remains an art. It has been shown that relatively simple models may suffice for determination of response maxima of building structures with uniform distributions of mass and stiffness. However, further development and verification needs to be done for building systems with atypical configurations, or for structures subjected to multidirectional base motions.

ACKNOWLEDGEMENTS

The work described in this paper represents efforts of the author and others on the earthquake-engineering research staff at the University of Illinois at Urbana-Champaign. Continued support of this program has been provided by the National Science Foundation. Funding was also provided for development of analytical models by the Office of Naval Technology for work done by the author while visiting the Naval Civil Engineering Laboratory, in Port Hueneme, California.

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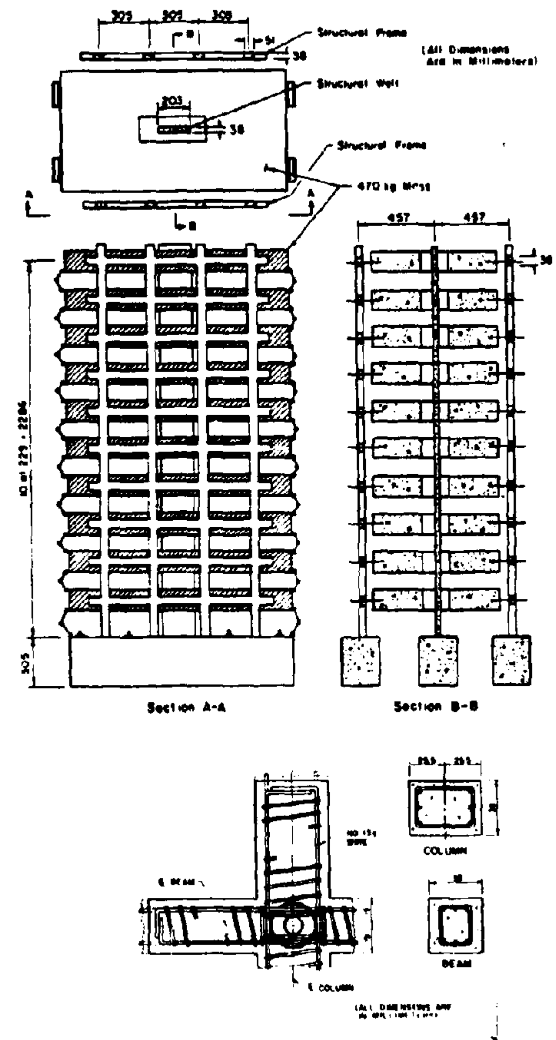


Fig. 2.1 Description of Test Structure

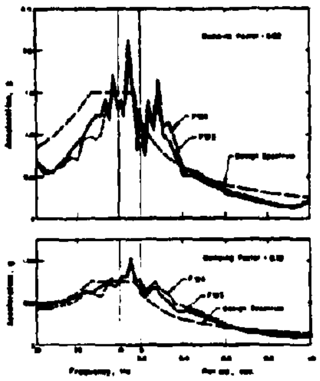


Fig. 2.2 Spectral-Response Curves for Simulated Motions

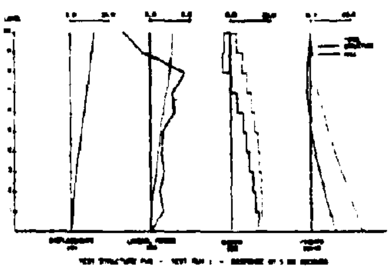
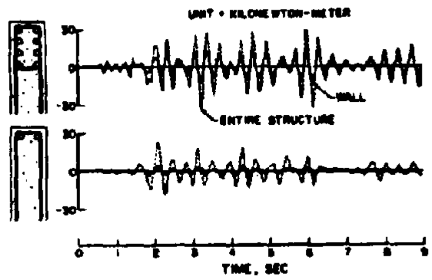
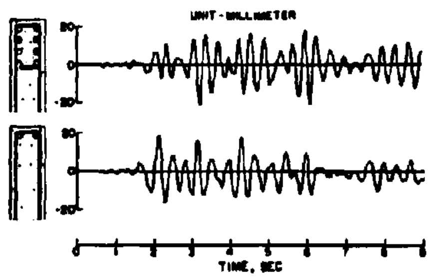


Fig. 2.3 Measurements of Lateral Response



MEASURED BASE-MOMENT RESPONSE

Fig. 2.4 Comparison of Lateral Forces for Elastic and Inelastic Designs



MEASURED DISPLACEMENT RESPONSE

Fig. 2.5 Comparison of Lateral Deflections for Elastic and Inelastic Designs

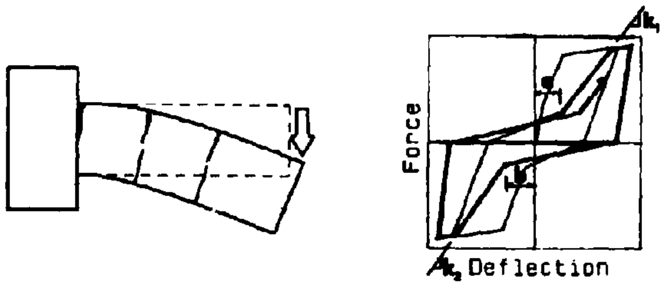


Fig. 3.1 General Hysteretic Relation for Reinforced Concrete Member

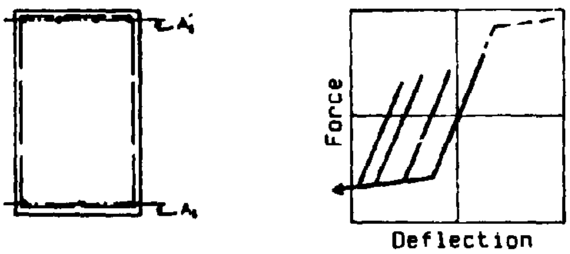


Fig. 3.2 Accumulation Effects for Asymmetrical Strengths

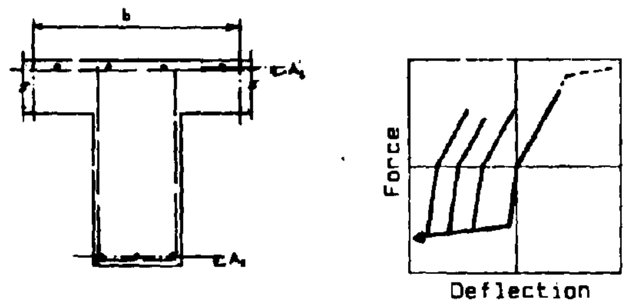


Fig. 3.3 Accumulation Effects for Asymmetrical Strengths and Stiffnesses

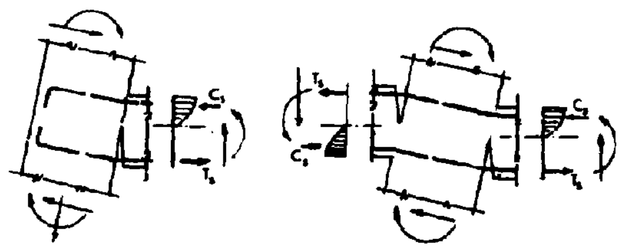
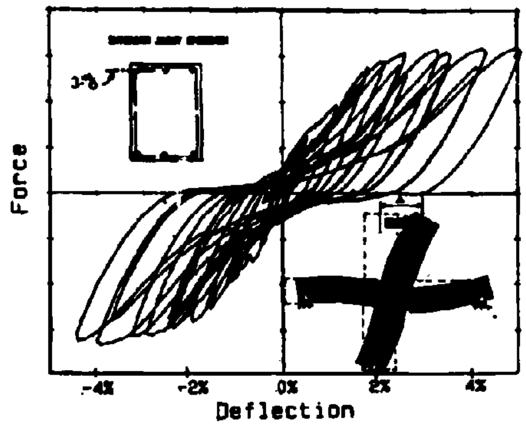


Fig. 3.4 Free-Body Diagrams for Beam-Column Joints



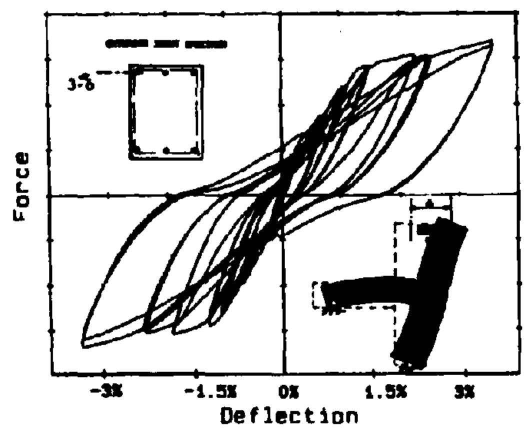


Fig. 3.5 Measured Force-Deflection Curves for Interior and Exterior Beam-Column Joints

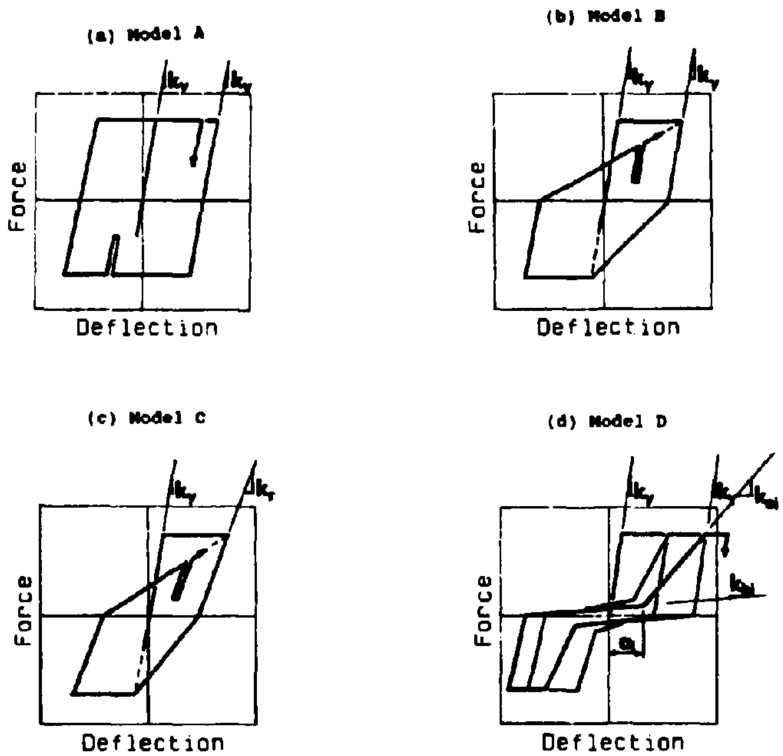


Fig. 4.1 Hysteresis Models used in Analysis

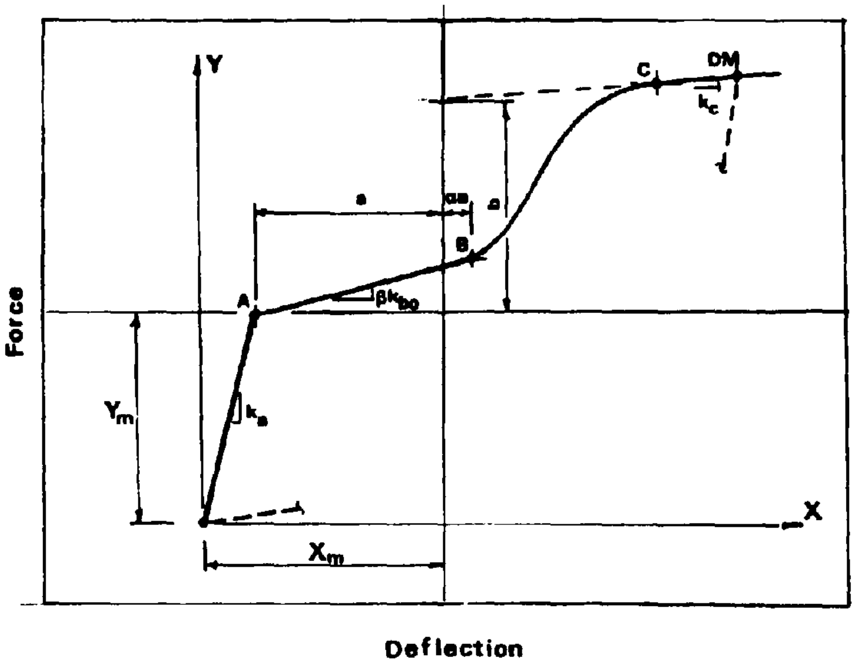


Fig. 4.2 Cubic-Spline Hysteresis Model

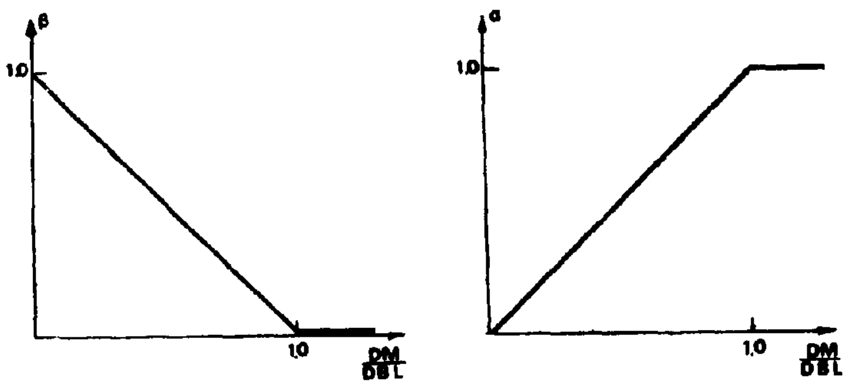


Fig. 4.3 Idealisations for Bond Deterioration

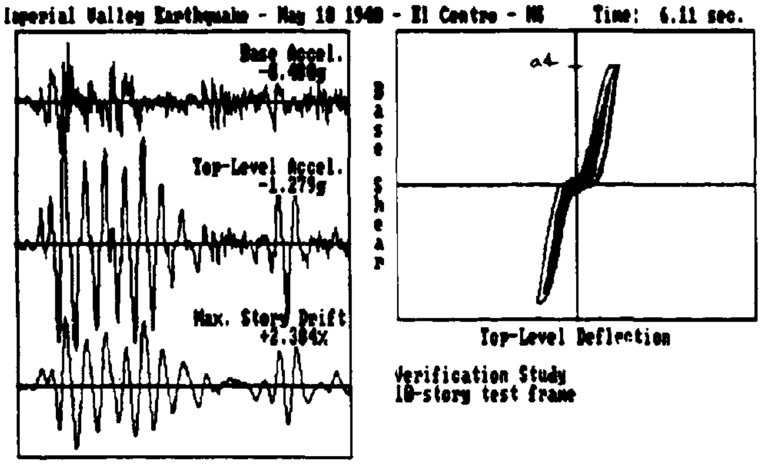


Fig. 6.1a Response Computed with Simple Model - Run 1

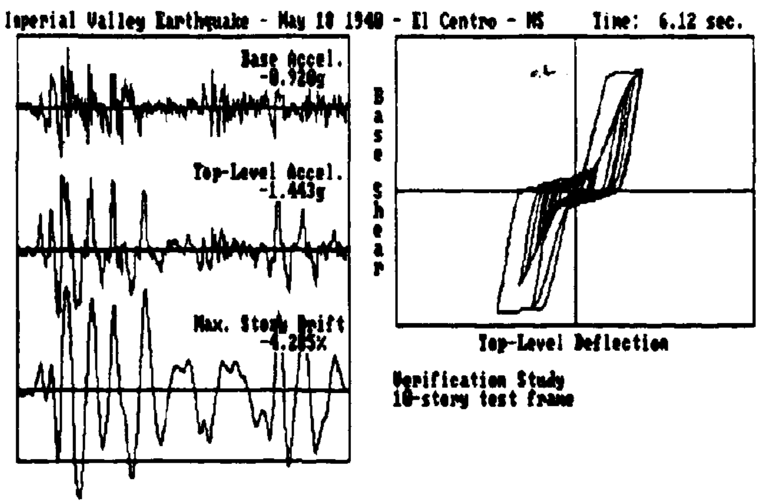
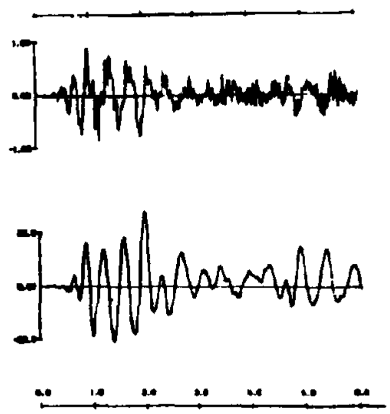
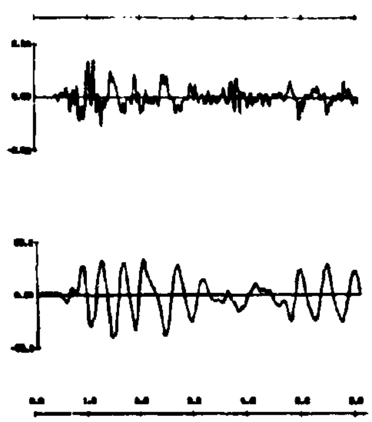


Fig. 6.1b Response Computed with Simple Model - Run 2



Run 1



Run 2

Fig. 6.2 Measured Response of Reduced-Scale Model

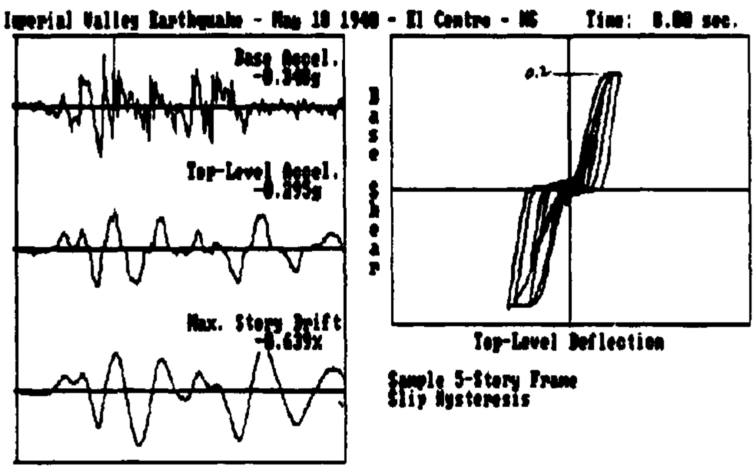


Fig. 7.1a Computed Response to El Centro, 0.348g

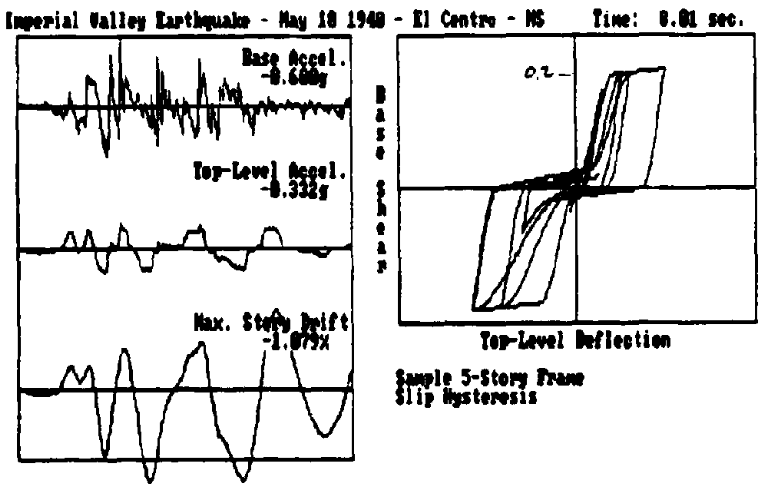


Fig. 7.1b Computed Response to El Centro, 0.600g

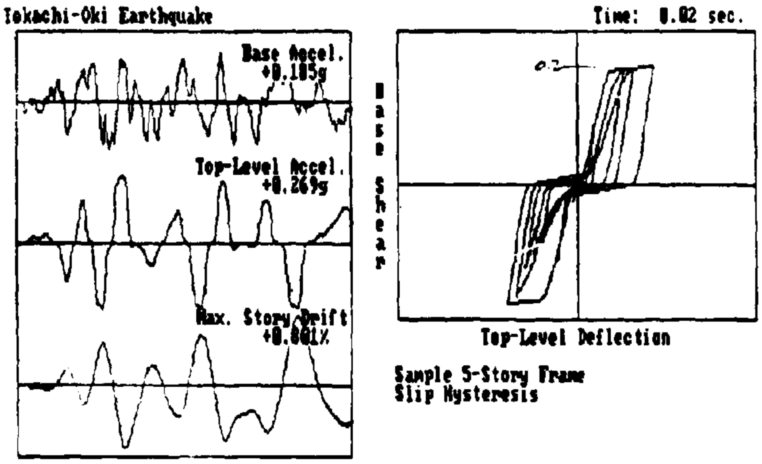


Fig. 7.2a Computed Response to Tokachi-Oki

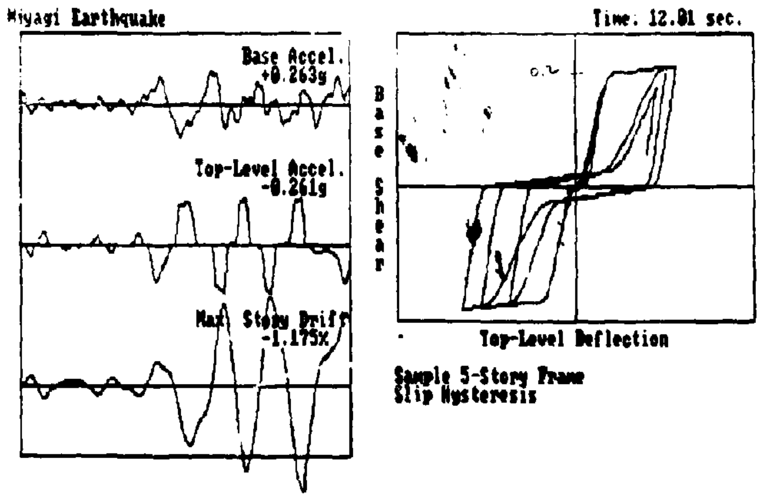


Fig. 7.2b Computed Response to Miyagi

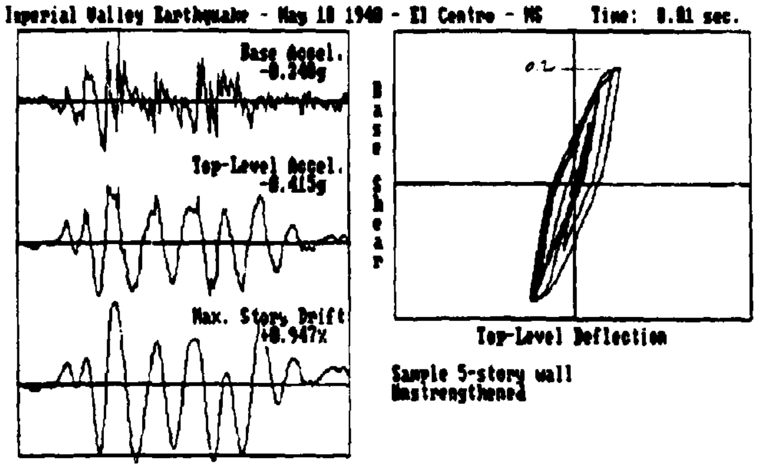


Fig. 7.3a Computed Response of Unstrengthened Structure

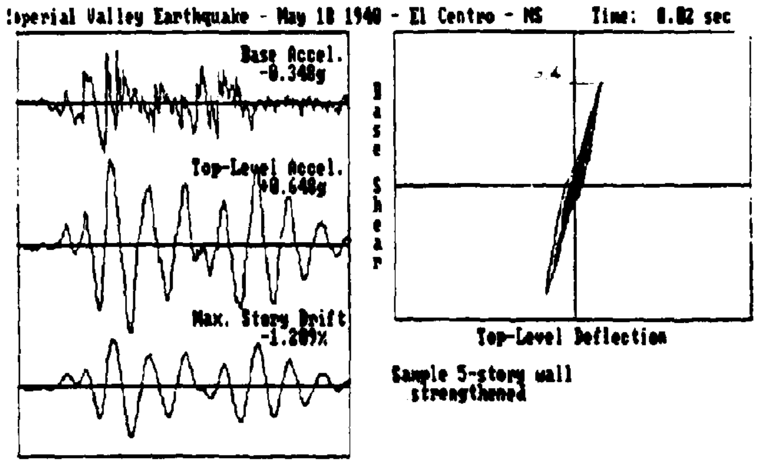


Fig. 7.3b Computed Response of Strengthened Structure

IV.2

A BIOGRAPHY OF A LARGE-SPAN STRUCTURE, PRE-AND POST-EARTHQUAKE, AFTER THE PROVISIONAL AND FINAL STRENGTHENING

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Mihai Stancu**
Olga Stancu***
Cristian Constantinescu****

1. INTRODUCTION

The object of this paper is an analysis of the structure of the main hall of the Exhibition of Achievements of National Economy (EREN). This structure is of particular interest because it is a large span structure, it may be analyzed quite clearly and convincingly due to its particular layout, and it was in fact the most analyzed structure of Bucharest due to its importance.

The studies carried out in this connection covered experimental, as well as computational, investigations. The full-scale experimental analyses were carried out at different stages, covering a period from 1976 (pre-earthquake) up to 1984 (after completion of final strengthening). In order to check the solution adopted for final strengthening, a computational analysis was carried out to compare the pre-and post-strengthening structural earthquake resistance.

This paper presents briefly the structure and its history and thereafter the outcome of investigations referred to.

2. DESCRIPTION OF THE STRUCTURE

The structure dealt with supports a 96-m. span dome that covers the main exhibition hall. The dome has a steel structure, the main part of which is represented by 32 trussed half-arches, which are braced, in the plane of the dome, by means of secondary members supporting the roofing too. The reinforced concrete structure, supporting the dome, consists of 32 couples of columns, of

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which the internal columns transport the gravity loads from the dome to the ground, while the external columns play a rather secondary role. The reinforced concrete structure was designed with moment transmitting connection in a tangential plane, while the connections in a radial plane do not create a proper frame.

The main load bearing structures for horizontal loads (wind loads, earthquake loads) must be looked for, therefore, in a tangential plane. It may be stated that this selection of load-bearing structure was reasonable. A general view of the structure is given in fig.2.1 and a view of a couple of columns is given in fig.2.2.

3.BIOGRAPHY OF THE STRUCTURE

The structure was built shortly after 1960. After some significant damage to the dome in 1963, the initial steel structure of the dome was replaced by the present one and some corresponding measures were adopted for the reinforced concrete supporting structure. The 1977 earthquake damaged both the reinforced concrete columns (at their upper story) and the glazing, which had to be replaced.

The damage, in form of locally cracked or crushed and exfoliated concrete, was visible at the lower and upper ends of the upper story of the columns, especially along the tangential direction. The damage was severe for the columns along the predominant earthquake direction, NNE-SSW and almost negligible for the direction ESE-WNW. No permanent horizontal displacements of the ring supporting the dome were observed.

A provisional strengthening solution consisted of bracingof the upper story (in fact of the spans) and of the internal columns (bearing the dome). This bracing was located in a tangential plane (the main reason for the choice of this bracing solution was represented by the fact that the most important increase in actual period occurred for the natural period of rotation in a horizontal plane, or of overall torsion). A view of this olution is given in fig.3.1.

A final repair and strengthening solution was thereafter thoroughly analyzed by ICPMC (Design and Research Institute for Building Materials) with the cooperation of INCERC. The solution was that of jacketing the main bearing columns and providing better moment resistant connections. The solution adopted is represented in fig.3.2.

The initial structure was designed to resist in the elastic range seismic forces corresponding to a seismic coefficient c_S = 0.047. The final strengthening solution was designed for a homologous value c_S= 0.119.

To give an idea of the ductility of the main loadbearing columns, the ratio of axial force to the homologous bearing capacity at the critical sections was 0.025 for the pre-earthquake stage and of some 0.018 for the stage determined by the final strengthening solution. This accounts for a high pre-earthquake ductility for the story when post-elastic behavior was bound to occur and still improved conditions for the present stage (note in this connection that the failure mechanism became different after the final strengthening, as mentioned in section 5).

To give a summary idea of the influence of final strengthening the shear forces (in kN) corresponding to the capable bending moments of the end sections of the three stories of a column are given in table 3.1.

Note that these values are not proportionally significant for the bearing capacity of the structure as a whole due to the non-proportional influence of horizontal members and to the modification of failure mechanism, discussed in section 5.4. of the paper.

Table 3.1.

Stage	Column	Story		
		1(lower)	2(medium)	3(upper)
1	2	3	4	5
Initial	Internal,radial direction	266	233	55
	Internal,tangential "	143	125	59
	External,any "	405	180	43
Final	Internal,radial direction	5094	7154	737
	Internal,tangential "	2363	3108	827
	External	- unchanged -		

4.BIOGRAPHY OF DYNAMIC CHARACTERISTICS

The dynamic characteristics were determined on the basis of ambient vibration techniques. The simultaneous recording of oscillations along radial and/or tangential direction,by means of seismometers placed on a same ring at points distanced by 90°,made it possible to obtain, by means of simple electrical superposition , records of motions of :

a) translation of a ring along the N-S direction ;
b) translation of a ring along the E-W direction;
c) rotation of a ring about its center (overall torsion);
d) ovalization of a ring.

The outcome of experimental data is summarized in table 4.1.

Table 4.1

Stage	Natural period(s) corresponding to direction			
	Translation N-S	Translation E-W	Rotation in plane	Ovalization
June 1976 (pre-earthquake)	0.60	0.60	0.41	0.35
March 1977(immediately post-earthquake)	1.08	0.98	0.94	0.36
April 1977(after provisional strengthening with bracing)	0.78	0.74	0.59	0.36
July 1982 (prior to final strengthening, bracing supplemented)	0.77	0.72	0.48	0.37
July 1984 (after final strengthening)	0.54-0.55	0.51-0.52	0.42-0.43	0.33-0.34

Besides the data presented,it must be mentioned that the preearthquake oscillations corresponded to a practically ideally axi simmetric structure. The effects of earthquake resulted not only in a higher flexibility, but also in a strong loss of the symmetry.The increase of oscillation period was stronger for the N-S direction than for the E-W direction and this was in agreement with the effects on other structures, which demonstrated a strong directivity of earthquake ground motion. Moreover, the ambient vibrations were oriented, in the post-earthquake stage, predominantly in a NNE-SSW direction, which coincided very well with the predominant direction of seismic action, as provided by the accelerograph and seismoscope records obtained at INCERC-Bucharest [1].

The provisional strengthening led to a limited restoration of symmetry,while the final strengthening was more efficient in this view, leading to a good symmetry restoration.

It may be noted also, on the basis of data of table 4.1., that the strongest relative post-earthuqake increase was for overall torsional oscillations. This more than twofold increase accounts for an approximately fivefold decrease of stiffness. When upper story (which was in fact damaged) is considered the decrease in stiffness was still stronger.

5. ENGINEERING ANALYSES

5.1. General

The engineering analyses carried out encompassed a qualitative analysis of the loadbearing reinforced concrete structure, a linear, multi-DOF computer analysis, as well as non-linear analyses on an idealized model. The latter analyses were oriented to determine the features of failure mechanisms and to give a picture of non-linear oscillations. A summary of this work is given in the following sections.

5.2. Characterization of the loadbearing structure.

A quantitative analysis of the reinforced concrete structure puts to evidence its repetitive character and, at the same time, the fact that one can analyze practically following loadbearing systems:

(a) a radial system, S_R (fig.5.1)., consisting of a couple of columns (an internal and an external one);

(b) a tangential system, S_T (fig.5.2), consisting (in an idealized manner) of a column and of the adjacent portions of horizontal rings, considered up to the points of zero bending moment (which may be assumed to have zero vertical displacements during the deformation).

The system S_R is basically a system of two cantilevers, obliged by the horizontal members to undergo equal horizontal displacements. This system has therefore a relatively low bearing capacity and is flexible, such that it cannot properly work together with the system S_T. The system S_T is provided with the capacity of transmitting bending moments at the nodes. Therefore it be es like a proper frame.

The final strengthening has kept the general features of the systems S_R and S_T, introducing quantitative changes (increases of the resistant moments at the same sections of the internal columns). The ratio of factual stiffnesses of the systems S_R and S_T was analyzed on the basis of the ratio of natural periods for horizontal translation and for rotation about a vertical axis, of the entire structure. Given the data of the first line of table 4.1, it turned out that the systems S_T are more than three times stiffer than a system S_R. It may be expected that, in case of stronger loading, this gap would still increase.

5.3. Linear computer analysis.

The linear computer analysis was performed for the ensemble of the steel dome and the reinforced concrete bearing structure. Given the symmetry with respects to the E-W vertical plane, only half of the structure was modeled, introducing alternatively, in the physical symmetry plane, kinematic conditions that accounted for deformation symmetric and antisymmetric respectively with respect to that plane. The number of nodes was 304, the number of members was 612 and the number of degrees of freedom exceeded 1300.

The computer analysis made it possible to build a comprehensive picture of the distribution of the internal forces under various loading assumptions. Among the loading hypotheses, some were related to vertical loads (dead load, snow load, live load, testing load) and some to horizontal loads (wind load along one of the directions N-S and E-W, seismic load along one of the horizontal translation directions and seismic load corresponding to overall torsion). It may be mentioned in this respect that the loading corresponding to the overall torsion was determined on the basis of a stochastic ground motion model which accounted explicity for the non-syncronous characterof disturbances applied at different ground-structure interfaces [2].

The results of the computer analysis showed that the distribution of resistant bearing moments was reasonable, for the pre-earthquake as well as for the post-strengthening stages.

The final strengthening led to an increase in the stiffness of the structure S_T that was relatively more important than for the structure S_R. The ratio of conventional torsion to translational seismic displacements decreased, by multiplication with the factor 0.8.

5.4. Analysis of post-elastic behavior.

The analysis of post-elastic behavior was related to the main idealized loadbearing system, S_T. The failure mechanisms coresponded to the drawings of fig.5.3. for the inital stage and fig. 5.4. for the post-strengthening stage. To compare the initial and the post-strengthening conditions, it may be mentioned that the critical acceleration for the upper story of the reinforced concrete structure increased from 2.1 m/s^2 to 4.2 m/s^2 due to the influence of strengthening.

In order to get a qualitative idea about the dynamic post-elastic behavior of the structure, a non-linear time-history analysis was carried out for the system S_T, represented as a 3 DOF system. The input was represented by an artificial accelerogram with a predominant frequency of 2 Hz. and with a peak accleration of 2 m/s^2 (that was due to the PGA recorded at INCERC [1]). The results of analysis are represented in fig.5.5 for accelerations and in fig.5.6 for displacements.

6. FINAL CONSIDERATIONS

The analysis of the structure referred to made it possible to draw out following final remarks :

1. The initial layout and structural design were generally reasonable, and corresponded to the regulations in force at the time of design (the design requirements were considerably increased by the new design code, endorsed after the earthquake).

2. The structural damage due to the 1977 earthquake affected the upper story of the reinforce loadbearing structure. In spite of the apparent damage and the considerable decrease of stiffness, the structure was in no way at the brink of failure, given the good ductility conditions and the absence of remaining displacements.

3. The final strengthening solution was reasonably adopted and led to a considerable increase of stiffness and strength, as well as to a modification of the failure mechanism.

4. The post-strengthening condition of the structure is apparently good, given the full-scale experimental results and the results of engineering calculations.

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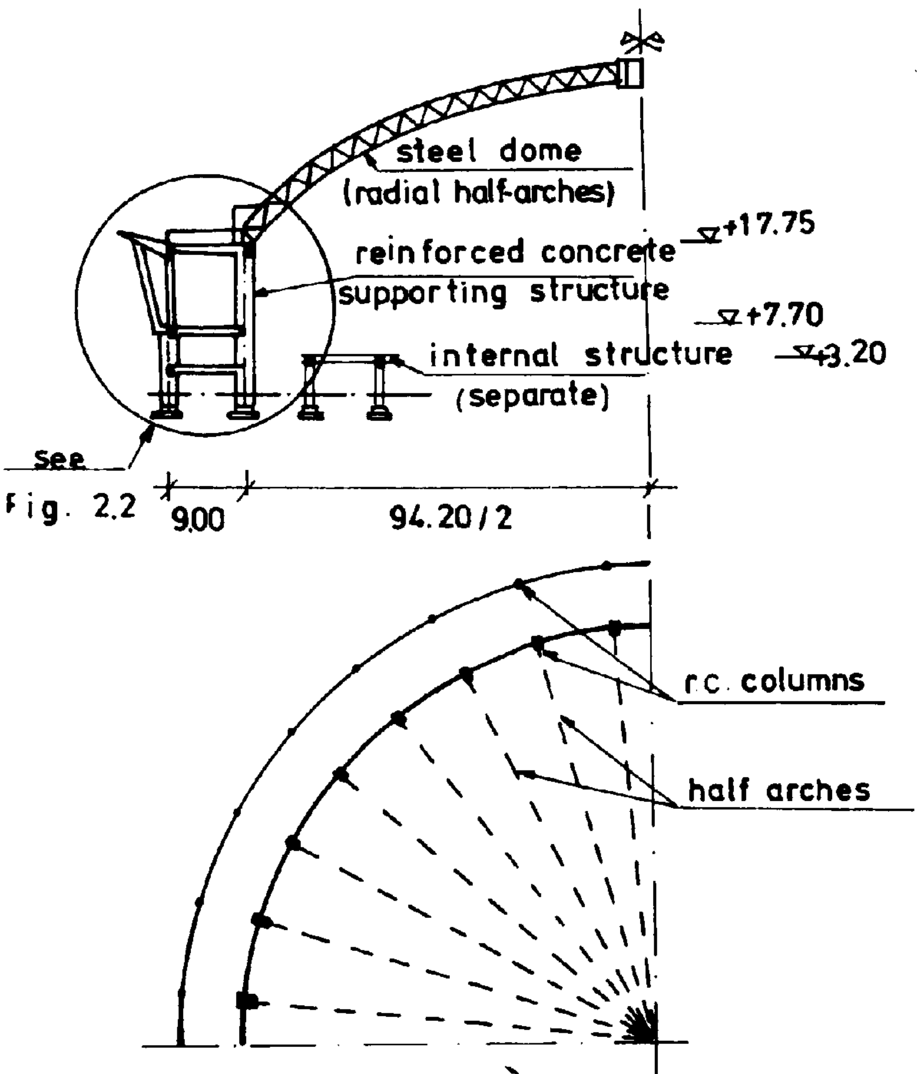

steel dome
(radial half-arches)
reinforced concrete
supporting structure
+17.75
+7.70
internal structure
(separate)
+3.20
see
Fig. 2.2
9.00
94.20 / 2
r.c. columns
half arches

Fig. 2.1

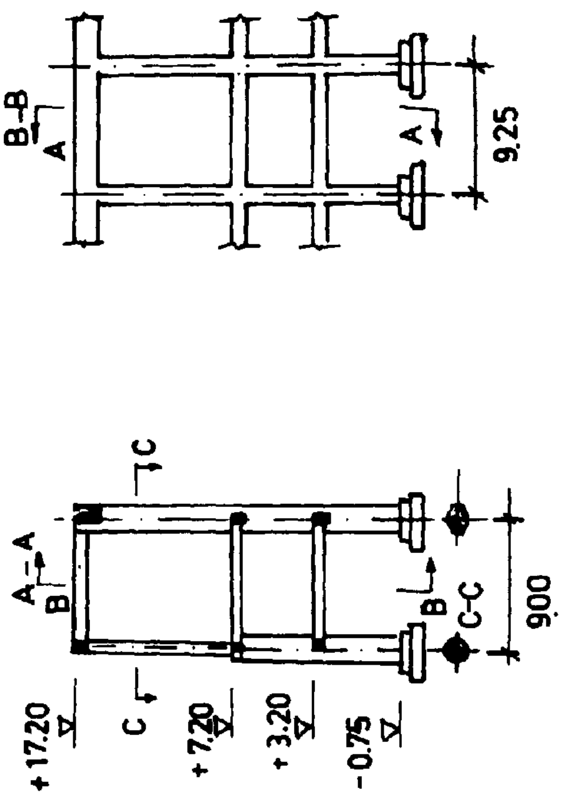


Fig.2.2

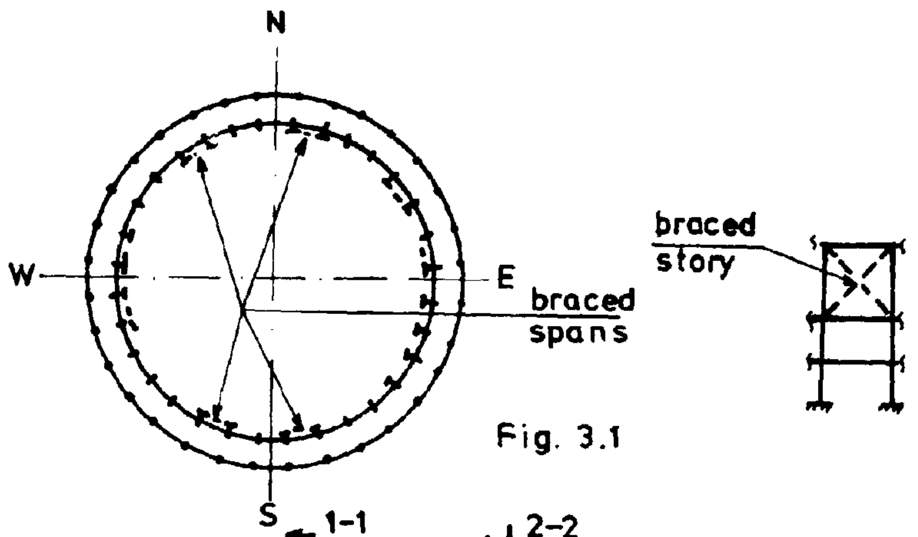


Fig. 3.1

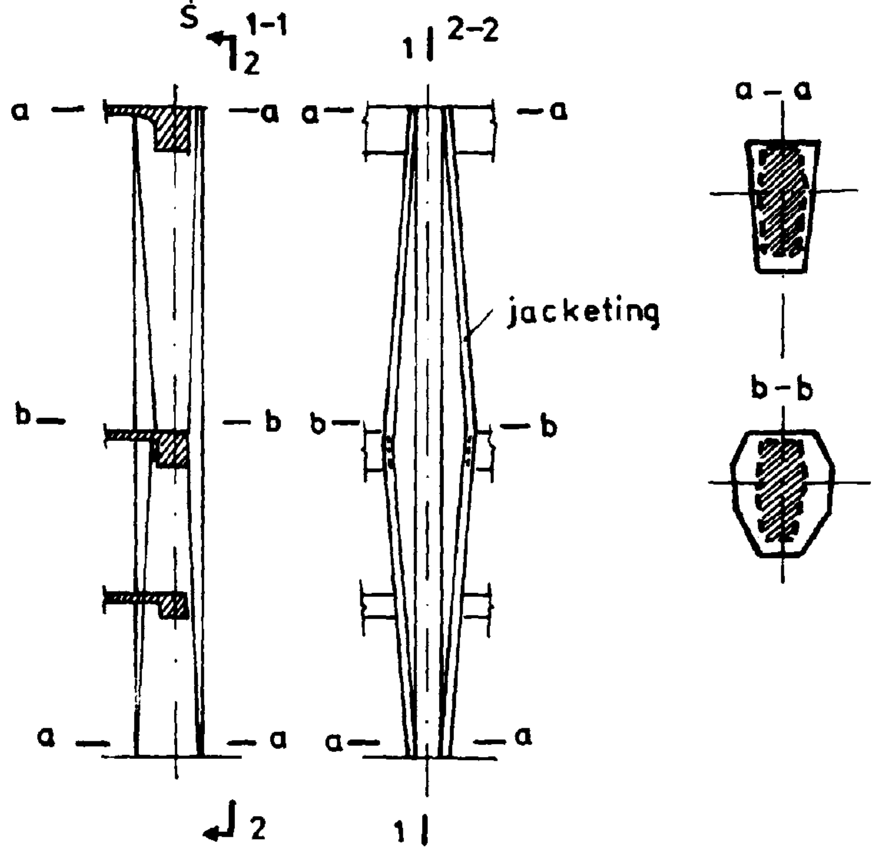
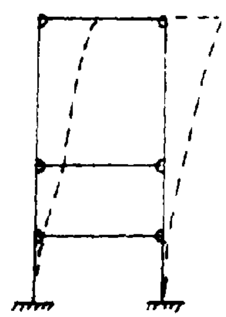
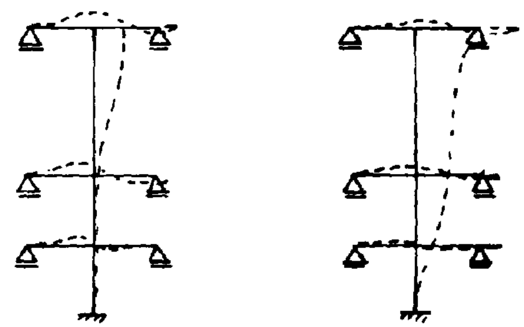


Fig. 3.2



S.R

Fig.5.1



S.T

Fig.5.2

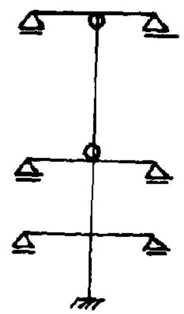


Fig.5.3

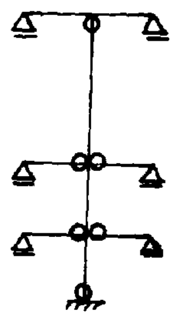


Fig.5.4

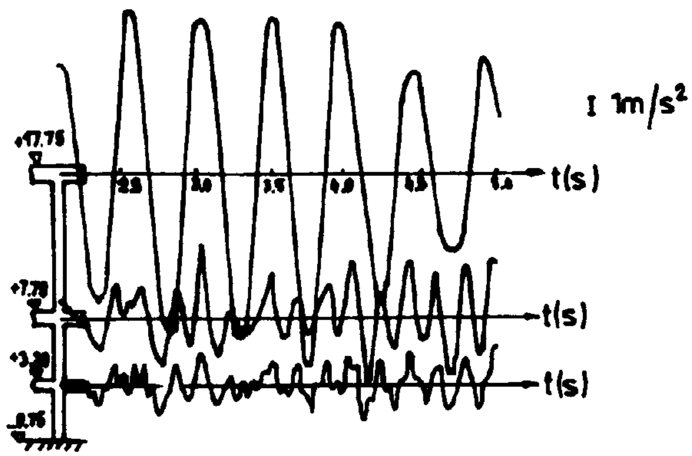


Fig. 5.5

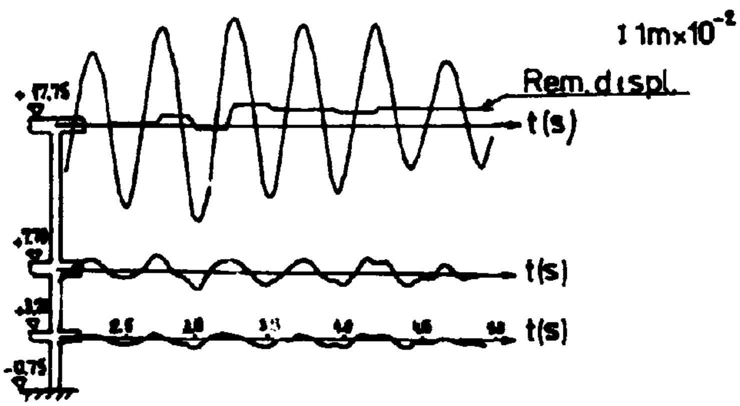


Fig. 5.6

IV.3 ARCHITECTURAL DEMANDS AND LIMITS IMPOSED BY THE ASEISMIC STRUCTURAL CONCEPT: A CASE STUDY STARTING FROM THE DAMAGE CAUSED BY THE MARCH 4, 1977, VRANCEA EARTHQUAKE ON A BUILDING TYPE WITH PECULIAR ARCHITECTURE AND STRUCTURE

Sever Georgescu[x]
Dan Rădulescu[xx]

I. RELATION BETWEEN ARCHITECT AND ENGINEER IN SEISMIC ZONES IN ROMANIA

More than 573.000 flats were erected within a period between 1948-1984 in Bucharest, most of them being in 5-11 storied buildings. The demands for building on a large scale required standard precast structures and mechanization of works. On the other hand the seismicity of Romania required reduced spans, moderate heights, structural uniformity of long buildings, etc. The architectural uniformity and the limited comfort provided by structures erected in the 1960's required new design solutions.

Since 1965 many typified tower structures used shear walls manly for seismic safety reasons. These structures have brightened up the urban area by using new architectural solutions.

By adjusting the technical procedures to functional and comfort necessities, new technologies for various industrialized buildings have been used.

II. THE CHARACTERISTICS OF THE ANALYZED BUILDING TYPE

The building described in this paper is twelve stories (with basement, ground floor, ten stories and technical floor). This is a shear wall structure with monolith reinforced concrete beams, columns and floors and was designed in 1962, according to the Seismic Design Code P.13/1963 for seismicity degree 7.

The building is designed with four towers (structures D_1, D_2, A, C, each having one flat on every level), while the fifth tower, the tower of the stair case, is erected in the central

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area, as a separate core.

The five towers have a joint shear wall at the basement in the staircase area besides the other four towers that have shear walls separated from the staircase by a 3 cm joint. A similar structure was erected next to that one.

The towers with apartemnts have been built with half a floor difference on height. The units D_1 and C are designed with staring rooms on the basement and units A and D_2 with garages on the ground level. As compared to the ground level, the foundation levels are of 2.20 m for units A and D_2 and of 2.90m for units C and D_1.

The design shows that the building is asymmetrically disposed, in the shape of a "butterfly", while the units, although of an almost rectangular shape, are asymmetrically structured with various rigidities along the two directions due to the distribution in plane of shear walls and of several cross beams meant to link the lateral shear walls on the other direction.

The tower with the staircase (this includes also the concrete lift well) is laterally bordered by four own reinforced concrete shear walls connected by lintels that reveal the vertical four rows of exterior windows for lighting the staircase and four rows of apartment access doors.

Thus, the shaft of the staircase has weakened corners, although the elevator reinforced concrete well and the stair flights and langings provides a certain rigidity.

The internal walls are masonry walls of 7.5 cm. This building is connected to the next one, built according to the same design project, by fire escape gangways between the seventh and the tenth floors.

III. THE BEHAVIOR OF THE ANALYZED CONSTRUCTION TYPE DURING THE MARCH 4,1977 EARTHQUAKE

Following the March 4, 1977 Vrancea earthquake (time 9.22 p.m. M=7.2, I_0 = VIII, I_{BUCH} = VIII, direction NE-SW), the building had serious damage in the basement, and the staircase shearwalls had less serious damage such as cracks at the lintels over the apartments doors.

The shear walls had cracks mainly in the basement. Inclined cracks with a 3-5 mm width were observed together with the damage of concrete and reinforcement as follows /3/:

- at tower D_1 : shear walls D_{10} and D_5;

- at tower A, shear wall D_4
- at tower D_2: shear wall D_{10}, column S_2 and shear wall D_2;
- at tower C: shear wall D_4 and D_5.

It was observed that, in most cases, cracks had not reached the pedestal of the shear wall. Generally, the shear walls had no cracks on the height except for the isolated cases.

Beams and lintels in apartments had vertical and inclined cracks with openings of 1-3 mm.

The floor slabs with relatively high dimensions (up to 4.50 x 8.50 m) suffered cracks with 1 mm width and in several places cracks up to 3 mmm width.

The staircase had the most serious functional damage affecting the post-seismic evacuation of people:

- perimetral shear walls had no damages;
- lintels parallel to the N-S direction were undamaged;
- lintels perpendicular to the North-South direction suffered cracks or bending failure or bending failure at both ends or failures caused by shearing force (in an X shape);
- the staircase flights had cracks of 1-3 mm width on the inner side and along the joining with the landings;
- the landings had cracks of 2-5 mm width on 2-3 rows and along the joining with the broadsteps;
- the landing bases had cracks of 2-5 mm width on 1 - 3 rows along the line of the flights and transversally as well. At landings the concrete was loosened near the space between the flights on an area of 1.25 x 1 m. One of the conditions that caused that damage at all levels was the two lamp recesses provided on the intrados of each landing. The project stipulated only two holes with the diameter of 25 mm. During construction the project was misunderstood and recesses with a diameter of 30 cm were left in the landing base, cutting the reinforcement and then placing the gypsum shells. The reinforced concrete used for casting the flights and landing bases presented segregations, inclusion of other elements such as wood chips and presented also visible working joints.

The internal masonry walls of 7.5 cm thickness had severe x-shaped cracks and were damaged by crushing the mortar or bricks.

The external masonry walls of 30 cm thickness (made of hollows bricks) placed under the end beam- of the rooms and windows had inclined cracks and were detached from the beams. No degradations were observed at the 7.5 cm internal insulation (made of bricks and performed at several external shear walls) and according to the recommendations in force after the earthquake checking have been performed since the earthquake.

V. ARCHITECTURAL AND ENGINEERING PROBLEMS RELATED TO THE BEHAVIOR AT EARTHQUAKES OF BUILDING WITH COUPLED TOWERS

5.1. ARCHITECTURAL PROBLEMS

Residential buildings with apartments distributed on separate tower units make possible natural lighting all through the day, reduce the sound impact with the neighbors and avoid the inside corridors of the long buildings. They have the disadvantage of increased energy losses on three sides of the apartments. To provide the natural lighting of the stairs, the coupled towers are shifted. With a view to reduce the number of neighbors on the landing, the floor slabs are shifted with half a story.

Buildings of the V,Y,H,U, shapes have been frequently erected. In order to obtain natural lighting of the staircase on four sides for the buildings previously described, the coupling of towers was shifted resulting a "butterfly" shaped plan.

5.2. ENGINEERING PROBLEMS

. Structural engineers are concerned with the structural solutions from which the designer has to choose the one that meets the functional and architectural requirements.

. This solution has to provide a degree of antiseismic resistance. It is possible to erect buildings, according to the same construction recommendations, with different behavior in an earthquake.

. The research team, that analyzed after the earthquake the building behavior, disposed of construction plans of the building (without calculation notes). Nevertheless P.13/1963code that has been in force since 1969, is very well-known.

On the zoning map attached to this code, Bucharest is marked with the seismic degree of 7. A shear wall tower building of this height was designed according to P.13/1963 code for seismic coefficients of about 4% and at present for those of about 10%. Thus, the differences between the design seismic forces and the real intensity of the earthquake led to much of the observed damage.

The purpose of our analysis is to establish possible qualitative improvements in the building behavior based on the study of constructions designed with the same codes.

. Following the structural analysis, in 1969 the designer observed that the towers suggested by the architect, had various sizes and the shear walls and columns used had different rigidities. For these cases, the stipulations in force recommended the separation of building by means of aseismic joints from the top down to the basement.

Using the experience available at that time a 3cm joint was chosen.

Indeed the four lateral towers behaved differently during the March 4, 1977 earthquake than the tower staircase (the foundation was the same only up to the basement level). After the earthquake tower D_2 had a permanent residual displacement toward the exterior of about 25mm and tower C of about 10 mm versus the staircase.

. After several inspections it was established that the joint was partially closed with polywood plates left from shuttering the shear walls. But the structural separation couldn' t avoid the out of phase oscillation of the towers.

Due to the relative height of the towers, the rigidity differences and the lack of the adequate joint (its dimension was not stipulated by P.13/1963 code), the units collided with the staircase tower

. The measurements of the dynamic characteristics performed after the earthquake showed differences between units, namely natural periods at torsions were very much different from those units with almost the same lay-out, such as D_1 and D_2.

. The central unit was most severely subjected to strong motion near flights, landing base slabs and lintels as the weakest points. Since there were no rational construction systems (tube corners were weakened by doors and windows), the lintels could not take over the high effort differences and the lateral shear walls were not loaded.

On the opposite diagonal direction, the rigid core of the monolith walls of the two elevator shafts had a spare resistance protections the elements. Thus, excepting the core of the elevator shafts the staircase shear walls proved to be useless.

. The breaking of the landing bases in the adjacent areas was strongly favored by the lamp holes ten times larger than the ones stipulated in the project.

. The dynamic measurements performed in 1977 showed that the damage state of the staircase tower led to the non-linearities in the natural oscillation periods, proving the suggested strengthening methods as good.

It was possible that the stresses which occurred in the staircase tower could have overhelmed the design values due to the shifting of flood slabs with half a level.

- The fire escape gangway linked unit C with the next similar one and that joining somehow influenced the behavior of the building, however that quantification was a more difficult task.

The neighboring building erected according to a similar design underwent similar damage.

It was very useful to analyze the behavior of two other buildings erected in the same period, with tower systems at about 500m distance from the foregoing analyzed buildings.

The buildings consisted of eleven story units of four towers each, but structurally coupled in two by two (with a common shear wall) and then linked between each other by means of the staircase elements (landing bases, flights and lift wells). The towers did not have the same visibility and free perimeter but the architect accepted that solution. Every unit of that type was in contact with one of the next four towers through an aseismic joint. Each building had two units of four towers separated by two aseismic joints. The working of several joints and cracks in the shear walls were observed on March 4, 1977, due to the reduced design forces at that time and to the wrong used slipping forms, together with the working of certain cracks on the staircase landing base, but not as large as those occurred at tge first type of building. Neither the vertical displacement of towers nor the existence in the landing base of lamp holes, similar to those discussed at the first type of building, caused loadings to break.

As an additional factor in these buildings, the increased foundation depth, by the existence of three basements, was worth mentioning.

At the same time, the more reduced natural periods for units in the second case could determine higher calculus dynamic amplification coefficients (β) due to the spectral curve with a maximum β value only up to 0.3 s according to P.13/63 and thus led to increased seismic forces.

The analysis of the behavior, during 1977 earthquake in the city of Jassy, of other buildings with similar flexible lay out and coupled towers, where the staircase is not a structural unit separated by a joint /4/, proved that the unit most subjected to the earthquake was the staircase. The landing bases and the shear walls of the staircase cracked in the most disadvantageous direction, generally diagonal in plan.

That was a characteristic phenomenon for linking rigid cores only through flexible elements (lintels, beams, slabs) ; that phenomenon occurred inside of the towers A_1, D_1, D_2 because of the damage of floor slabs in the first described buildings.

At the same time, the damage of the basement shear walls was described in the first case in the neighborhood of certain openings in the vertical structure. The mentioned remarks proved the different method of interaction between soil and shear wall structure versus the interaction between soil and framed structures.

As the value of the additional deflection given by the foundation rotation (important for relative rigid shear wall buildings) was in an inverse ratio to the inertial moment of the foundation base area, it was much more avisable to use the solution of entirely coupled towers with a foundation that worked at the same time for all towers, or else to chose foundation systems that did not give exaggerated rotations.

. Taking into account the general knowledge in the field, accumulated by the end of 1969, whether it was mentioned or not in the building codes, the construction characteristics chosen for the first case were influenced by the reduced design forces given by the code for relatively flexible buildings, by the lack of data on the real resistance, by the behavior of shear walls and joining elements, by the exaggerated belief in the rigidity characteristic of shear wall constructions and by disregarding the contribution of dynamic deformabilities of foundation ground. (σ =2.5 kg/cm^2), all these being represented by the solutions with shifting, differentiated surface foundation and seismic joint between units. The present code P.100/1981 suggests to avoid excessive tronsoming of constructions.

5.3. SIMILAR CASES ON THE BEHAVIOR OF COUPLED TOWER BUILDINGS RECORDED IN THE WORLD

. The present international experience on the behavior during earthquakes of buildings of this type, recommended by architects and consisting of connected or separated units, shifted or not on the vertical has drawn attention to certain similar and specific problems.

. During the Caracas earthquake - Venezuela 1967- many framed buildings collapsed or were damaged, generally those buildings with a flexible ground floor among which several had an H-shaped flexible lay-out. The main units were connected with the smaller ones that included the staircase and the elevators and were provided or not with an seismic joint between them.

The joint between the two buildings, San Jose and Palace Corein that collapsed, had been provided in the central area, and for the other buildings such as Covent Garden, Pasaquire,San Bosco etc. which were only damaged, the staircase towers were of more reduced dimensions as compared witht the main towers /5/.

The Union building, although consisting of nonsymmetrical structures, was connected by two staircase units toward the end of the main towers. Maybe due to this fact or to the structural continuity between all units, the building was moderately damaged but it did not collapse.

Taking into account the construction similarity of many buildings in Caracas, the importance of the distribution method and of the method of joining the main units with the staircase towers has become obvious and may be correlated with damage.

We do not know to what extent the detailed information about these constructions are known by the engineers in our country who designed similar buildings during 1968-1969.

. Another case of compound buildings was furhher studied, central building of the Olive View Hospital damaged by the 1971 earthquake in San Fernando , USA /6/; the central unit was damaged as were three out of the four towers of the staircase placed at the side ends of the central units.

It is noteworthy to mention the intensity of the 1971 earthquake, the behavior of the foundation ground and of certain main structural elements, as well as the system with two flexible stories at the base of the building.

Apparantly, the presence of an seismic joint of 10 cm between the towers and the main building was much more disadvantageous than useful. Although the main building, with transversally connected buildings, butterfly-shaped, was damaged and distorded due to the two flexible sotires, it did not collapse while two of the four towers collapsed and one was strongly inclined - The problem of dimensioning the aseismic joints is still un - solved even in countries with advanced design codes.

Similar cases were recorded after the Romanian March 4 , 1977 earthquake on the teritory of Svichtov town in Bulgaria /7,8/.

A residential reinforced concrete building on R.Avramov St. at Svishtov built disregarding the seismic code, consisted of two towers asymmetrically connected in a Z-shape through the staircase, each tower being shifted on its height with half a level.

The building collapsed because the designers had not taken into account the seismic forces, the severe torsions of the non-synchronized oscillations for the two units, the stress of the staircase at various levels caused by the dislevelment and the flexible ground floor and by the aseismic non-conformity of frame joints.

Referring to the mixed constructions (columns and shea r walls) we should mention the case of several buildings from Bulgaria, Svishtov consisting of two main towers coupled with the staircase tower. Eight story structures were designed for a non-seismic degree, erected using the method of packed lifted floor slabs and then joined by monolithical works or by welded vertical elements (marginal columns and shear walls). The staircase was a monolith built with sliding forms.

A main tower consisted of two towers coupled at the corners. The main towers were distributed on both sides of the staircase and shifted on its height with half a level. The floor slabs were only adjacent to the staircase.

Following the March 4, 1977 Romania earthquake diagonal cracks occurred in the walls and damage in the filling masonry The staircase shear walls had cracks produced at the working joints and lintels during the sliding process but the structure was not damaged.

At the working joints between the precast floor slabs and the staircase central nucleus, cracks were observed but nevertheless the staircase was still functional after the earthquake. The lack ofm joints was apparently useful.

Bulgarian specialists said that the seismic energy was disippated in general by the infillihg masonry of 25 cm thickness present on the contour frames which worked with vertical shear wall saving the structure. That means that the structure worked on a unitary method.

Out of this type, only two had the same type of damage; the other 3 buildings had less serious damage at a seismic intensity of VII-VIII MSK at Svishtov.

The analysis of the type of buildings erected during 1960-1970 showed that the coupling process had not an unitary method.

The Romanian and international studies proved once more that the buildings made of units with various functionality, structurally coupled or adjacent, are an area for cooperation between the engineer and the architect aiming to optimize the architectural solution and structural safety.

CONCLUSIONS

1. The structural solutions for buildings consisting of towers with various functionalities required a specific analysis of their behavior during earthquake.

2. The use of general recommendations for aseismic joints included in the seismic design code (meant to avoid the effect of non-synchronized oscillations of units with various rigidities should be analyzed from case to case in order to assess the requirements, the dimensions size, and adequate constructive measurements in order to avoid the undesired effects by the mere application of the method.

3. The Romanian and international experience concerning the earthquake behavior of buildings made of coupled units emphasized the importance of structural solutions for the joining units correlated with the joint type and with the erection technology of the structures themselves. The following aspects were very much unfavourable: asymmetrical coupling,free joining of tall units, vertical shifting of functional stories together with shifting and reduction of the foundation depth, uncontrolled transmission of high stresses only through slabs, beams or lintels, etc.

4. Framed structures,especially those with flexible ground floors, were most affected, even those with coupled towers.Shear wall structures may become as flexible due to an exaggeration of architectural functionality requirements, requiring the merging of different functions in the same tower that might provide another controllable and safe structural behavior during earthquakes.

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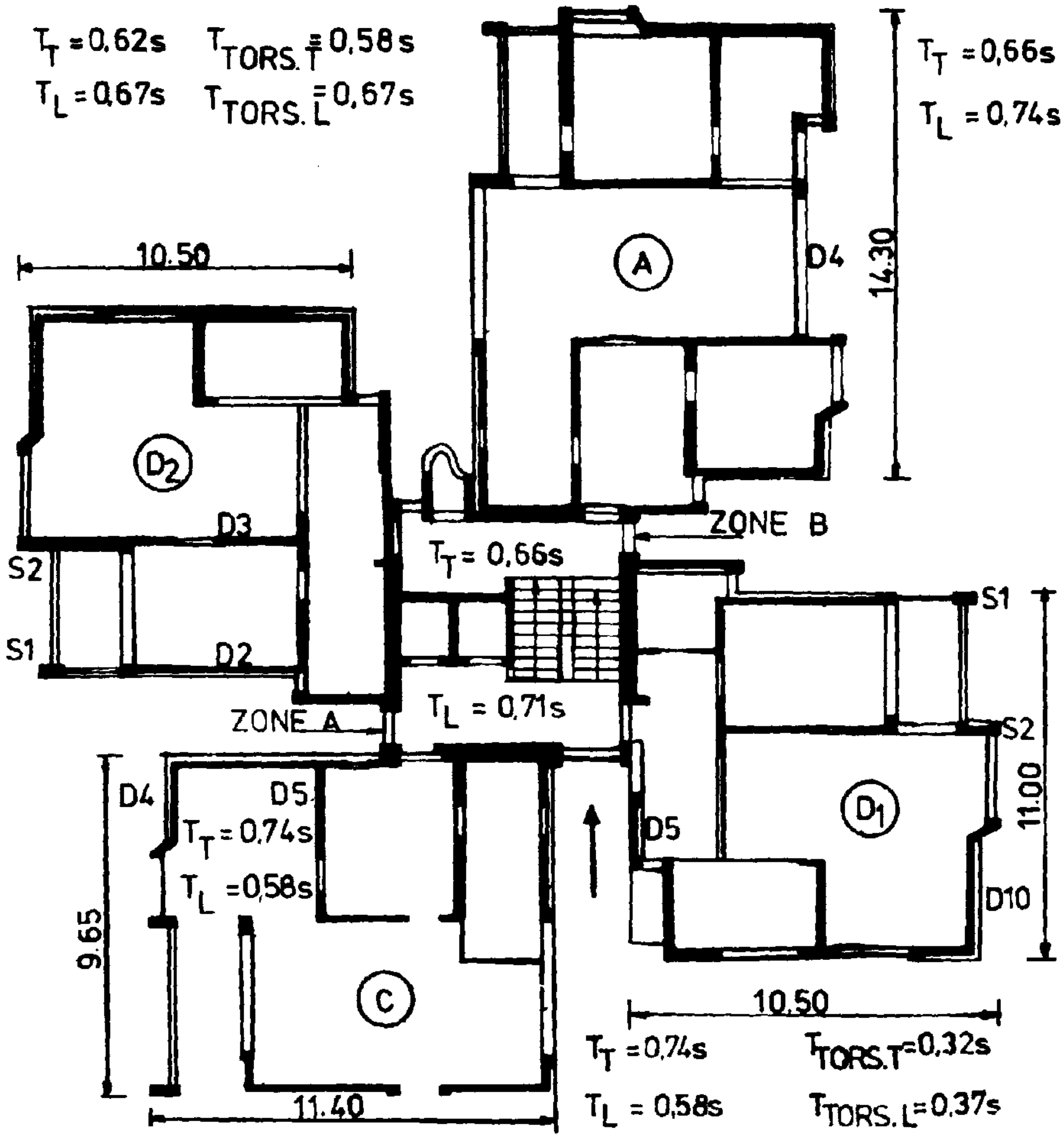
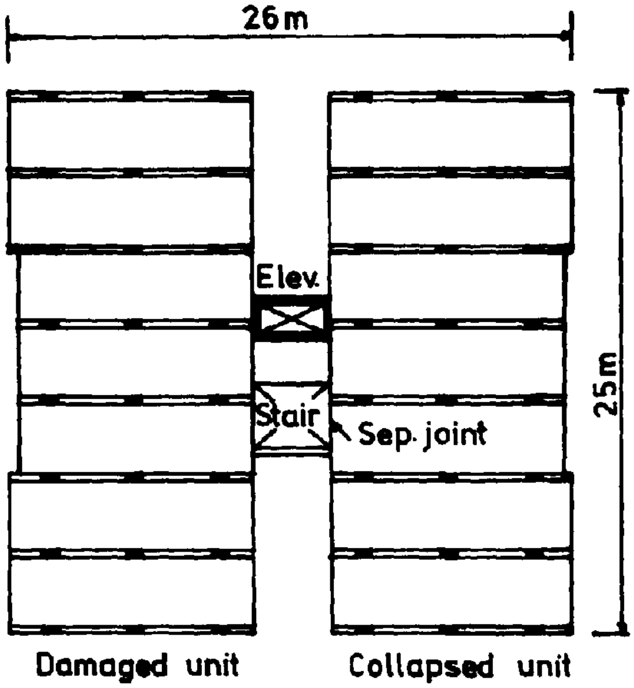
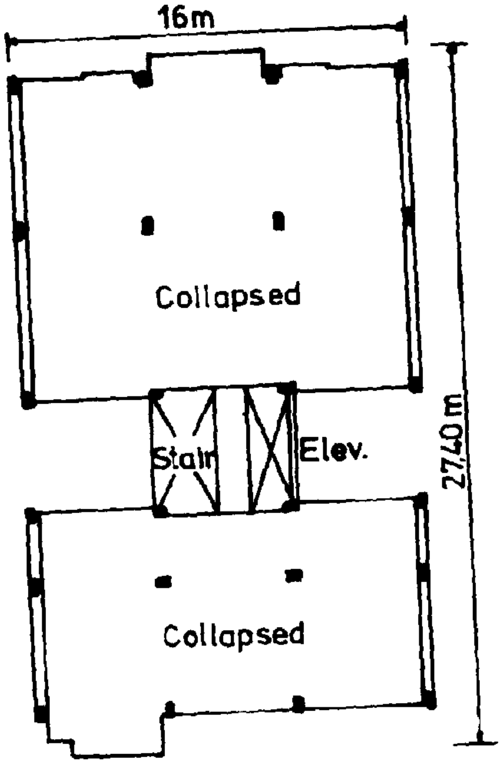


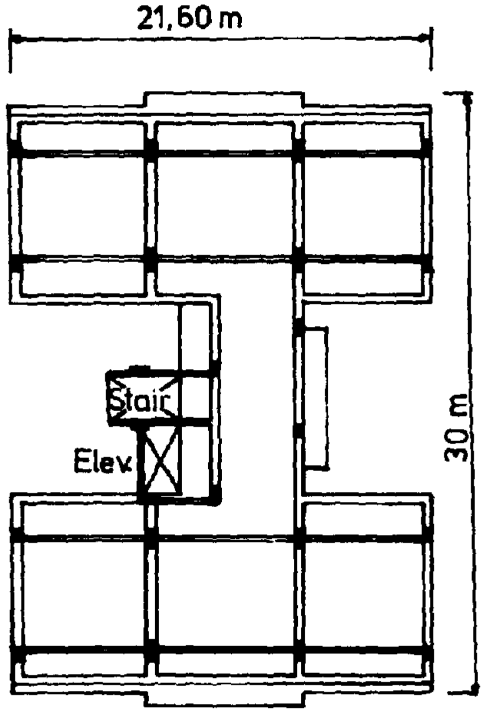
BUILDING WITH SEPARATED UNITS
DAMAGED AT 4.03.1977 IN BUCHAREST



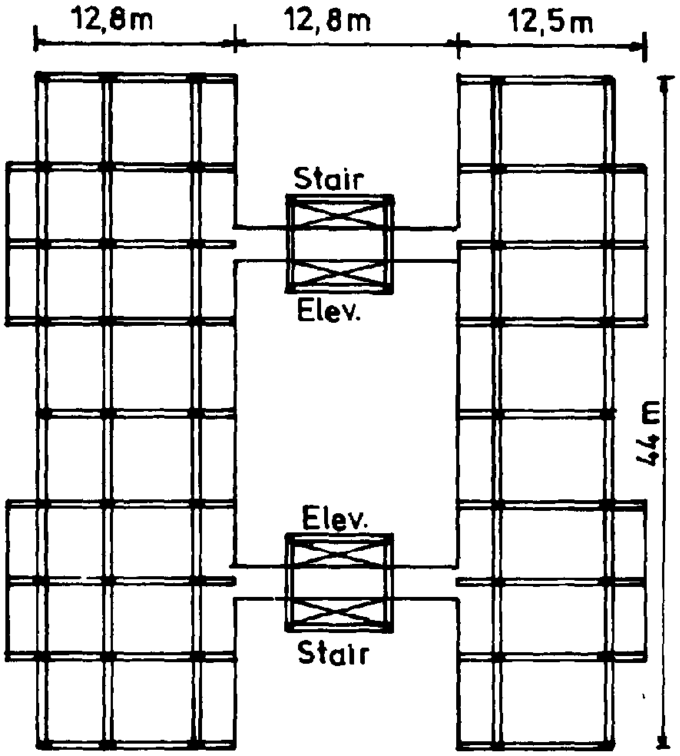
TIPICAL FLOOR PLAN
PALACE CORVIN BUILDING
10 story
CARACAS 1967



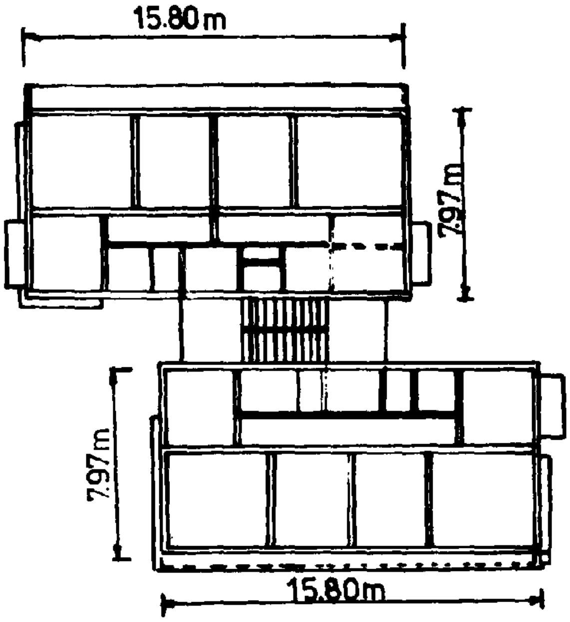
SAN JOSE BUILDING
10 story
CARACAS 1967



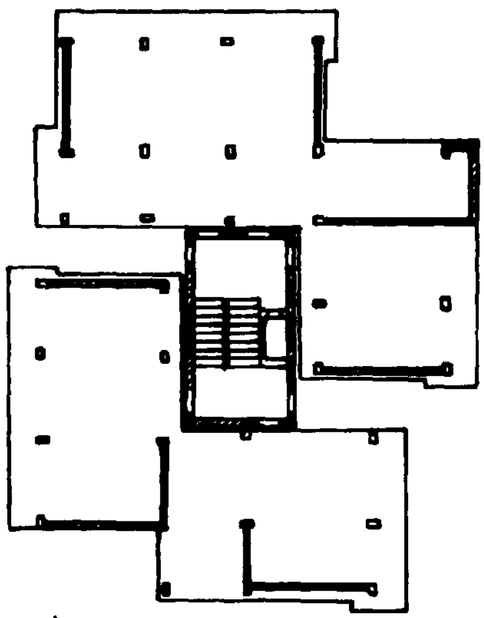
PASAQUIRE BUILDING
12 story
CARACAS 1967



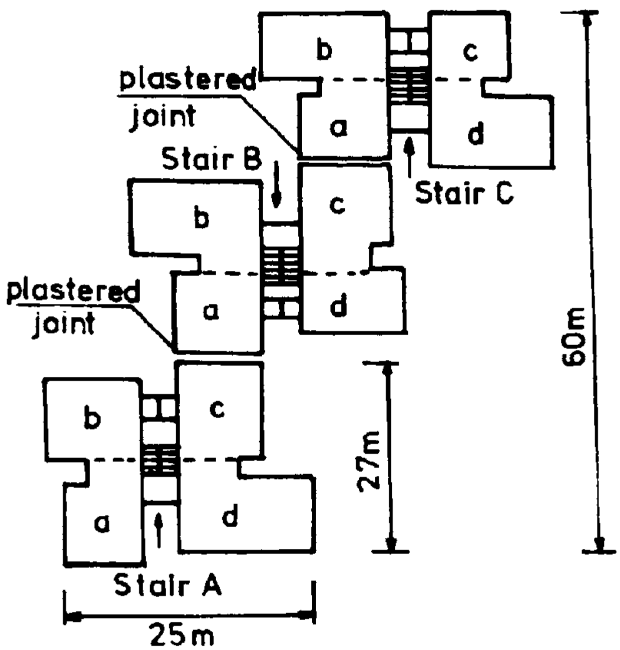
UNION BUILDING
17 story
CARACAS 1967



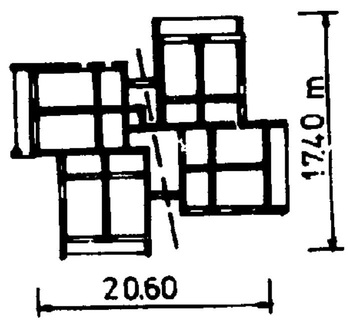
Building with assymetric units collapsed at 4.03.1977 in Svishtov, Bulgaria



Building with lift-slab Moderate damage at 4.03.1977 in Svishtov, Bulgaria



Buildings consisting of units coupled 2 by 2 in Bucharest
Moderate damage at 4.03.1977



Buildings consisting of units coupled 2 by 2 in Jassy
Moderate damage at 4.03.1977

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SINGLE STORY INDUSTRIAL HALLS

Two structural models representing fully prefabricated single story buildings have been investigated by means of a large capacity shaking table. The main difference between the two tested models (and between the prototype structures represented) consisted in the roof solution. In the case of the first model the roof was made of reinforced concrete surface members alternating with opening for lighting apertures. The roof of the second model included only longitudinally and transversally at precast beams providing a relatively flexible diaphragm at the roof level.

No two dimensional bearing member was used at the roof structure, and the openings between the beams were used either as light steel lighting apertures or opaque areas alternatedly disposed.

Both structures tested were 3/10 scale models, incorporating the main characteristics of typical single story industrial halls . Their dimensions were 9.0 x 7.2 m (fig.1) and an average height of 1.8 m. Thus the prototype were fragments of actual structures having two bays of 15 m in transversal direction (fig.2) and 2 bays of 12 m in longitudinal direction along the central column axis (fig.3) and an average usable height of 6 m. Along the marginal column axis, in longitudinal direction four bays of 6 m each were provided. The structure consisted mainly of: (1) precast columns of reinforced concrete (B 300) clamped in precast isolated foundations; (2) precast main beams G Ψ 12-15-2 on the central column axis, and G6-T on marginal column axis (B 500); (3) transversal beams GT-15-3 of reinforced concrete (B 600) at distance of 3 m. Welded joints were used throughout the structure.

The models were tested in one horizontal direction parallel to the longitudinal direction. The maximum base accelerations obtained during the tests were 5.4 m/s^2 for both models. The maximum horizontal displacements at the roof level were 72 mm at the first model and 90 mm at the second one. The acceleration amplification at the roof level was about 5^ % larger for the second model.

The tests have revealed a great amount of roof deformability especially in the case of the second model. The design of such types of structures under lateral loadings should be performed with a proper consideration of the roof deformability. As regards the particular tested solutions, a recommendation was made to increase the cross-section dimension of the columns with 10% in each direction, in excess to the values determined according to the code provisions (1). It should be noted that due to reduced roof weight, usually small column

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dimensions are required from a conventional design. On the other hand, the tests have shown that overall torsion is significant and is generally larger than that recommended by the code for such structures. Therefore a proper detailing of the corner columns is of paramount importance for the seismic performance (2).

FURTHER STUDIES ON SINGLE-STORY STRUCTURES

The investigation of single-story structures was continued by testing a new structural model for checking the design criteria and studying in more detail the ductilities of columns under strong earthquake excitation. Another objective of this study was to develop some experimental hysteretic curves for columns in various stages of behavior up to failure. The prototype structure was an industrial hall having two bays of 18 m in the transversal direction and a variable number of bays in the longitudinal direction. The height was 4.2 m. The test model was reduced at a length scale of 1/1,4 to make use of the entire capacity of the seismic simulator. The main geometric characteristics and dimensions of the model are shown in fig.4. It included two central columns, S 1, two marginal columns, S 2, and two corner columns, S 4. As the main concern in this study was the column behavior, the original roof solution consisting of two-dimensional bearing members resting on reinforced concrete beams was replaced by a segmental slab loaded with ballast to simulate the gravity loads of the prototype. The values of axial loads in model columns were 374kN for S 1, 212 kN for S 2 and 137 kN for S 4. The relatively large scale of the model, allowed for reinforcement details close to those of the prototype columns.

The test showed that up to base accelerations corresponding to a seismic intensity of 6 on MM Scale the model remained mainly within the elastic range. The first cracks occured at the column bottom sections at base accelarations corresponding to a seismic intensity of about 6.5.

For higher actuating intensities (7.0-7.5 on MM Scale) the cracks extended from the base along the height of the columns. For a correct interpretation of the results it should be noted that the model columns were rigidly fixed on the shaking table, in other words the foundation flexibility was not simulated in the experimental program. The main response parameters recorded during the tests were accelerations and displacements at selected points in the structure and hysteretic curves (as force-displacement or M- Ø relationship) in various stages of earthquake excitation.

The capable ductilities of the columns were function of

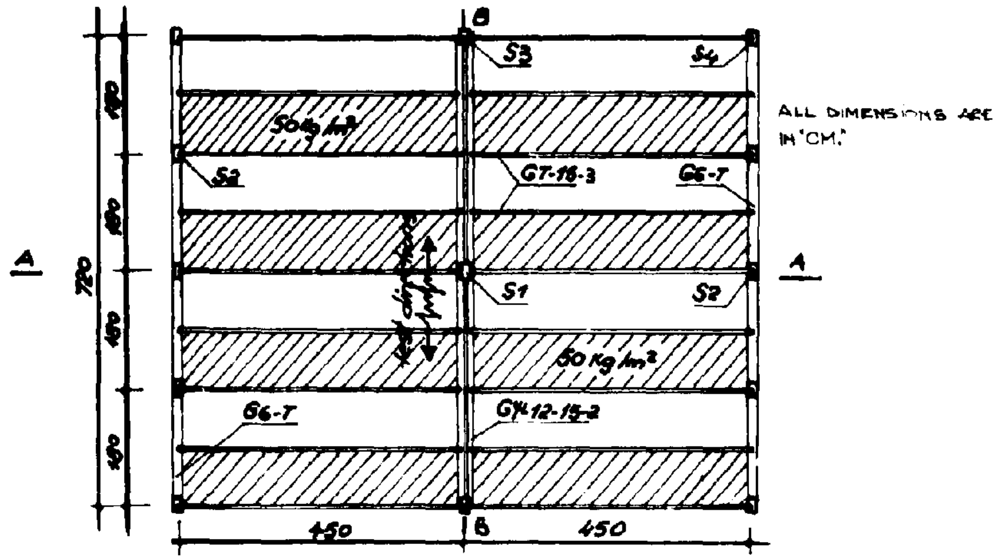


Fig.1 - ROOF PLAN, MODEL OF A SINGLE-STORY BUILDING

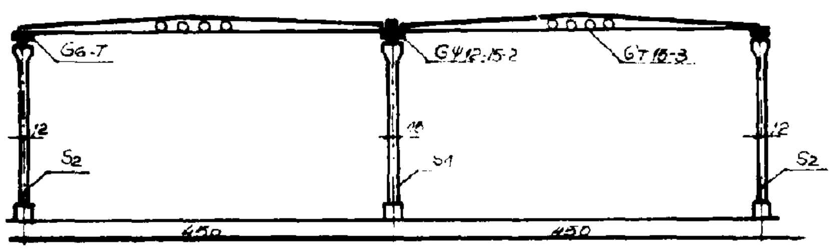


Fig.2 - CROSS SECTION A-A

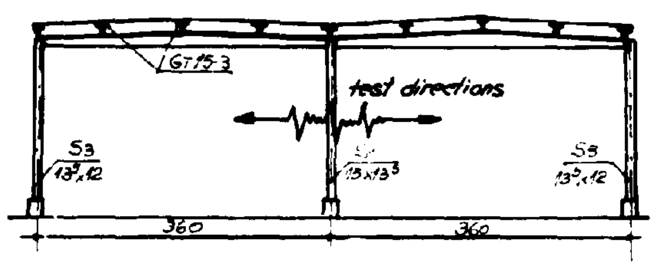


Fig.3 - LONGITUDINAL SECTION B-B

the ratio σ_b/R_i, where σ_b is the actual stress in the concrete and Ri is the concrete class. The displacement ductility coefficients determined experimentally were 20-25% higher than those resulted from a conventional analysis. However, the measured ductilities corresponding to curvatures were closer to the calculated values.

SEISMIC BEHAVIOR OF A THREE DIMENSIONAL MODEL OF A TWO STORY R.C. STRUCTURE

A relatively large scale model of a two story r.c. structure consisting mainly of precast elements was erected and tested up to failure by means of the same shaking table. The length scale was 1/2.5. The model had three bays in the actuating direction and two bays on the normal direction. The plan dimensions of the prototype were 12 x 18 m. One distinctive characteristic of the structural system was the omission of some intermediate columns at the second story to allow larger clear spaces at this level (fig.5). The model was tested with strong base motions simulating one horizontal component of the earthquake records. Figure 6 is a view of the model on the shaking table. The only monolithical members of the structure were the longitudinal beams and reinforced cast-in-place topping at the floor level.

The maximum acceleration recorded at the roof level was 6.1 m/s^2 and the maximum lateral displacement during the last test run was 78 mm, representing approximately 1/30 from the story height (Table 1). The tests showed that under severe seismic excitation large amount of damage occured into the component members but the structure withstood these base excitation without collapse.

The failure occured by formation of a sway mechanism at the second level. It is interesting to note that the roof consisted of large surface loadbearing members which were not rigidly connected to the columns. In other words the connection between columns and roof was provided by flexible joints, so that the mentioned failure mechanism developed as soon as plastic hinges occured at the bottom end of the columns at the second level. The test emphasized some ways to improve the seismic behavior of the structure. Due to pronounced differences between relative lateral displacements at the two levels, it appeared reasonable to reduce the cross-section dimensions of the central columns (extending only over the first level height) and to increase the cross-section of the perimetral columns (extending over the whole height of the structure) in order to provide a more uniform distribution of the stiffnesses along the structure height. On the other hand a suggestion was made to study the possibility of designing the central columns as

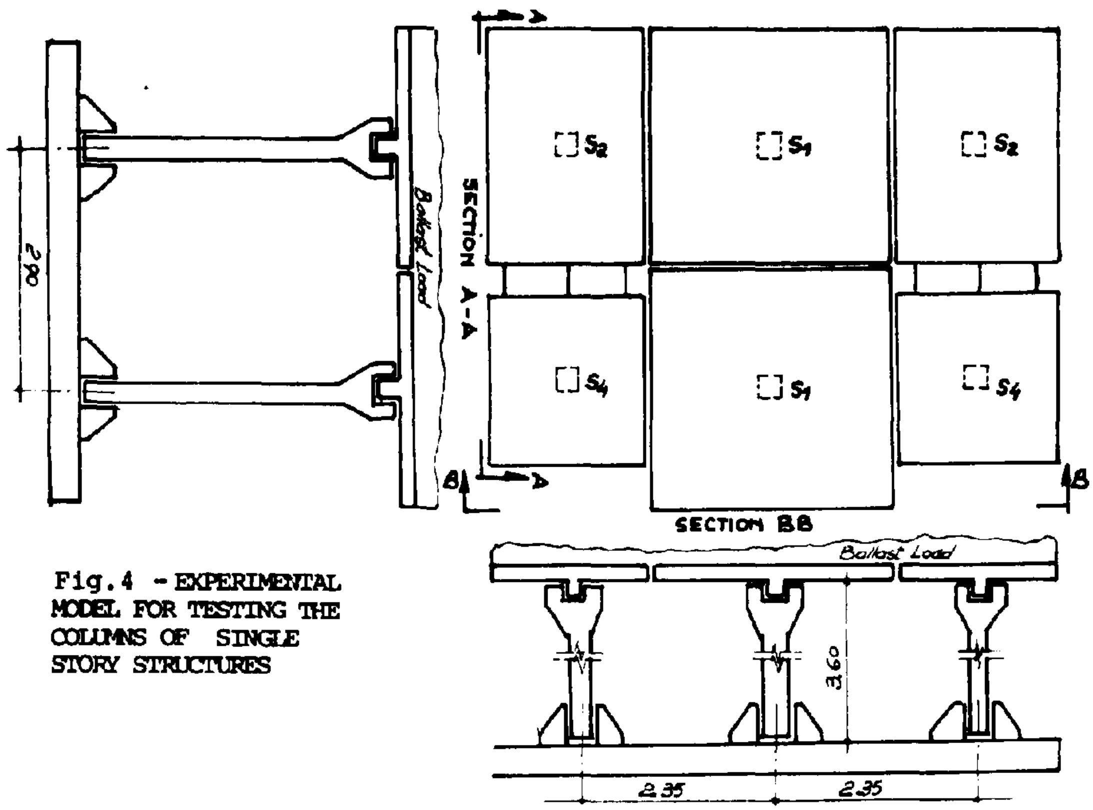


Fig.4 - EXPERIMENTAL MODEL FOR TESTING THE COLUMNS OF SINGLE STORY STRUCTURES

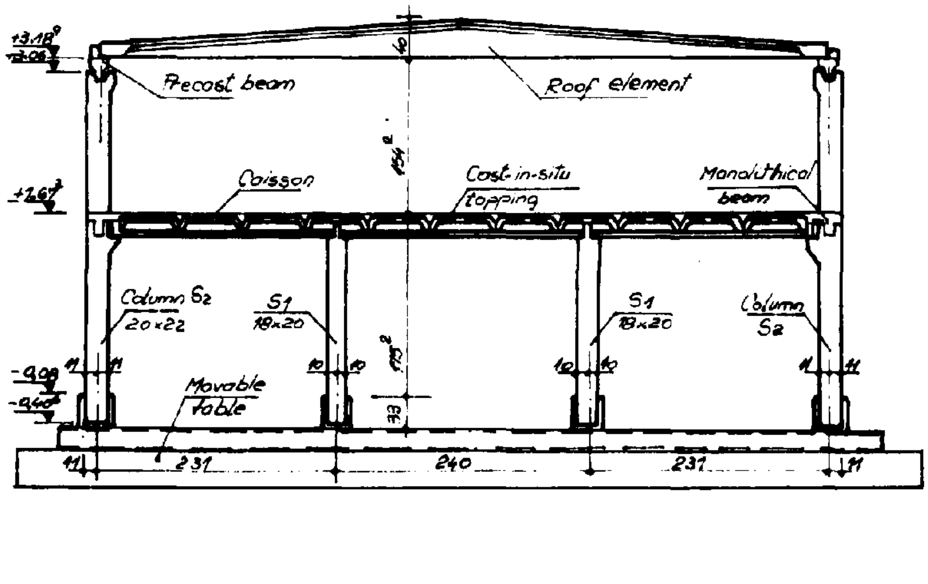


Fig.5 - TWO STORY STRUCTURAL MODEL - CROSS SECTION

pendular members (hinged at both ends) and to increase properly the stiffness of the remaining columns. As the failure was accompanied by concrete crushing and reinforcement buckling in the column zones above floor slab, it was recommended to provide a supplimentary transversal reinforcement in these zones and to improve the joint details.

The seismic response of the model was compared with responses predicted by various analythical methods /3/. The variation of the lateral flexibility coefficient $\delta 22$ with the fundamental period may be observed in fig.7. The maximum stresses of the corner column, S 4, in a section above floor slab during various seismic tests are shown in fig.8.

SEISMIC BEHAVIOR OF A THREE STORY MODEL OF A REINFORCED CONCRETE INDUSTRIAL BUILDING

The investigation sinthetically presented in this section refers to the study of the seismic response of a three-dimensional model of a three-story industrial structure in both elastic and postelastic range. The prototype is a two bays by two bays portion of an actual structure comprising eight bays of 6 m in longitudinal direction and two bays of 9 m in transversal direction. The structure is composed of precast columns as continuing units along the entire height, precast transversal beams supporting caisson elements at a current floor slab, prestressed surface elements and their corresponding beams at the roof level. Most of the component members are precast units excepting the longitudinal beams and topcasting at the floor slab.

The experimental investigations were carried out on a reinforced concrete scaled model including two end bays within an actual structure. The length scale was 1/3. Other similitude scales are as follows. Elasticity modulus, volumetric density, stress, strain, force distributed on unity area and acceleration scales : 1/1. Force and total mass scales: 1/9. Lateral displacement scale :1/3. Time and velocity scales: $1/\sqrt{3}$. Moment of inertia scale: 1/81. A transversal section through model structure is shown in fig.9.

Subsidiary nonstructural weights were attached at roof and at each floor slab to compensate for low stress level in the test model when considering only its dead-weight.

Figure 10 shows the experimental structure on the shaking table prior to performing the tests.

The major objectives of the experimental investigation under the following: (1) To study the behavic of the model un-

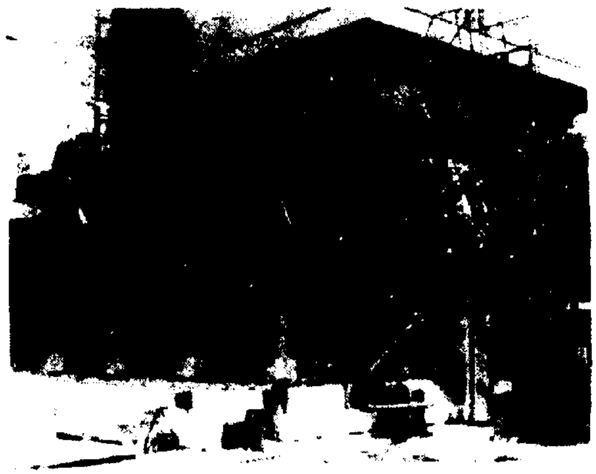


Fig.6 - VIEW OF THE TWO STORY MODEL ON THE SHAKING TABLE

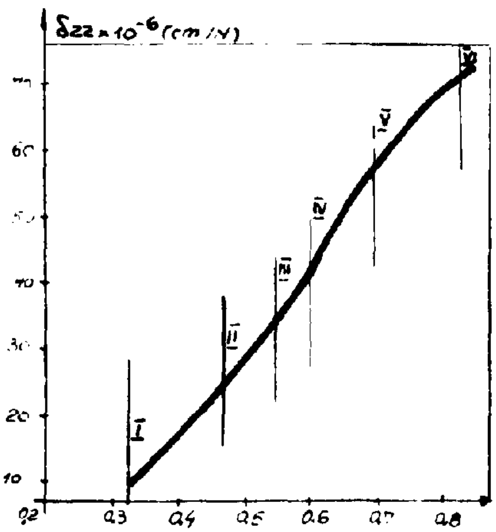
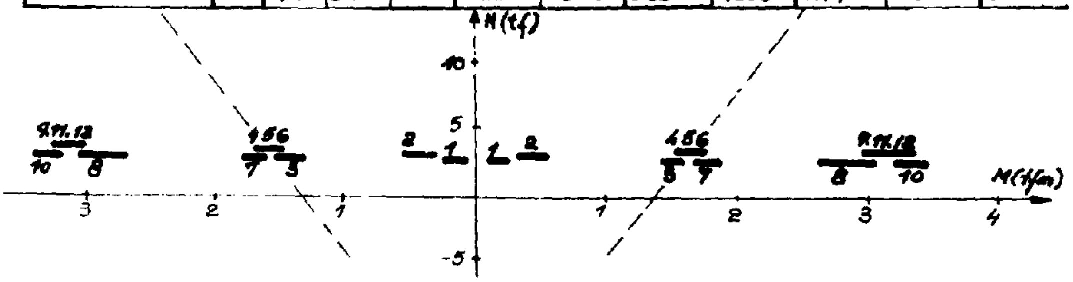


Fig. 7 - FLEXIBILITY COEFFICIENT 22 VERSUS FUNDAMENTAL PERIOD.

TABLE 1 - Maximum seismic response (the two story model-nonsyncronized values)

Behaviour stage	Test No	Max. accelerations a_0	a_1	a_2	Max. displacements (mm) Level 1	Level 2	Fundam. period (sec) T_1	T_2	ΔY/H Level 1	Level 2	a_2/a_0
Quasielastic	1	0.18	0.20	0.35	0.42	0.95	0.33	0.118	1:4000	1:3075	1.94
	2	0.32	0.40	0.70	1.10	2.90	0.38	0.139	1:1527	1:905	2.18
Cracking	3	0.86	1.40	2.80	4.80	13.90	0.42	0.149	1:350	1:179	3.25
	4	1.05	2.25	3.05	5.85	15.85	0.46	0.165	1:287	1:163	2.90
	5	1.20	2.50	3.00	7.75	22.40	0.50	0.180	1:216	1:111	2.50
	6	1.40	2.50	3.10	8.58	24.90	0.52	0.187	1:196	1:99	2.21
	7	1.60	2.70	3.00	9.15	26.25	0.53	0.191	1:184	1:95	1.87
Crack development and damage	8	2.40	4.60	5.50	16.80	52.50	0.59	0.212	1:100	1:46	2.29
	9	2.50	4.60	5.60	17.50	54.50	0.61	0.219	1:96	1:44	2.24
	10	3.00	5.50	6.10	24.60	71.35	0.69	0.240	1:78	1:33	2.03
Failure	11	3.20	5.20	5.90	20.10	70.00	0.72	0.257	1:84	1:32	1.84
	12	3.10	5.50	6.00	22.80	78.00	0.83	0.288	1:74	1:30	1.93



FIG; 8 - MAXIMUM STRESSES OF CORNER COLUMNS, S4, AT LEVEL 2 UNDER SEISMIC EXCITATIONS der low, moderate and high intensity seismic loadings and to compare the elastic and postelastic responses with those predicted by various analytical methods; (2) To evaluate the three-dimensional interaction of precast members of the test structure under strong seismic excitations; (3) To study the strength and stiffness degradation along postelastic range up to failure; (4) To get experimental data regarding the variation of accelerations and displacement along the structure height and (5) To observe the cracking process and failure mode of a spatial structure of this type under earthquake loadings.

Three types of tests were performed on the model, namely: (1) Static tests (horizontal load applied successively at floor levels) to evaluate the structure stiffness at selected stages of behavior; (2) Free-vibration tests to determine damping and fundamental period at low amplitudes and (3) Earthquake simulation tests to induce various levels of stress within the structure starting with relatively low excitations and going on with successively increasing intensities of base motion. It should be noted that damage into the component members was induced only under earthquake simulation loading. The seismic actuating program of the model included nine tests with base accelerations ranging from 0.5 to 2.76 m/s^2. A selection of maximum responses is given in Table 2. To account for the reduced length scale, all tests were conducted with the base motions speeded up by a factor of 1.73 according to the similitude requirements.

The experimental investigation correlated with analytical study on the model structure emphasized four distinctive stages of behavior up to failure.

1. Quasielastic stage (seismic tests I and II) was characterized by maximum base accelerations of 0.54 and 0.79 m/s^2 respectively and maximum lateral displacement at the roof level of about 7.7 mm during the seismic test I. The base shears and overturning moments were lower than those corresponding to design distribution of the conventional seismic forces.

2. Cracking stage (seismic tests III and IV) was characterized by initiation and development of cracks in joints mainly along surface contact between precast elements.

Maximum base accelerations were in the range 1.3 to 1.6 m/s^2 with amplification of 1.11-2.13 along the structure height. Maximum lateral displacement recorded was of 12.78 mm (Table 2).

3. Stage of extensive development of cracks and occurrence of some structural damage (seismic tests V - VIII) was characterized by the extension of cracks from joints to the beams and column sections and was followed by concrete crushing in some column sections. Although maximum base accele -

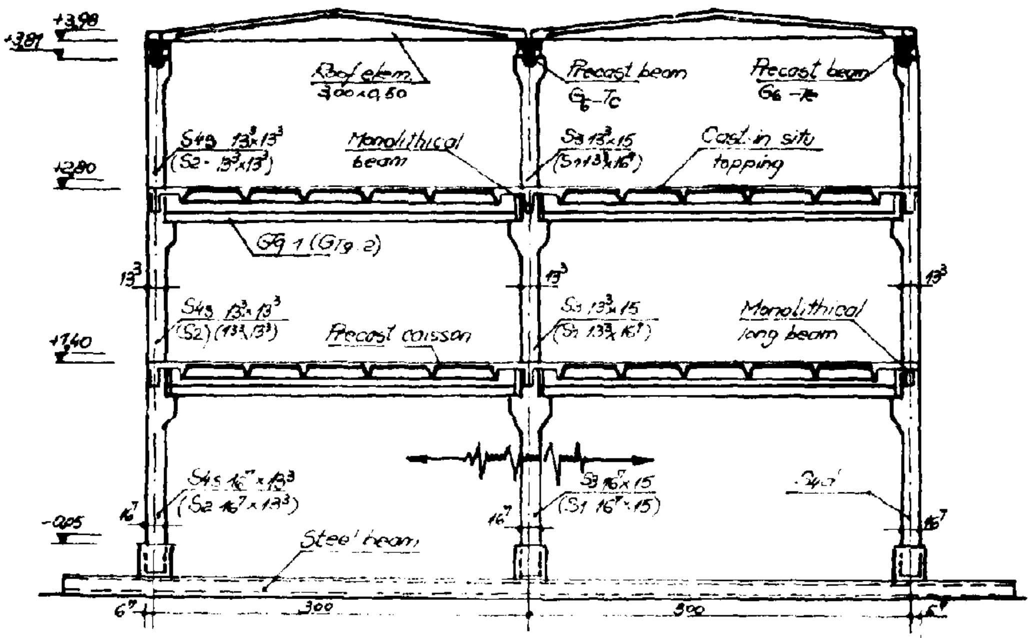


Fig. 9 - THREE STORY MODEL - TRANSVERSAL SECTION

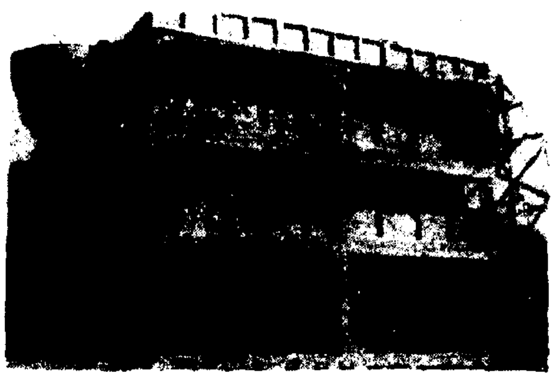


Fig.10 - VIEW OF THE MODEL OF THE SHAKING TABLE

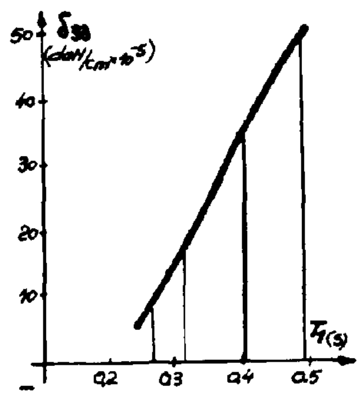


Fig.11 - FLEXIBILITY COEFFICIENT δ_{33} VERSUS FUNDAMENTAL PERIOD

Table 2 - Maximum seismic response of the three story model

Table 2 - Maximum seismic response of the three story model

Behaviour stage	Test No	Maximum accelerations (m/s²)				Max. displacements (mm)			Fundam. period (sec)			a_3/a_0	a_3/a_0
		a_0	a_1	a_2	a_3	Level 1	Level 2	Level 3	T_1	T_2	T_3		
Quasielastic	I	0.54	0.84	2.30	1.95	3.18	5.12	7.66	0.263	0.124	0.084	4.25	3.60
	II	0.79	1.08	0.96	–	2.22	3.61	4.88				1.21	–
Cracking	III	1.31	2.17	2.06	2.79	6.36	7.72	11.62	0.346	0.167	0.110	1.57	2.13
	IV	1.61	2.63	1.79	2.93	4.61	10.84	12.78				1.11	1.82
Crack development and damage	V	1.84	3.56	3.85	7.11	14.95	28.09	41.12				2.09	3.86
	VI	1.21	2.75	2.75	3.65	16.70	35.52	46.46	0.457	0.230	0.144	2.27	3.02
	VII	2.29	5.39	3.46	5.11	36.04	48.16	55.75	0.516	0.266	0.161	1.48	2.23
Failure	VIII	2.09	3.25	3.38	6.86	36.57	59.60	66.21				1.62	3.28
	IX	2.76	2.79	2.39	5.44	31.76	72.25	111.00	0.616	0.298	0.196	0.86	1.97

ration did not exceed 2 3 m/s^2 its amplification on the height was appreciable thus loading to stresses corresponding appro - ximately to a seismic excitation of intensity VIII on the MSK Scale.

4. Failure stage was reached during the final run of seismic test IX and was characterized by severe spalling and crushing of concrete and by a significant deterioration of the lateral stiffness of the structure.

Maximum acceleration at the top level was 5.44 m/s^2 while interstory drift was 1/50 - 1/40 from the story height.

Figure 11 shows the variation of flexibility coefficient δ_{33} with the fundamental period of the structure. The analy - tical investigation carried out on the model structure by means of a computer program able to take into account the three di - mensional interaction of component members under various loa - ding distributions of forces as resulted from the tests empha - sized a relatively close correlation between experimental and calculated values of stresses, more pronounced differences be- ing observed as regards the structure flexibility. This is due to difficulties encountered when introducing the progressive de- gradation of the members into the computation process.

Both the experimental and analytical studies enabled a better understanding of the elastic and postelastic behavior of the test structure and some recommendations have been made to improve the overall performance of the building. These re- commendations reffered mainly to the variation of column cross sections, reinforcement ratios from one story to the other and to some modifications as regards the precast beams and their connections with columns at floor slab (4).

SEISMIC TEST OF A FOUR STORY MODEL STRUCTURE

The prototype structure is characterized by a large span (18 m) in transversal direction and bays of 3 m in longitudinal direction. The free story height is of 3.2 m. The structure in- cludes both reinforced concrete members and prestressed mem - bers. The columns are nonprestressed precast members provided with cantilevers in longitudinal direction connected to form longitudinal beams. The roof and current floor slab consist essentially of prestressed elements of T shape.

The model was designed and constructed at a length scale of 1/2.5 and comprised 6 bays of 1.2 m in longitudinal direc- tion and a span of 7.2 m in transversal direction. The main geometric dimensions of the model are shown in figs. 12 and 13.

Before applying the seismic actuation the lateral deformation of the structure under relatively low static forces applied horizontally at the floor levels, was similar to that of a framed structure.

After the development of the first cracks, the lateral deformations increased about 1.25 times and the degree of rigity between girder and column decreased about 20%. The first cracks developed in the floor members in the vicinity of joints. Under increasing intensities of base motions these crackings extended both in columns and connection zones between longitudinal beams and columns.

A selection of the maximum floor acceleration and displacement during the excitation program is presented in Table 3. The analysis of experimental data indicated that first level diaphragm maintained within the elastic range of behavior up to the seismic test No.5, undergoing translational and rotational deformation of relatively low values. During the subsequent seismic excitations the overall deformation of the floor slab, apart from translational and rotational deformations presented flexural deformations and indicated significant inelastic excursions. On the other hand the test emphasized the necessity of improving the joint between column and L shaped beam as well as the increasing of floor unit shear reinforcement.

SEISMIC TEST OF A FIVE STORY STRUCTURE WITH CENTRAL CORE

The overall plan dimensions of an actual structure of this type are 36 x 36 m. The experimental investigation was carried out on a structural model reproducing only a fragment of the actual structure as may be seen in fig.14 . The length scale of the model was chosen 1/4.5. The main objective of the tests was to obtain experimental data on the overall seismic response, behavior of joining zones and floor slabs. A special attention was paid to the behavior of the central core and to the interaction of the peripheral frames and core.

The static tests included two phases. The first phase comprised the study of core adequacy and the active zones have been determined considering separate cross-sections(shaped as L, T or +) subjected to bending and shear. The second phase included the checking of those cross-sections in the assemble of the structural model.

The test of the model on the shaking table followed a standard procedure used in the previous tests described above.The principal experimental data obtained during the seismic tests were story accelerations and displacements, the angular deformations between girder and core and the sliding displacements between core and floor unit.

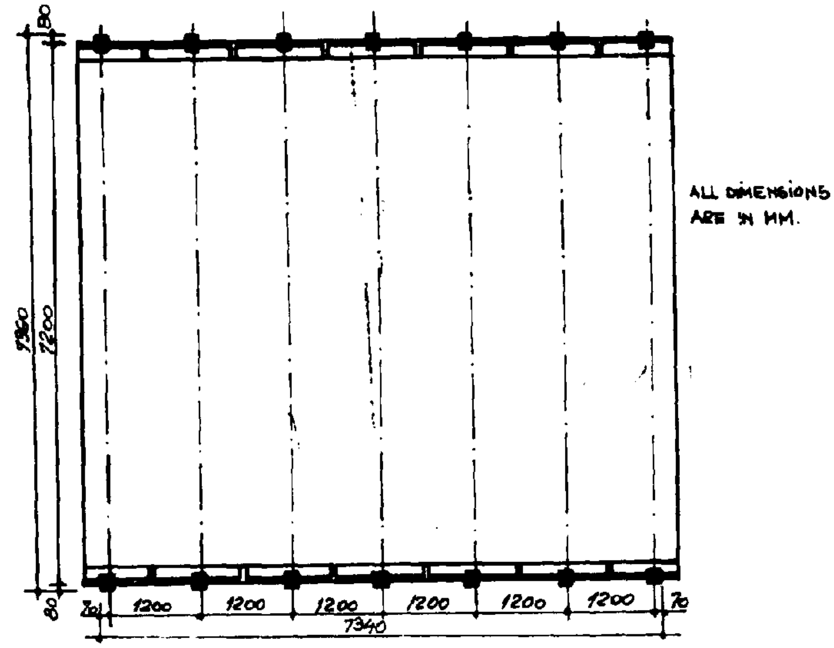


Fig.12 - FULLY PRECAST FOUR STORY STRUCTURED MODEL
PLAN SECTION

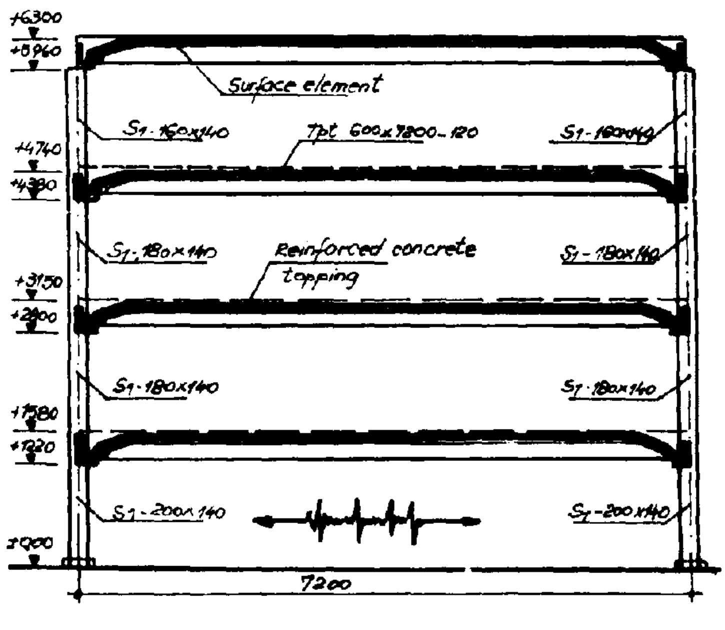


Fig. 13 - TRANSVERSAL SECTION

Table 3 - Maximum seismic response of the four story model (nonsyncronized values)

Test		Maximum accelerations (m/s²)					Maximum displacements (mm)				Foundam. period		
No	caracteristics	a_0	a_1	a_2	a_3	a_4	d_1	d_2	d_3	d_4	T Transv (sec)	T Long (sec)	T Tors. (sec)
1	D2 - 10 sec	1.20	1.19	1.18	1.19	2.07	1.15	4.76	5.82	4.83	0.283	0.328	0.325
2	D2 - 5 sec	1.88	1.25	1.60	1.38	2.29	2.39	7.64	9.35	10.68			
3	D2 - 5 sec	1.38	1.07	1.22	1.23	2.48	2.08	5.67	7.09	8.21			
4	O2 - 5 sec	1.73	1.10	1.30	1.22	2.29	–	--	—	—			
5	O2 - 5 sec	4.84	1.07	1.41	1.29	2.48	3.69	8.66	10.46	12.22			
6	B1 - 15 sec	4.94	2.07	2.25	1.81	2.59	8.75	17.45	21.50	24.11	0.307	0.347	0.347
7	B1 - 15 sec	1.31	1.13	1.32	0.62	1.33	2.75	5.10	5.81	9.37			
8	B1 - 15 sec	1.16	1.51	1.51	1.56	1.94	6.13	11.47	13.25	16.24			
9	B1 - 12 sec	2.26	1.88	1.60	1.64	2.85	8.00	14.27	16.50	18.75			
10	B1 - 7 sec	3.02	2.02	2.82	2.05	4.19	8.75	16.05	19.52	21.59			
11	A2 - 19 sec	1.41	1.78	2.11	1.93	2.41	8.75	17.32	19.75	22.59			
12	A2 - 13 sec	1.91	1.55	2.49	2.01	3.17	8.75	17.32	19.98	18.41			
13	A2 - 19 sec	4.03	3.96	2.97	2.67	4.45	14.38	27.01	29.98	33.48	0.329	0.370	0.368
14	IS - 13 sec	4.23	3.16	3.53	2.92	4.06	18.13	31.85	37.42	45.20			
15	IS - 13 sec	2.62	2.59	3.76	3.16	3.81	15.65	32.36	39.74	97.09			
16	4-III/77 - 21 sec	3.32	2.35	2.96	2.87	5.08	13.39	29.56	35.79	80.77			
17	IS - 19 sec	4.23	3.06	4.38	4.31	5.08	20.28	42.68	52.28	58.59	0.340	0.373	0.378

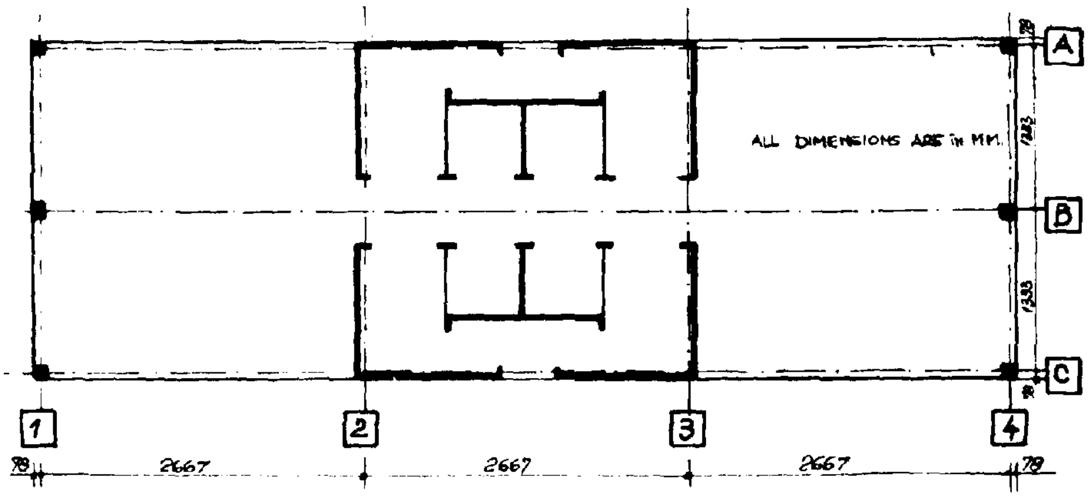


Fig. 14 - MODEL OF A CENTRAL CORE STRUCTURE - PLAN SECTION

The cracking process started in the core and extended in the peripheral frames.

The test showed that the structure as a whole and the core in particular is correctly designed from the aseismic point of view. However, on the basis of the results obtained some weak points have been identified and some recommendation were made to improve the structure performance under severe seismic excitations. Some of those recommendations referred to the connection

The experimental investigation showed that the strength and the response are very dependent on the core characteristics and the details between peripheral frames and core.

The validity of most design concept was proved through the performed tests. It should be noted that buildings of si - milar configurations withstood the Romanian earthquake of March 4, 1977 with minimum damage.

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IV.5 ASPECTS OF DYNAMIC INTERACTION BETWEEN STRUCTURAL AND NONSTRUCTURAL MEMBERS

Diaconu Daniel[x]
Cosmulescu Paul[x]

ABSTRACT

This paper presents the results of theoretical and experimental investigations carried out to establish the role of interaction between structural frame and infilling masonry walls to the strength and stability of building structures during earthquakes.

The interaction mechanism and aspects in connection with qualitative importance of interaction in case of strong horizontal loads are evidenced. The stiffness variation with structural degradation is also pointed out.

The comparative global seismic response of a building without infilling masonry is presented together with that of a building with infilling masonry. Also, a comparison is made between experimental data and theoretical results.

The experimental dynamic studies were carried out on a 1/5 scale models tested by means of a seismic shaking table of 140 tons capacity.

1. INTRODUCTION

The study on the interaction between resistent structure and infill panels under lateral actions, especially sesimic ones, as well as the analysis of soil structure interaction are a relatively new field of investigation. Theoretical formulations in the literature are generally valid for elastic range, i.e. within a limited domain of frame - masonry wall interaction, in case of monotonous increase or alternating loads. Nonstructural elements, although playing an unimportant role in the absence of seismic motions, could become structurally effective in case of strong horizontal loadings. The optimum distribution of infill panels both in plane and along structure height, may

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lead to substantial increase in the building strength and rigidity, while an unfavourable panel distribution may lead to contrary effects.

2. RESULTS OF EXPERIMENTAL TESTS

Experimental tests were carried out on a 1/5 scale models (Figs.1 and 2) representing r.c.structures with eight levels, with and without infill panels.

Through the dynamic tests of seismic type the principal parameters describing the overall elastic and especially post - elastic behavior of r.c.frames were determined as follows:

- the degradation of structural strength;
- the degradation of rigidity;
- variation of dynamic characteristics;
- the energy dissipation;
- failure mechanism etc,

Experimental dynamic programs were carried out with increasing intensities, so that the tested structures followed all degradation stages to the ultimate strength. Both models were loaded so that stress scale was 1/1.

Experimental programms at base action level are presented in Tables I and II. In Table III dynamic characteristic changes for all degradation stages, for both structures are presented. It can be seen from variation mode of dynamic characteristics (Figs. 3 and 4) that damping increases with the decrease of frequencies. The response recorded during seismic actioning programm revealed that for infill panel structure (SwP), various degradation stages occured at higher levels of base acceleration a_o, as compared with the structure without masonry panels. (SwoP).

The decrease in oscillation period was more accentuated for SwP as compared with SwoP (Table III and Figs.3 and 4).

The lateral displacements due to static horizontal forces of 2 KN applied succesively at 1st, 4th and 9th levels, exhibited smaller values for SwP than for SwoP(Fig.5), although base acceleration levels necessary for reaching every degradation stage were greater in case of SwP.

In case of dynamic excitation, the instantaneous story accelerations and displacements have also smaller values for Swp in comparison with SwoP (Figs. 6 and 7). The distribution manner of accelerations (Fig.6) shows that amplification ratios a_4/a_o and a_9/a_o for SwoP were greater for all degradation stages, i.e. the structural rigidity was smaller than SwP. The role of infill panels became important in limiting the lateral displacements especially in postelastic range (Fig.7) for the entire building height.

The failure mode of the r.c.structure was similar for both models, but occured at different energetic levels ($a_o^{failure}$ = 4.25 m/s^2 for SwoP, $a_o^{failure}$ = 5.3 m/s^2 for SwP).

Initially, some cracks occured in beams at their ends from 2nd to 4th levels, and then extended along the structure height and accentuated during the increasing levels of actioning program . Towards the final part of the testing programme cracks developed on column extremity sections begining from lower levels 3 and 4 (a_o = 3,27 m/s^2 for SwoP and a_o =4.12m/ s^2 for SwP). In case of SwP, after a period of full interaction within elastic range up to base acceleration, a_o = 1.6 m/s^2, the masonry panels separated of the contours. Then both horizontal and diagonal cracks developed and finally lateral panel displacements occured. The 4th and 6th floor panels collapsed at a base acceleration of 5.3 m/s^2.

COMPARING THE EXPERIMENTAL AND ANALYTICAL RESULTS.

. By comparing the experimental data and analytical results it may be observed that lateral displacements for all the degradation stages are 2 - 3.5 times smaller in case of analytical calculations.

. Theoretical story shear forces are 2 - 3 times smaller than experimental ones.

. The calculated fundamental periods of structures with and without infill panels are by 5 - 7% smaller than experimental results. Experimental SwP has 21 - 33% smaller fundamental periods than SwoP (elastic and microcracking stages).In case of analytical calculation these decrease is of 17- 25%.

. From theoretical analysis of masonry panel the following aspects are emphasized:

- panel failure occurs due to horizontal shear stress at all levels excepting the 8th level;
- diagonal tension produces the failure of masonry panel from 1st to 6th level;
- the crushing of masonry corner does not occur.

The experimental data regarding the masonry behavior and failure, generally confirmed the analytical results.

4. CONCLUSIONS

. Masonry panels modify the structure seismic sesponse with respect to the stiffness degradation, variation of dynamic characteristics, the energy absorbtion level and features of structure failure.

. In the initial phase, the infill panels have larger rigidities than the surrounding frames, and degradation ratios greater than the resisting r.c. frames.

. Experimental data confirmed the analytical models used for infill panels.

. Initiation and development of structural damage is more rapid for SwoP than for Swp.

. The separation of infill panels from surrounding r.c. frames occurs at relatively small base acceleration (a_o = 1.6 m/s^2), for the whole height of the building.

. In case of Swp, due to additional rigidity of infill panels, the repartition coefficients of story shear forces relatively to axes of infill panels are 2 - 3 times larges than the coefficients corresponding to axes without infill panels.

. Due to the presence of masonry, the static and dynamic lateral displacements diminished as follows:

- static actions, elastic stage (Fig.5)

SwP/SwoP = 0.18/0.27 = 0.67

- dynamic actions, failure (Fig.7)

SwP/SwoP = 77.29/86.85 = 0.89

. In the postelastic range, due to embedding of masonry at the frame corners, checking of beams and columns are required to concentrated forces developed through interaction.

. In the design analysis, the determination of story shear distribution is required both along the structure height and along axes, taking into account the large differences in stiffness among various damage stages. As a result of separation between infill panel and surrounding frame associated with cracking under relatively low base accelerations, a substantial change in rigidity distribution along the height and in a horizontal plane is produced.

Table I

Structure Without Infill Panels.

Test nr.	a_0 m/s^2	a_4 m/s^2	a_9 m/s^2	d_4 mm	d_9 mm
1 Elastic	0,783	1,20	1,717	2,73	8,21
3 Microcra-cking	2,57	4,15	5,19	7,25	11,90
6 Cracking	2,98	4,62	5,82	40,6	61,1
8 Development of cracks	3,27	5,38	6,36	49,7	72,5
11 Failure	4,25	6,87	7,12	59,3	86,85

Table II

Structure With Infill Panels

Test nr.	a_0 m/s^2	a_4 m/s^2	a_9 m/s^2	d_4 mm	d_9 mm
1 Elastic	0,76	1,20	1,60	1,63	7,01
3 Microcracking	3,50	4,12	4,93	6,78	9,88
6 Cracking	3,63	4,39	5,07	31,13	67,5
7 Development of cracks	4,12	4,61	5,56	40,21	67,6
10 Failure	5,30	6,11	6,74	44,72	77,29

Table III

Dynamic Characteristics Change

Stage	Direction	Period		Damping	
		sw.P	swP	sw.P	swP
Elastic	Transv.	0,297	0,200	2,10	1,20
	Longit	0,297	0,211	1,80	1,40
	Tors	0,296	0,200	1,88	1,10
Microcracking	Transv.	0,328	0,222	2,40	2,10
	Longit.	0,308	0,244	1,99	1,80
	Tors.	0,328	0,222	1,96	1,90
Cracking	Transv.	0,302	0,250	2,90	3,55
	Longit	0,319	0,265	2,50	2,54
	Tors.	0.357	0,250	3,00	3,15
Development of cracks	Transv.	0,430	0,333	3,40	3,80
	Longit.	0,369	0,294	3,50	3,85
	Tors.	0,421	0,286	3,50	3,75
Failure	Transv.	0,433	0,341	3,93	4,85
	Longit.	0,350	0,300	3,60	4,15
	Tors.	0,442	0,367	3,55	4,27

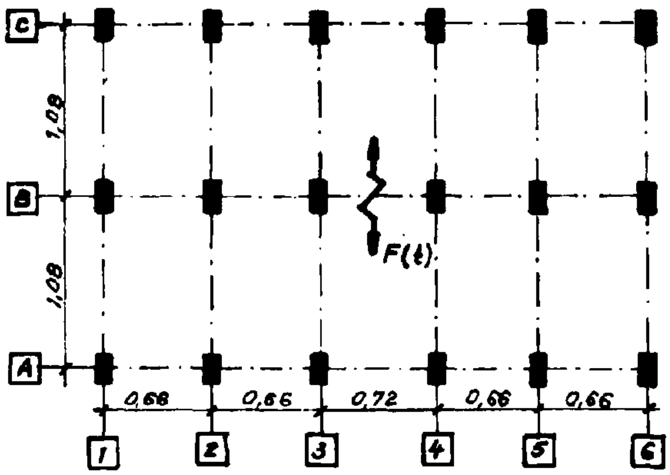


Fig. 1

Transverse Section Structure Without Infill Panels

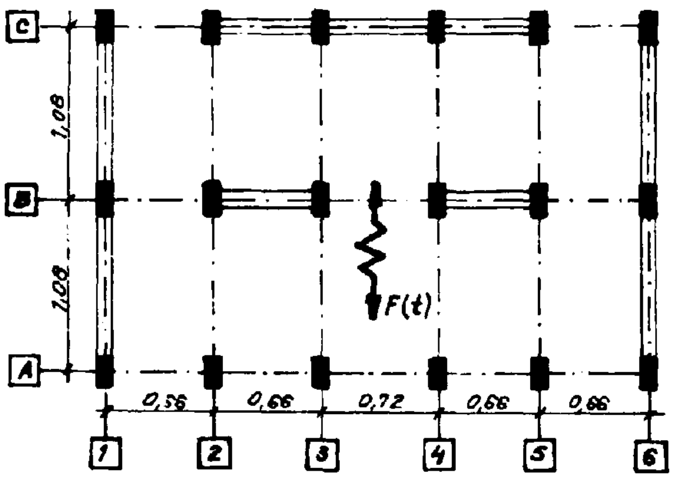


Fig. 2a

Transverse Section Disposition of Infill Panels at Ground Level

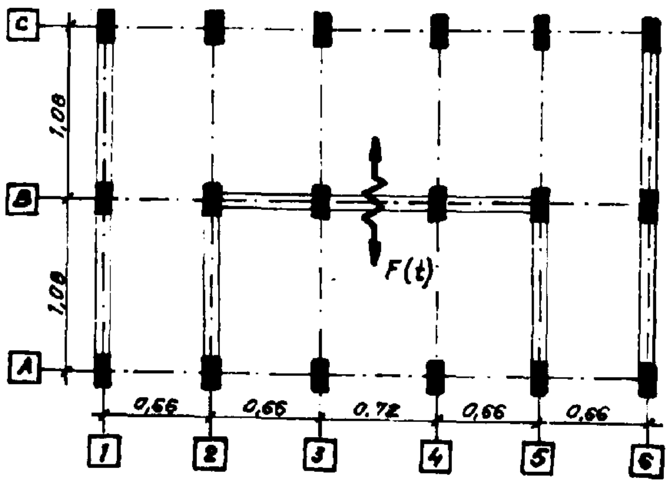


Fig. 2b

Transverse Section Disposition of Infill Panels at Current Level

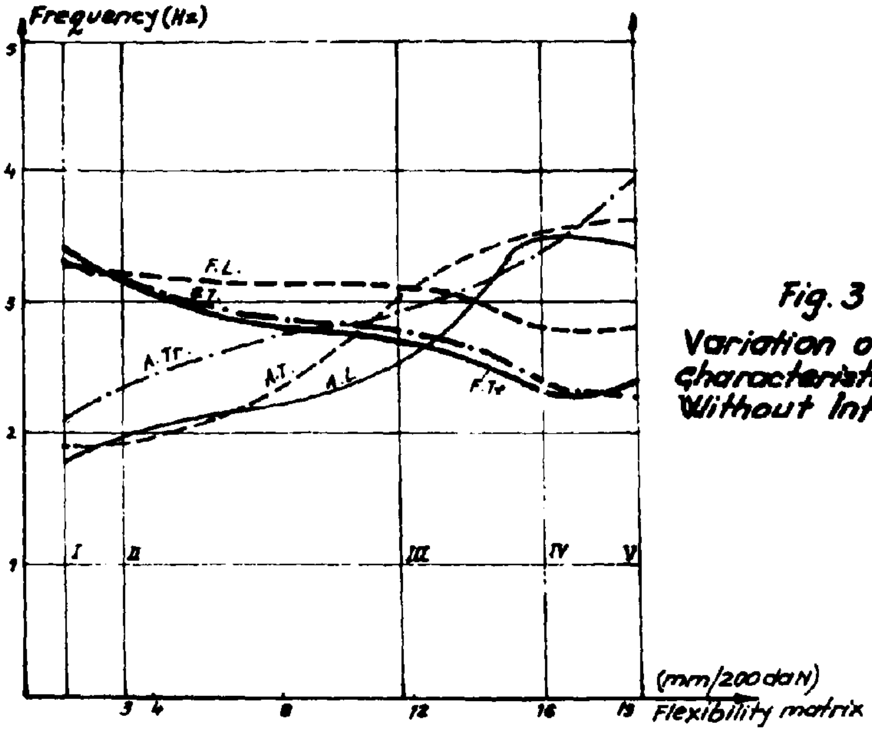


Fig. 3
Variation of Dynamic Characteristics. Structure Without Infill Panels

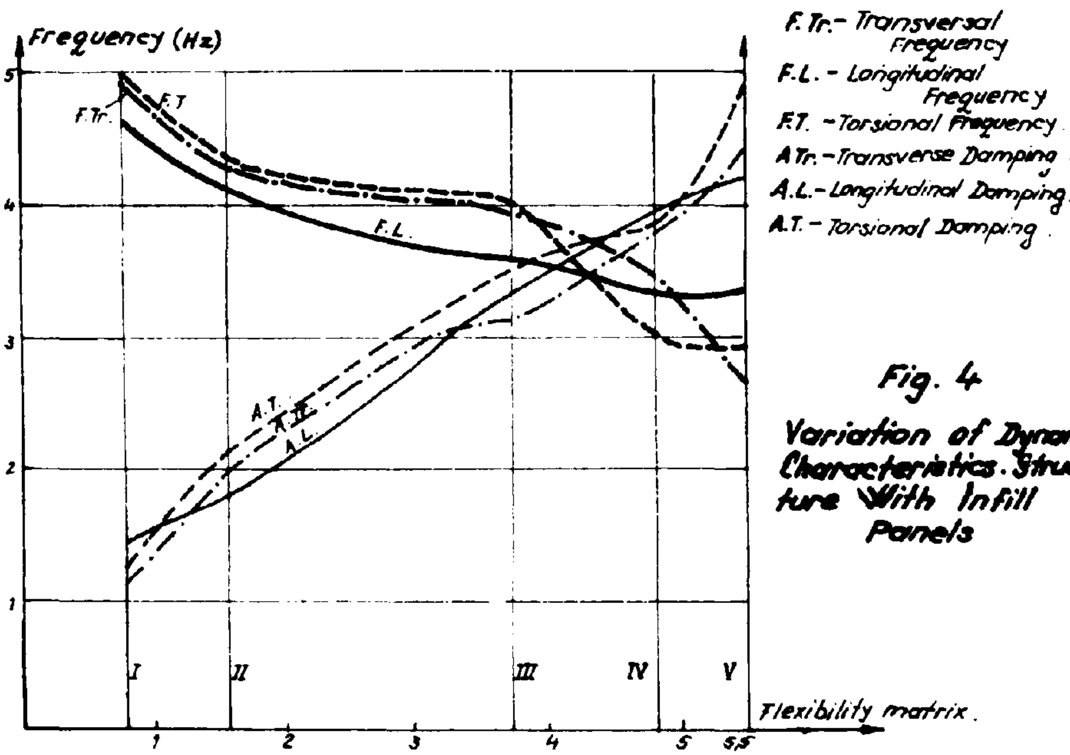


Fig. 4

Variation of Dynamic Characteristics. Structure With Infill Panels

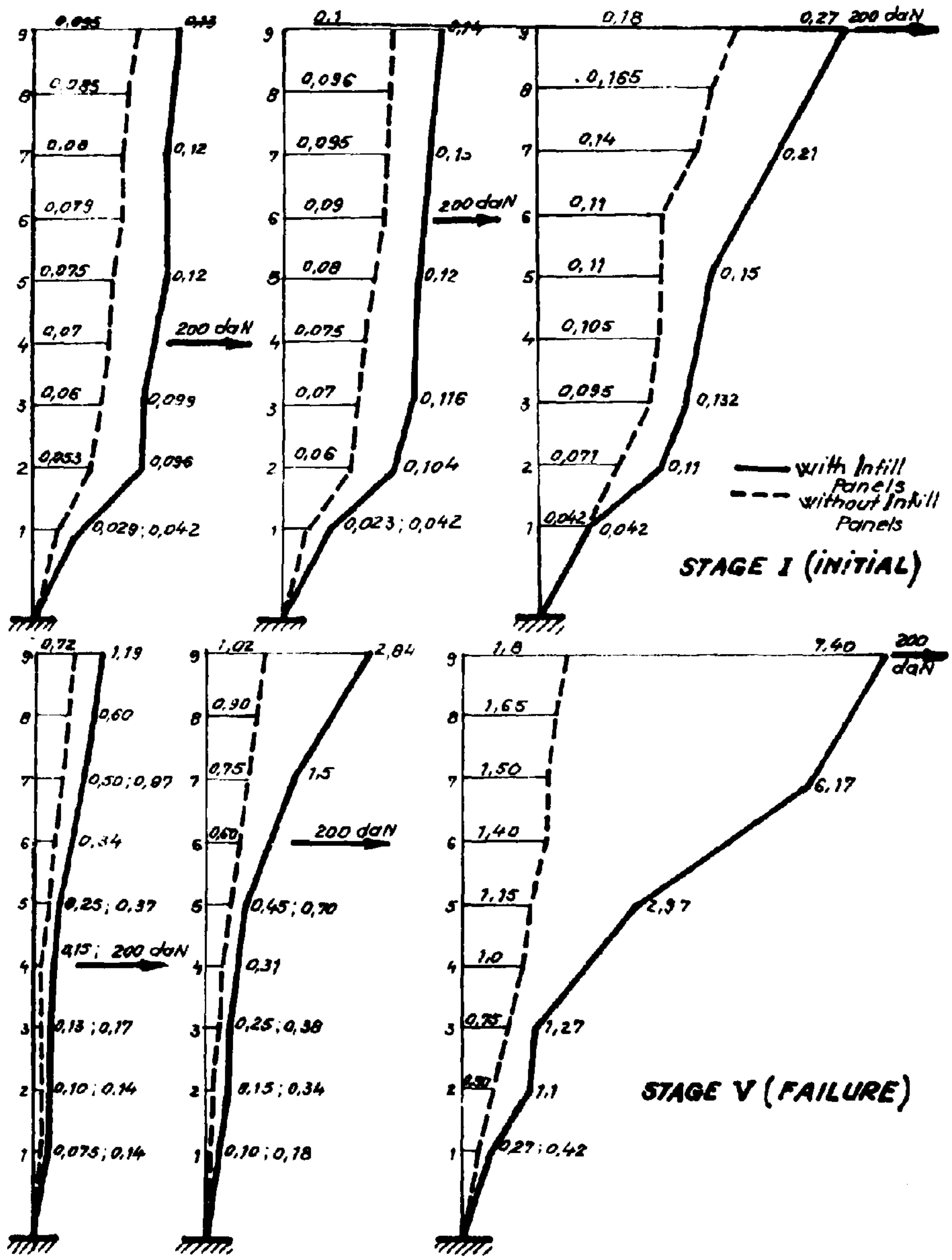


Fig. 5 - Static Deformed Shapes. (mm)

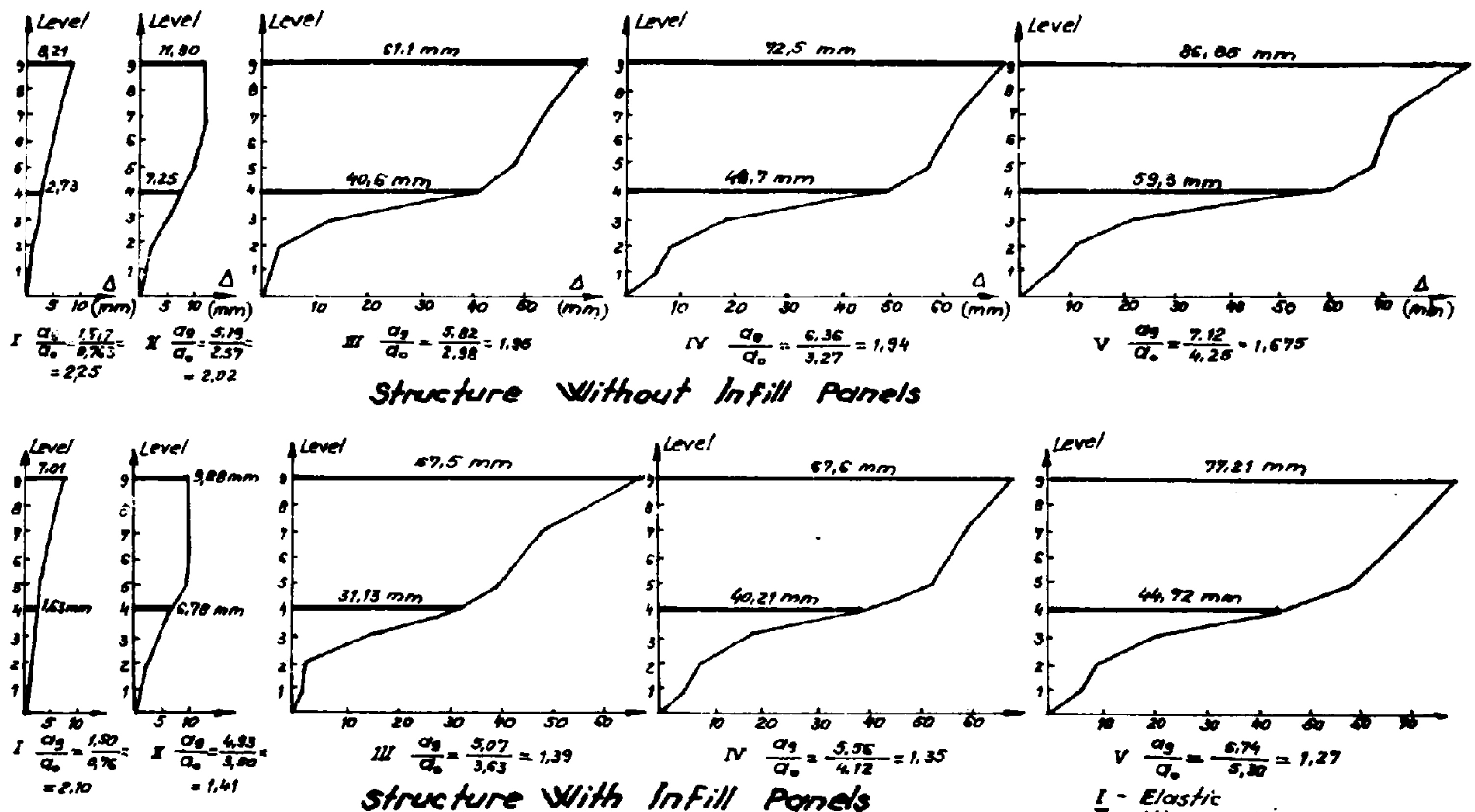
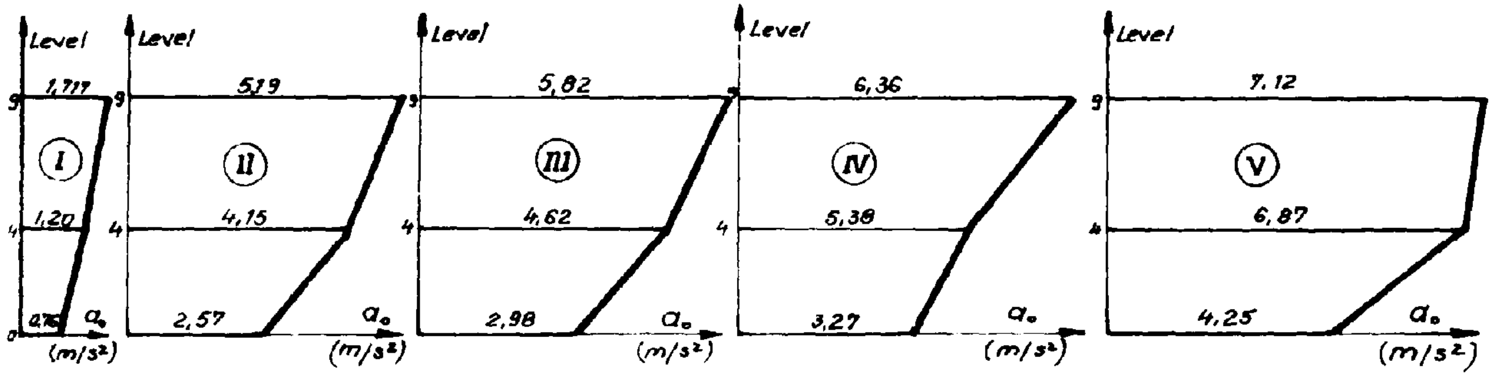
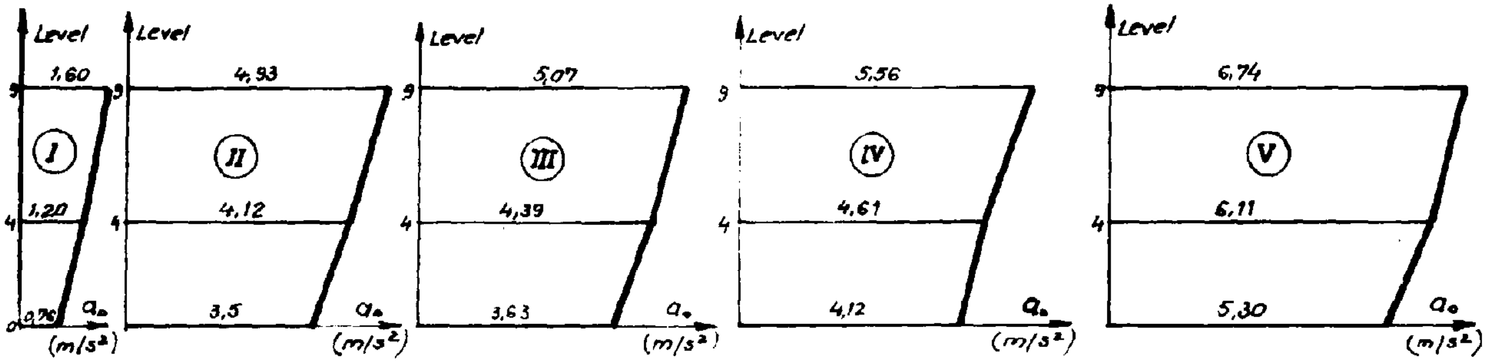


Fig. 6 - Instantaneous Dynamic Displacements.

I - Elastic
II - Microcracking
III - Cracking
IV - Development of cracks
V - Failure



Structure Without Infill Panels



Structure With Infill Panels

Fig. 7 - Acceleration Distribution.

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IV.6 PECULIARITIES IN THE SEISMIC BEHAVIOR OF VARIOUS STRUCTURAL SYSTEMS MADE OF ENTIRELY PRECAST LARGE PANELS

D.Diaconu x)
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St.Marinescu x)

The results of research works carried out on different types of large panel structures with 5 to 11 stories (honeycomb, cellular or vertical core systems) are presented in this paper. The experimental tests were performed using shaking tables of medium or large capacities.

Various behavior characteristics of large panel buildings subjected to the March 4, 1977 earthquake are presented. The seismic adequacy criteria of earthquake resistant large panel structures with flexible functional layout are also emphasized.

The large panel buildings usually are complexes of wall and floor precast units forming box-type structures. Unit arrangement is one of the most important aspects of large panel structures. In the case of buildings in seismic areas, at least two resistance lines corresponding to each plane direction are considered. Structural integrity should be granted by proper horizontal and vertical ties between the units.

Seismic damages generally occurred in the joining zones, while the large panels were less damaged. Several entirely precast large panel systems are used in Romania for ground floor + 4 stories - ground floor + 9 stories buildings. The March 4, 1977 earthquake proved that the aseismically designed large panel structures were subjected to rather unimportant damages (1, 4). The satisfactory behavior of such structures is mainly granted by the fact that design observes the aseismic prescriptions, especially as concerns joints. Besides, the tests performed on several large panel joining systems, full scale structure parts and low-scale reinforced concrete models seriously improved the building systems under use, Considering the advantages in terms of shorter building time, precast large panel structures are extensively used in the seismic areas of Romania; may we also mention a research program including new tests performed on shaking tables.

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This paper introduces several characteristics of the seismic behavior of large panel building systems, based on tests performed on the existing stock, joints and low-scale models.

TESTS ON THE DYNAMIC CHARACTERISTICS OF LARGE PANEL BUILDINGS.

The tests on existing structures were performed in order to determine the oscillation eigen periods as well as the critical damping rate and provided the following conclusions:

- The fundamental periods of ground floor + 4 stories are lower than 0.28 s longitudinally and lower than 0.30 s transversely. The critical damping rates are between 9.0% and 10.7 %. The periods of mode 2 are of about 2.25 times lower than the fundamental periods and the average torsion periods are of ab. 0.25 s;
- For ground floor + 7 stories the fundamental periods are lower than 0.38 s longitudinally and lower than 0.40 s transversely;
- The foundation soil basically determines the fundamental period (up to 20 - 30%);
- The ground floor stiffness characteristics in the case of commercial ground floor buildings has a serious influence on the oscillation periods as compared to similar story structures.

TESTS ON THE BEHAVIOR OF SEVERAL TYPES OF JOINTS UNDER STATIC AND DYNAMIC LOADING.

Many types of panel joints may be used in building, including wet joints with various reinforcing solutions, but literature mentions only a few tests on such joints under loading simulating earthquakes. The tests performed by the specialists at ICCPDC - Iaşi Branch covered both static tests under alternating cyclic loadings and dynamic tests under seismic loadings. The variable parameters that were considered are: a) the loading type; b) the aggregate type (sand and gravel, lightweight aggregate); c) the geometry of key joints; d) the amount and detail of reinforcement in joining areas. For the study of the structural behavior of vertical and horizontal joints, full - scale units or large-scale models were used (usually 1/2).

The tests allowed the comparison between the response of the units tested under static and dynamic loading as well as the improvement of joining details of real structures, starting from the resistance and ductility characteristics of various types of joints. Parts of full-scale structures were also tested in order to obtain data on the comparative behavior of several types of joints in box-type structures.

SEISMIC BEHAVIOR OF GROUND FLOOR + 4 STORIES

STRUCTURES WITH NON-BEARING FAÇADE WALLS

Tests were performed on the aseismic behavior of a new structural system where the resistance function was separated from the insulating one.

The experimental stage included the testing under seismic loading of a 1/5 length scale model (see fig.1). This structure has one longitudinal shear wall; non-structural walls and boundary columns belonging to the cross shear walls exist on the sides adjoining the façade. The structural model included the main characteristics of an actual ground floor + 4 storeys model (typified) and was tested using an average shaking table (about 15 tons).

As difficulties occurred in connection with the additional weight on the floor, the weight values corresponding to each storey were lower than those required to generate axial stress levels in the model as well as in the prototype. Nevertheless, this disadvantage was surpassed by increasing the earthquake input motion /2/. The resulting acceleration scale was ab.1/2.2. The equipment used in the seismic response recording included sensors for measuring the relative displacement and the absolute acceleration parallel to the longitudinal axis of the structure. These sensors were fixed on a stiff steel frame mounted on the movable platform. This model structure had an essentially elastic behavior up to 1.85 m/s^2 fundamental acceleration values; no cracks occurred up to this loading level in the compounds.

The first cracks occurred and developed in the central shear wall under base excitation of 1.9 - 2.2 m/s^2. Failure occurred at maximum 3.66 m/s^2 fundamental acceleration values, namely the sliding of the shear walls under the 1st story floor. Under further seismic loading, up to 9.1 m/s^2, this effect was stronger and the cracks in the cross shear walls extended. The behavior of horizontal and vertical joints proved satisfactory.

The variation of acceleration on structure height provided various responses for the two types of vertical units (shear walls and peripheral frames./ At the beginning, the central shear wall takes over a great part/ 83%) of the overall seismic load, but as cracks develop, the base shear force values corresponding to the two peripheral frames increase again and ultimately each frame takes over ab.38% and the shear wall 24% differential deformations on the two extremities as against the longitudinal shear wall. The floor stiffness influences not only the seismic action but also the damages occurring in the cross shear walls as well as the seismic design of such structures.

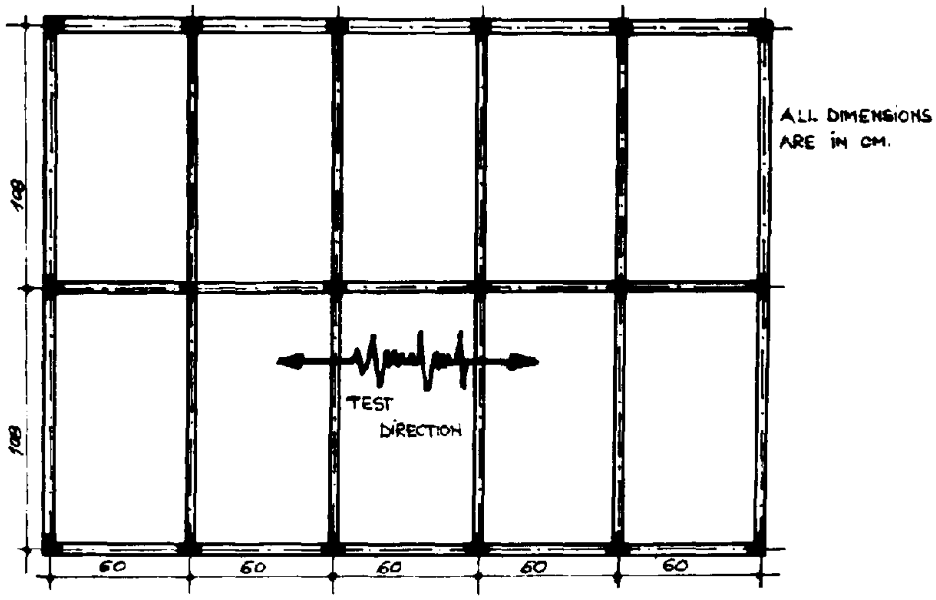

Fig. 1-FLOOR PLAN OF A FIVE STORY STRUCTURAL MODEL WITH NONBEARING FACADE WALLS.

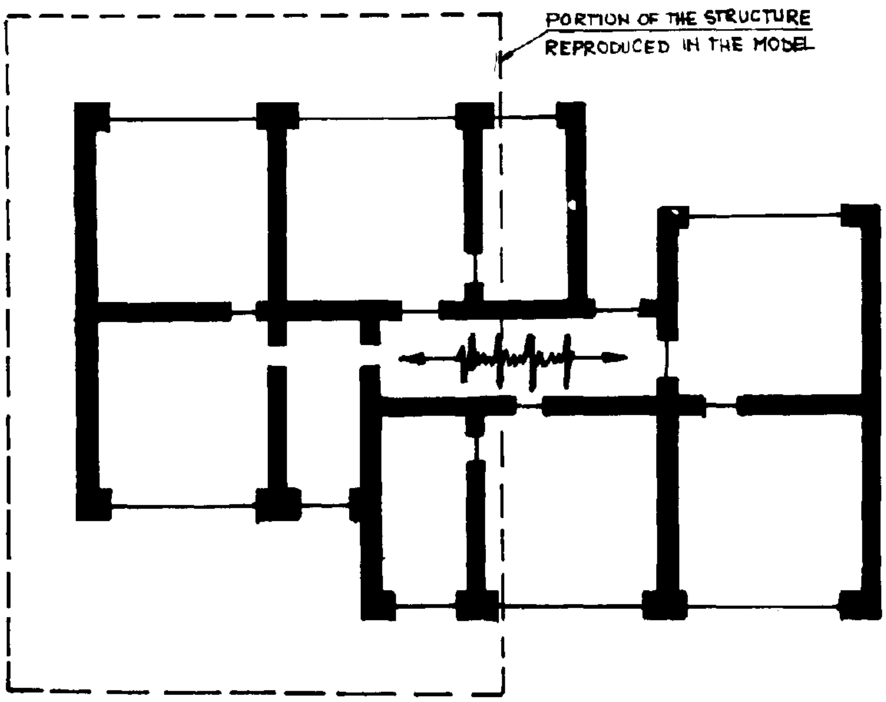

Fig. 2-FLOOR PLAN OF A FIVE STORY CELLULAR STRUCTURE.

FURTHER STUDIES ON THE SEISMIC BEHAVIOR OF GROUND FLOOR + 4 STORY LARGE PANEL BUILDINGS.

A major concern in improving the plan layout of large panel buildings is the increase of the architectural flexibility of spaces. In this respect cellular systems are preferred to honeycomb structures. The seismic adequacy of three types of cellular systems was studied experimentally on 1/5 scaled models, tested under similar conditions as the above described structure. Figure 2 shows schematically one of these structures in which a cell has the dimensions of 6 m x 6 m. The model had five stories and reproduced only a portion of the whole plan of the structure. All components were precast and were provided with contour denture.

The keyed joints contained projecting loop bars as well as some welded connections. The reinforcement bars used in the model had diameters of 1 - 4 mm. Each floor slab was loaded with additional weights of 5500 N except the roof slab where the added weight was 3600 N. The other two models had different plan layouts but essentially the same mass distribution and similar joint types. The tests revealed the quasielastic stage of structural behavior under base motions with intensities of 1.8 - 2.8 m/s^2. The starting and development of cracks as well as some damages in locations of stress concentrations occurred in the subsequent test up to maximum accelerations of 5.8 m/s^2. In this stage the maximum relative displacement measured in horizontal joints was 0.52 mm. Cracks were also noticed in column sections. The variation of seismic force distribution in various structural vertical members depends on the degradation stage of lateral stiffness and on the instantaneous dynamic deformation. The failure of the tested models was characterized by ruptures in horizontal joints at the bottom stories and significant cracks in perimeter columns and piers under base excitations of intensities up to 6.3 m/s^2. As regards the behavior of floor slabs, the following remarks can be made: Under strong seismic excitations the floor slab behaves as a shear wall of finite rigidity resting on vertical members of various rigidities. The changes in these rigidities during seismic action results in a significant redistribution of horizontal forces among vertical members. All models reproduced the condition of prototypes where no topping was provided at floor slabs.

SEISMIC BEHAVIOR OF NINE AND TEN STORY PANEL BUILDINGS.

A typical floor plan of a nine story large panel building is shown in Fig.3. The vertical and horizontal load - carrying system consists of simple and double cores interconnected by shear walls at each level. The joints are of key type and incorporate some particularities as regards their reinforcement. The problems of joint design were presented before /1/ and are not to be considered in this paper. A 1/4 scale model that reproduces a fragment of the structure consisting of two cores of E1 type (Fig. 6) was designed and is to be tested by means of an earthquake simulator.

Another panel system consisting of complexes of boxshaped cores and floor slabs is shown in Fig.4. The investigations carried out on this system included analytical studies on elastic and nonelastic seismic response, static loading tests on joints and seismic loading test on a reinforced concrete scale model /3/ The model had nine stories and represented approximately a 1/4 scale core within an actual building. The plan dimensions of the model structure are given in Fig.7. Nonstructural masses used to increase the inertia force during the tests (725 kg at a current level) were lower than those which would have been required for a full-scale model of the core. Therefore, large intensities of base excitation were necessary to induce nonelastic deformations and failure of the test structure. The input excitations were the artificial earthquakes D2 an B2 generated by Jenning and Housner and an input, IS, incorporating features of some local earthquake records. The model was subjected to base motions of successively increasing accelerations ranging from 0.2 m/s^2 to 6.5 m/s^2. Remarks on the seismic response were derived from relative displacement and absolute acceleration waveforms as well as from strain gage records. The visual observations of damage occurred within the component panels indicated that failure was caused by: (1) excessive degradation of horizontal joints and sliding along vertical joints especially at the first two levels; (2) concrete crushing in piers at the first story; (3) significant cracks in the walls parallel to the test direction at the bottom levels; (4) rupture of vertical reinforcement at corner joints in sections located under the first story floor. The results of this test indicated that with a careful detailing, particularly in the joint areas, large panel cores can be used as an effective earthquake resistant system. It should be noted that symmetrical ensembles of such cores within an actual structure would respond in a better manner than an isolated core.

Analytical studies of seismic response of the test structure, considering the stiffness and strength degradation in the more stressed lower stories showed adequate correlation with the experimental results. A complete model of a ten story structure whose floor plan is shown in Fig.5 was constructed and tested by means of the same shaking table. As in the previous case the length scale was chosen to be 1/4. The plan and elevation of the test model may be observed in Fig.8. The test revealed that as soon as the structure entered the nonelastic range, the lateral deformations increased 2.4 times as compared to maximum elastic deformations and the component cores were subjected to differentiated stiffness degradations. In addition, out of phase oscillations of the cores were observed and the influence of the second mode in the total response increased. A gradual cracking of the panels was initiated from the corner zones and the connecting floor slabs transformed into link members hinged at their ends.

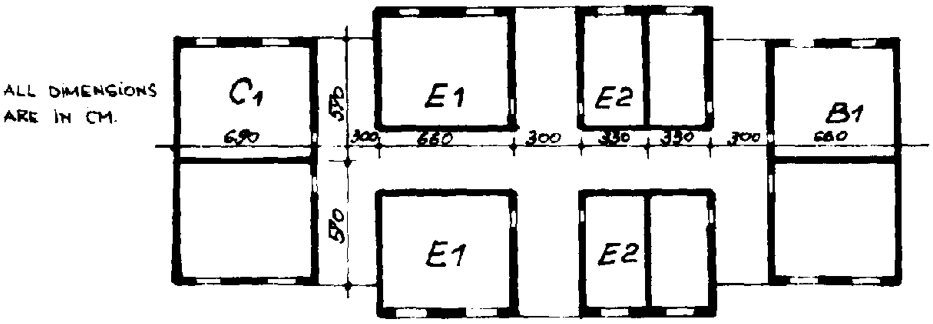

Fig. 3- TYPICAL FLOOR PLAN OF NINE STORY LARGE PANEL STRUCTURE.

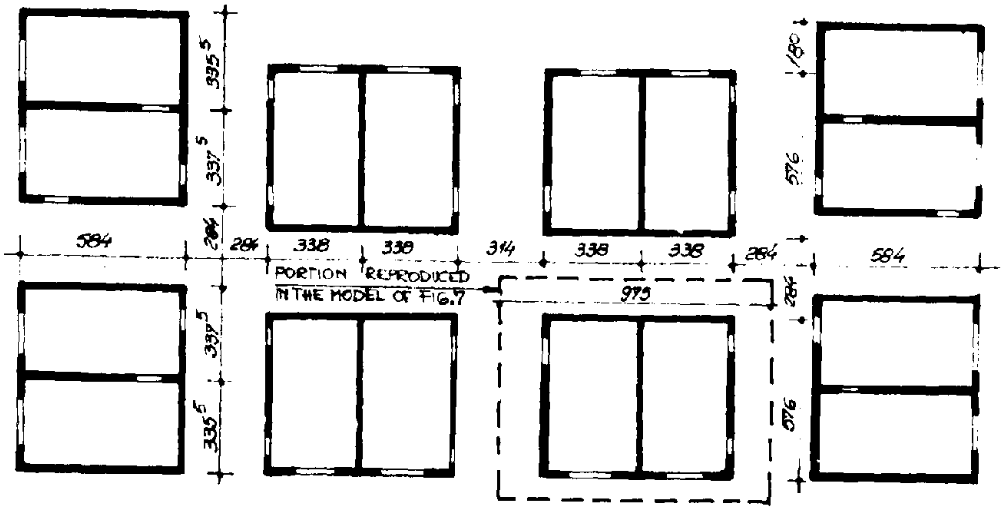

Fig. 4- ARRANGEMENT OF CORES WITHIN THE PLAN OF A NINE STORY STRUCTURE.

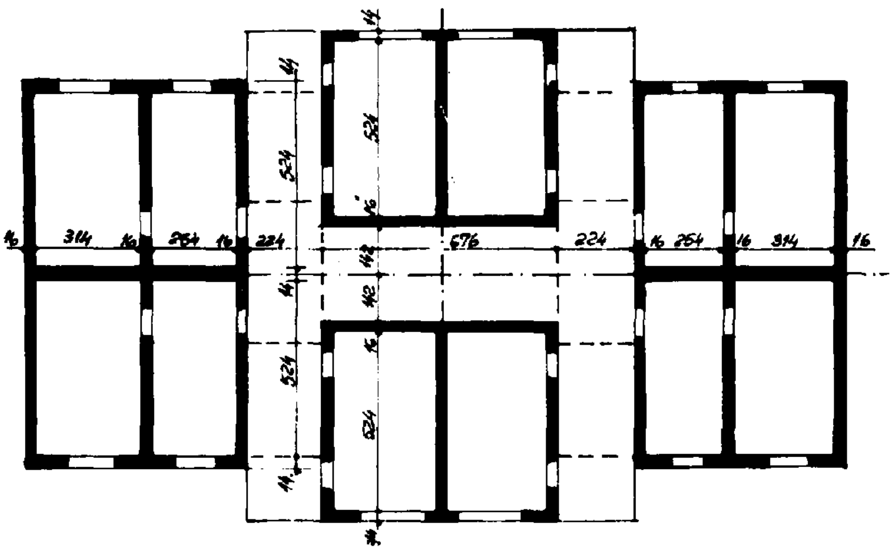

Fig. 5- FLOOR PLAN OF TEN STORY BUILDING.

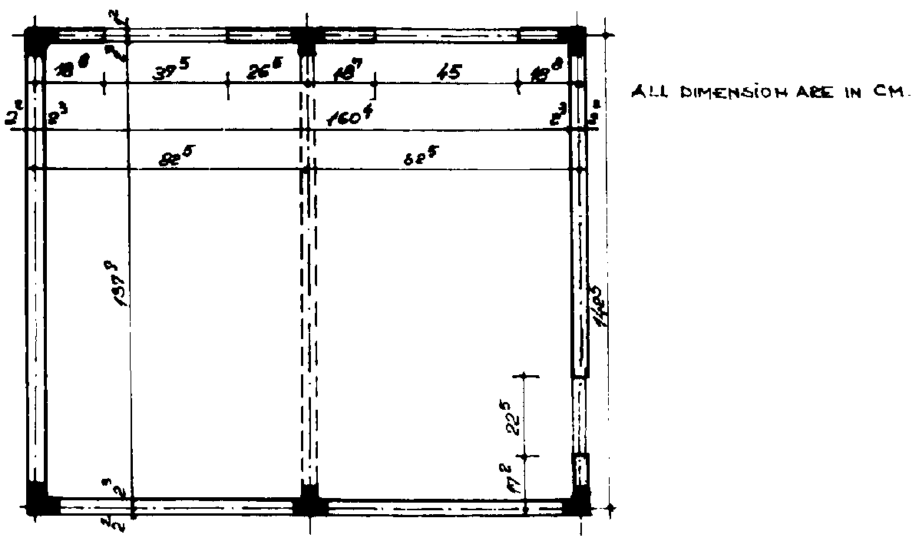

Fig.6- FLOOR PLAN OF A MODEL OF E1 CORE IN .3

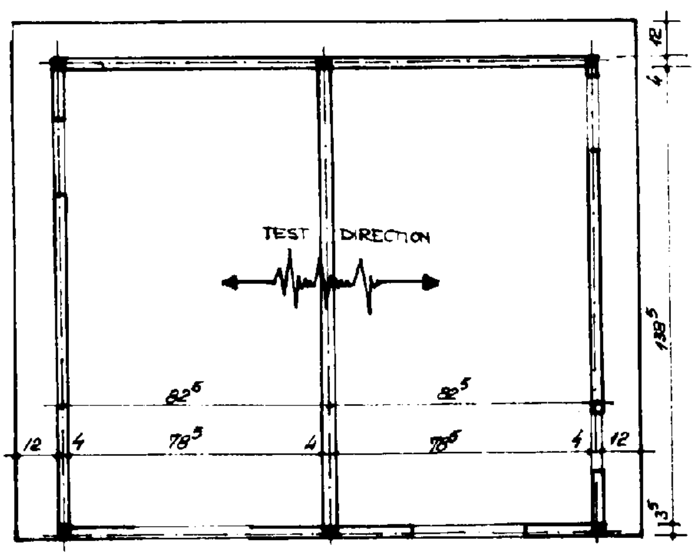

Fig.7- FLOOR PLAN OF A TYPICAL CORE MODEL WITHIN THE STRUCTURE IN FIG. 4

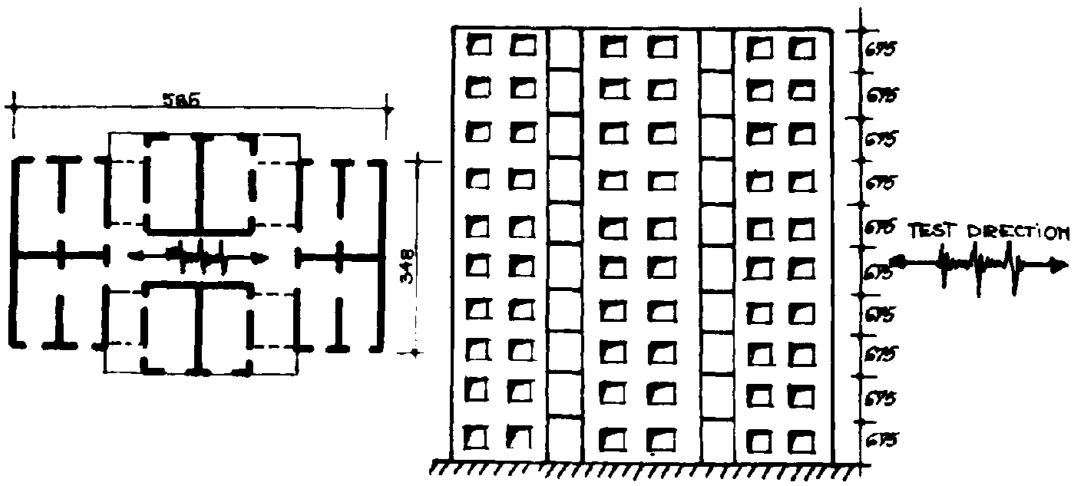

Fig. 8- PLAN AND ELEVATION OF A 1/4 SCALE MODEL OF THE STRUCTURE IN FIG.5

Under subsequent excitations of increasing intensities the described phenomena amplified and vertical cracks developed in the central core. The failure was caused by partial ruptures of first level piers, local yielding of horizontal joints between cores, sliding of reinforcement bars at corner sections in the first story and excessive cracks of vertical joints at the corner zones of the central cores.

CONCLUSIONS

A considerable amount of research on the seismic design of precast concrete large panel buildings has been conducted in Romania lately. This paper is meant to present some of the investigations carried out at the Building Research Center of Iaşi. These investigations have been both analytical and experimental involving typical joint specimens, full-scale portions of structure and complete structural models. Particular emphasis has been given to the arrangement of components, the nature of the interconnection between the walls and floors, the nonelastic behavior of structures designed to observe the elastic levels and the integrity of joints. These studies proved the validity of some concepts of seismic design, improved and simplified some details and developed guidelines in seismic analysis.

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IV.7 SEISMIC RISK REDUCTION USING PRESTRESSING IN REINFORCED CONCRETE STRUCTURES

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ABSTRACT

The real pattern of seismic loading of buildings during strong earthquakes implies, in the current design practice, two main mechanisms: the mechanism of resistance and the mechanism of survival.

For buildings in zones of higher seismic hazard, survival mechanisms protect against collapse and consequently avoid the loss of human lives.

From a structural point of view, after the earthquake the structure can be found in a damaged state that is more or less advanced, but difficult to anticipate in the initial design stage. Material and financial expenses are necessary for the total structural and functional recovery of the buildings.

An efficient method to reduce significantly the structural damage of concrete structures in the prestressing of typical cross-sections. Thus, during alternate loadings, a constant bearing capacity and rigidity factor could be maintained.

The paper deals with the analyses of the conceptual elements of structures using precast members assembled by prestressing.

Some possibilities of reducing the seismic risk of existing buildings by ulterior prestressing are also presented.

All over the world, for structures built in regions with high seismicity, prestressed concrete (P.C.) has not been used - as a rule, opinions being different. There are countries where prestressed concrete structures in high seismic zones are pro -

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hibited, while in other countries their use is allowed but with special requirements.

The progress in this direction was stopped or, at least delayed, due to: prestressed concrete's brittleness, in complete knowledge of the behavior using alternative loadings of the over-prestressed sections and the reduced disipation capacity of seismic energy induced in the structure.

An attentive analysis started in the seventies, concerning this aspects, has shown that the P.C. could meet earthquake-proof design requirements.

Nowadays, taking into account the results of the last 15-20 years, one can estimate that the utilization of P.C. in earthquake resistant structures leads to a reduction in seismic risk, decreasing the vulnerability of the structure as compared with those made using reinforced concrete (R.C.).

At the same time, it is possible that post - earthquake reconstruction of a P.C. structure will be less expensive than the post-earthquake reconstruction of a structuring made using R.C.

The correlation of the qualities given by prestressing with a certain type of structure and a certain type of earthquake, probable for the site, could lead to an optimized situation.

The paper intends to present conceptual elements obtained lately in the studies carried out by the authors.

SEISMIC RISK REDUCTION BY THE DECREASE OF THE VULNERABILITY COEFFICIENT

The development of P.C. structures in seismic hazard zones is based on several advantages compared to the R.C.structures.

The P.C.provides increased resistance to base shear force due to the reduction of the main tensile stresses by an additional axial compression.

In the vertical load bearing members, this effect appears without increasing the gravitational forces (that amplifies the base shear force) - an obvious advantage.

Tangential stresses due to the general torsion of the building (that practically cannot be avoided or at the least limited to acceptable values) are taken over in superior circumstances.

In the case of P.C. structures, in order to avoid brittle failure, failure capacity stress must be proportioned as it relate to the cracking stress.

This requirement makes possible to obtain(in comparison with R.C.structures), an accurate checking of acceptable loading range, during a certain seismic sequence when the cracking state is exceeded.

At the same time, in order to avoid the loss of the prestressing, during some moderate earthquakes, a more accurate checking of stresses in steel and concrete is necessary.

P.C.provides an increased capacity to maintain the initial stiffness along the seismic sequence due to the cracking associated with the neutral axis displacement, and therefore, reducing the rotation imposed to the cross section.

The main difference in the resistamt mechanism of P. C. structure is that, in the case of P.C. the internal energy is entirely dissipated into the elastic range, while at R.C. the dissipation is produced by an inelastic response.

The reinforcement ratio of P.C. sections can be reduced, to a mechanical reinforcement ratio of 0,2 - 0,35 providing the necessary ductility.

Meanwhile P.C. allows a gradual reduction of the stiffness associated with the deformation stage, mostly of the non-remanent character.

The energy hysteretic dissipation capacity of P.C. is compensated by 30% increased in deformability as compared to R.C.

Up to the cracking stage P.C. presents a reduced damping ratio under 2 % and after cracking ca. 6%.

Thus acceptable seismic loadings increase by 20 - 30 % ascompared to R.C. resulting in supplementary safety.

These aspects that represent in fact the characteristic differences between P.C. and R.C. structures resistant to earthquakes, make the vulnerability coefficient (with possible values from 0 to 1) tend mostly to 0.

As a result, in a zone of the same seismic hazard,(considering as elements at risk buildings and human lives) a reduced effective risk will result when the prestressing is used.

A practical way of reducing buildings vulnerability under seismic loads, can be obtained by calibrating the prestressing force so that certain behavior levels should result purposefully, under control and by calculation. These levels shall be correlated with the conditions of post seismic recovery established at the initial design stage.

In this respect the following suggestions are made:

- For earthquakes of reduced intensity an elastic behavior of the structure is required with total recovery after the earthquake, maintaining the prestressing effect during the earthquake.

- For base earthquake it is possible to allow plastic hinges in the main structural units (e.g. in the framed structures the plastic hinges could be admitted in beams); the building recovery is done relatively easily by local interventions to several zones of the structure.

- For violent earthquakes, the structure is allowed to work in the plastic range. Therefore, a limited number of plastic hinges are allowed at the vital members of the survival mechanism of the structure (e.g., the columns in a framed structure) if the capacity of plastic sections to provide rotational ductility. without reaching the limit deformation of the material is checked by computation.

In such circumstances, the survival of the structure, avoiding the loss of human life, is required. In this case the rehabilitation or demolition of the building after the earthquake is optional depending on the damage level.

ON THE POSSIBILITY OF BUILDING RECOVERY WITH MINIMUM COSTS

The R.C. buildings that were allowed an inelastic response during string earthquakes require increased costs for recovery because:

- the resistant sections may be in the plastic hinge stage, so that repair works are necessary;
- as a whole, after a violent earthquake, the structure presents s loss of stiffness that must be recovered.

The recovery of the initial stiffness requires the strengthening or the repair of the structural units or even the introduction of new subsystems integrated in the initial structure.

As a result of the incompatibility of the deformations between the structure and non structure members (the structure works in the inelastic range) the non-structural units suffer a strong damage or even a complete loss of the utility.

Consequently recovery costs might reach 70-80% of the initial costs of the building.

The P.C. structures avoid this shortcoming because the resistant sections working in the elastic range during long intervals of the seismic motion have a total recovery of the load-bearing capacity.

After the loss of prestressing the pretensioned reinforcement acts as tensional-bars, providing the joint work of the whole structural units; the existence of these tensional bars allows a better protection of the non-structural units as compared to the R.C. structures.

The P.C. can conserve its initial qualities as compared to the R.C., because the microcracking process in the mass of the material is stopped or avoided in the presence of the prestressing.

Due to the permanent compression of the cross section the corrosion process is substantially reduced as compared to the R.C.

The R.C. holds an inferior position concerning the cracking mechanism because this is related mainly to the action of the permanent loads.

Under these loads the P.C. members are in a forced compression stage according to the specific P.C. design rules, practically leading to the impossibility of cracking in this stage.

At the same time, the cracking stage represents a normal process of function of the R.C. for, in this way, the reinforcement starts working.

In P.C. the cracking stage following the hygrothermal deformations is practically avoided by prestressing.

In R.C., this kind of internal stress due to some reasons become significant in the cracking process.

As a consequence the microcracking and cracking processes gradually open the way for corrosive factors in the concrete mass; therefore the loss of strength and of the postelastic deformation capacity.

The P.C.works under service conditions results in a state of forced compression, avoiding this damaging mechanism acting before the earthquake.

PRESTRESSING AS AN EFFICIENT METHOD OF REPAIRING OR STRENGTHENING OF R.C. BUILDING DAMAGED BY EARTHQUAKES

A R.C. structure may require after the earthquake re - pair or strengthening works.

Prestressing technologies allow both operations

After the reconstruction of the damaged cross sections (by concreting, jacketing etc.) a favourable state of stresses can introduce allowing the construction to withstand future seismic motions.

At the same time, the reinforcement used for post-tension acts as tension - bars leading to a joint work of the whole structure.

Lately a so called "additional prestressing" has been developed.

That means an intervention directly into the cross sec - tions by the action of am external forces system.

Basically "the additional prestressing" could be acheived by two technological procedures:

- using supplementary post-tensioned reinforcement;
- by imposed deformation .

Using additional reinforcement an increased flexural strength can be obtained for uncracked and cracked sections.

The reinforcement ducts can be linear (when the effi - ciency is minimum) or polygonal (when an increase of shear force capacity is possible).

In these cases a preliminary checking of the section ca- pacity at supplementary compression is necessary.

The buckling checking is necessary also for the slender members subjected to post-tènsion as well as for all cases when tensioned reinforcement is not entirely connected to the repaired members along them.

As a rule, in analyses required for the repair or streng- thening of R.C. structures by prestressing, there are two dis - tinct situations, namely:

Case 1 - the existing cracks are under allowable values for R.C., then , the most economic solution consists of considering the prestressing force as an external constant force, checking the R.C. section strength as subjected to eccentric compression.

Case 2 - Cracks have exaggerated openings; then there are doubts concerning the quality and the amount of steel and the class of the concrete used. In this case, the prestressing shall be dimensioned without taking into account the existing non-prestressed reinforcement. Thus, the R.C. member is transformed in a P.C. member with external reinforcement.

The repair or the strengthening of R.C. members in order to recover or to increase shear force capacity is made as a rule by the vertical stirrups or by longitudinal tendons inclined near the bearing areas in order to obtain a shear force in an opposite sense.

The second solution of introducing "additional prestressing" consists of providing imposed deformation placing the tendons with uneven bearings. In this way bending moments result acting reversely to the moments given by the exterior loads and consequently deformations adverse to those given by the earthquake.

STRUCTURAL POSSIBILITIES OF P.C. BUILDINGS IN SEISMIC AREAS

The studies indicated that partial prestressing- including tensioned and nontensioned reinforcement - is more convenient tham total prestressing, but even in this case reduction in energy absorbtion and system ductility may occur.

Therefore, prestressing is useful and favourable for the structures where the transfer or the seismic forces is based mainly on the shear force.

In other words, the use of prestressing for flexible framed structures seems to be less recommended as compared with the rigid and semirigid structures (framed shear - walls, monolith shear - walls, large panels).

Prestressing could be useful also for high rised multi - storied frame structures.

Using the beam-column joint local prestressing framed - structures become stiffer than the R.C. frames, providing an overall hysteretic dissipation of the energy induced by the earthquake.

The large panel structures fit better to the prestressing idea due to the specific type of joints (designed to transmitthe shear forces) which could be simplified as technology and im - proved as safety degree by prestressing.

The prestressing is also a rational idea for central core structures in both precast and monolithical variants.

The reduced dimensions of the central core associated with the insufficient vertical load on the core (specific to this type of structure) may lead on the core (specific to this type of structure) may lead to an unfavourable state of stresses. By prestressing, this situation is avoided.

Concerning the horizontal prestressing at the floor slab level, its effect is also favourable, taking into account the disk role of the slab in the uniform repartition of the loads towards the vertical elements.

The maximum efficiency of the prestressing in a structure results when a convenient state of stresses is created on both directions, for instance:

- the beam-column joints of frames prestressed in trans - verse and in longitudinal direction;
- the shear-wall structures with prestressing in both vertical and horizontal joints.

The proper utilization of prestressing in a structure implies knowledge of the probable type of earthquake on the respective area (shallow or intermediate, having higher accelera - tions, velocities or displacements).

If the seismic motion is characterized by a higher acce - leration (the velocity and displacement being more reduced) the rigid and semirigid structures are mostly affected.

When the velocity component is dominant, the flexible structure are submitted to greater loads.

Finally, if in the seismic motion the displacement is predominant, the overloading concerns the very flexible struc - tures mostly because of the second order effects.

Thus, for a particular probable earthquake in location the structure can be deliberately designed using a system with minimum vulnerability.

Considering the earthquake of March 4, 1977, the first temptation considering the above mentioned facts is to chose rigid structures because in 1977 the velocity component was quite increased while the acceleration presented a moderate value.

However in the present state of the development of seismological techniques the probability of a certain type of motion might be better evaluated only for special structures design.

Therefore the structure must be prepared to withstand a variety of types of seismic motions.

According to the type of earthquake expected on a certain size one can chose the type of structure, and if this fulfills the possibilities offered by prestressing, it will be efficiently applied.

For instance, the constructions with structural walls could be prestressed if, on that particular area strong earthquakes are to be expected.

Another utilization of prestressing is represented by the beam-column joints in framed structures or even the whole frame structure in case that expected earthquakes may be characterized by higher velocities or displacements.

IV.8 DYNAMIC ANALYSIS OF MULTISTORY BUILDING STRUCTURES WITH CORES OF PRECAST COMPONENTS ASSEMBLED BY PRESTRESSING

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1. INTRODUCTION

The main requirements related to the design and construction of present-day appartment buildings, that strongly influence are: functional flexibility, industrialization of construction process and - obviously - structural safety, with emphasis put on seismic protection.

Starting from the idea, almost unanimously accepted nowa - days, that modern housing plans should be suitable to varied and frequently changing ways of life, there is an increasing ten - dency toward flexible housing space. Among the most important factors contributing to functional flexibility, the layout of the structural system should be taken into consideration, insofar as its vertical and horizontal members are removed from inside the residential units.

As far as industrialization is concerned, one of the most efficient solutions known to-date is the prefabrication of structural components, associated with high productivity techniques for assemblage and for other works carried out on construction site.

In case of buildings located in earthquake-prone areas, specific problems must be solved in relation to structural safety, in order to achieve proper seismic protection. There is a trend in modern design to apply the concept of so-called hazard sce - narios /1/, described by means of the functional and structural features of the building, together with the characteristics of disturbing phenomena. To each such scenario shall be associated

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a design situation defined in relation to the nature and intensity of actions, by an appropriate model of structural behavior and the corresponding performance criteria. Hence, one can make a choice concerning the safety level which should be assigned to the building, so that for every probability of occurence of a specific disturbing event will occur, the construction can exhibit a satisfactory response in relation to the serviceability requirements.

The present paper gives a general description of the layout of a structural system trying to meet to a higher degree the requirements hereabove mentioned and discusses the most significant results obtained from its dynamic analysis to seismic actions.

2. LAYOUT OF STRUCTURAL SYSTEM

The main feature of the structural system presently studied /2/ consists of gathering its vertical components, namely shearwalls, into several cores (tubes). The horizontal structural components are long-span floors supported by cores, to whom they convey all loads (Fig.1).

This solution offers promise in fulfilling the requirements of functional flexibility for residential buildings, by:

- grouping vertical structural components solely in cores located at the extremities of the apartments and producing floors with large free areas, suitable for a variety of housing plans;
- removing any vertical structural member from the facades, between the cores;
- achieving flat soffit floors, without beams;
- locating sanitary equipment inside the cores, thus considerably enhancing the plan's freedom.

As an example, some possible plan arrangements for 3 and 4 room apartments in a building having this kind of structural system are shown in Fig.2.

From the industrialization viewpoint, this system is characterized by complete prefabrication of structural components, making use of two categories of precast units:

- spatial (three-dimensional) reinforced concrete units for cores;
- plane (bi-dimensional) prestressed concrete units for floors.

On any story, each core is made up of two box-type precast units, assembled to each other on site. The assemblage of the pile thus obtained is achieved by vertical prestressing. To this end high-tensile wire tendons are threaded through ducts properly located within the core units cross-section. The tendons are anchored into the understructure and post-tensioned from the upper part of the core.

A horizontal prestressing is also introduced at each floor level, by post-tensioning several tendons threaded through ducts provided within the precast floor units, as well as within the core floors.

From the viewpoint of practical application it is expected that - by making use of highly efficient industrialized techniques for producing and assembling its precast components - the prototype of this structural system might offer the possibility of erecting a variety of buildings with largely improved plan flexi-xibility, having different number of stories and located in areas with different seismic characteristics. From this latter aspect derives the interest presented by studying the conditions required by the proposed structural system in order to exhibit appropriate behavior to either deep of shallow-type earthquakes, as they might occur in different regions of Romanian territory.

3. LAYOUT OF PROTOTYPE CORE

The use of prestressing in the case of earthquake resistant structural systems - especially for vertical members, with a vital role in achieving seismic protection - has been applied on a rather small scale to-date in residential building construction and the available information is scarce. There are only a few isolated cases in our country where prestressing has been used for multistory building structures, so that there is a lack of experience in this field. Developing design studies, as well as theoretical and experimental research work on this matter, has been therefore of particular interest.

In order to prepare the way for implementation of structural systems of the type described in the previous chapter, an experimental model of a core made up of precast box-type components assembled by vertical prestressing has been completed, aiming at a thoroughful investigation of its seismic behavior by testing on a powerful shaking-table. The 1:3 scale model reproduces a prototype core for a 9-story residential building, whose description is briefly given hereinafter.

The geometry of the prototype core (Fig.3) is very similar to that of an intermediate core belonging to the structure illustrated in Fig.1. The precast components are box-like, consisting of three walls and the bottom floor. Their assemblage in pairs at any story is achieved by reinforced concrete joints.

The pile of core precast components is assembled by post-tensioning vertical cables, properly located to get, as far as possible, favorable stress-states in relation to the effect of external actions, both gravitational and seismic.

The materials provided for the prototype core are: concrete class Bc 35 (characteristic compression strength 28 N/mm^2), non-prestressing reinforcement grade PC 60 and OB 37 (characteristic strenght 410 and 255 N/mm^2, respectively), prestressing reinforcement grade SBP-I (specified tensile strength 1600 N/mm^2).

4. DYNAMIC ANALYSIS OF PROTOTYPE CORE

Then dynamic characteristics (eigenvectors) of the prototype core have been obtained by means of CASE computer program /3/, developed on the basis of ETABS program originally issued at University of California, Berkeley.

For this analysis, the core structure has been considered as being made of vertical sub-structures (shear-walls) connected to each other by floors, taken as infinitely rigid in their own plane. At any story level, the dynamic behavior of the structure is then defined by three degrees of freedom, namely two translations and a rotation in the floor plane. Because of the simmetry of the prototype core structure about two orthogonal axes, the eigenvector components resulting from a three-dimensional modal analysis are entirely decoupled. Consequently, the eigenvalues separately computed by a plane analysis for each of the principal directions are practically identical to those obtained from a three-dimensional analysis performed on a tube scheme.

The value of the fundamental natural periods, computed by taking into consideration two situations concerning the stiffness of coupling beams, namely $E = 0.60E_{concrete}$ and $E = 0.15\ E_{concrete}$, are given in Table 1.

Table 1

Type of analysis	$\frac{E_{coupl.beams}}{E_{concrete}}$	Fundamental period (s) X	Y	Q
Plane, transv. direction (Y)	0.60		0.314	
	0.15		0.362	
Plane, long. direction (X)	0.60	0.437		
	0.15	0.437		
Three-dimensional	0.60	0.436	0.312	0.273
	0.15	0.436	0.359	0.295

The purpose of the dynamic analysis carried out for the prototype core, was to serve a first step toward the utilization of the structural system described in chapter 2 for residential buildings located in regions with varied seismo-tectonic characteristics.

The study was organized on the basis of a set of relations between the ground motion intensity, the response structure and its method of analysis /4/. In a similar manner the multi- test concept of seismic design operates /5/, by considering a sequence of analysis levels, in correlation to certain damage scenarios.

Taking into account the macroseismic characteristics of the Romanian territory, a zone having the degree of seismic intensity equal to 8 on MSK-64 scale has been chosen as a reference point. Accordingly, the structural system should be designed so that its behavior corresponds to several scenarios, as it is shown below.

Scenario no.1 : when subjected to an earthquake with lower intensity in relation to the nominal one assigned to the respective site (i.e.degree 7, with $a_{max} \cong 0.12g$), the structure should exhibit an elastic response, so that complete serviceability of the building may be preserved.

Scenario no.2: when subjected to an earthquake having an intensity equal to the nominal one (i.e.degree 8, with $a_{max} \cong 0.20$ g), the structure should exhibit a controlled inelastic response, aimed at keeping the vital structural components -that is, prestressed concrete shear-walls, for the present case - within the elastic range and accepting inelastic excursions for other structural members - that is, reinforced concrete coupling beams. An alternation of the building serviceability might be expected.

Scenario no.3: when subjected to an earthquake with higher intensity in relation to the nominal one (i.e.degree 9, with $a_{max} \cong 0.30$ g), the structure on the whole might enter the inelastic range. One should keep under control the ductility demand of vital structural components, in order to avoid collapse.Under these circumstances the buildings serviceability might be broken off.

The present paper deals with the dynamic analysis corresponding to scenarios no.1 and 2 only. Owing to the fact that the study related to scenario no.3 - by considering the inelastic behavior of prestressed shear-walls too - as well as the model testing on shaking table, are now in progress, the respective findings will be reported elsewhere.

The dynamic analysis of the prototype core has been carried out by means of ANELISE computer program /6/, developed at the Design Institute for Typified Buildings in Bucharest . It performs a time-history linear and non-linear structural analysis, seismic ground motions being applied under the form of digitalized accelerograms, either actually recorded or artificial. For a better understanding of the possible behavior of a structure under seismic actions, accelerograms have been used either for deep earthquakes (Vrancea 1977 , N-S component) or for shallow earthquakes (EL CENTRO 1940, N-S component) both being scaled for two peak values, 0.20g and 0.12 g respectively. The step-by-step integration process has been extended over the first 10 seconds of the respective ground motion, using a 0.02 s time - interval.

A hysteretic bi-linear M-O relationship has been adopted for coupling beams (Fig.4). The yielding moment is computed on the basis of plane sections hypothesis and of characteristic curves for concrete and steel reinforcement shown in Fig.5.

Several cases of reinforcement - selected within the range of rational detailing - have been considered for coupling beams and, depending on them , the main parameters of the structural response have been computed. In every case, the reinforcement of coupling beams has been taken identical over the building height. Which corresponds to the actual situation of using identical precast core components.

The values of the yielding moment M_y^{cb} corresponding to the reinforcing solutions of the coupling beams are given in Table 2.

Table 2

Case	A	B	C	D	E	F	G
Bars number and diameter (mm)	2Ø12	2Ø16	2Ø20	2Ø22	2Ø25	4Ø20	4Ø22
$A_s = A'_s$ (mm^2)	226	402	628	760	982	1256	1520
M_y^{cb} (kNm)	71.5	126.5	196.5	237	305	393	474

Calculations have been made for cases B - G, for the structure subjected to earthquakes with a_{max} = 0.20 g, the case A proving incompatible with a satisfactory response. The most significant results revealed by the dynamic analysis performed on the transversal direction (Y) of the prototype core are briefly discussed hereinafter.

The values of the response parameters differ from one earthquake to another, being larger for El Centro, due to the fact that the fundamental period of the structure (0.36a) falls within a zone of higher amplification in the spectrum of this earthquake. The response to the same ground motion is modified in relation to the inelastic excursions of coupling beams,which are dependent on their reinforcement.

The value of the core maximum base shear V decreases along with the reduction of the reinforcement of coupling beams (Fig.6). This decrease appears more marked for El Centro (31%) than for the Vrancea earthquake (11%), so that in the case of the smallest reinforcement (case B) the respective values come very close to each other. It should be noted that, in this case, an augmentation of the base shear appears for the Vrancea earthquake, a phenomenon that might be ascribed to the increase of structure flexibility, due to deep excursions of coupling beams into the inelastic range, which in turn shifts the response within a zone of spectral amplification around 0.4 - 0.5 s.

The variation of the maximum values of interstory displacements over the building height (Fig.7) is obviously influenced by the identical reinforcement of coupling beams on all stories. Depending on the reinforcement case, the differences between extreme values are 33% and 19% for Vrancea and El Centro earthquakes, respectively.

The maximum bending moment M_w at the base of each coupling shear-wall exhibits a variation (Fig.8) quite similar to that of core base-shear. By reducing the reinforcement of coupling beams from case G toward case C, a decrease of the bending moment M_w (with 19% for El Centro and with 5% only, for Vrancea earthquake) is recorded. Further reduction of reinforcement of the coupling beams (caseB) leads almost to a standstill for El Centro, while for Vrancea there appears a rather sharp increase, as a consequence of the amplification effect already mentioned.

The maximum axial force N_w at the base of each coupling shear-wall, caused by horizontal seismic action, present a continuous decrease (by 43% for Vrancea and 53% for El Centro earthquake) along with the reduction of coupling beams reinforcement (Fig.9).

As far as rotational ductility demands of coupling beams is concerned, the values are quite small and generally show little variation over the building height, for each case taken individually (Fig.10). An exception is encountered for the smallest reinforcement considered (case B), where considerably larger values appear, yet without exceeding the possibilities of rational detailing of coupling beams. For the same case of reinforcement, the maximum values of rotational ductility factor μ_{θ} of the coupling beams are 1.25 - 1.91 times larger for El Centro than for Vrancea (Fig.11).

A separate analysis has been carried out for the structure subjected to same earthquakes, but with a_{max} = 0.12 g, the calculations being made for cases A - E of coupling beams reinforcement (see Table 2). The results obtained from the dynamic analysis show that the variation of the main response parameters is qualitatively similar (Fig.12-17) to that previously recorded in case of earthquakes with a_{max} = 0.20 g.

5. CONCLUSIONS

The findings of this phase of the study confirm the fact that, by correct modelling of the structure and of its elastic and inelastic deformation characteristics, together with the use of an appropriate computer program for non-linear analysis, it becomes quite possible to make accurate predictions about the behavior of a building subjected to seismic actions of different kinds and with different intensities.

By operating changes on certain characteristics of the structure, it is possible to get some optimum values for its response parameters to a specific seismic excitation, according to accepted scenarios. If one aims to ensure seismic protection for a building against the action of different types of earthquakes, an envelope of the dynamic responses should be taken into consideration. The optimization process then becomes more difficult and its results might have mainly qualitative value.

With regard to the structural system which forms the object of this study, the results already obtained prove that it is potentially able to ensure the behavior described by the first two scenarios and the parameters of its elastic and inelastic response to seismic actions of the type considered may be kept within prescribed limits. The second phase of the study will consider the third scenario, by investigating the inelastic behavior of prestressed concrete structural vertical components too and - possibly - the soil - structure interaction. It will certainly provide additional valuable data to complete the picture of seismic performances of this category of structures.

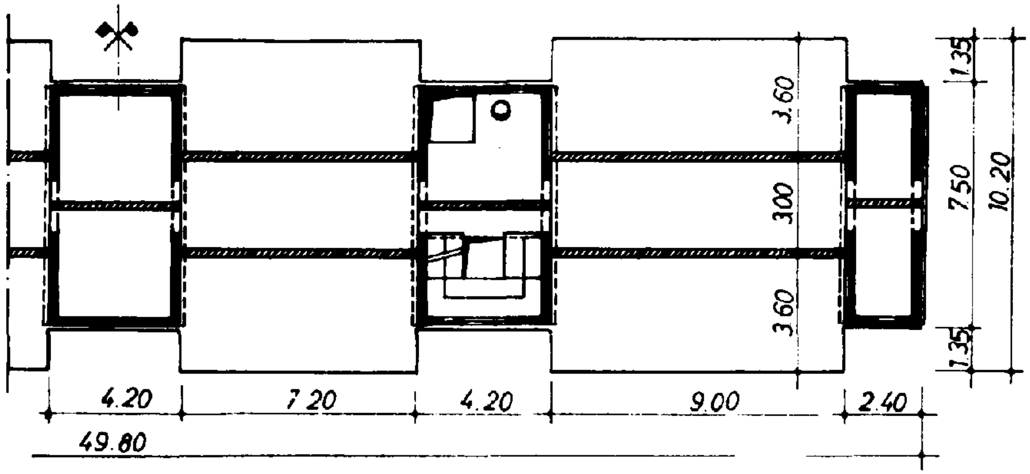

Fig.1: General Layout of Structural System

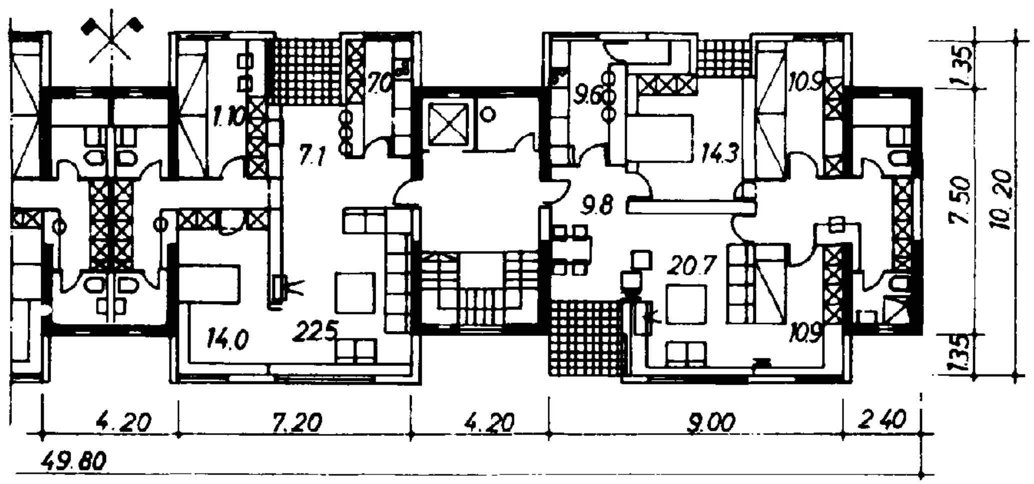

Fig.2: Typical Floor Plan for Residential Building (Example)

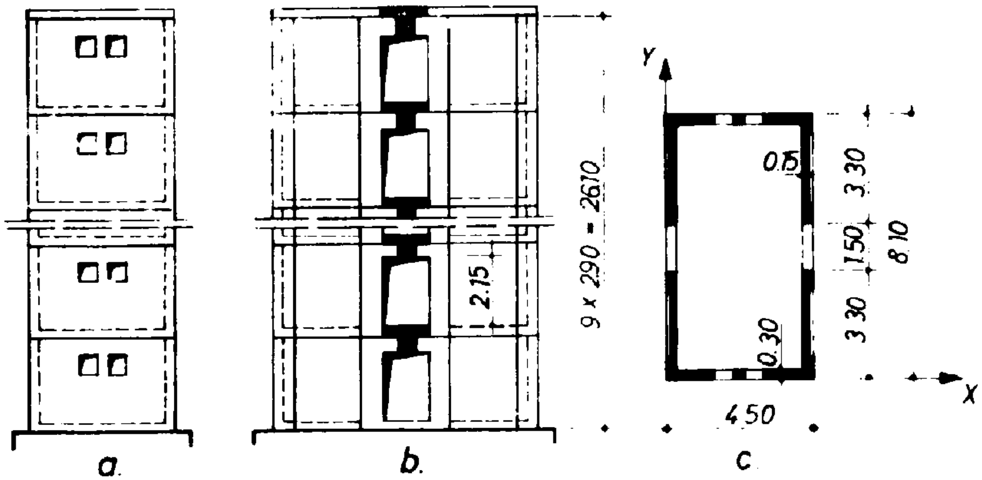

Fig 3: Prototype Core: a,b - Side Views ; c - Horizontal Section

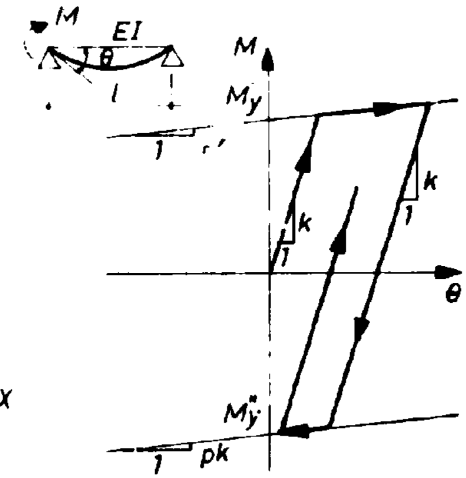

Fig. 4: M-θ Relationship for Coupling Beams

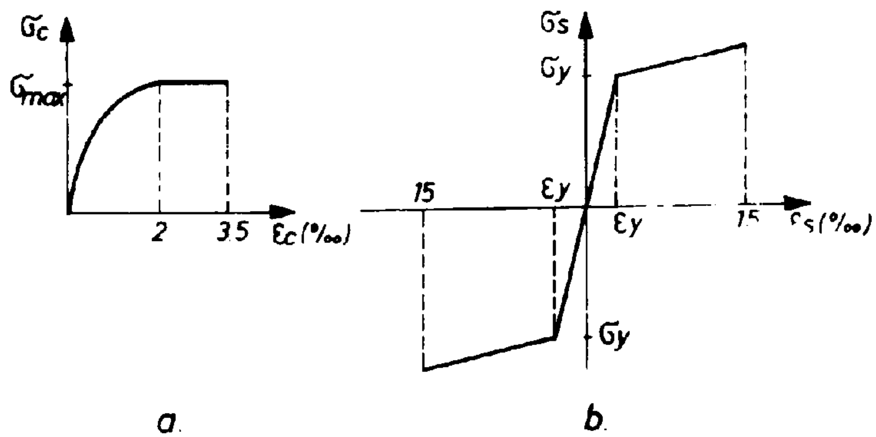

Fig.5: σ-ε Relationships: a - Concrete ; b - Reinforcement

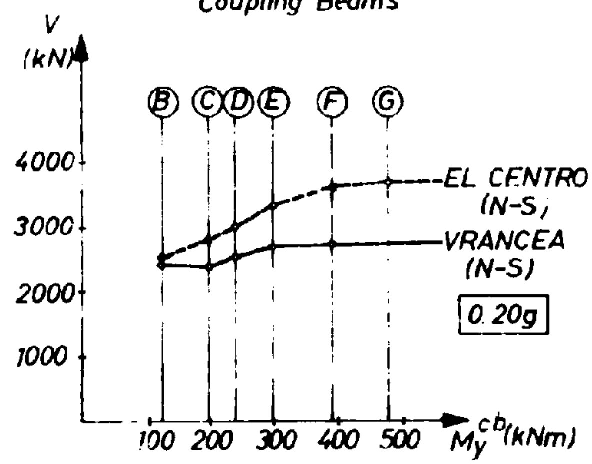

Fig 6: Max Base Shear for Core

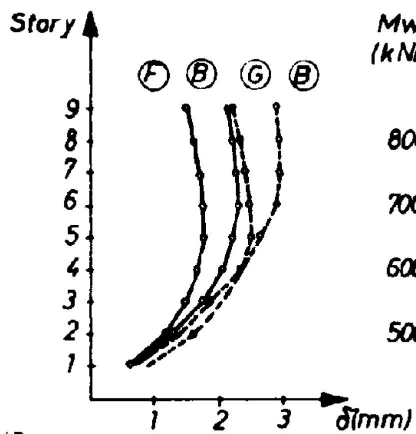

Fig.7: Max. Interstory Displacements

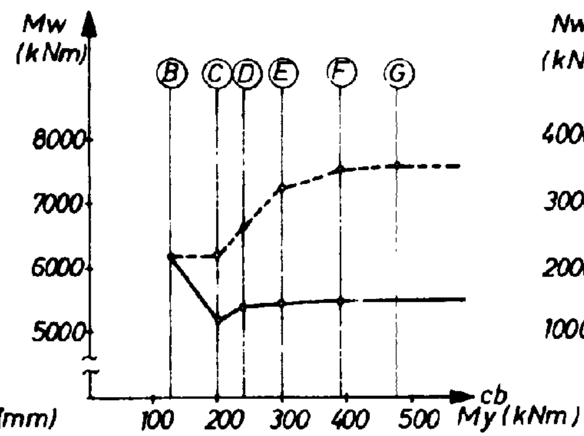

Fig.8: Max. Base Bending Moment for Shear-Wall

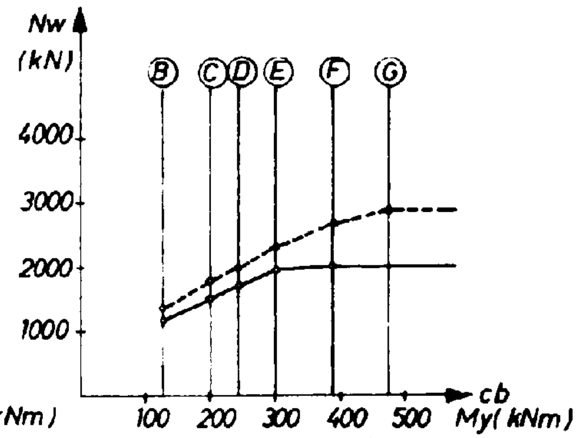

Fig.9: Max. Base Axial Force for Shear-Wall

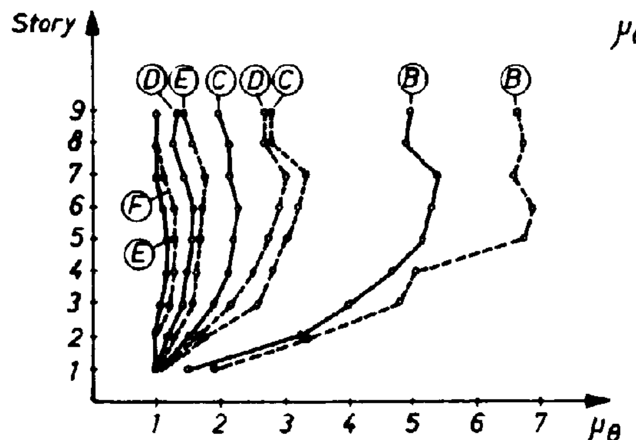

Fig.10: Rot. Ductility Demand for Coupling Beams

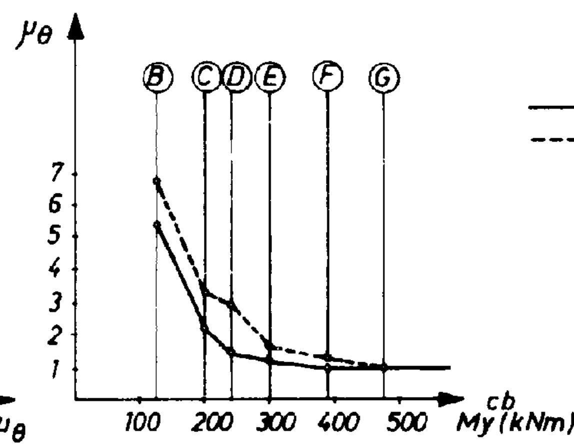

Fig.11: Max. Values of Rot. Ductility Demand for Coupling Beams

—— VRANCEA N-S

---- EL CENTRO N-S

0.20 g

Cases B.....G see Table 2

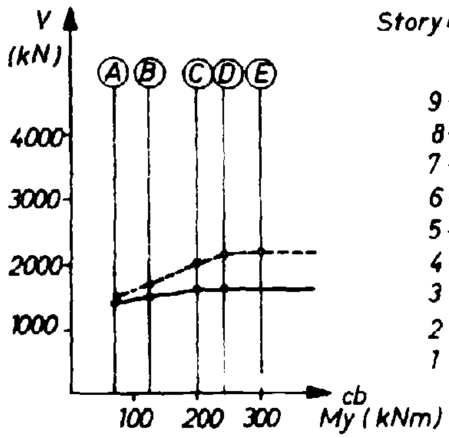

Fig. 12: Max. Base Shear for Core

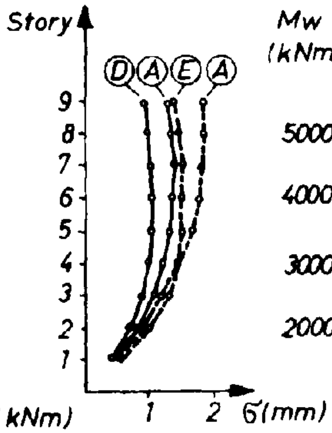

Fig. 13: Max. Interstory Displacement

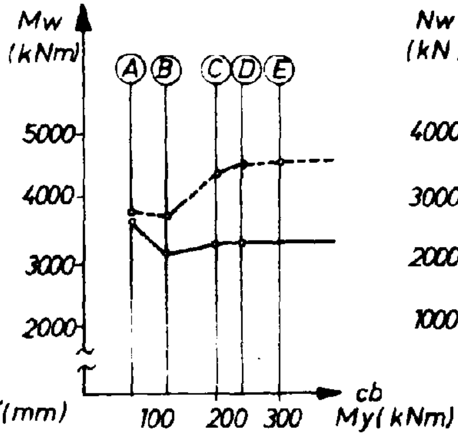

Fig. 14: Max. Base Bending Moment for Shear-Wall

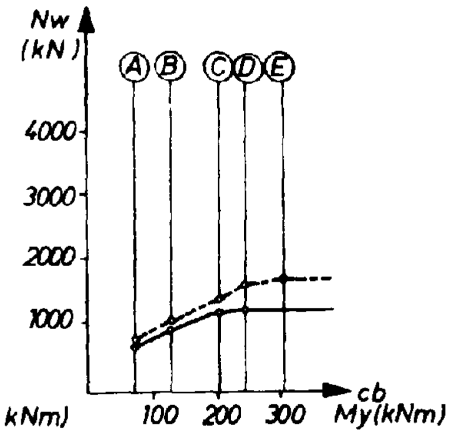

Fig. 15: Max. Base Axial Force for Shear-Wall

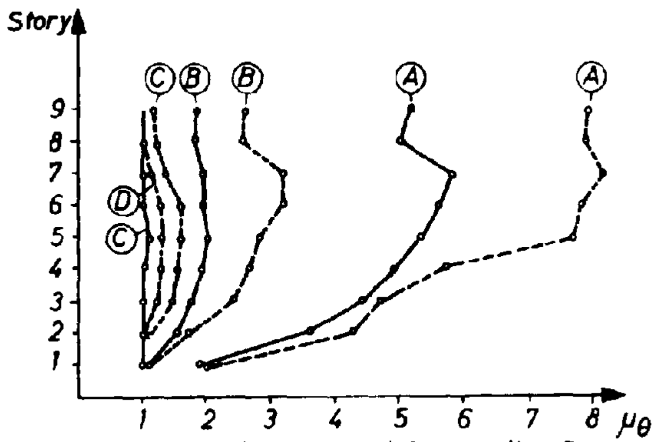

Fig. 16: Rot. Ductility Demand for Coupling Beams

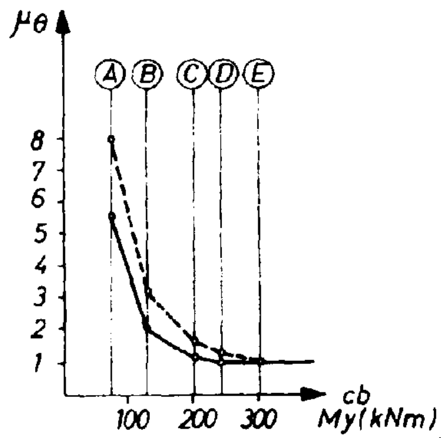

Fig. 17: Max. Values of Rot. Ductility Demand for Coupling Beams

——— VRANCEA N-S

- - - - EL CENTRO N-S

$\boxed{0.12\,g}$

Cases A ... E see Table 2

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IV.9 ANALYSIS OF THE SEISMIC RESPONSE OF A 9 - STORY LARGE PANEL BUILDING

Dan Constantinescu[x]

INTRODUCTION

Precast large panels are widely used on buildings in Romania /1/. This structural system performed rather wall during the earthquake of March 4, 1977 /2/.

The design of large panel structures is based in Romania, as anywhere else in the world, on an elastic response to conventional seismic forces /3/. The current trend is to comprehend better the postelastic behavior of such systems. The experimental investigations on shaking platforme of scale models have proved insufficient to explain satisfactorily all the aspects involved /4/. The paper presents the most significant results of research which in a first tentative attempt to analyse theorethically the inelastic seismic response of a large panel building /5/.

The research deals with a 9-story building (fig.1) which is fully prefabricated with large panels /6/. The seiamic resisting structure is provided by the vertical cores and by the floor diaphragms. It is all made of precast large panels which are interconnected throughout the perimeter. The wall large panels place in berween the cores have no seismic resistance role. A large panel for the core wall is provided with shear keys throughout its height (fig,2). Typical connections are also depicted in fig.2.

The research comprises the five analyses in fig.3. Each analysis provides its own findings as well as the input data for a subsequent analysis.

The first analysis refers to the relationship between the base shear force P and the lateral displacement Δ of the eight floor. The static horizontal forces P_i on every structural structural subsystem act at the floor levels, are distributed on the vertical according to the fundamental mode of vibration and increase from zero up to the attainement of the first ultimate state of a subsystem of structure (fig.4). Due to the lack of

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an appropriate computer program (the development of such a program is in progress in Romania /7/), the horizontal forces were assumed to act only along the direction marked by an arrow in fig.1, the torsion of the structure was neglected and the structure response was hand.computed using two extreme hypotheses : one in which all the coupling beams were assumed nondeformable (denoted here by BND) and another in which plastic hinges were assumed to occur at all coupling beam ends from the very start of the horizontal loading on structure (denoted here by BP).The cantilever vertical cores C - C6 are the basic structural subsystems in the BND assumption, while the coupled structural walls within the cores are the basic structural subsystems in the BP assumptions (fig.5). The relationships $P_I - \Delta$ in the BND assumption and $P_{ij} - \Delta$ in the BP assumption are developed from integrating the cross-section $M-\phi$ relationships along the vertical of the basic structural subsystems (fig.6). The response of the large panel structure is assumed similar to that of a cast-in-place structure (see fig.7).This assumption is discussed within the final analysis presented in the paper.

The $P-\Delta$ relationship of the structure arises from the superposition of the $P_i - \Delta$ relationships of the structural subsystems (fig.8). It provides conclusions concerning the structure stiffness, the yield force P_y and the ultimate displacement Δu. At the same time it provides input data for the static and dynamic analyses of the soil-structure ensemble.

The second analysis in fig.3 is concerned with the relationship between the base bending moment M and the rotation θ on the soil foundation. The computer program DYNMAT /9/ was used. The footing was assumed nondeformable, while the soil was considered to behave as a perfect elastoplastic semispace having the limit stresses according to fig. 9a. Thus the stress distribution on the soil foundation takes the shapes in fig.9 b - d as the nemding moment M increases. Two types of soil were considered: a dry sand and a firm clay. Characteristics typical to Bucharest were chosen./10/.

The $M - \theta$ relationship provides conclusions concerning the rotation stiffness of soil foundation and the magnitude of horizontal forces on the structure from which the soil response can no longer be assumed as linear. At hte same time, it provides input data for the static and dynamic analyses of the soil-structure. ensemble.

The third analysis in fig.3 is concerned with the relationship between the static horizontal force P on the structure and the total lateral displacement Δ_T of the eighth floor. The latter arises from the superposition of the displacement Δ due to the structure deformation and the displacement Δ_f due to the rotation θ on the soil foundation (fig.10).The $P - \Delta_f$

relationship is in fact a linear transformation of the M - τ relationship.

The analysis of the static response of the soil- structure ensemble to lateral loading provides conclusions on the total stiffness of the ensemble as well as on which of the two subsystems, i.e. structure or soil foundation, is likely to yield first.

The fourth analysis in fig.3 is concerned with the dynamic seismic response of hte soil-structure system. In fact the analysis is performed on an equivalent SDOF system (fig.11) that is, a dynamic inverted pendulum having a mass equivalent to the structure and the same P - Δ and P - Δ_f relationships as the structure and the soil foundation, respectively /10/. The parametric analysis carried out takes into account the two assumptions on the structure's response (i.e. BND and BP), the situation in which the structure is fixed at its base or rorates on the soil foundation (sand or clay) and the horizontal N- S components of the accelerograms of El Centro, May 1940, and Bucharest, March 4, 1977, earthquakes. Conclusions are obtained with regard to the expected maximum value, Δ_{max} of the structure displacement induced by the earthquake as well as on the effect that each parameter has on the dynamic response of structure.

The design of the building and indeed the analyses reported here are based on the assumption that a prefabricated structure responds to an earthquake as if it were cast-in-place or, in other words, as if no plastic shear displacement could occur in any horizontal and vertical connection between wall large panels (see fig.7). This assumption is analyses within the final part of the paper.

The most significant results of the analyses in fig. 3 are presented and discussed within the following parts of the paper.

ANALYSIS OF THE STATIC RESPONSE OF STRUCTURE AND SOIL FOUNDATION

The results of the first three analyses in fig.3 are depicted in figs. 12-14. The bilinear approximations of the P- Δ and M - θ relationships (continuous lines in figs.12 and 13 respectively) are used to analyze the static and dynamic response of the soil-structure system.

The graphs in fig.12 give to the following conclusions:
1. Both the elastic and the tangent postelastic stiffnes s

are much the same in the two assumptions on the structure response (BND and BP). However, the difference between the two corresponding yield forces P_y is quite large.

2. The design seismic force P_s, which is about 8% of the total weight of building, is much smaller than the yield force P_y in the BP assumption. That justifies the elastic analysis on which the design is based. The large difference between P_s in fig.12 is largely due to the way in which the limit state method is prescribed in Romania /11/.

3. The computed ultimate displacements Δ_u corresponding to the BND and BP assumptions are much the same (about 7...8 cm) The Δ_u value is crucial since it must not be exceeded by the maximum displacement Δ_{max} induced by the earthquake in order to avoid the collapse. Indeed, the smaller Δ_{max} is compared with Δ_u, the less damaged is the structure. It is worth emphasizing that the Δ_u values computed here are quite conservative evaluations of the actual values /5/.

The graphs in fig.13 show that, in the case of clay, the elastic stiffness to the rotation on soil foundation is larger and the yield moment is less than in the case of sand. These findings will prove essential when the dynamic response is analyzed.

A global image on the response of soil-structure system, to lateral loading is depicted in fig.14. The superposition of the bilinear approximations of the $P - \Delta$ and $P - \Delta_f$ relationships yields a trilinear $P - \Delta_T$ relationship. Throughout the first linear zone both the structure and the soil foundation respond elastically. Then one of the two subsystems elastically while the other responds inelastically and so forth. Only the first two linear zones of the $P - \Delta_T$ relationship are depicted in fig.14.

The graphs in fig.14 give rise to the following conclusions:

a. When the building rests on sand

- The yield force P_y of a structure in any of the BND and BP assumptions is much less than the yield force associated with the soil response. Hence, while the structure responds inelastically to a strong motion earthquake the soil response is elastic.

- The elastic stiffness of the soil-structure system is much less than that of the structure assumed fixed at its base. Consequently, the elastic fundamental period of vibration of the soil-structure system is much larger than that of the fixed-base structure (1.91 sec.as compared with 0.48) and therefore the dynamic response in the two situations can differ significantly.

b. When the building rests on clay

- The yield force P_y of the structure in the BP assumption is much less than the yield force associated with the soil response. However, both structure and soil yield in the BND assumption.

- The elastic stiffness of the soil-structure system is but moderately larger than that of the fixed.base structure. Consequently the elastic fundamental period of vibration i n little increased by the rotation on soil foundation (from 0.48 sec to 0.83 sec) and that can better or worsen the dynamic response of structure to a certain earthquake.

ANALYSIS OF THE SEISMIC RESPONSE

The dynamic response of structure is depicted by means of the hysteretic $P - \Delta$ relationship (figs.15 and 16). The response of the fixed-base structure is depicted by a continuous line while the response of structure resting on clay is depicted by a discontinuous line. When the structure rests on sand the seismic response is either elastic or has minor plastic incursions. That is why the response of a structure resting on sand is depicted in fig.15b only (by dotted line).

The most relevant conclusions are:

1. The structure's response to the Bucharest accelerogram is less conservative that that to El Centro accelerogram. That comes into conflict with the belief of most researchers who consider the Bucharest earthquake irelevant for stiff structures like the one analyzed here /8/.

2. The reduction of the yield force P_y of the structure (BP compared to BND) may decrease the maximum displacement induced by the earthquake.

3. The rotation of the structure in the soil foundation during the earthquake can yield a reduction of the maximum displacement of structure induced by the earthquake.

A comprehensive discussion on the above conclusions is given if Ref./10/ on the basis of the spectral characteristi c s of the earthquakes and of the softening effects which the yielding of structure and/or the rotation on soil foundation have on the dynamic response of structure as a component of the soil structure system.

ANALYSIS OF THE SHEAR FORCES ALONG THE HORIZONTAL AND VERTICAL CONNECTIONS

The maximum shear forces Q_n and Q_y induced by the earthquake along the horizontal and vertical connections can be assessed with the approach depicted in fig.17a. Starting from a maximum value, Δ_{max} of the structure displacement, the shear force P_1 at the base of any structural subsystem(i.e.core or shear wall) can be developed (e.g., in the BND assumption, P_i for any vertical core follows from the graphs in fig.12a). Then P_i is distributed on the vertical of the structural subsystem and the horizontal forces P_{ij} acting at each floor level are obtained. The shear force Q_h along an horizontal connection is in fact the resultant of the forces P_{ij} above. The bending moments M at any floor level can also be computed with the help of P_{ij} forces and hence the normal stress distributions. A vertical shear force Q_v arises from the equilibrium of the element marked by disgonals in fig.17b. The shear force in any coupling beam can be computed similar to Q_v. It is apparent that the maximum value of the force P_i results when the attainement of the ultimate bending moment at the base of the structural subsystem is considered.

The paper presents the shear forces of the cores C2 and C4. The base shear forces Q_h are depicted in fig.18 while the vertical shear forces along the constructions and in the coupling beams are depicted in figs.19 and 20. Q_{v1} and Q_{v2} denote the shear forces Q_v on the height of the first two storys. $\bar{Q}_{vm}$ denote the mean value on a story height of the shear force Q_{vm} on the total height of structure. The bracketed values in figs. 19 and 20 represent the shear stress $\tau = Q_v$ / A where A is the gross area of the concrete cross section on which Q_v acts. Fig. 19 also provides the friction horizontal force Q_{hf} (friction factor is taken equal to 0.3) while figs. 19 and 20 provide the ultimate values Q_{vu} of Q_v along a story heigh vertical connection Q_{vu} is computed acccrding to Ref./12/. The value Q'_{vu} corresponds to the design stregnth of concrete and steel, while Q''_{vu} corresponds to the mean (effective) strengths. The values Q_{v1}, Q_{v2} and Q_{vm} should be compared with Q''_{vu}. It is worth emphasizing that the formula prescribed for Q_{vu} in /12/ is based on tests carried out with static loading. A more realistic evaluation of Q_{vu} should rely on tests with dynamically reversed loading /12/.

The values depicted in figs. 18-20 give rise to the following conclusions:

1. The shear forces Q_h are less than the friction forces Q_{hf}. Thus friction alone can prevent the occurrence of the plastic mechanis, depicted in fig.7c.

2. The maximum computed values of the shear force Q_v are largely exceeded by the ultimate values Q''_{vu} and thus the occurrence of the plastic mechanism depicted in fig.7b is also prevented.

These two conclusions seems to justify the assumption that the large panel structure exhibits a seismic response similar to that of a cast-in-place structure.

3. Large shear forces in the coupling beams occur only when there is no cantilever wall to oppose the shear deformation (see vertical section B-B in fig.19). That seems to suggest that cores incorporating a cantilever shear wall opposing the seismic action can behave according to the BND assumption. As a matter of fact it is apparent in fig.1 that the designer had just that in mind when he arranged the median (cantilever) walls within the vertical cores.

4. By comparing the shear forces Q_{v1}, Q_{v2} and Q_{vT} it is apparent that the maximum values occur always on the second story or above. That is in agreement with the way the coupling beams are damaged during a strong motion earthquake.

CONCLUDING REMARKS

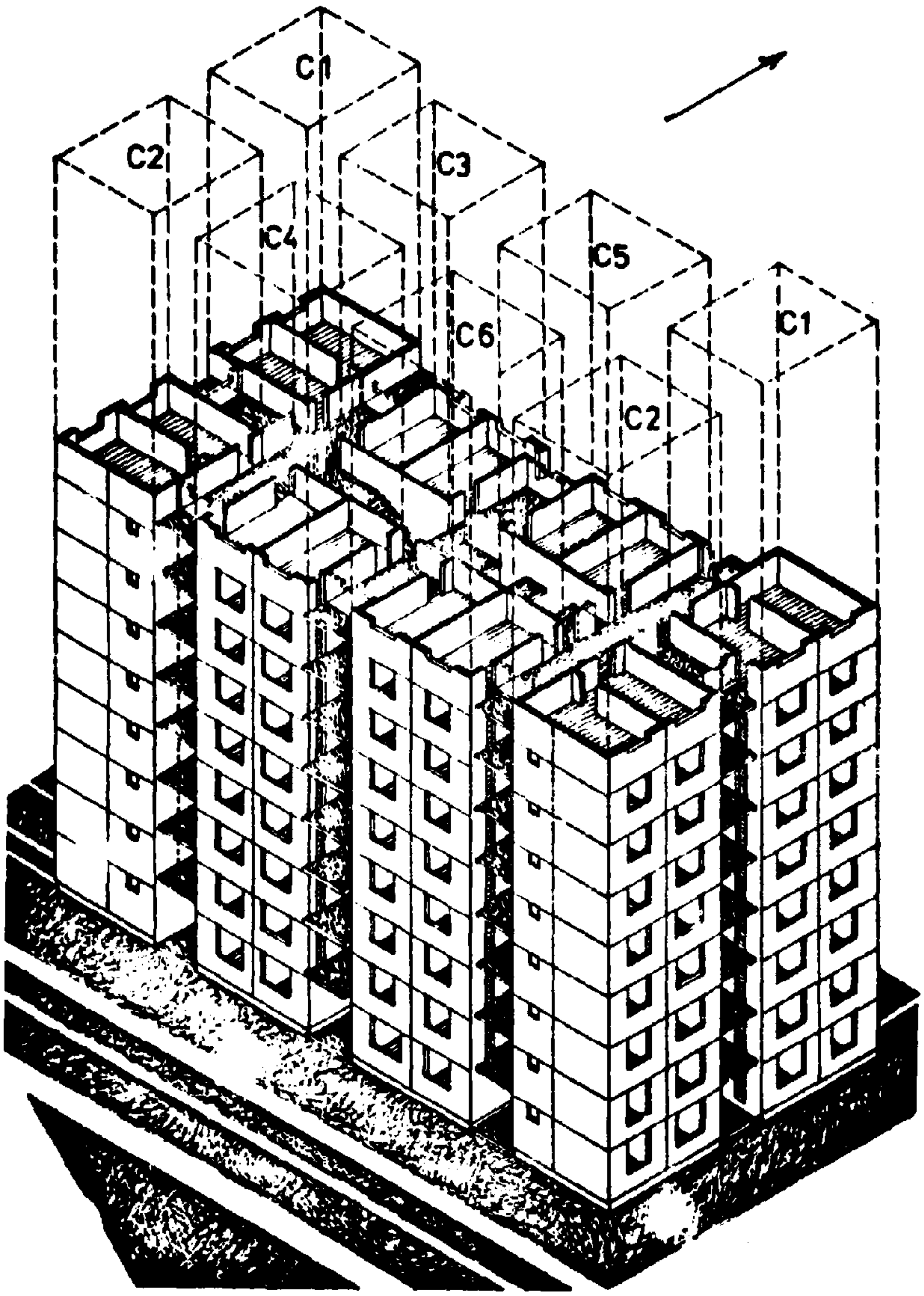
Besides the conclusions presented so far, the paper also emphasize some overall aspects which are of major interest to structural engineers in Romania as well as elsewhere.

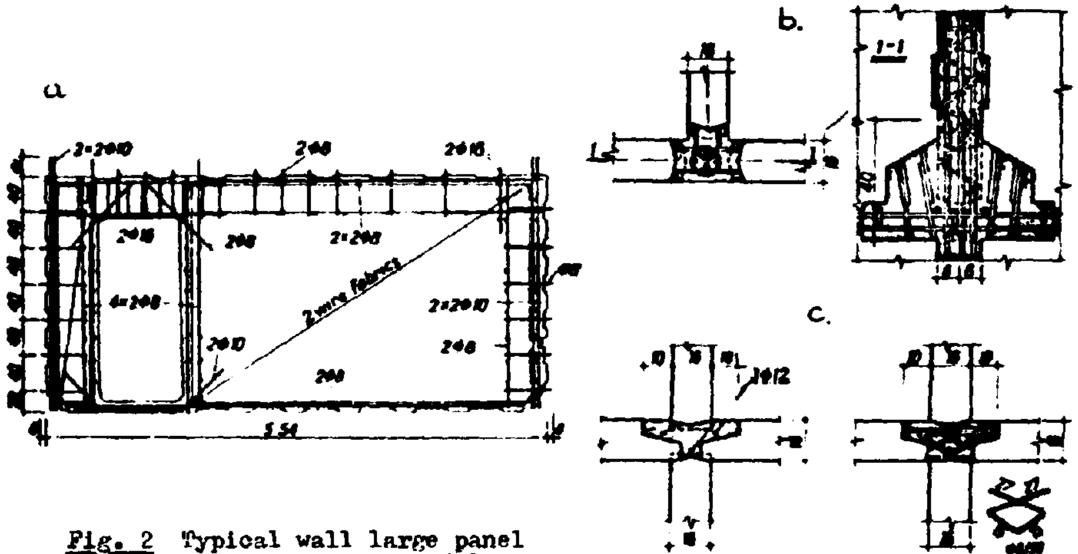
First, the paper outlines the possibility of analyzing the inelastic seismic response of a prefabricated large panel structure as throughly as it is currently done with cast-in-place shear wall structures. The current design of large panel structures. The current design of large panel structures in Romania is based on an elastic response philosophy /12/ while the design of cast-in-place shear wall structures accounts for the inelastic response to strong motion earthquakes /14/. There is an imperative need to approach the design of the two rather similar structures with a common philosophy.

Second, the major effect which the deformability of the soil foundation has on the dynamic response of structure is emphasized. A comprehensive discussion of this aspect is presented in Ref./10/.

Finally, the significance of accounting for the spatial behavior when dealing with the seismic response of structures is outlined. For instance, to neglect or underestimate the co-operation of structural walls opposing the seismic action when orthogonal structural walls are present may decisively alter the final results.

Fig. 1 The 9-storey large panel building





<u>Fig. 2</u> Typical wall large panel
with door opening (a).
Vertical (b) and horizontal (c) connections.

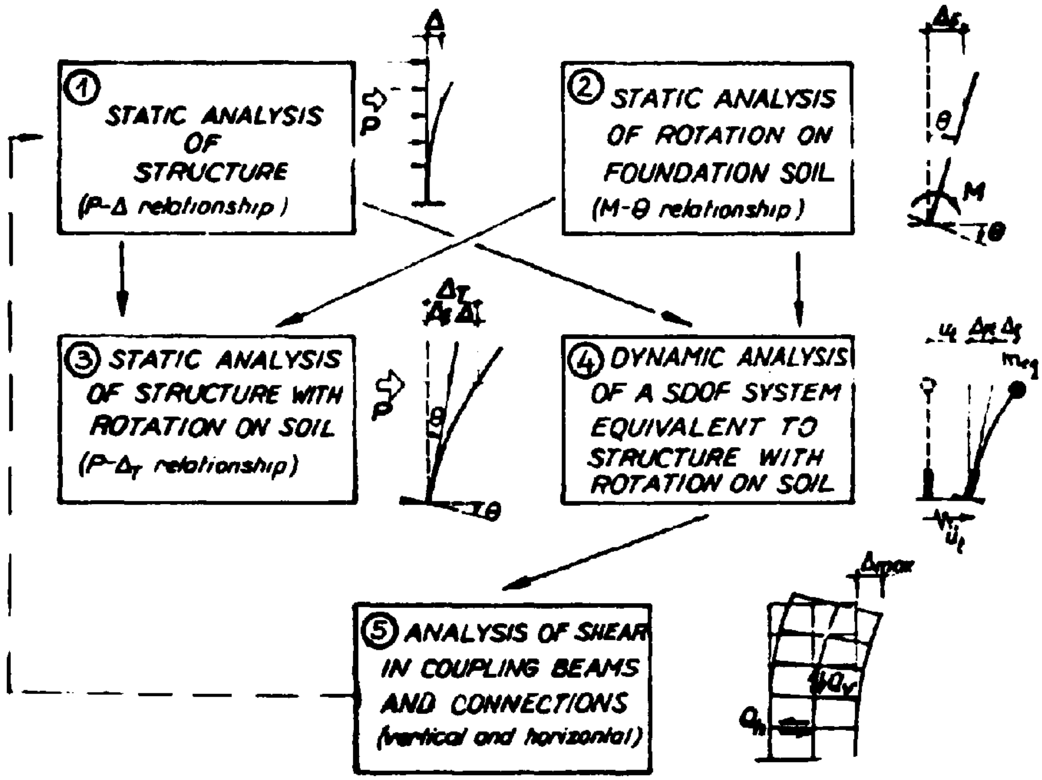


Fig. 3 The analyses carried out within the research

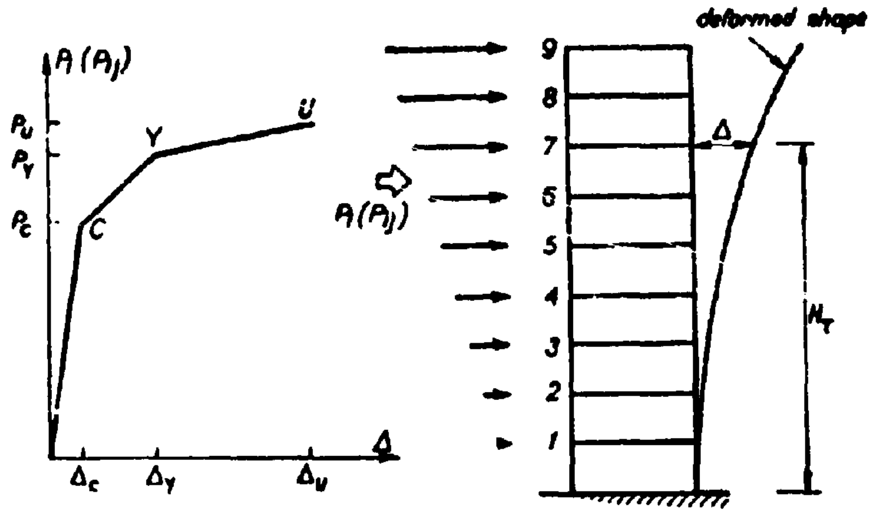


Fig. 4 P - Δ relationship for a structural subsystem

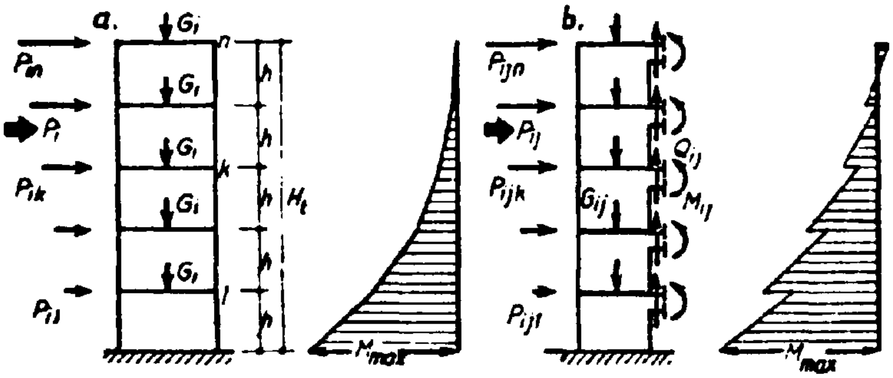


Fig. 5 The load and bending moment distribution on a cantilever core (a) and on a coupled shear wall (b)

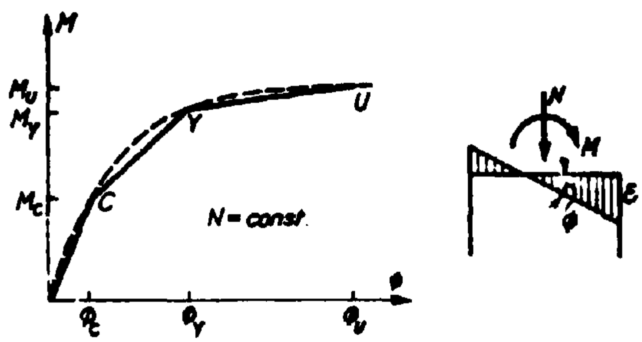


Fig. 6 M - ϕ relationship for a RC cross-section

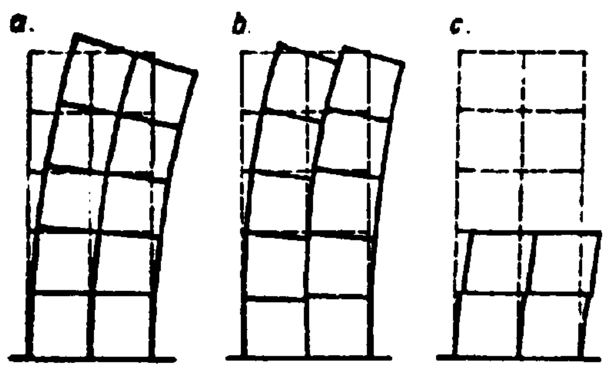


Fig. 7 The lateral deformation of a large panel shear wall when the response is similar to that of a cast-in-place structure(a) and when shear failures develop along the vertical (b) or horizontal (c) connections [8]

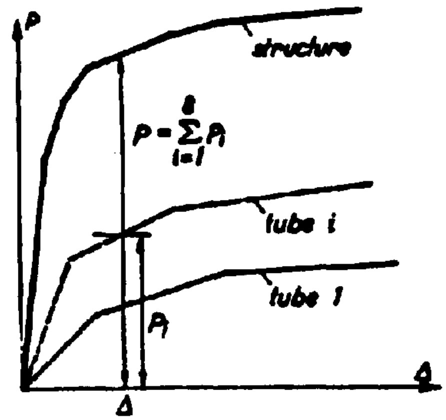


Fig. 8 The superposition of $P_i - \Delta$ relationships of cores developing the P-Δ relationship of structure

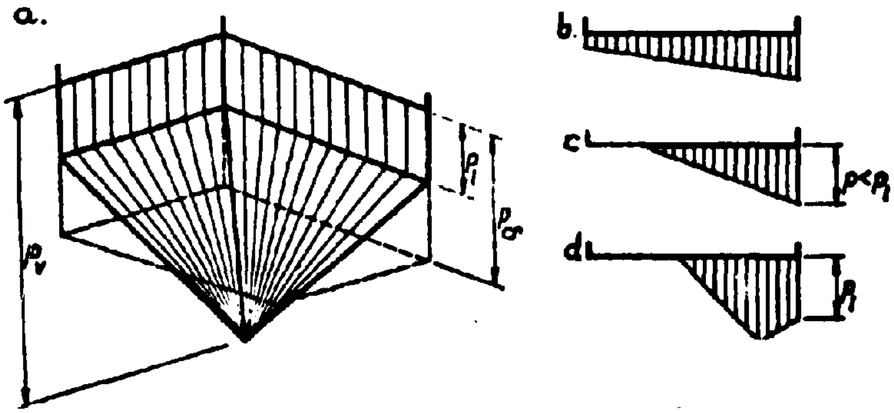


Fig. 9 The limit stresses over the footing surface (a) and possible stress distributions (b..d)

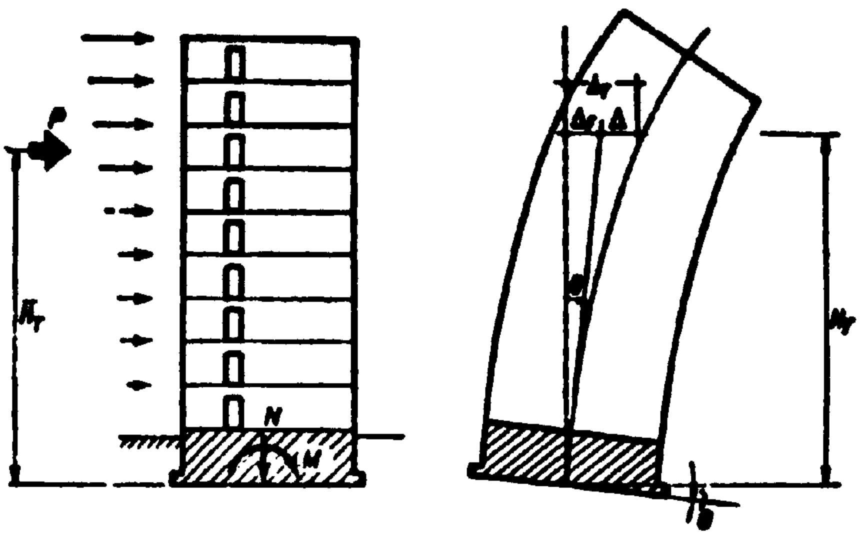


Fig. 10 The lateral deformation of soil-structure system

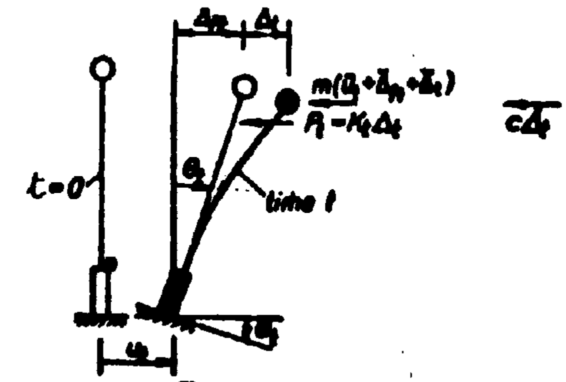


Fig. 11 The inverted dynamic pendulum equivalent to soil-structure system

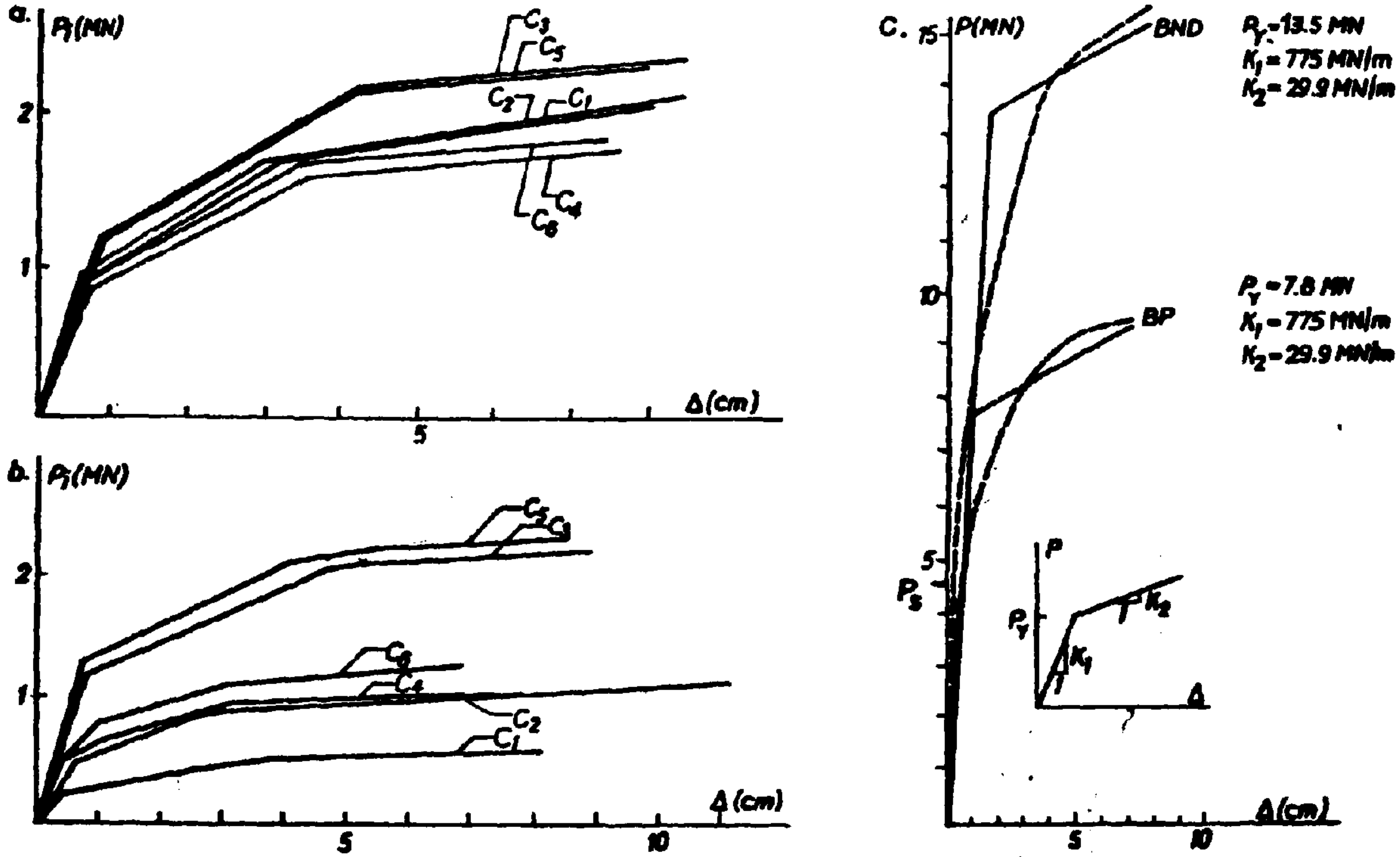


Fig. 12 P_i - Δ relationships of the cores C1 ... C6 in the BND (a) and BP (b) assumptions and the corresponding P - Δ relationships of the structure (discontinuous line) (c)

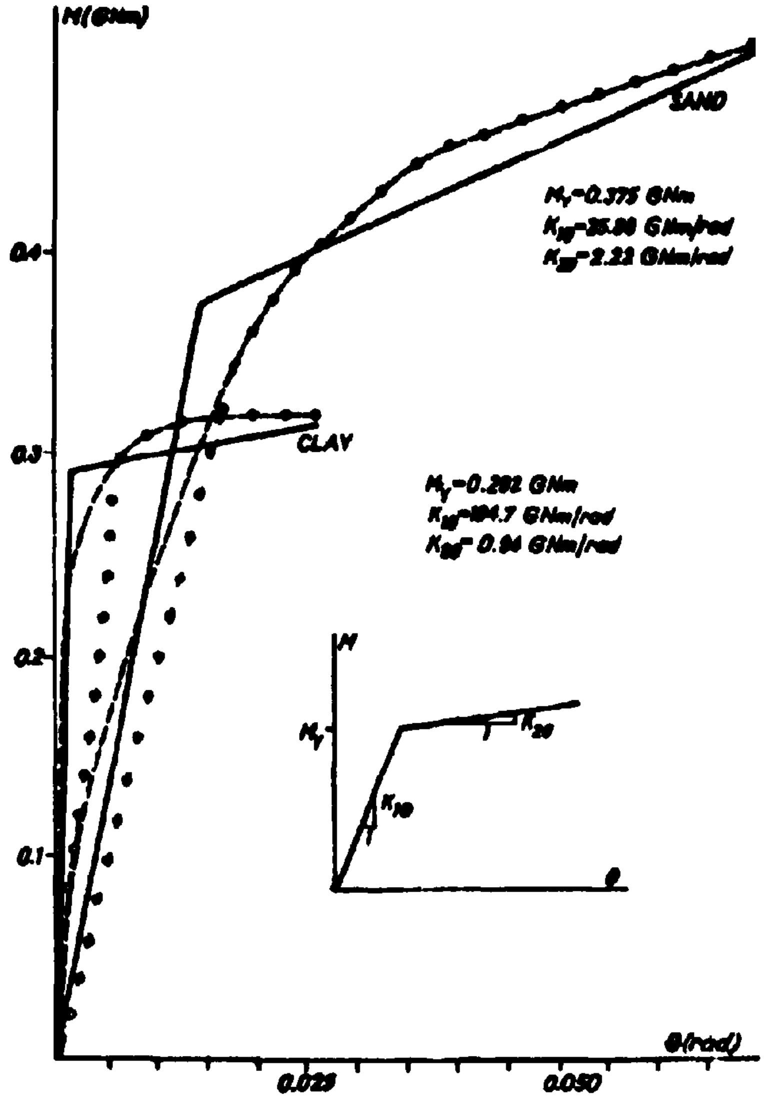


Fig. 13 M - Θ relationships (discontinuous line) for the two types of soil

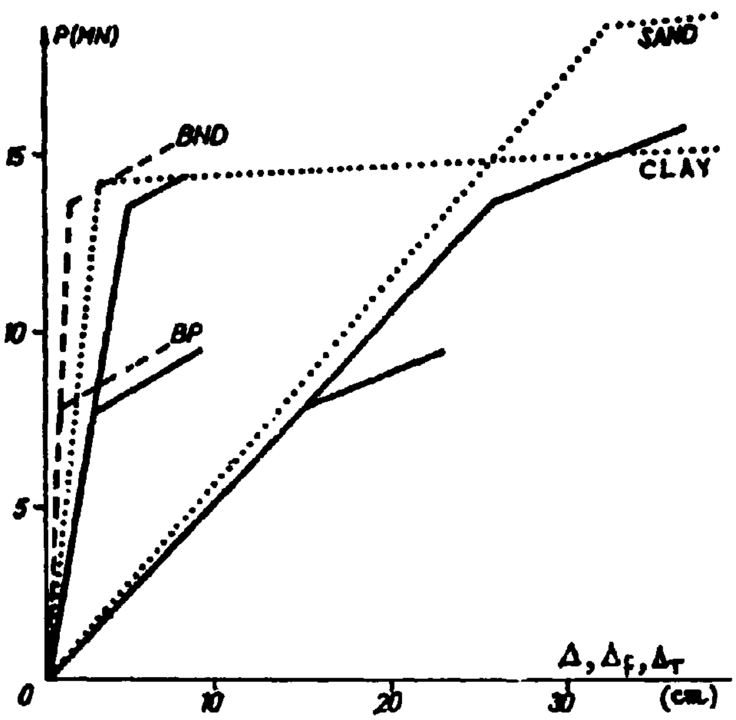


Fig. 14 The P-Δ_T relationships of the soil-structure system (continuous line) developed by superposing the P-Δ relationships of structure (discontinuous line) and the P-Δ_f relationships of soil foundation (dotted line)

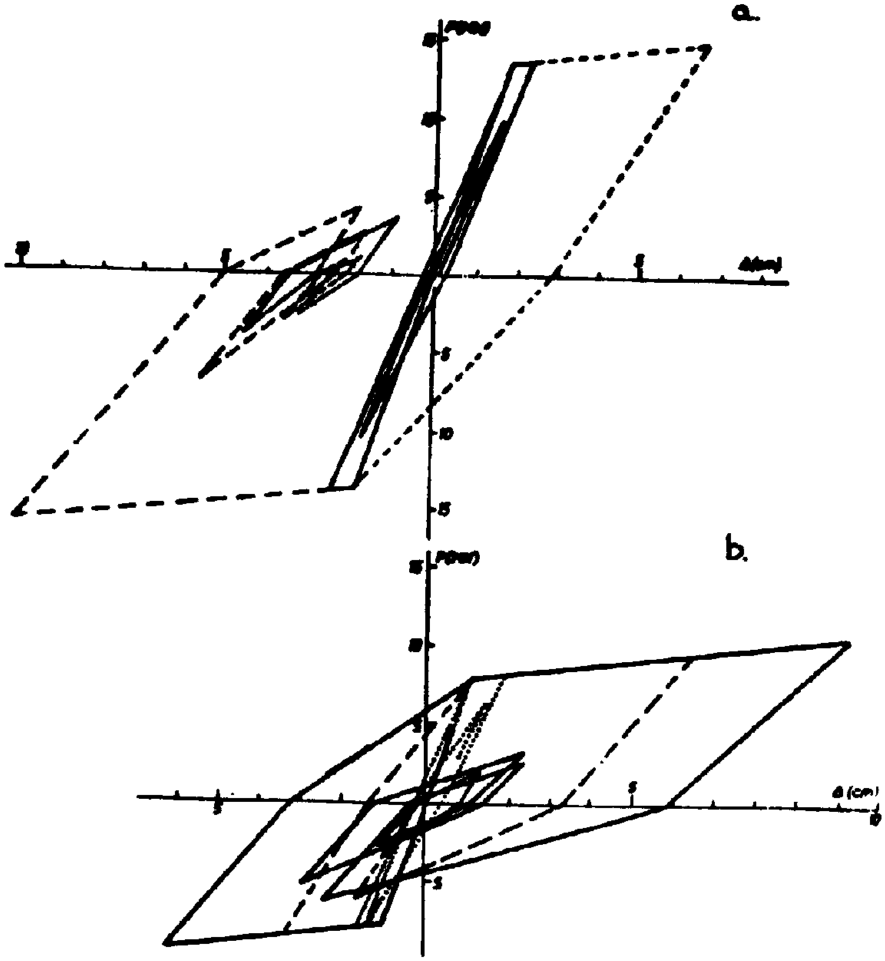


Fig. 15 The hysteretic response of structure to the Bucharest earthquake when the END (a) and HP (b) assumptions are considered. The structure is base-fixed (continuous line) or rotates on clay (discontinuous line)

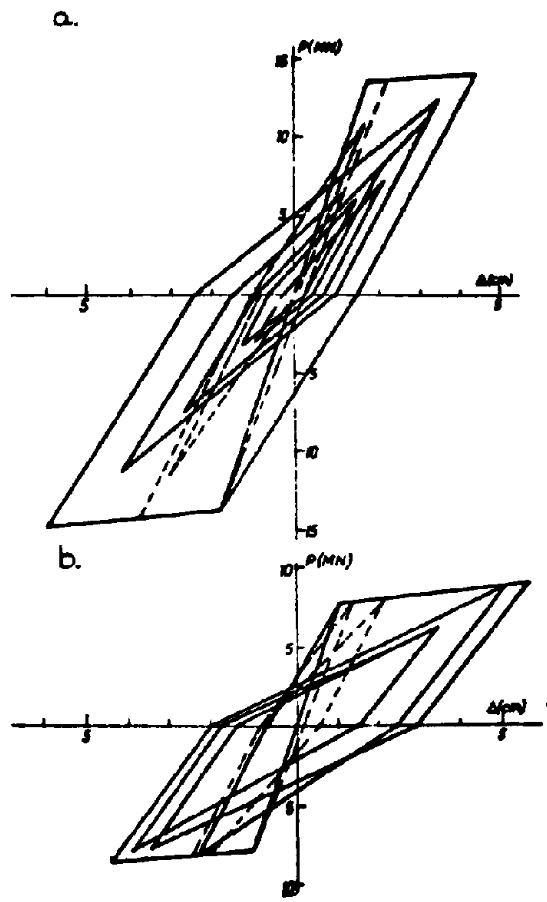


Fig. 16 The hysteretic response of structure to the El Centro earthquake when the BND (a) and BP (b) assumptions are considered. The structure is base-fixed (continuous line) or rotates on clay (discontinuous line)

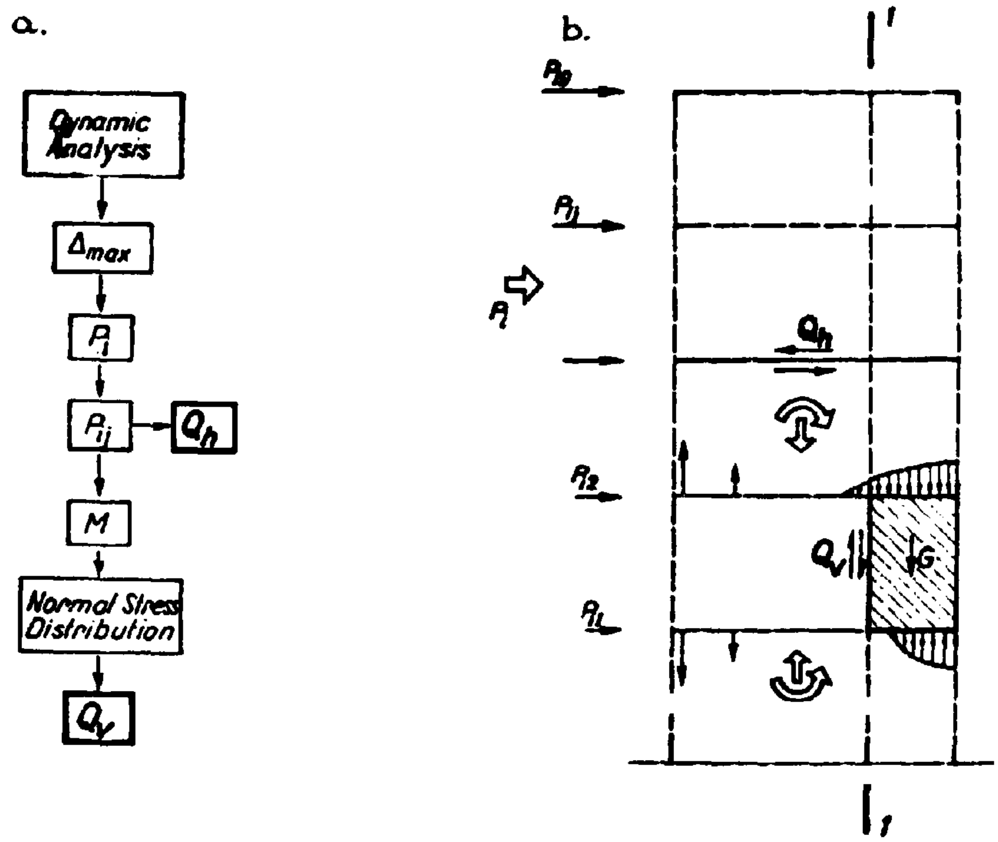


Fig. 17 a,b Approach to evaluate the horizontal and vertical shear forces induced by the earthquake

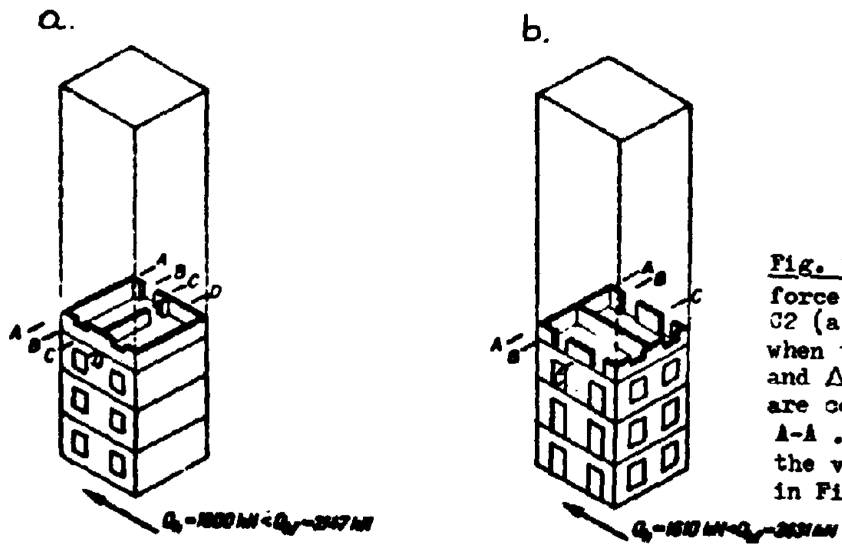
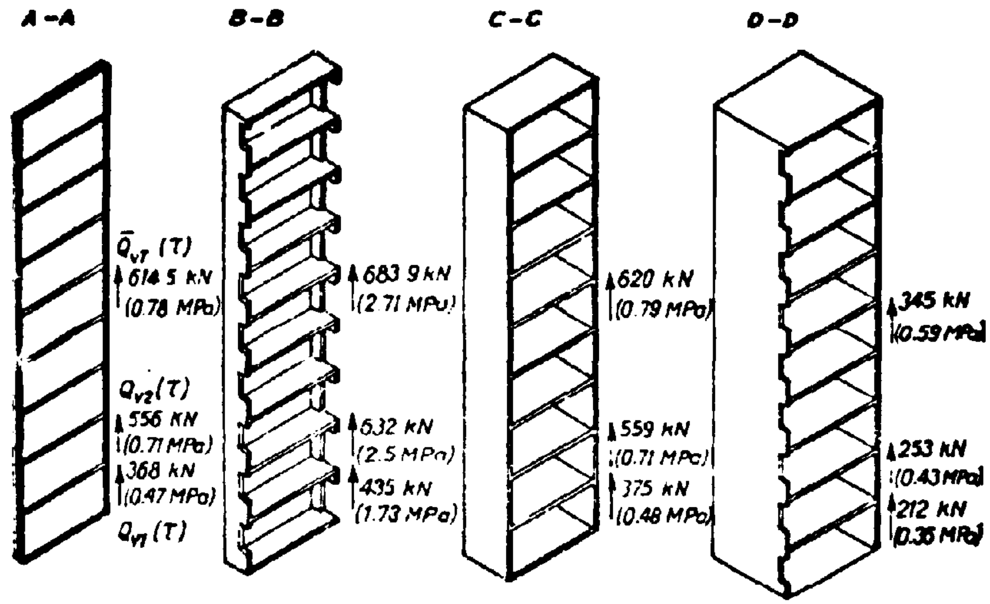
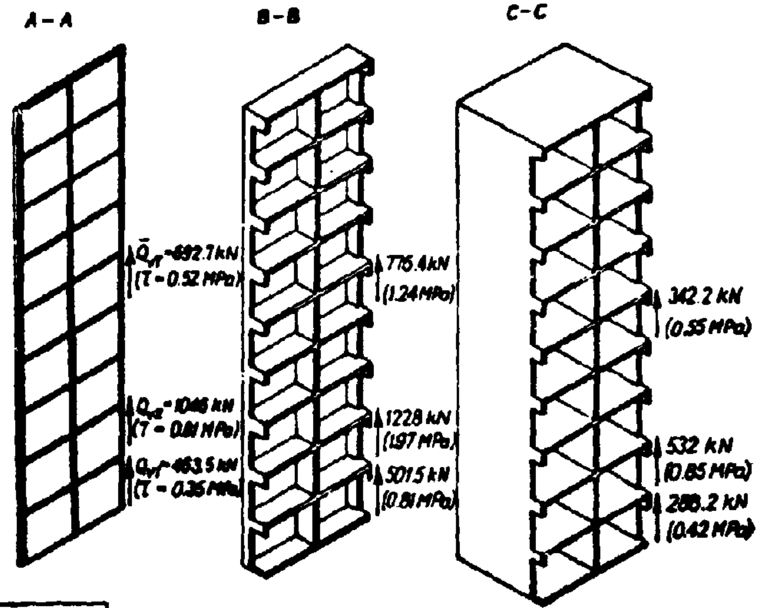


Fig. 18 The base shear force Q_h for the cores C2 (a) and C4 (b) when the END assumption and Δ_{max} = 5.5 cm are considered . A-A ... D-D denote the vertical sections in Figs. 19 and 20



Q'_w = 1037 kN
Q''_w = 1667 kN

Fig. 19 Vertical shear forces Q_{v1}, Q_{v2} and $\bar{Q}_{vT}$ on a storey heigth of the core C2 when the BND assumption and Δ_{max} = 5.5 cm are considered



Q'_{vv} = 1314.4 kN
Q''_{vv} = 2146.7 kN

<u>Fig. 20</u> Vertical shear forces Q_{v1}, Q_{v2} and $\bar{Q}_{vT}$ on a storey heigth of the core C4 when the BND assumption and Δ_{max}=5.5 cm are considered

IV.10 SPATIAL INTERACTION EFFECTS ON BEHAVIOR OF SINGLE-STORY MILL BUILDINGS SUBJECTED TO SEISMIC ACTION

Dragoş Georgescu[x]

Some aspects of the Romanian code P 100-81, now being revised, concerning the effect of a general torsional moment taking into account the asincron nature of seismic action, are discussed in the paper.

Some suggestions are made for the new edition of the code P 100, refering to both defining torsional moment in e - lastic bracing roof structures and calculating such structures.

INTRODUCTION

The Romanian Code P 100-81, now being revised, takes into account the asincron nature of the seismic action, by introdu - cing in the analysis a general torsional moment.

In the case of a single-story mill building, whose masses G at the roof level are distributed as in fig.1.a., the calculus scheme is indicated in fig.1.b.

The shear force S and the torsional moment M are:

$$S = c\sum G \quad (1)$$

$$M = Se \quad (2)$$

where c is the global seismic coefficient, $e = \frac{L}{20}$, when $L > B$

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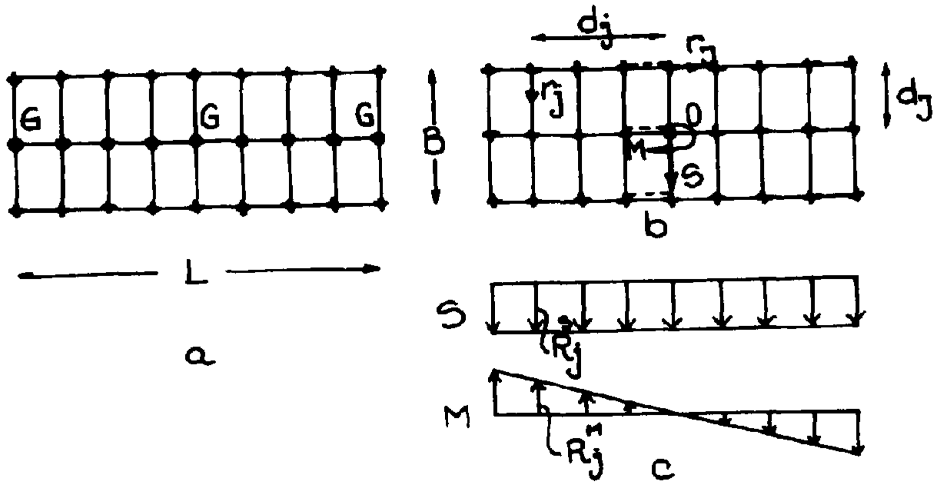

Fig.1

This calculation scheme takes into account a disphragm roof infinitely rigid, which usually is calculated according to the rigid beam on elastic supports method. The shear force S is to be distributed to the transversal frames according to their rigidities r_j and the torsional moment M is to be taken by transversal frames and by vertical bracing, according to their rigidities r_j, r_J and their distances d_j, d_J to the torsional centre O (fig.1,c). It results:

$$R_j^S = S \frac{r_j}{\sum r_j} \tag{3}$$

$$R_j^M = M \frac{r_j \, d_j}{\sum r_j \, d_j^2 + \sum r_J \, d_J^2} \tag{4}$$

$$R_j = R_j^S + R_j^M \tag{5}$$

In the case of the elastic bracing roof structures this analysis method by applying a torsional moment M in the center of the structure can not be applied, since it leads to wrong results. Both the definition of the torsional moment and the analysis method are discussed in the paper. Some suggestions are made for the new edition of the code P 100.

SPATIAL ANALYSIS OF THE BRACING STRUCTURE

In (1) a spatial interaction coefficient α of the structure (fig.2,a) is determined:

$$\alpha = \frac{r_c}{r_{cv}} \tag{6}$$

in which r_c is the rigidity of the transversal frames (fig.2, b) and r_{cv} the conventional rigidity of the bracings (fig.2,c). For diaphragm roofs

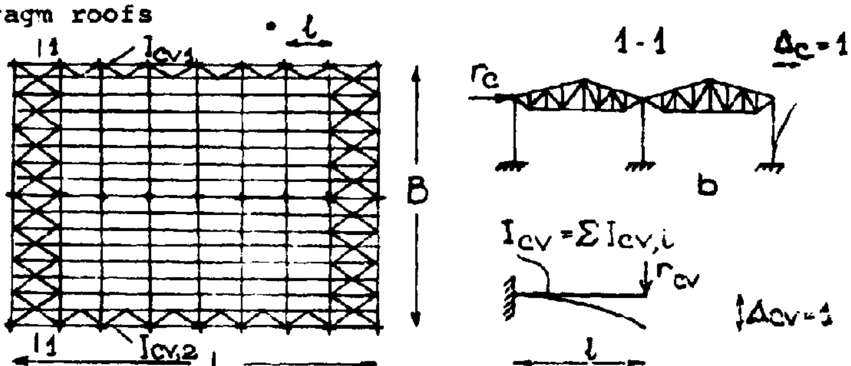

Fig.2

The calculation scheme of a continuous beam on elastic supports results (fig.3.a).

Distribution of the rigidity r_c on the distance l between the trnasversal frames one reaches the scheme of a beam on continuous elastic medium (fig.3.b) with the rigidity:

$$K = \frac{r_c}{l} \tag{7}$$

Accepting the Winkler hypothesis, the following differential equation results:

$$\frac{d^4 y}{d x^4} + 4\beta^4 y = 0 \tag{8}$$

where

$$\beta = \sqrt[4]{\frac{K}{4EI_{cv}}} \tag{9}$$

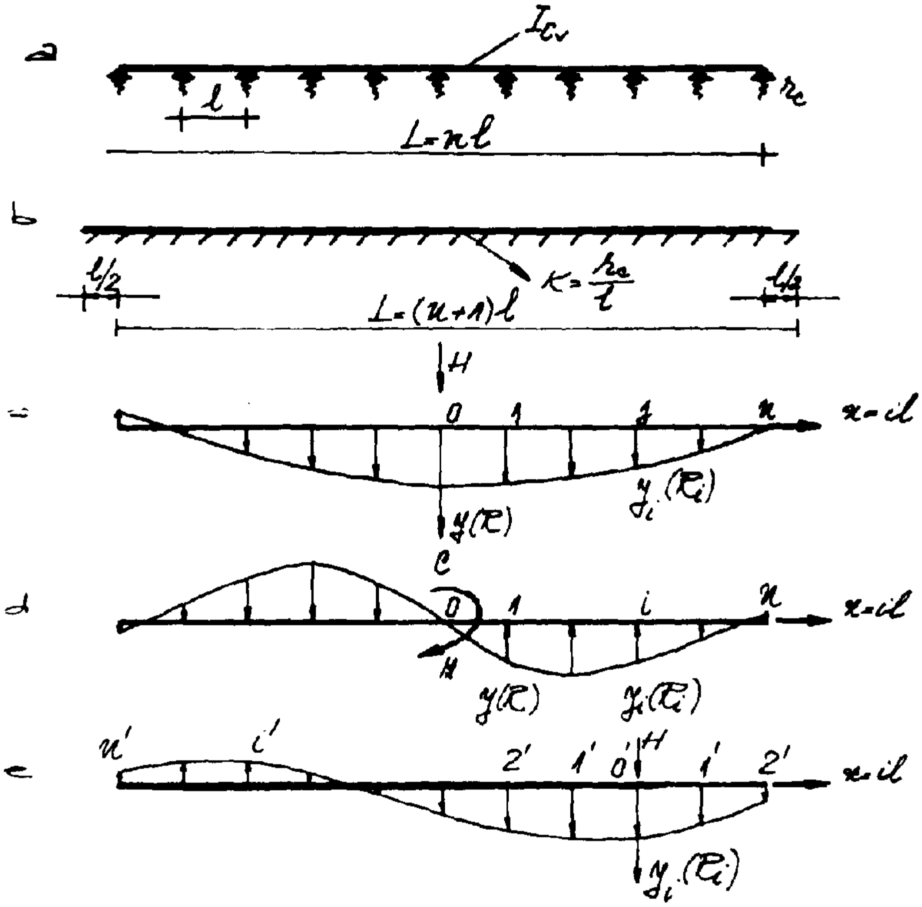

Fig.3

For the loading in fig.3,c is obtained:

$$y = \frac{H}{2K} \beta \varphi_1 \ (\beta x) \tag{10}$$

Introducing in Eq 10 $m = \beta l, x = il$ and $Kl = r_c$, it results /1/:

$$y_i = \frac{Hm}{2 r_c} \varphi_1 (im) \tag{11}$$

where

$$\varphi_1 (im) = e^{-im} (\cos im + \sin im) \tag{12}$$

As a result of Winkler's hypothesis concerning the pro - pertionality between the reactions and deflections y_i and ta- king into account $\delta_{11} = 1/r_c$ where δ_{11} is the flexibility co- efficient of the transversal frames, the following equation is obtained:

$$R_i = \frac{y_i}{\delta_{11}} = \mu H \varphi_1 (im) \tag{13}$$

where:

$$\mu = \frac{m}{2} = 0,466 \sqrt[4]{\frac{1}{\alpha}} \tag{14}$$

For the loading in fig.3,d y_i and R_i are:

$$y_i = \frac{\mu M}{2 r_c} \beta \varphi_2 (im) \tag{15}$$

$$R_i = \frac{4 \mu^2 M}{l} \varphi_2 (im) \tag{16}$$

In Eq 15 and 16:

$$\varphi_2 (im) = e^{-im} \sin. \ im \tag{17}$$

For the loading in fig.3,e y_i and $R_i(1)$ aee :

$$y_i = \frac{Hm}{2 r_c} \gamma \tag{18}$$

$$R_i = \mu H \gamma \tag{19}$$

In Eq 18 and 19:

$$\gamma = \left\{ \varphi_1 (im) + 2 \varphi_4 \left(\frac{c}{l} m\right) \varphi_4 \left[m\left(i + \frac{c}{l}\right)\right] + \varphi_3 \left(\frac{c}{l} m\right) \varphi_3 \left[m\left(i + \frac{c}{l}\right)\right] \right\} \tag{20}$$

where φ_1 is expressed by Eq 12 and φ_3 and φ_4 are:

$$\varphi_3 = e^{-im}(\cos im - \sin im) \tag{21}$$

$$\varphi_4 = e^{-im} \cos im \tag{22}$$

ANALYSIS OF THE BRACING STRUCTURES SUBJECTED TO SEISMIC FORCES.

For the diaphragm roof structures the calculus schemes in fig.4.a and 4.b leads to the same values of the reactions R_i and deflections q_i ; there is no need to explain how the torsional moment M is to be produced.

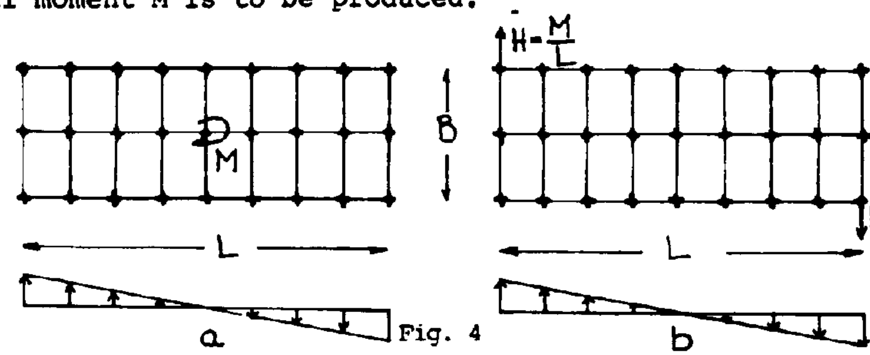

Fig. 4

On the contrary, for the bracing roof structures the schemes in fig.5,a and fig.5,b are completely different and that is why the application of the Eq 2 in this case may lead to wrong results.

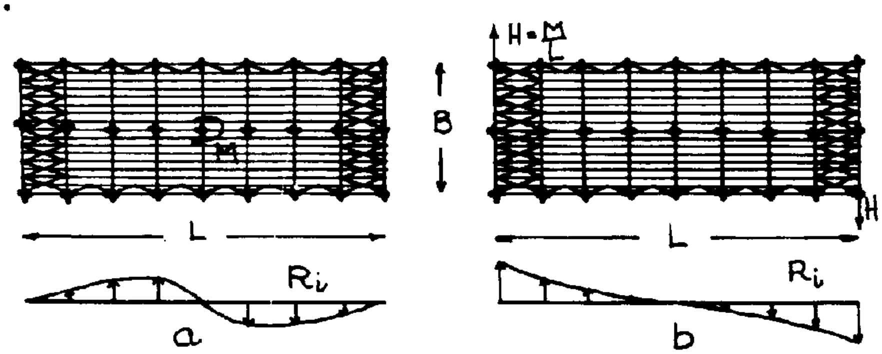

Fig. 5

For such structures it is obviously necessary to specify the starting point of a torsional moment M .

Figures 5,a and 5,b suggest that the origin of the torsional moment M is a couple of forces H applied on the extreme transversal frames.

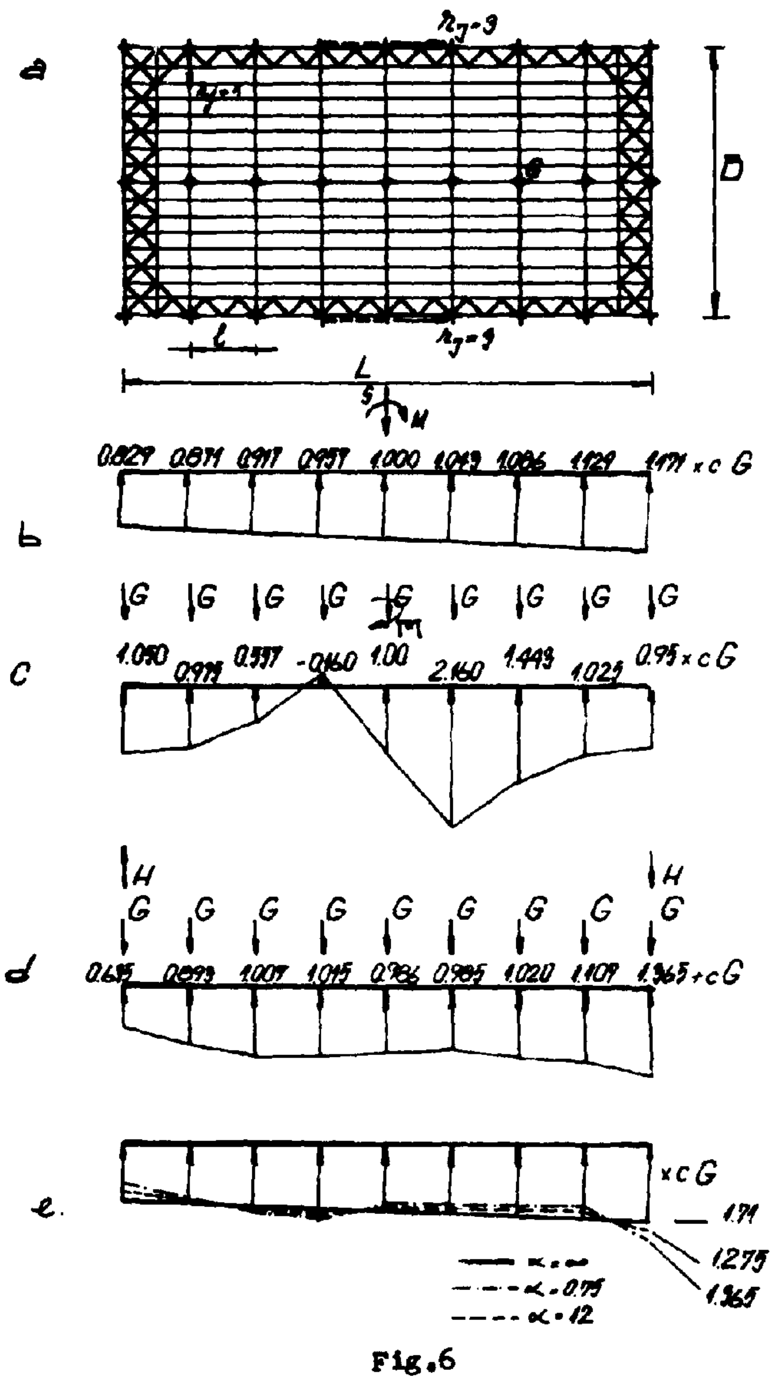

Fig.6

The forces H can be considered as a part of the seismic forces cG acting on this frames:

$$H = \eta c G \tag{23}$$

The coefficient can be calculated from the equation:

$$H = \eta c G L = n c G \frac{L}{20} \tag{24}$$

obtaining

$$\eta = \frac{n}{20} \tag{25}$$

where n is the number of the masses G acting at the roof level (fig.1,a).

Table 1 gives the values η for different values α

Table 1

η	[illegible]	8	10	12	14	[illegible]	18	20
ζ	030	0.40	0.50	0.60	0.70	[illegible]	0.90	[illegible]

For the case in fig. 6,a the reactions R_1 are calculated:

- according to the rigid roof philosophy Eq 3 and 4 (fig.6,b)
- in accordance to the elastic roof philosophy and for $\alpha = 0.75$ both loading the structure by means of a torsional moment M applied in the center of structures Eq 4(fig. 6 c) and by a couple of H forces applied on the extreme portals Eq 19 (fig.6d). Figure 6,e shows values R_1 obtained by applying Eq 19 for different values.

It is to observe:

1. The calculus scheme in fig. 6,d reasonably describes the actual behavior of the structure, since the calculus scheme in fig. 6,c is not to be applied.

2. The higher the coefficient α , the closer the behavior of bracing roofs to that of rigid roofs.

CONCLUSIONS

1. In order to conduct a good analysis of the elastic bracing roof structures, the method of calculating the torsional moment M, in the Romanian code P 100-81, must be changed. The suggestion is made in the paper to replace, the general moment M applied in the center of the structures with an equivalent couple of forces H applied on the extreme frames permits the analysis of,

of, both rigid roof structures and elastic roof structure.

2. For the analysis of the elastic bracing roof structures the beam on continuous elastic medium method described in the paper is applicable.

3. The author intends to continue the studies by including in the analysis of the elastic roof structures the effect of the vertical bracings in the plane of columns.

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IV.11 SEISMIC RISK EVALUATION FOR BUILDINGS INCLUDING SOIL-STRUCTURE INTERACTION EFFECTS

Dan Mircea Ghiocel[x]
Adrian Popovici[xx]
Dan Ghiocel[xxx]

At the present time a strong tendency exists to approach the seismic analysis of structures within a stochastic frame - work. Stochastic modelling allows more rigorous analyses and offers the possibility of seismic risk assessment. In this way the safety level of structures, can be controlled and maintained at a minimum level, corresponding to the acceptable risk of human society.

For embedded and buried buildings the seismic response is essentially influenced by the dynamic characteristics of soil deposit.

The paper presents a probabilistic seismic analysis including the stochastic idealization of both earthquake motion and soil properties. The seismic motion at the free surface of the soil deposit is idealized as a nonstationary Gaussian stochastic process, while the soil dynamic characteristics are modelled as random variables with different probability distributions.

Two case studies are presented: a large underground building of a metro station, investigated with and without adjacent buildings, and a shallow embedded massive building.

The analyses are carried out by finite element computational models. Nonlinear and hysteretic behavior of the soil is included. The results obtained have shown the strong influence of soil dynamic properties. The presence of the multistory buildings in the vecinity of the metro station severely affect the seismic response of the buried structure-.

For comparison, deterministic analyses were carried out for both case studies. The results suggest the necessity

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of parametric studies in the case of deterministic approaches.

INTRODUCTION

At the present time a strong tendency exists to approach the seismic analysis of structures within a stochastic frame work. Stochastic models consider the random nature of earthquake ground motions, permitting a profound seismic analysis based on more rigorous assumptions concerning the idealization of excitation and the evaluation of structural response. Moreover, stoc - hastic analyses offer the possibility of seismic risk assessment in terms of probability. In this way, the safety level of structurescan be controlled and maintained at a minimum level, corresponding to the acceptable risk to human society.

Generally, many authors preffer probabilistic seismic analyses to stochastic modelling of earthquake ground motions,other parameters of structural model being assumed as deterministic.The maximum response of a structure is idealized as a random variables with extreme probability distributions of expressed by mean, coefficient of variation and upper fractile values.

For relatively massive buildings with rigid structures, and especially or embedded and buried buildings, the seismic response is essentially influenced by the deformability and dissipative characteristics of foundation soil. In these situations, the computational model has to incorporate the seismic soil-structure interaction effects. The significant uncertainties concerning the dynamic characteristics of soil deposit, involve the extension of stochastic modelling on soil properties.

The paper presents the results of two probabilistic seismic analyses including the stochastic idealization of both earthquake motion and dynamic characteristics of soil.

COMPUTATIONAL MODEL

The seismic excitation was modelled as a nonstationary Gaussian stochastic process, defined by a set of artificially generated accelerograms based on the Ruiz-Penzien digital simulation technique.

The dynamic characteristics of soil-rigidity and damping were considered as random variables with uniform or normal probability distributions, simulated by Monte-Carlo methods.

The probabilistic seismic analyses included three steps, as follows:

(1) The assessment of a stochastic seismic signal at the free surface level of soil, related to specific site conditions;
(2) The evaluation of earthquake ground motion with depth;
(3) The estimation of probabilistic structural response, including seismic soil-structure interaction effects.

The seismic analysis was carried out by a finite element computational model implemented in FLUSH code. The assumption of vertical propagation of S and P seismic waves was accepted.

The nonlinear and hysteretic behavior of soil was modelled by the Seed-Idriss equivalent linear procedure. Transmitting and viscous boundaries were used to simulate the semi-infinite nature of soil. Using the complex response method, the nonproportionality of damping matrix of soil- structure system was included.

To point out the influence of the variability of soil properties on the seismic response of a structure, the stochastic modelling was adopted for only seismic input; mean values were introduced for soil parameters.

Supplementary seismic analyses were performed in order to allow a comparison between deterministic and probabilistic response of structures; mean values and extreme values (mean values + 30-40%) were considered.

Some special aspects referring to the perturbations of seismic arriving motions and structure-soil-structure dynamic interaction in dense urban areas are discussed.

CASE STUDIES. RESULTS AND DISCUSSIONS

The first case study refers to a large underground building of a metro station in Bucharest. The reinforced concrete structure is of box type with three floors(figure1). The finite element computational model is presented in figure 2.

Two situations of the metro station were investigated: (1) withou adjacent buildings (2) with adjacent buildings of both side.

The soil deposit consisted of quasi-horizontal layers, The vertical dimensions of the soils finite elements were chosen to correctly transmit through the mesh the vertically propagating seismic shear waves S up to a maximum of 10 Hz.

The transmitting boundaries of the discrete model were placed at a distance of more than 100 ft far from lateral structural walls, in order to include the effects of secondary nonlinearity of surrounding soil behavior due to seismic soil-structure interaction.

The set of simulated accelerograms at the station site were scaled to a maximum acceleration value of 0.2 g, corresponding to the seismic intensity map for Bucharest. The shear modulus of soil was modelled as a random variable, with the mean value calculated from the experimental observations and the coefficient of variation 30%.

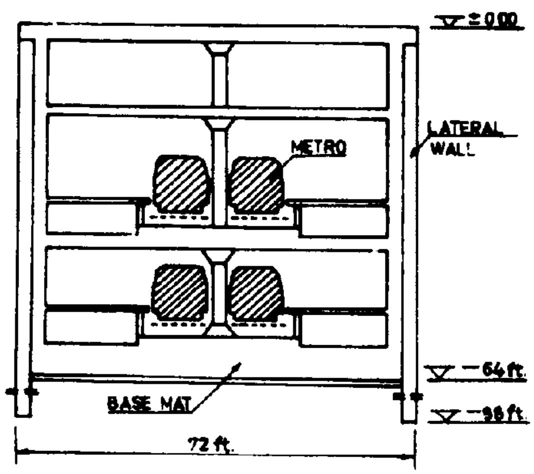

Figure 1
Cross-section of underground metro station

The results show the strong influence of dynamic soil deformability on the seismic response of a buried structure. The effect of kinematic soil- structure interaction was especially significant for soft clay layers. At levels of these layers, in the vicinity of lateral structural walls, some maximum shear strains were smaller with more than 5 % from those computed in the free field.

For the probabilistic seismic response of the structure expressed in bending moments and shear forces, the coefficients of variation had values ranging in the interval 20-60%. For assumptions of the stochastic idealization of only seismic excitation, the values of coefficients of variation were strongly reduced between 5 and 10%.

If random variation of soil parameters were included within the probabilistic analysis, the structural response would be amplified for upper fractile values 95% with 25-75%.

For comparison, deterministic seismic analysis was carried out; the excitation was defined by the accelerogram recorded at INCERC-Bucharest station, in the NS direction during the March 4, 1977 Vrancea earthquake. The maximum acceleration value was 0.22g. For soil shear rigidity, extreme values with mean values

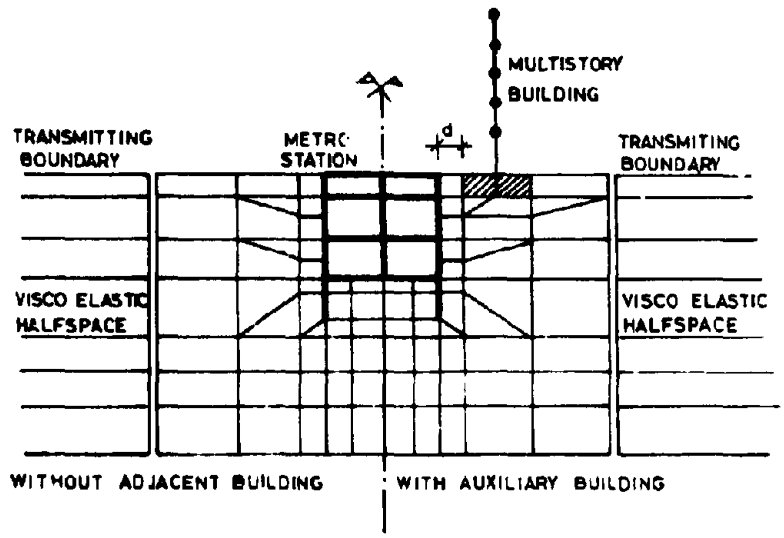

Figure 2

Schematic view of finite element computational model-FLUSH code.

+ 33% were considered.

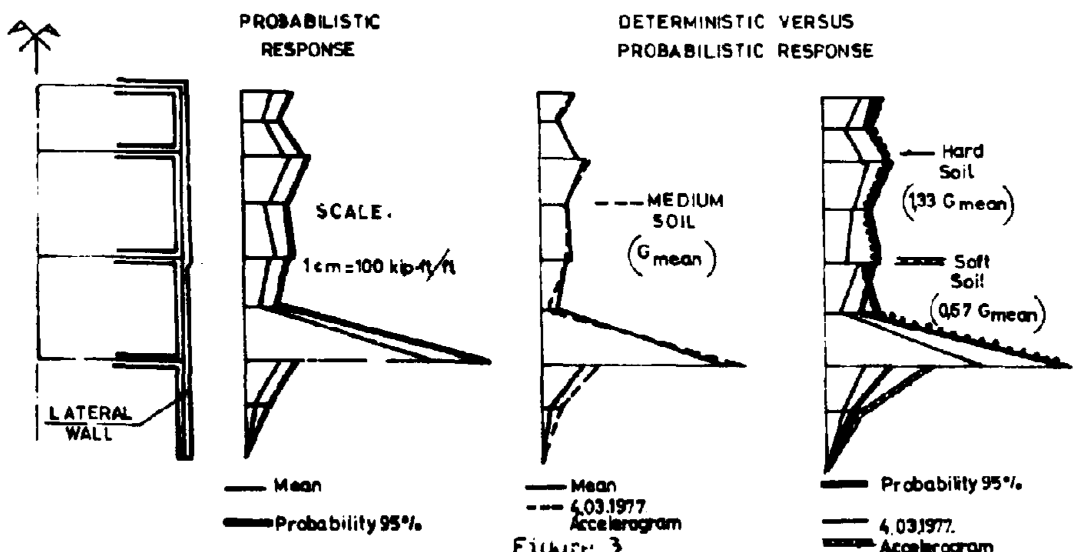

Figure 3

Maximum bending moments (absolute values) along lateral wall

Figure 3 presents comparative plots of maximum bending moments in the lateral wall, obtained from the probabilistic analysis, with probability of 95% and deterministic analysis.The diagrama suggest the importance of soil parametric studies in seismic analyses of buried structures. For mean values of soil rigidity, the deterministic structural response was close to the mean of probabilistic response. If the extreme values of soil were considered, the maximum response obtained by the deterministic approach would be placed in the vicinity of probabilistic maximum response defined with the upper fractile 95% . This result was partially generated by the frequency content of the March 4, 1977, accelerogram, rich in spectral components at low frequencies. The frequency content is specific for the site where the accelerogram was recorded, but is not probable for metro station site. Spectral amplitudes of 1977 earthquake at low frequencies correspond to upper fractile values of the probabilistic spectrum station site (figure 4).

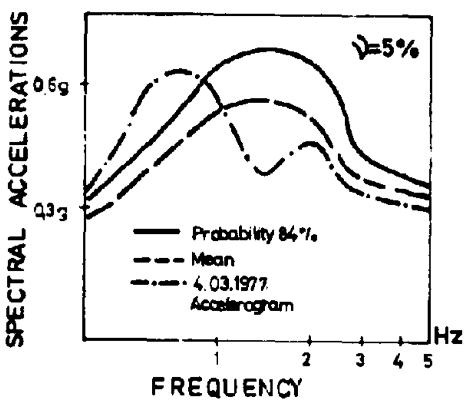

Figure 4
Response spectra

The presence of a multistory building near the metro station significantly influenced the seismic response of the buried building (figure 5). The reverse influence of the metro station on the seismic structural response of the adjacent multistory buildings was not significant: maximum bending moments increased less than 10%. This assertion might not be true for horizontally or inclined propagating seismic waves, in which case the underground station would act as disturbing obstacle for seismic arriving waves.

The second case study presents a massive shallow embedded structure of a nuclear reactor building.

The seismic excitation was modelled, as in the first case study, by a set of artificially generated accelerograms, specific to local soil conditions of the site. The dynamic characteristics of soil, shear modulus, and the damping factor, were idealized as random variables for three assumptions: uniform distributions in the intervals defined by mean values + 10 % and normal distributions defined with the probability 84% .and

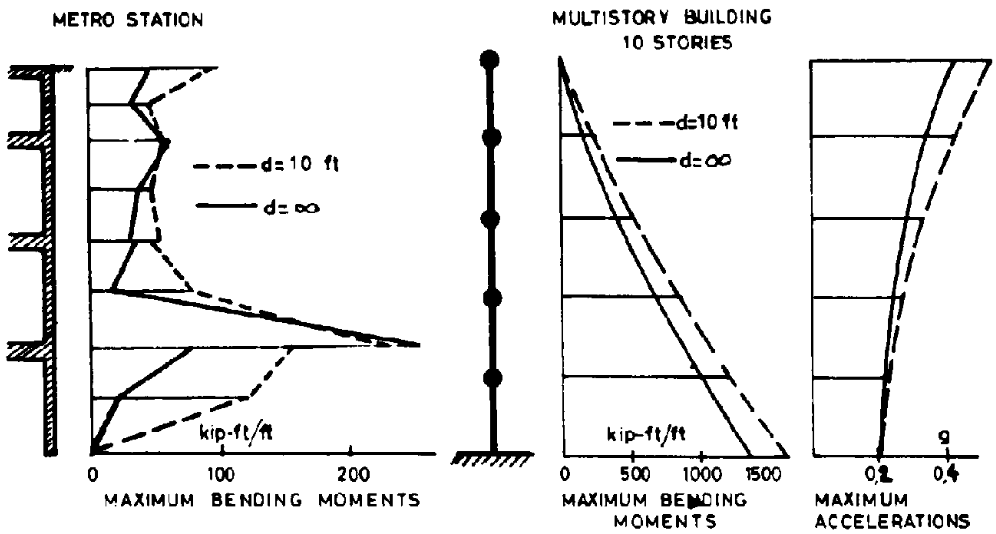

Figure 5
Effects of metro station-soil-adjacent building seismic interaction

98 % staying within the limits mean values + 40%.

The coefficients of variation of probabilistic maximum structural response have presented values up to 25-33 %, depending on the assumptions accepted in soil parameters stochastic idealization. If the probabilistic response of structure were defined with a probability of 90%, the random variation of soil in the analysis would have increased the response with 15-25 %.

For comparison, a deterministic seismic analysis based on design spectrum compatible accelerograms was performed; extreme values of the soil dynamic properties were considered (mean values + 40 %).

The deterministic response was greater than the probabilistic response, upper fractile 90%, with 20-30%.

FINAL REMARKS

1. The comparison of the results obtained by deterministic and probabilistic analyses confirms the conservative tendency of deterministic seismic analyses.

2. The sensitivity of seismic risk of a structure at the variation of design parameters can not be evaluated by a deterministic approach with certainty.

3. For rigid massive buildings, and especially for embedded or buried buildings, the influence of dynamic characteristics of the surrounding soil on the structural seismic response is essential.

In the case of a deterministic approach, parametric studies are necessary; using extreme values (mean + 30-40%) for soil dynamic properties, reasonable values of maximum response close to upper fractile values of probabilistic response of a stochastic approach (90-95%) are obtained.

4. In dense urban areas the effects of seismic interaction between adjacent buildings can significantly affect the response of structures. For the case studies presented here in, the presence of the multistory building in the vicinity of the metro station severely influenced the response of the buried structure.

5. Further studies will refer to the effects of surface and inclined seismic waves on the seismic response of the structures nonsynchronous character of soil motion along the longitudinal axis of the metro station, etc.

IV.12 STUDIES CONCERNING THE BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS WITH OPENINGS UNDER ALTERNATING HORIZONTAL LOADING

Gh.Ciuhandu[x]
Afion Mihăescu[xx]

The paper describes an ample experimental program carried out between 1982-1984 at INCERC - Timişoara Branch in co-operation with the Reinforced Concrete and Buildings Chair of the Timişoara Faculty of Civil Engineering.

Experimental research in the design stage, carried out on five models (1: 2.75) of shear walls with openings, provided results regarding ductility, the amount of energy absorbed, stiffness and bearing capacity of the shear walls tested, lengths of the zones of plastic hinges in coupling beams and walls.

The results of this experimental program allowed recommendations to be formulated in addition to provisions made by the Romanian Instructions P 85-82 for this type of shear walls.

1. INTRODUCTION

Monolith RC shear-wall structures are at present one of the most widely adopted structural solutions for multistory buildings.

The behavior of these structures under dynamic horizontal loading due to wind or earthquakes is a factor where the integrity of the building is concerned; since the value of the dynamic loads acting on a structure is proportional to the stiffness of the structure, the dynamic load on a shear-wall structure is usually considerable, even though it is compensated by an increased strength capacity of the shear walls.

As to the energy induced by the dynamic load, shear- wall structures absorb this energy especially in the strongly affected areas near the base; in the case of shear walls provided with openings, a kind of storage and subsequent dissipation of energy is favoured by the presence of coupling beams that are likely to

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crack and plastic . This would provide sufficient safety during service.

The ability of a structure to undergo deformation in the plastic range, or the ability to store energy, respectively, characterizes a factor called ductility; from this point of view, shear-wall structures are less ductile than, for example, framed structures. Increased ductility of shear-wall structures, in order to provide better chances of survival in case of heavy seismic loading, is at present a major concern of most theoretical and experimental research, and it is also dealt with in this paper.

As to the behavior of monolith shear walls under seismic loading, the experience gained by Romanian experts in the 1977 earthquake has shown that a strictly elastic behavior of shear-wall structures under heavy seismic action cannot provide a safe and efficient criterion for the design and construction of such structures in earthquake prone areas. Once this objective was achieved, it became more and more necessary to work out experimental programs that would provide data on the actual behavior up to breakdown of shear walls under seismic loading, and would thus provide a better means od post-elastic analysis and a more adequate configuration of the shear walls.

This paper presents a study of monolith shear wall models with openings subjected to constant gravitational loading and increasing, alternating horizontal loads that were applied in cycles up to failure.

2. THE EXPERIMENTAL PROGRAM. CHARACTERISTICS, METHODS.

Experimental research mainly focused on aspects concerning the post-elastic behavior of shear walls with openings:

(a) the cracking and degrading of shear walls under alternating horizontal loads increasing up to failure

(b) the manner of fatigue and values of bearing capacity;

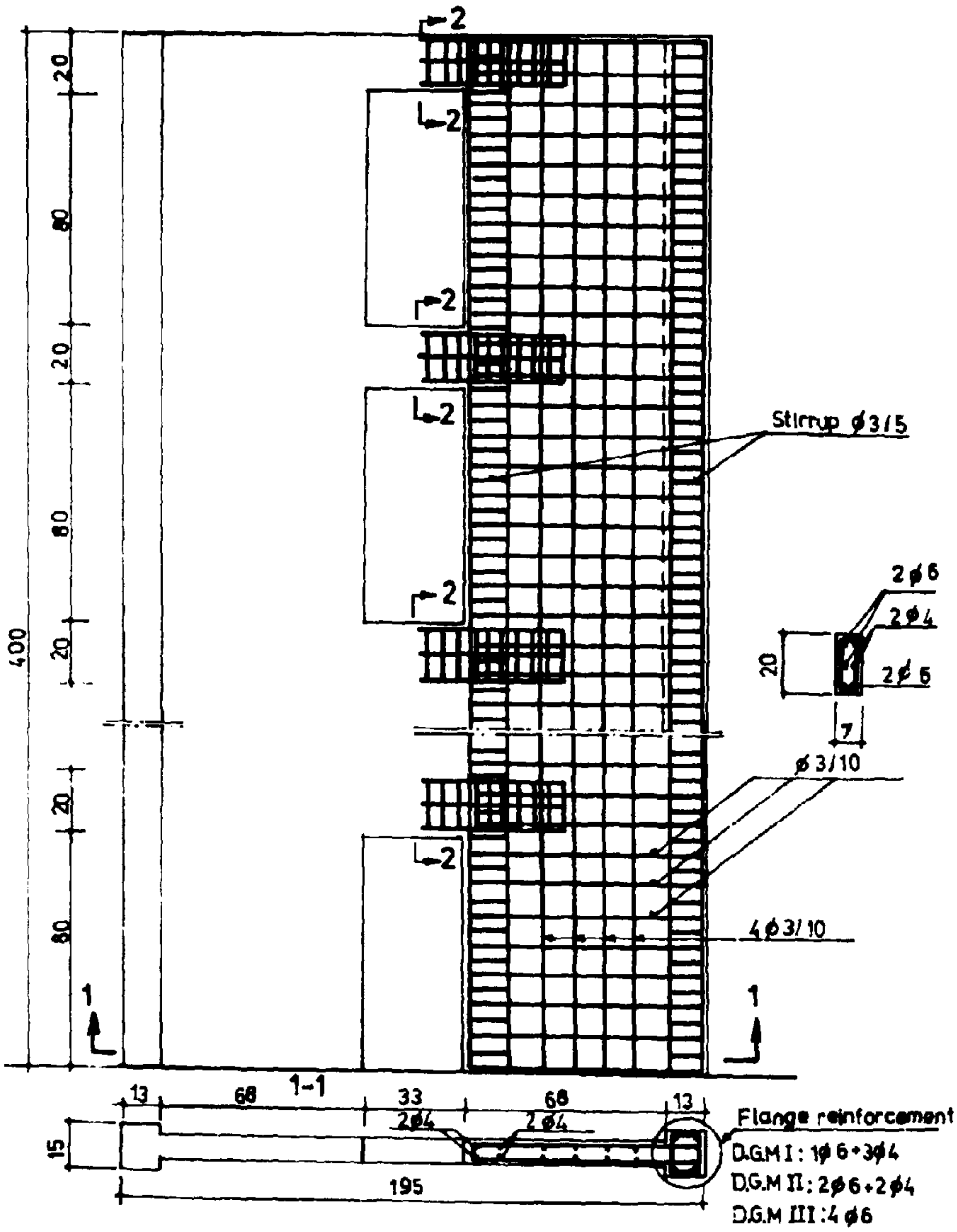
(c) formation and order of occurrence of plastic hinges;

(d) the influence of the steel ratio of the longitudinal reinforcement in flanges on the global behavior of the shear wall;

(e) the influence of concrete confinement due to tightening of the stirrups in flanges and at the ends where door openings were provided;

(f) experimental assessment of such characteristics as the ductility, energy absorbtion and stiffness of the shear walls.

The prototype adopted was that of a monolith shear wall of a four-story residential building (H=11m) with a cell - like



2
2
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2
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80
20
80
20
20
80
400
Stirrup ϕ3/5
2ϕ6
2ϕ4
2ϕ6
20
7
ϕ3/10
4ϕ3/10
1
1
13
68
1-1
33
68
13
2ϕ4
2ϕ4
15
195
Flange reinforcement
D.G.M I : 1ϕ6+3ϕ4
D.G.M II : 2ϕ6+2ϕ4
D.G.M III : 4ϕ6

Fig.1

structure. The 5.40 m long and 0.20 m deep shear wall was provided with a row of symmetrical openings.

The dimensions chosen for the openings were 0.90x2.20 m ; to the normal height of the opening the sum of the possible depths of the floor slabs was added, assuming either subsequent casting of the floors or the use of prefabricated slabs.

A story height of 2.75 m and the above dimensions of the openings yielded coupling beams 0.55 m high and 0.90 m wide.

The prototype was made more stable by stiffening its ends with flanges of 0.35 x 0.40 m containing, alternatively, three kinds of longitudinal reinforcement and two kinds of transversal reinforcement, respectively.

Two kinds of reinforcements were used in the coupling beams: longitudinal reinforcement and stirrups and diagonal reinforcement, respectively.

Five models (geometrical scale 1 : 2.75) of this prototype were used for the experiments. They included shear walls provided with symmetrical openings, having flanges with various types of reinforcement and coupling beams reinforced either longitudinally (DGM I - DGM IV) or diagonally (DGM V). Figs. 1 and 2 provide the geometrical characteristics and the configuration of the experimental models.

Modelling of the reinforcement of the tested shear walls was carried out according to instructions provided by /1/ and included the following: double-layer grid reinforcement in the usual ratio, differentiated bar diameters in the web and in the flanges, identical reinforcement (with longitudinal or diagonal bars) of the coupling beams; the first three models differed only in the reinforcement ratio of the flanges (DGM I - 0.o0%; DGM II - 0.14%; DGM III - 0.20%. See fig.1).

Shear wall model IV (DGM IV) has essentially the geometrical and configurational characteristics of model II (DGM II); it differed from this in a closer spacing of the stirrups in the flanges and at the ends near the openings of the walls which was adopted in view of concrete confinement.

Model V differs from Model IV only in the type of reinforcement adopted for the coupling beams i.e., diagonal instead of longitudinal, but with the same ratio of reinforcing steel as in the coupling beams of Model IV (Fig.2).

OB 37 steel bars were used for reinforcement, and concrete strength was in the 175 - 256 daN/cm^2 range.

The models were cast in situ in a vertical position,star -

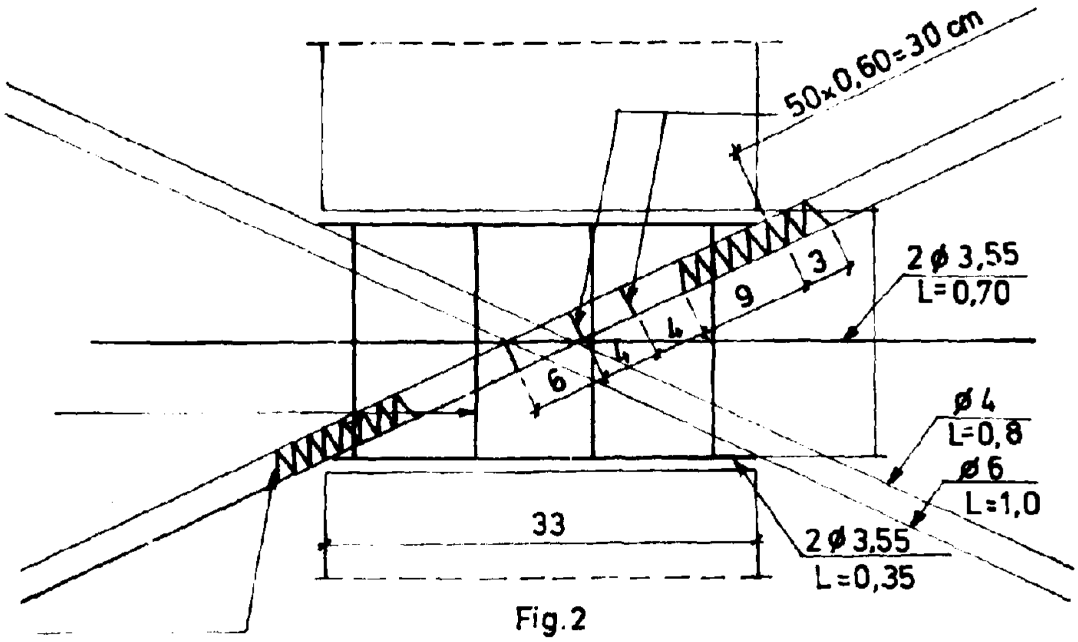

Fig.2

ting from the foundation and continuing over the four storeys in the proper shear wall at a working rate of one storey per two days.

Testing of the models was carried out on a test stand according to the loading scheme indicated in Fig.3.

To better simulate the triangular distribution of the conventional static load of an earthquake, horizontal loads were applied statically, in cycles and alternatively as three concentrated forces of the same intensity P, acting along the axes of coupling beams 1,3 and 4. Gravitational loads were applied prior to horizontal loading using two presses located in the centers of gravity of the walls (at their top) which induced a unit compressive force of 15 daN/cm^2 that was constant over the total height of the model throughout the testing period.

3. BEHAVIOR OF THE EXPERIMENTAL MODELS

The models'behavior, assessed from a characteristic crack pattern (Fig.4) and from visual observation throughout testing,

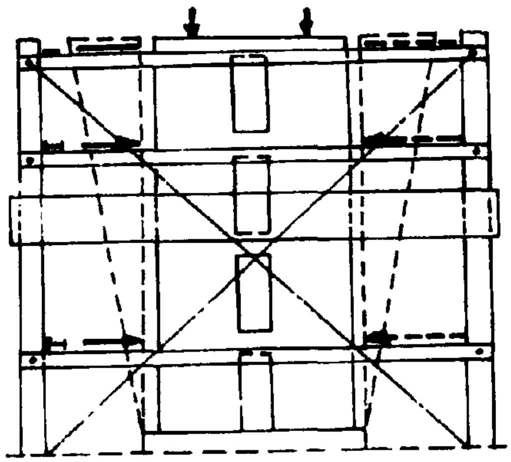

Fig.3

was characterized by the following features, in the order of their occurrence:

- an initial elastic behavior ending with the appearance , of cracks at the ends of the coupling beams, due to bending, extending over the entire height of the beams;
- yielding of the stretched reinforcement in the most strongly stressed beam (usually beam 2);
- appearance and widening of horizontal cracks in the critical section at the base of the shear wall, due to general bending, concomitantly with the occurrence of yielding in the stretched reinforcement of the flanges.
- onset of creep in the coupling beams by the development of diagonal cracks due to shearing in the fields of the beams (particularly in the fårst three floors); in this situation the beams can no longer receive the additional shearing forces and any further increase of the horizontal loading is taken over by the walls;
- onset of creep in the walls by the development of horizontal cracks due to general bending of the shear wall especially on the first floor;
- buckling of the compressed reinforcement in the flanges of the non-confined members, at the final cycles of horizontal loading;
- appearance and development, at all tested members, of horizontal cracks at the casting joints between floors 1 and 2.

The aspects described above generally conform with to Paulay's procedure for a step-by-step evaluation of the elasto- plastic

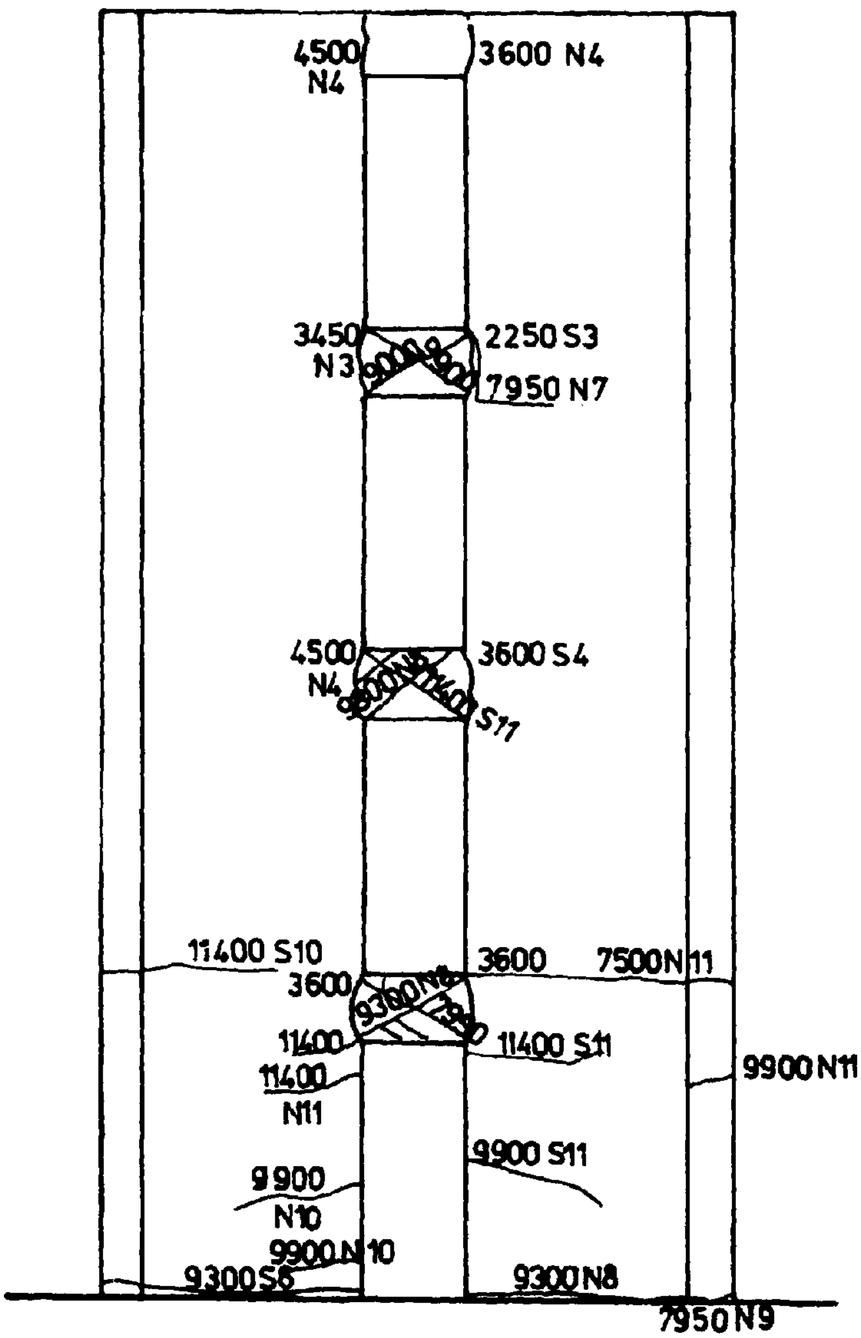
4500 N4
3600 N4
3450 N3
2250 S3
7950 N7
4500 N4
3600 S4
S11
11400 S10
3600
3600
7500 N 11
11400
11400 S11
11400 N11
9900 N11
9900 S11
9900 N10
9900 N 10
9300 S6
9300 N8
7950 N9

Fig.4

behavior of shear walls with openings /2/.

During this process of degradation, plastic hinges appeared and developed in the coupling beams and walls. In the case of a reinforced concrete member, plastic hinges are known to form in those sections where yielding of the stretched reinforcing bars is produced, concomitantly with the appearance of plastic deformations in the compressed concrete due to micro-cracking in the cement stone. In the case of the tested members, even though yielding of certain stretched reinforcing bars in the section near the base of the shear wall occasionally and sporadically preceded yielding of the bars in the coupling beams, cracking and degradation of the concrete always began and advanced more rapidly at the extremities of the beams than at the shear wall base. Thus, the two defining conditions for plastic hinges to be formed in a reinforced concrete member-yielding of the stretched reinforcing bars and plasticization of the compressed concrete - were in all cases first encountered at the coupling beams'ends and then at the base of the walls of the tested members.

Maintaining this particular order in the formation of plastic hinges is an important requirement when an increased capacity for energy dissipation will be obtained through the post-elastic deformation of the coupling beams, and when a final elastic line of the shear wall is to be secured on this basis, thus providing a chance for the latter to "survive" a heavy earthquake.

The practiced of designing the coupling beams as the main energy dissipators before the walls become plastic, ensures an elastic behavior of the latter up to relatively high values of horizontal loads, and thus provides a high degree of protection against damage. The practical experience gained by Romanian experts after the 1977 earthquake has shown tnan the damage done by an earthquake can be remedied comparatively easy and less expensively in the beams than in the walls.

Confinement of the flanges and at the ends near the openings of the walls of members DGM IV and DGM V prevented, in these cases, buckling of the compressed reinforcement; in the non-confined members, buckling could not be prevented. Also, confinement of the concrete postponed the failure of the respective shear walls by maintaining the integrity of the concrete in the flanges and at the ends (especially in the narrow compressed area at the base of the tension wall), which allowed additional shear forces to be transmitted to the foundation block.

Though the behavior of the coupling beams had not been specially observed, it could be seen that the beams with diagonal reinforcement of the fourth shear wall (DGM IV) were less degraded than the conventionally reinforced beams of shear walls DGM I to DGM IV.

4. CONCLUSIONS

Starting from displacements, rotations and specific deformations measured with apparata placed on the models, a number of conclusions can be reached regarding the characteristics of ductility, energy absorbtion and stiffness of the tested members - decisive aspects for shear wall behavior under seismic loading.

The principal conclusions of practical interest regarding ductility characteristics are:

(a) Sectional ductility at the base of the walls decreases with increasing ratios of reinforcement in the stretched zone;

Concrete confinement in the flanges and at the ends near the openings increases sectional ductility by inreasing the specific ultimate strain (deformation) of the concrete in excess of the value given by the standard (0.0035) /3/.

(b) The rotating capacity of the plastic hinges increases with the ratio of reinforcement in the flanges and by confining the latter, a procedure increasing the bearing capacity of the concrete under compression.

(c) In the yield state, the length of the plastic zone in the walls (that is characterized by cracks and degradations of bond and by the yielding of the longitudinal reinforcement in the flanges and at the ends) extends from the base up to the top over a height of about 0.15 H, where H is the total height of the shear wall.

(d) Shear wall ductility decreases as the ratio of reinforcement increases. It is considerably (over three times) increased by concrete confimement in the flanges and at the ends near the openings. Shear wall ductility also benefits from the diagonal reinforcement of the coupling beams.

(e) Coupling beam ductility is not very high, which confirms the hypothesis that in beams with a span/height ratio < 2 the effect of shear forces influences the yield mechanism of the beam, reducing its ductility.

The length of the plastic zone for each beam end can be taken to be 1/2 the span of the beam, since the tensile stresses in the reinforcing bars extend over the entire span and the whole beam is in the zone of plastic deformation.

The following resulted from the analysis of experimental data regarding the amount of energy absorbed:

(a) The amount of energy absorbed by the member increases with increasing ratio of reinforcement in the flanges

(b) Although confinement of concrete reduces the amount of energy absorbed at the same value of the current horizontal load, the confined member absorbs 50% more energy at failure than its non-confined counterpart.

(c) Diagonal reinforcement in the coupling beams increases the amount of energy absorbed per loading cycle ; this amount is larger than in the shear walls with longitudinally reinforced beams.

(d) The absolute value of energy absorbed in the beams increases with increasing ratios of reinforcement in the flanges.

The overall stiffness of the tested shear walls decreases gradually, corresponding to the qualitative aspects of shear wall behavior described at the begining of Section 3. The quantitative characteristics of the latter, expressed in percents of bending stiffness in the uncracked state, are as follows /4/:

- at the end of its elastic behavior, the shear wall retaire about 60% of its initial stiffness in the uncracked state
- yielding of the stretched reinforcement in the most strongly stressed beam occurs at a stiffness representing 40 - 50%.
- the onset of creep in the beams corresponds to a shear wall stiffness of about 15-25%
- the onset of creep of the walls occurs when the overall shear wall stiffness has become 7.5 - 15%.
- at failure, the bending stiffness of the shear wall is of about 5% its initial stiffness in the uncracked state.

Consequently, shear wall configuration in accordance with the specific requirements for adequate seismic behavior requires the following:

- Ductilization of the plastic zone at the base of the walls by confining the concrete in flanges and at the ends by means of additional stirrups; this also ensures adequate stability of the compressed bars in the narrow compressed zone of the tension wall
- Diagonal reinforcement of the coupling beams, this provides increased ductility and better dissipation of earthquake-induced energy.
- From the structural point of view, it is essential in earthquake prone areas that the coupling beams be "weaker"members than the walls, or in other words, that they should plasticize before the walls.

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UNITS OF MEASURE

1 cm	= 0.394 in
°C	= (°F - 32) / 1.8
1 daN	= 2.248 lb force
1 daN/cm^2	= 14.5 psi
°F	= 1.8 °C + 32
1 ft	= 0.3048 m
1 ft^2	= 0.0929 m^2
1 GNm	= 0.738 x 10^6 kip-ft
1 ha	= 2.47 acres
1 in	= 2.54 cm
1 kg	= 2.2 lb
1 kg/cm^2	= 14.2 psi
1 kg/m^2	= 0.205 lb/sq. ft
1 kip-ft	= 0.7376 mNm
1 km	= 0.621 miles
1 km^2	= 0.386 sq. miles
1 kN	= 0.225 kips
1 kNm	= 0.738 kip-ft
1 m	= 3.28 ft
1 m/s^2	= 3.28 ft/sec^2
1 m^2	= 10.76 ft^2
1 mile	= 1.61 km
1 mm	= 0.039 in
1 mm^2	= 0.00155 in^2
1 MN	= 225 kips
1 MN/m	= 68.6 kips/ft
1 MPa	= 145 psi
1 N/mm^2	= 145 psi
1 sq.mile	= 2.59 km^2