OCCASION

This publication has been made available to the public on the occasion of the 50th anniversary of the United Nations Industrial Development Organisation.

DISCLAIMER

FAIR USE POLICY

CONTACT

Please contact publications@unido.org for further information concerning UNIDO publications.

For more information about UNIDO, please visit us at www.unido.org

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
Vienna International Centre, P.O. Box 300, 1400 Vienna, Austria
Tel: (+43-1) 26026-0 · www.unido.org · unido@unido.org

RESTRICTED

15

DP/ID/SER.A/727
6 August 1986

ENGLISH

TECHNICAL ADVICE ON

ASEISMIC CONSTRUCTION STRENGTHENING AND REPAIR OF BUILDINGS

SI/MEX/85/804

MEXICO

Technical report: Findings and recommendations of a team of earthquake specialists*

Prepared for the Government of Mexico by the United Nations Industrial Development Organization, acting as executing agency for the United Nations Development Programme

Based on the work of
J. G. Bouwkamp, G. G. Penelis, M. Ö. Erdik,
M. Velkov, E. Dulácska and H. Sandi

United Nations Industrial Development Organization
Vienna

*This document has been reproduced without formal editing.

V.86-59030
3109T

CONTENTS

I. FINDINGS AND RECOMMENDATIONS ON THE MICHOACAN-GUERRERO EARTHQUAKE OF 19 SEPTEMBER 1985 - GENERAL OBSERVATIONS AND DAMAGES TO STEEL STRUCTURES

BY

J. G. BOUWKAMP

TABLE OF CONTENTS

1. INTRODUCTION

The following report covers the findings of the subject mission, based on observations of damages in Mexico City, resulting from the September 19, 1985, Guerrero - Michoacan earthquake, reports received during the mission, discussions with engineers and researchers in Mexico City and information obtained by the consultant from other sources prior to the mission. The mission took place in Mexico City during the two-week period of December 1-15, 1985.

As team leader (ref. UNIDO telegram no. 97438 of November 27, 1985), consultant coordinated the efforts of the UNIDO team of experts, consisting of Messrs. J.G. Bouwkamp, E. Dulacska, M. Erdik, G. Penelis and M. Velkov. The members of the team had all been involved in the UNDP-UNIDO Project RER/79/015, on Earthquake Resistant Design in the Balkan Region.

Considering the particular aspects of the earthquake groundmotion in Mexico City, the type of damages and the structural systems involved, the building inventory and construction practices as well as the needs for repair of damaged structures and possible strengthening of both damaged and undamaged buildings, including historic monuments, it was decided that the advisory capacities of the team could be deployed most effectively by having each member address one or more of the specific problem areas. In this manner the team would be able to offer to mexican engineers and researchers as well as governmental organizations advise in a number of areas without duplication of effort.

After briefings by Mr. Silva-Aranda, UNDP Resident Representative and Mr. E. Csorba, UNDP Consutant, on December 2 and a visit to the severely damaged areas, task assignments were defined, reflecting both advisory needs and specific expertise of the individual consultants. Accordingly, the team members agreed on the following task assignments:

Gulacska	masonry construction; analysis and design as well as repair and strengthening methods,
Erdik	strong ground motion, soil-structure interaction (foundation response), strong motion instrumentation,
Penelis	strengthening of historic monuments and post-earthquake rehabilitation measures,
Velkov	prefabricated and in-situ reinforced concrete structures — research, analysis, design and retrofitting of r.c. elements and structures,
Bouwkamp	National and Regional Project Management - Balkan experience. Post-earthquake damage assessment. Research program and equipment development.

The additional topics were added because an efficient project management is absolutely necessary in guiding both national and regional efforts of possible future UNDP funded projects in earthquake engineering. Furthermore, in order to derive the maximum benefits from Regional Projects it is essential that unified post-earthquake damage assessment procedures are adopted by all participating countries. This would permit not only efficient and immediate regional post-earthquake emergency assistance efforts by damage-inspection teams from other countries, but also, subsequently, the use of such damage data in assessing the potential damages a simular earthquake could have caused when other areas in the region would have experienced similar ground motions. In fact, by adopting post-earthquake damage evaluation procedures which have already been adopted elswhere (Balkan region), the damage-data could be used interregionally for potential damage assessment studies, provided construction practices and building systems in the area under study are similar to those present in the area hit by the earthquake.

2. MISSION PROGRAM

The mission programs in Mexico City consisted of the following activities:

December 2, 1985 am - meeting at UNDP offices, briefing by Mr. Silva-Aranda, UNDP Resident Representative

	pm	- visit to UNAM,National Autonomous University of Mexico and meeting with Professor L. Esteva - Civil Engineering - visit to Chamber of Construction
December 3, 1985	am pm	- all day visit to damaged areas in Mexico City.
December 4, 1985	am	- meeting with Professor R. Meli and members of IAEE Sub-Commitee on Earthquake Performance of Rural Structures (members: Prof. Arnand Araya, India, Dr. Teddy Boen, Indonesia, Dr. Yuji Ishiyama, Japan, Ing. Julio Vargas Neumann, Vice Minister, Peru, and Dr. Charles Scawthorn, USA. - visit to the Structural Research Laboratories of the Civil Engineering Institute, including the Shaking Table Laboratory at UNAM
	pm	- meeting with Professor Meli and members of the IAEE Sub-Committie - Presentation by Prof. Meli on Effects of the September 19, 1985 Earthquake on Buildings in Mexico City.
December 5, 1985	am	- meeting with Professor Meli to finalize December 6, 1985 Seminar by UNIDO Experts.
	pm	- meeting at Chamber of Construction for viewing of video movie on post-earthquake damages immediatly after the earthquake.
December 6, 1985	am pm	- seminar by UNIDO Experts on pertinent topics relevant to Mexico City earthquake damage observations.
December 7, 1985	am pm	- visits to damaged appartment buildings in the Nonoalco-Tlaltelolco section of Mexico City.
December 8, 1985		Sunday
December 9, 1985	am	- meeting with UNAM faculty and other engineers
	pm	- visit to damaged structures in Mexico City
December 10, 1985	pm	- presentation at Chamber of Construction
December 11, 1985	pm	- presentation at Chamber of Construction
December 12, 13, 14, 1985		other pertinent business and return of experts.

The December 6, 1985 Seminar was developed considering both the advisory needs, as assessed by Mr. Bouwkamp, and the expertise of the UNIDO consultants. The following program was presented:

9.00 - 9.45	Bouwkamp	- The Balkan Project, objectives, scope and management.
		- Post-Earthquake Damage Assessment.
9.45 - 10.15	Erdik	- Behavior of Prefab. Structures (soilstructure interaction).
		- Retrofitting - a case study.
10.15 - 11.15	Velkov	- Moment-Resistant R.C. Frame Structures (research and analysis).
11.30 - 12.00	Dulacska	- Stone Masonry Structures
12.00 - 13.00	Penelis	- Repair and Strengthening of Historic Monuments.
13.00 - 14.00	Velkov	- Prefabricated R.C. Structures
14.00 - 14.30		Discussion.

The above program was developed because the earthquake of September 19, 1985 exhibited extensive soil-structure interactive effects (Erdik) and showed severe damages to reinforced concrete frame and column-slab type structures (Velkov). Damages to masonry structures seemed to have been limited, nevertheless a presentation by Mr. Dulacska was considered appropriate. Although damages to historic buildings were not immediately noticable, information received implied that significant damages did have occured. Hence, a presentation by Mr. Penelis on the subject was considered potentially beneficial.

A presentation by Mr. Velkov on prefabricated R.C. Structures was considered to be of particular interest to engineers as this type of construction could well be considered for future use in Mexico City.With the same low construction depth as the waffle-slab floor system used in Mexico City, prefab. systems are expected - based on observations in the Balkan region - to perform structurally far better than the typical waffle-slab systems. Admittedly, the foundation design of the more rigid prefab systems require particular attention because of the pour soil conditions in Mexico City.

The presentation by Bouwkamp reflected the experience of the Balkan Project RER/79/015, both in terms of organization and technical coverage. Also, it placed emphasis on the necessity of developing regionally unified post-earthquake damage evaluation procedures and post-earthquake emergency assistance programs.

The December 10, 1985 presentations at the Chamber of Construction were aimed at informing the construction sector on some of the potential applications of new technologies, both in material sciences and construction. On December 11, 1985, the presentations focussed on significant considerations in post-earthquake repair procedures with particular emphasis on policy aspects of post-earthquake repair procedures and on the consequences of repair and strengthening procedures, including retrofitting. Accordingly, the following program was formulated:

December 10, 1985.

Csorba - New Technologies in Building Materials, Production, Prefabrication and Construction.

Velkov - System Development, Research and Behavior of Prefabricated R.C Large-Panel Structures.

December 11, 1985

Penelis - Post-Earthquake Rehabilitation Measures following the June 20, 1978, Thessaloniki Earthquake.

Velkov - Design and Retrofitting of R.C. Elements and Structures

Because soil amplification and soil-structure interaction effects had been a primary cause of the failures observed in the Mexico City region and the development of strong-motion instrumentation arrays may be an essential part of any future research effort, Mr. Erdik met seperately with both Professors J. Prince and G. Romo. Also, in order to provide an information transfer on post-earthquake rehabilitation measures and policies as well as on the repair of historic monuments, Mr. Penelis met with Mr. Alexandro

Rivas Vidal, Coordinative Office of Technical Operation, Mexico City, and with representatives of the Office of Dr. Sonja Lombardo, Directorate for Historical Monuments, National Historical and Antropological Institute. Mr. Vidal's organization is responsible for city-wide post-earthquake demolition, repair and strengthening measures.

Mr. Bouwkamp met seperately with Professor Oscar de Buen, leading structural engineer, responsible for the design of most high-rise steel buildings in Mexico City and designer of the two 15 and 21 story high steel office towers which had collapsed on September 19. The meeting took place at 21.00 hrs on December 6, 1985 at UNAM.

3. THE MEXICO MICHOACAN-GUERRERO EARTHQUAKE OF SEPTEMBER 19, 1985

3.1 Earthquake Characteristics (1-5)

The earthquake occured at 7.15 am. local time (13 hrs, 17 min., 44 secs GMT) and had, according to Instituto de Ingeniera, I.d.I., UNAM, an epicenter in the Pacific Ocean (17.68^0 N (latitude) and 102.47^0 W (longitude)), offshore the Mexican coast at the border between the States of Michoacan and Guerrero (river Las Balsas). On the other hand, the US National Earthquake Information Service (NEIS), placed the epicenter on shore, west of the mouth of the river Las Balsas, at a distance of about 50 km NNE of the first location, Fig. 1. The focal depth (hypocenter) was 33km. Initial estimates of the Richter magnitude, according to the I.d.I. and NEIS, were M_S= 7.8 and 8.1, respectively.

The earthquake, one of the best recorded events, was caused by the subduction of the Cocos Plate under the North American Plate. The fault rupture length has been estimated at 150 miles or 240 km. and was located at the SE edge of the so-called Michoacan Seismic Gap. In the coastal region, 18 out of 20 strongmotion accelerographs recorded the event. These instruments were installed under a cooperative program between the I.de I., UNAM, and the University of California, San Diego. The program is sponsored by the National Science Foundation, Washington, D.C., and contemplates the installation of 30 strongmotion instruments in total. In addition to the 20 accelerographs noted above, 15 more strongmotion instruments had been installed at the time of the earthquake and were located in dams in the State of Guerrero.

3.2 Ground Motions recorded in the Epicentral Area and in Mexico City.

One of the significant records in the epicentral region was obtained at Zaculata, located in the delta of the Las Balsas river, Figure 2. The record seems to indicate the occurence of two earthquakes with an interval of about 30 seconds; the first with a NS peak acceleration of about 28%g, the second with about 20%g, Figure 3. Spectral data, shown in Figure 4, indicate considerable excitation up to natural periods of 1 second.

( ) see reference list

In Mexico City, located about 400 km. away, an initial count indicated that at least 15 instruments recorded the earthquake ground motions. The locations of eight of these instruments are shown in Figure 5. The maximum peak acceleration, velocity and displacement values of these eight records are presented in Table 1. In general, results indicated that the maximum acceleration recorded at the University (UNAM) was approximately 4% g., or about 15% of the max. acceleration recorded at Zaculata, in the epicentral region. In fact, according to information, records on bed rock near Mexico City seem to have shown max. accelerations of between 1% and 2% g.; values reflecting the common reduction of the ground motion intensities associated with large epicentral distances. According to Table 1., the maximum acceleration recorded in Mexico City was about 17% g. and occured at the Ministry of Transportation (SCT).

Considering the results recorded at the University and SCT, it should be noted that the University instrument is located in a region of firm soils, while the SCT site is located on the soft clays of the old lake bed on which a major part of Mexico City has been built. The results clearly illustrate the influence of the soil conditions on the ground motions; an amplification of four.

Equally important is the ~~~t that the instrument at the University, see Figure 6, showed a recorc ~ predominant period of about 2 seconds, reflecting in fact the basi ~lic motion of the underlying bed rock; the record also illustrates ~er'od lengthening effect of large-distance earthquakes (from 1 sec. at Zaculata to 2 secs at Mexico City). Considering this periodic ground motion, it is not surprising that the record of the above noted SCT instrument also showed a predominant period of about two seconds as illustrated by the response spectra shown in Figure 7. With recorded motions of 60 to 180 seconds duration, the damage potential of the Mexico earthquake for structures with natural periods between 1 and 2 seconds is exceptionally large. From Figure 8, showing the SCT ground motion record, it can be observed that the motion is virtually harmonic with a period of about 2 seconds, and that the intensity of the ground motion in the EW direction exceeds 10% g for about 20 seconds, thus resulting in about 10 cycles of strong motion. This type of sustained motion has in principle a large damage potential and is significantly more severe than motions recorded in earlier earthquqkes in Mexico City.

3.3 Evaluation of Damage Potential of Recorded Ground Motions

Comparing the ground accelerations recorded at the instrument installed at SCT, with data from earlier earthquakes (Table 2), it is obvious that the 1985 data is practically five times larger than those recorded previously during the earthquakes of 1979 and 1980 (3). Furthermore, it is about 3.4 times larger than the maximum estimated ground acceleration of the July 1957 earthquake (6).

The damage potential of the ground motion recorded at the SCT site, is further illustrated by the calculated linear elastic response spectra for different percentages of damping. The EW response spectrum for 5% damping shows for a fundamental period of 2 secs a maximum acceleration of 0.984 g, or almost six times the recorded peak ground acceleration of 0.168 g. This amplification is significantly larger than normally reflected in other codes; in the US the amplification factor for 5% damping and a period of 2 seconds is only two .

The damage potential is further illustrated by comparing the intensity of the linear elastic response as noted above, with the design response spectrum of the present earthquake code for Mexico City (7). As shown in Fig. 9, the calculated linear elastic response of structures with periods of vibration between 1.8 and 3 seconds, using the SCT EW record, exceeded the Code provision by a factor of four (base-shear coefficient C =V/W of 0.98 versus 0.24). Considering furthermore that a reinforced-concrete, ductile, frame-type structure can be designed using a ductility factor Q = 4, the design base shear coefficient C of 0.24 can be reduced to 0.06. This would reflect a design level of only 1/16th of the level of excitation which could be expected for a building with a fundamental period of 2 secs when subjected to the SCT EW ground motion.

The damage potential has been further accentuated by the duration of the ground motion. Based on several records, major shaking during ten or more cycles of motion may have been common. The EW record at SCT, see Table 2, indicated that 16 cycles of motion with accelerations of 50% or more of the recorded peak ground acceleration have been registred. This phenomenon is potentially so devastating as it causes virtually the same number of deformation reversals in buildings having closely the same fundamental periods as the period of ground shaking. Considering the poor standard of construction and earthquake characteristics, such strong shaking may well have led to serious stiffeness deterioration. Structures with fundamental periods of initially 0.8 to 1.2 seconds, could, after a few number of cycles of strong excitation, experience a lengthening of the fundamental period through stiffness degradation. This effect would cause the structure to undergo increasingly larger motions while moving closer to the predominant period of excitation as a result of a continuing loss of stiffeness. This would result in a tuning effect, causing a virtual resonance condition.

Finally, the poor soil and foundation conditions, may have also contributed to the observed failures. The fundamental periods of buildings, may in fact have been larger than analysed for purposes of design. A twelve story building with a period calculated at about 1.0 second, may in reality have had a fundamental period of close to 1.3 or 1.4 seconds, an increase of 30 to 40% above the design value. This would further increase the potentially damaging effect of the earthquake.

4. BUILDING PERFORMANCE -- DAMAGE OBSERVATIONS.

4.1 General

Failures and severe damages to buildings in Mexico City have been dramatic. However, considering the total number of structures , estimated at 1.5 million, percentage wise a relatively small number of buildings have actually been affected. Estimates of collapsed or severely damaged buildings may represent only about one half of one precent of the total inventory. Such numbers are misleading because the damages were concentrated in a relatively small section of the city (23 km^2). The central region of Mexico City and the areas with both moderate and severe damages are shown in Fig. 10. The severely damaged region is built on the highly compressible clays of the old lake bed, with - as shown in Figures 11 and 12 - depths to the first layer between 26 and 32 metres and depths to the second layer of hard soil between 30 and 46 metres (9). The damage susceptability of the central section of the City is clearly illustrated by the zones of damages observed in the earlier 1957 and 1979 earthquakes.

It is important to note that in the moderately damaged area, and even in the area with severe damages, many buildings sustained only very little or no damage at all. In general building performance was excellent for buildings with less than 3 stories. With the exception of school buildings even buildings of 5 stories in height performed well. Likewise, very tall buildings such as the 43-story "Latino America" tower and the 54-story, 211 m high, "PEMEX" tower performed excellently, without the slightest damage. In fact these observations agree with the response spectral information that buildings with fundamental periods between 1 and 3 seconds, or structurally between 10 and 25 some stories, could be expected to be predominantly susceptable to serious damages. Because of the relatively small number of structures in this catagory, as compared to the total inventory in the central region of the City, the damages, although serious. were in fact sporadic.

In regions with high compressive soil layers at only small depths, buildings did not suffer serious damage because the level of excitation was small because of insufficient amplification of the basically low-intensity bedrock motions. In regions with high compressible soils at depths larger than those in the central region, damages were considerable smaller because the fundamental period of these deeper layers was higher than the 2 seconds of the bed rock ground motion; thus preventing soil amplification. An other factor contributing to the lack of damages in the outer region was the absence of significant numbers of taller buildings.

4.2. Damage Characteristics

The damage characteristics according to structural types, year of construction and number of stories are presented in Table 3.

These data indicate that reinforced concrete (R.C.) frame-type buildings and buildings with waffle slabs performed the worst and constituted about 80% of the buildings which collapsed or were severely damaged. Thereby was observed, that in comparison to the inventory of these types of buildings, the damage density for waffle-slab structures was twice as high than for buildings with frame-type, beam-column, earthquake load resisting systems. Considering Table 3 further, it is important to note also, that of the structures which collapsed or were severely damaged and which were built in the period since 1976, waffle-slab failures alone constituted about 70% (21+18 or 39 versus 56). R.C. framed-type structures built in the same period accounted for almost 20% of the collapsed or severely damaged structures built since 1976. This observation is particularly important because a new code for seismic resistant design was introduced in 1976. Clearly, this code needed modification. In fact, such changes were introduced shortly after the earthquake, with the stipulation that the design coefficient C (see Fig. 9) should be increased from 0.24 to 0.40 for structures located in the central area of Mexico City.

Furthermore, as could be expected from the earthquake spectral characteristics, most damages and collapses occured in buildings ranging from 6 to 15 stories in height (161+34 = 195 versus 330, or 60%). These buildings most likely experienced progressive stiffeness losses thereby moving towards the potentially critical predominant ground excitation period of 2 seconds.

Finally, it should be noted that collapsed and severely damaged buildings up to a height of 5 stories, accounted for less than 1% of all buildings in this catagory in the zone with heavy damages. Assuming a failure percentage of about 0.5%, the total of all buildings in this catagory would be about 25,000. Similar percentages of about 15%, 15% and 8% for structures with story characteristics of 6-10, 11-15 and over 15 stories, respectively, would constitute about the following total number of buildings in each of those catagories, namely: 1000, 200 and 70, respectively. The survey by the staff of the I.deI. would thus imply that the total number of buildings which collapsed or were severely damaged in the heavy damaged region (330), amounted to about 1.2% of all buildings (26,300) in that region.

In reflecting on the relative performance of steel versus concrete buildings, a brief review of the causes which led to the collapse two high-rise buildings seems in order. These buildings, built in 1957, were basically designed for wind. Nevertheless an effective base shear coefficient of 6% was achieved. This value could be considered representative for normal earthquake design conditions. However, contrary to present-day design provisions, ductility requirements and P-delta effects had not been considered in the design. Of course the virtual coincidence with the fundamental period of the building and the period of excitation (2 seconds) created a most severe earthquake loading condition. In fact of the five closely standing office towers of 15, 21, 21, 21 and 15 stories, one of the outer 21 story towers collapsed and fell on the adjacent 15-story structure, causing its collapse as well. The three remaining structures, a'though severely damaged, remained standing. The structures were all similar in construction, consisting of welded box-type columns and truss-type floor girders forming the lateral-load resisting frames. Due to the severe shaking, the large moments in the columns caused heavy shear loads in the floor trusses, causing failure of many web truss members. This in turn weakened the system and most likely caused an overload in the columns. As a result, welded field joints in the columns at the third or fourth floor level may have developed local plate buckling which initiated the final collapse of the 21-story structure. In fact, similar local buckling failures could be noted in one of the still standing 21-story buildings.

4.3. Types of Structural Failures

The predominant failures causing or contributing to the structural collapse or resulted into severe damages, are presented in Table 4. This table shows the percentages of failures which led or contributed to the collapse of buildings catagorized in Table 3. In most cases more than one single type of failure caused the collapse or the severe damages of these buildings.

Reviewing the data clearly points to the fact that corner buildings were particularly susceptable to failure (42%). Furthermore, failure in the upper floors and major failures at intermediate floor levels seemed to have been observed equally as major causes in about 42% of the cases.

Failure of corner buildings were typically caused by irregularities in the plan layout resulting in unbalanced stiffeness at the ground floor level and critical torsional effects under earthquake excitation. The pancake-type failures of the upper floors were undoubtedly due to a combination of several factors, such as extreme horizontal accelerations due to the long period shaking and rigid-body rotation of the total structure due to poor foundation conditions, inadequate code requirements for lateral load distribution in the upper floors, excessive live loads exceeding considerably the design live load values used in the design and low quality of construction of the upper floors subsequently, by other contractors.

Failure of the intermediate floors were due in part to hammering effects of close standing buildings of different height. The same phenomenon also contributed to the upper floor failures of the shorter adjacent building. Otherwise, intermediate floor failures were most likely caused by the insufficient strength of the buildings at these levels due to sudden large changes in the column sizes. This phenomenon resulted in an insufficient flexural yield strength of the columns as compared to the beams and even waffle slabs, thus causing failure of the columns at that level. The presence of high axial loads and the lack of sufficient column stirrups at the top and bottom of the columns led to critical interaction of high axial loads and column moments which resulted in a brittle failure of these columns.

Although according to the I.deI, UNAM, survey (Table 4) punching-shear failures of waffle slabs caused or contributed to the collapse of only 4% of the buildings, many failures of such structures occurred as noted earlier in Table 3.

5. RECOMMENDATIONS

Considering the damaging impact of the Michoacon-Guerrero earthquake of September 19, 1985 on the buildings in Mexico City in particular and the vulnerability of south America to devastating earthquakes in general, national and regional research efforts are recommended. Each earthquake is in first instance of national importance. However, the data and lessons learned from each event are of utmost importance for the international community at large and should be dissemminated as such. For that same reason, also the results of national research efforts related to the specifics of a national earthquake disaster, should be made available to the regional engineering community. Such information transfer would potentially reduce the earthquake hazard far beyond the national borders of the stricken country. In order to meet these objectives, a frame work for research data transfer is of vital importance. Such transfer could best be guaranteed through a regional network of research institutions which would address common problems in earthquake engineering through well coordinated programs of research. Integrated research efforts would permit addressing regional problems and, through mobilizing the regional expertise, permit resolving a broad spectrum of earthquake related problems through the efforts of many. Cooperative regional research programs would also permit the development of disaster assistance teams which could operate regionally and could be called upon to assist national teams in post-earthquake emergency services and engineering assistance. Such efforts would significantly enhance the regional capabilities to respond effectively in providing emergency disaster assistance in general.

Considering the specifics of the Mexico City damages, the following research efforts are recommended to be undertaken:

1. The deveopment of strong-motion instrument arrays to study soil amplification in the greater Mexico City area. Such information would be of great significance worlwide in evaluating the soil effects on the surface motions resulting from given bed-rock earthquake excitations.
2. Forced and ambient vibration studies of exsisting buildings to evaluate the basic structural and soil-structure interaction effects. This information would permit assessing the anticipated dynamic response of the structure and its potential behavior under strong ground motions under consideration of foundation and soil characteristics. In general, such

data would be essential not only in determining the strength of existing buildings and the effectiveniss of possible retrofitting measures, but also in developing improved earthquake design codes and detailed design procedures for structural systems and elements.

3. Studies on the behavior of typical construction elements and details (such as beam-column joints and infill walss) under cyclic loads to evaluate the resistance and ductility under earthquake load conditions. Such afforts are to be focussed on thereinforced concrete waffle-slab and frame-type structures in order to develop appropriate design requirements for new construction but also strenthening procedures to increase the earthquake load resistance of existing buildings.

4. In case the economic potential of prefabricating systems were to be established, research on such systems, appropriately adjusted to the regional construction capabilities, could be most beneficial in possibly replacing the waffle-slab type systems in the region.

In the above experimental research studies, quasi-static and pseudo-dynamic testing procedures should be used, requiring displacement controlled, double-acting load cylinders and high-speed data acquisition systems. In case of pseudo-dynamic tests of subassemblages, computer controlled closed-loop test procedures need to be developed. Considering the available testing equipment at the Instituto de Ingeniera, UNAM, the special equipment necessary to carry out the above listed studies is virtually non-existant. The shaking table facility at the Instituto de Ingeniera is medium in size and could be used effectively in carrying out real-time dynamic tests on structural assemblies (multi-story four-column frames).

Future research efforts supported under the auspicies of the United Nations Development Programme, should be formulated in consideration of the US-Mexico joint research Programme sponsored by the US Science Foundation, Washington, DC. This organization will be administrating research efforts in the amount of $ 4,000,000 (US $) over the next couple of years. Such efforts will be carried out in US institutions, involving both US and Mexican engineers and researchers.

6. REFERENCES

(1) Mena, Enrique S., et al,"Analysis del Acelerograme "Zacatula" del Sismo del 19 de Setiembre de 1985", Instituto de Ingeniera, UNAM, Mexico, October 1985, Informe: IPS-10E.

(2) Prince, J., et al: "Acelerogramas en Ciuda Universitaria Del Sismo del 19 Septiembre de 1985", Instituto de Ingenieria, UNAM, Mexico, September 20, 1985, Informe: IPS-10A.

(3) Prince, J., et al,"Acelerograme en el Centro SCOP de la Secretaria de Comunicaciones y Transportes del Sismo del 19 de Setiembre de 1985", Instituto de Ingeniera, UNAM, Mexico, Setiembre 21, 1985, Informe: IPS-10B.

(4) Prince, J., et al,"Los Dos Acelerogramas Del Sismo de Setiembre 19 de 1985, Obtenidos en la Central de Abastos, en Mexico, D.F.", Instituto de Ingeniera, UNAM, Mexico, Setiembre 23, 1985, Informe:IPS-10C.

(5) Prince, J., et al," Espectros de las Componentes Horizontales Registradas por los Acelerografos Digitales de Mexico, D.F. del Sismo del 19 de Setiembre de 1985. Acelerogrameas en Viveros y en Tacabaya", Instituto de Ingeniera, UNAM, Mexico, Octubre 1, 1985. Informe: IPS-10D.

(6) Zeevart, L., "Foundation Engineering for Difficult Subsoil Conditions", Van Norstrand Reinhold Company, New York, 1973.

(7) "Reglamento de Construcciones Para el Districto Federal y Leyes Conexas", Fifth Edition, 1982 Ediciones Andrade, S.A. Mexico, D.F.

(8) Bouwkamp, J.G., et al, "Dynamic Properties of an Eight- (and Twelve-) Story Prefabricated Panel Building", Reports Nos UBC/EERC 80/30 (and 80/29), Earthquake Engineering Research Center, University of California, Berkeley, 1980.

(9) "Efectos de los Simos del 19 y 20 Setiembre de 1985 en las Construcciones de la Ciudad de Mexico", Preliminar Report prepared by the Instituto de Ingeniera de la UNAM, Mexico, September 29, 1985. Updated version October 1985 and verbal communications of December 4, 1985 as incorporated in Tables 3 and 4 of this report.

TABLE 1. SUMMARY OF SIGNIFICANT ACCELEROGRAPH RECORDS OBTAINED AT DIFFERENT LOCATIONS IN MEXICO CITY DURING THE SEPTEMBER 19, 1985 MICHOACAN-GUERRERO EARTHQUAKE

	DIRECTION	I.de I., UNAM			SCT	C. ABASTOS		VIVEROS	TACUBAYA
		CUMV	CUIP	CU01		CDAO	CDAF		
PEAK ACCELERATION (cm/sec^2)	NS	37	32	28	98	69	81	44	34
	EW	39	35	33	168	80	95	42	33
	V	20	22	22	36	36	27	18	19
PEAK VELOCITY (cm/sec)	NS	9	10	10	39	35	25	11	14
	EW	11	9	9	61	42	38	12	10
	V	8	8	8	9	11	9	6	8
PEAK DISPLACEMENT (cm)	NS	6	6	6	17	25	15	9	12
	EW	4	8	7	21	25	19	7	9
	V	5	7	7	7	9	8	7	8

TABLE 2. SUMMARY OF RECORDED INFORMATION ON THE RECORDS OBTAINED AT THE SCT STATION DURING THE SEPTEMBER 19, 1985 EARTHQUAKE

	DIRECTION	PEAK VALUE	NUMBER OF CYCLES EXCEEDING 50% OF PEAK VALUE
ACCELERATION (cm/sec2)	NS	98	20
	EW	168	16
	V	36	21
VELOCITY (cm/sec)	NS	39	9
	EW	61	13
	V	9	17
DISPLACEMENT (cm)	NS	17	9
	EW	21	14
	V	7	6

NOTE: during earlier earthquakes the following peak accelerattion have been recorded: October 24, 1980 34 cm/sec2 (NS)
March 14, 1979 30 cm/sec^2 (EW)

TABLE 3 : STATISTICS ON COLLAPSED AND SEVERELY DAMAGED BUILDINGS IN MEXICO CITY

		Year of construction			Number of floors				
Type of Structure	Type of Damage	←1957	57-76	1976→	≤ 5	6-10	11-15	> 15	Collapsed or very severely damaged
Concrete frames	Collapse very severe	27 16	51 23	4 6	27 10	46 28	8 6	1 1	82 45
Steel frames	Collapse very severe	7 1	3 1	0 0	4 0	3 0	1 2	2 0	10 2
Waffle slabs	Collapse very severe	8 4	62 22	21 18	36 5	49 26	5 12	1 1	91 44
Masonry	Collapse very severe	6 9	5 13	2 1	11 22	2 1	0 0	0 0	13 23
Others	Collapse very severe	4 0	8 4	2 2	12 2	2 4	0 0	0 0	14 6
TOTAL	Collapse and very severe	82	192	56	129	161	34	6	330
percent of damage of all buildings in heavy damaged zone →					< 1%	~ 15%	~ 15%	~ 8%	

Note: damage density in waffle-slab systems was twice as high as in concrete-frame systems

Mexico City Earthquake September 19, 1985

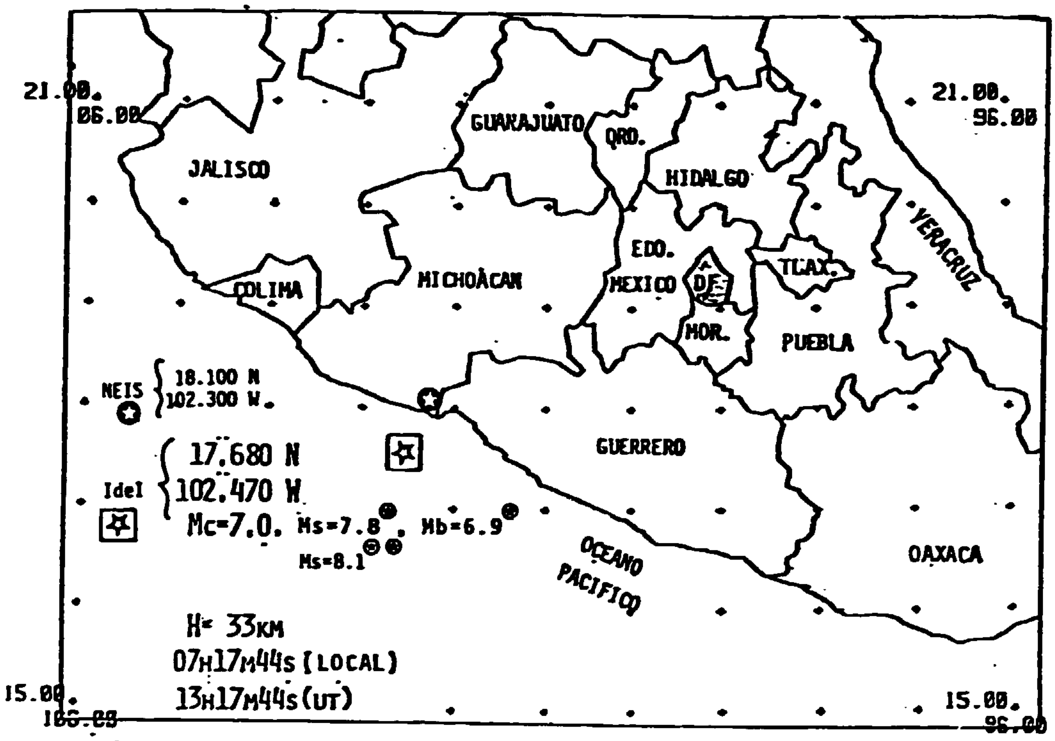
UNAM, December 4, 1985

TABLE 4: PREDOMINANT CAUSES OF FAILURE IN PERCENTAGES OF TOTAL

plan-assymmetry (nonsymmetric rigidity)	15%
corner building	42%
first-floor flexibility (soft story)	8%
short-column effect	3%
excessive overload (increased live loads)	9%
buildings with previous differential settlement	2%
foundation failure (local large settlement)	13%
pounding/hammering of close-standing buildings	15%
previous damages from earlier earthquakes	5%
punching of waffle slaos	4%
major failure in upper floors (often only upper most floor)	38%
major failure of intermediate floors	40%

NOTE: in numereous cases a combination of above causes lead to ultimate collapse or severe failure

Mexico City Earthquake, September 19, 1985



according to US National Earthquqke Information Service (NEIS)

corrected data of NEIS per 290985

Fig. 1 Epicentral Location of September 19, 1985 Michoacan-Guerrero Earthquqke (based on preliminary data)

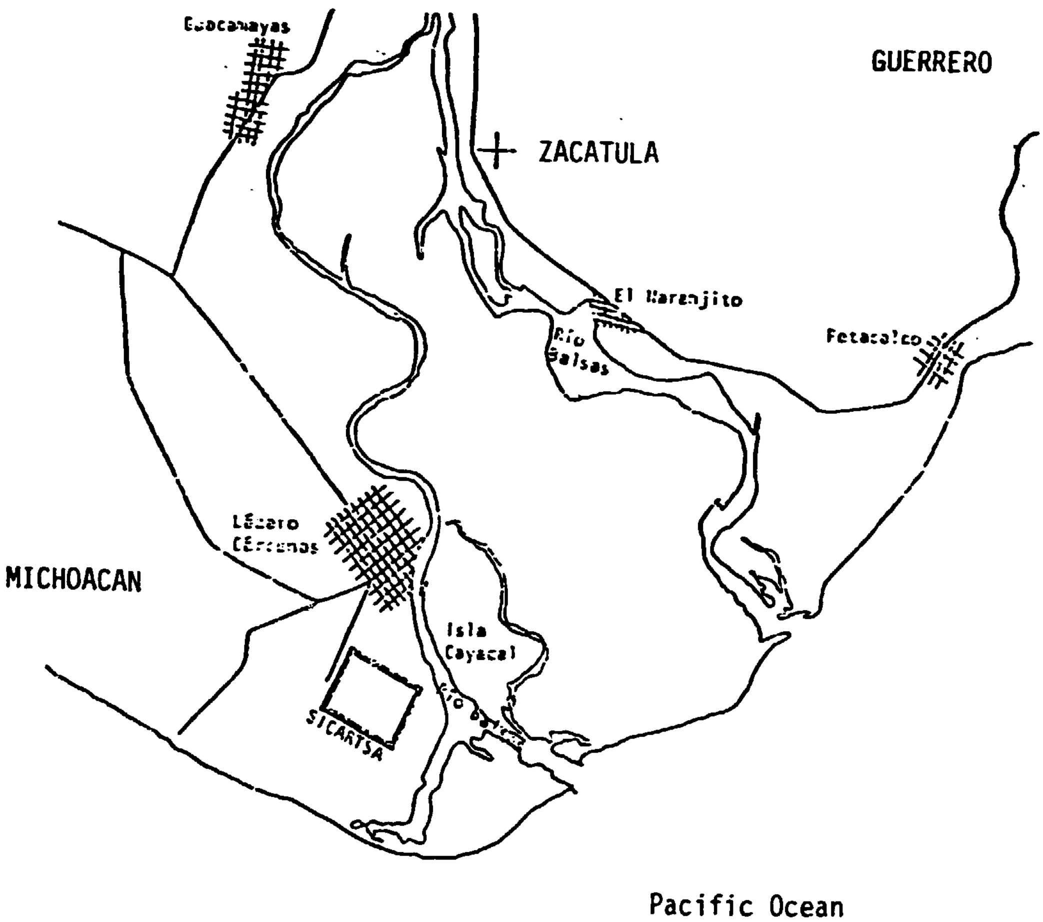


Fig. 2 Location of the Zacatula Strong-motion Station in the Delta of the Balsas River

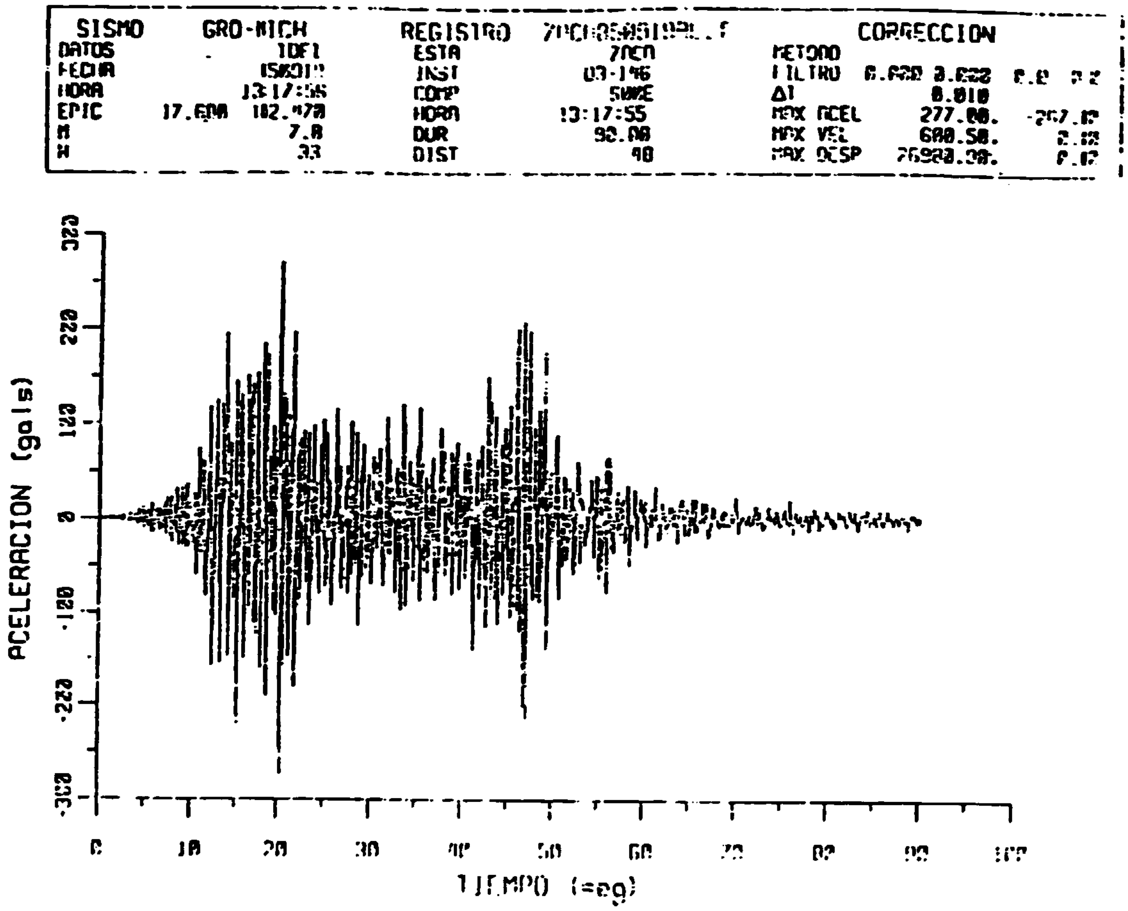


Fig. 3 N-S Acceleration Record of Zacutula Station record of the September 19,1985 main shock.

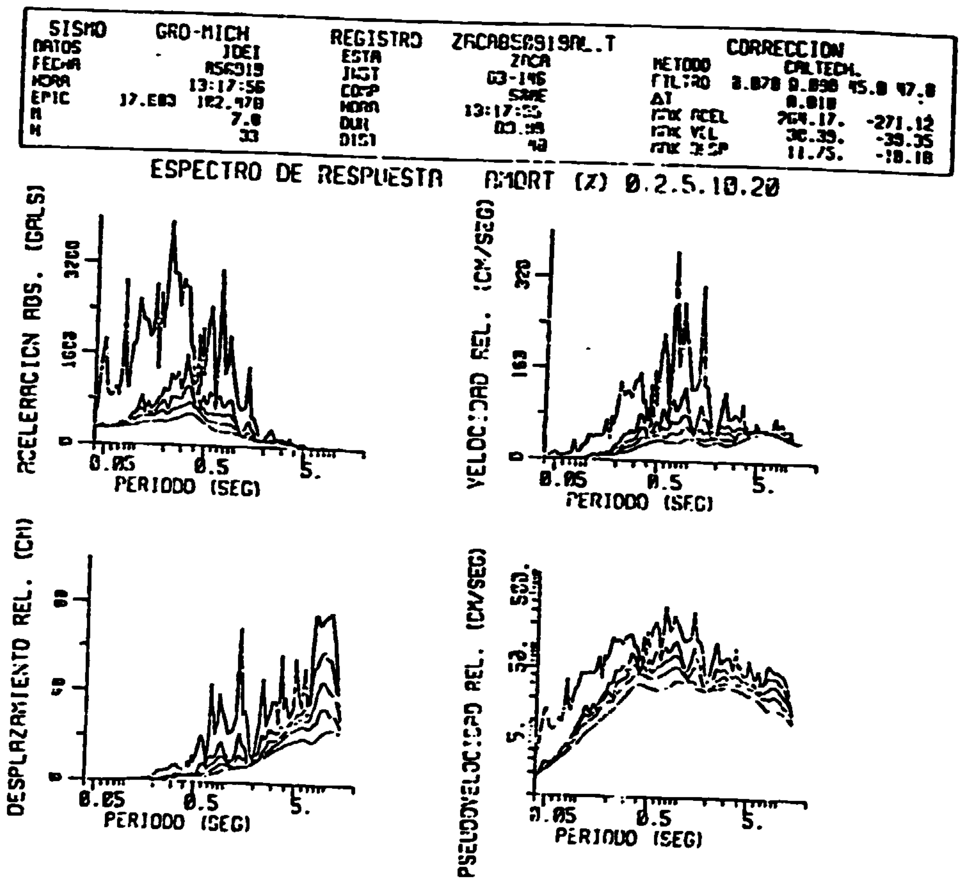


Fig. 4. Linear Response Spectra for the N-S acceleration component of the Zacatula, September 19, 1985 Record

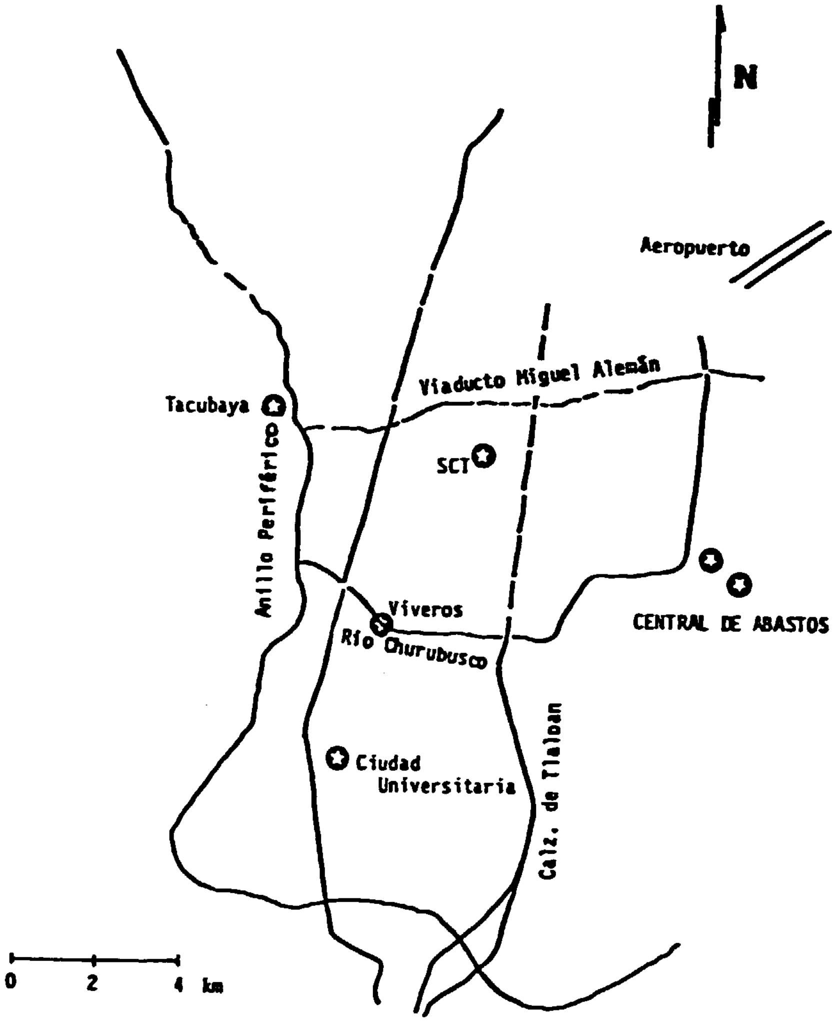


Fig. 5. Location of the Digital Strong-Motion Accelerographs in Mexico City

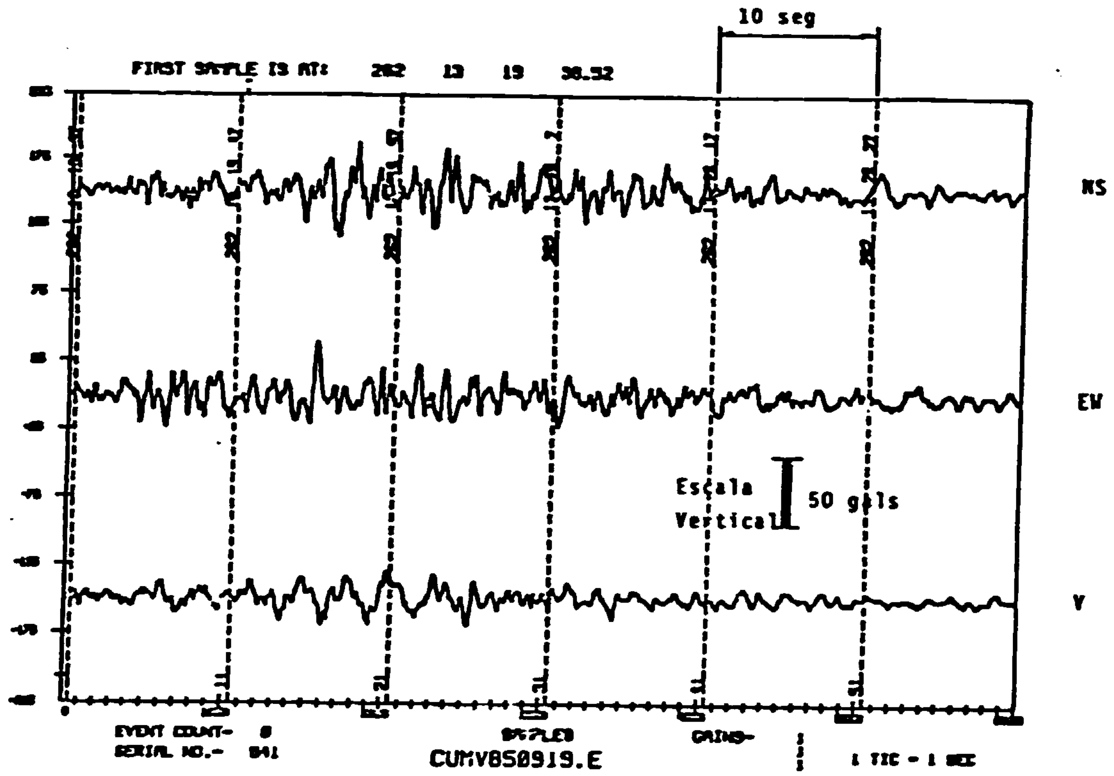


Fig. 6. Three- Directional Acceleration Records of the September 19, 1985 Earthquake, recorded at the Instituto de Ingenieria, UNAM, Mexico City

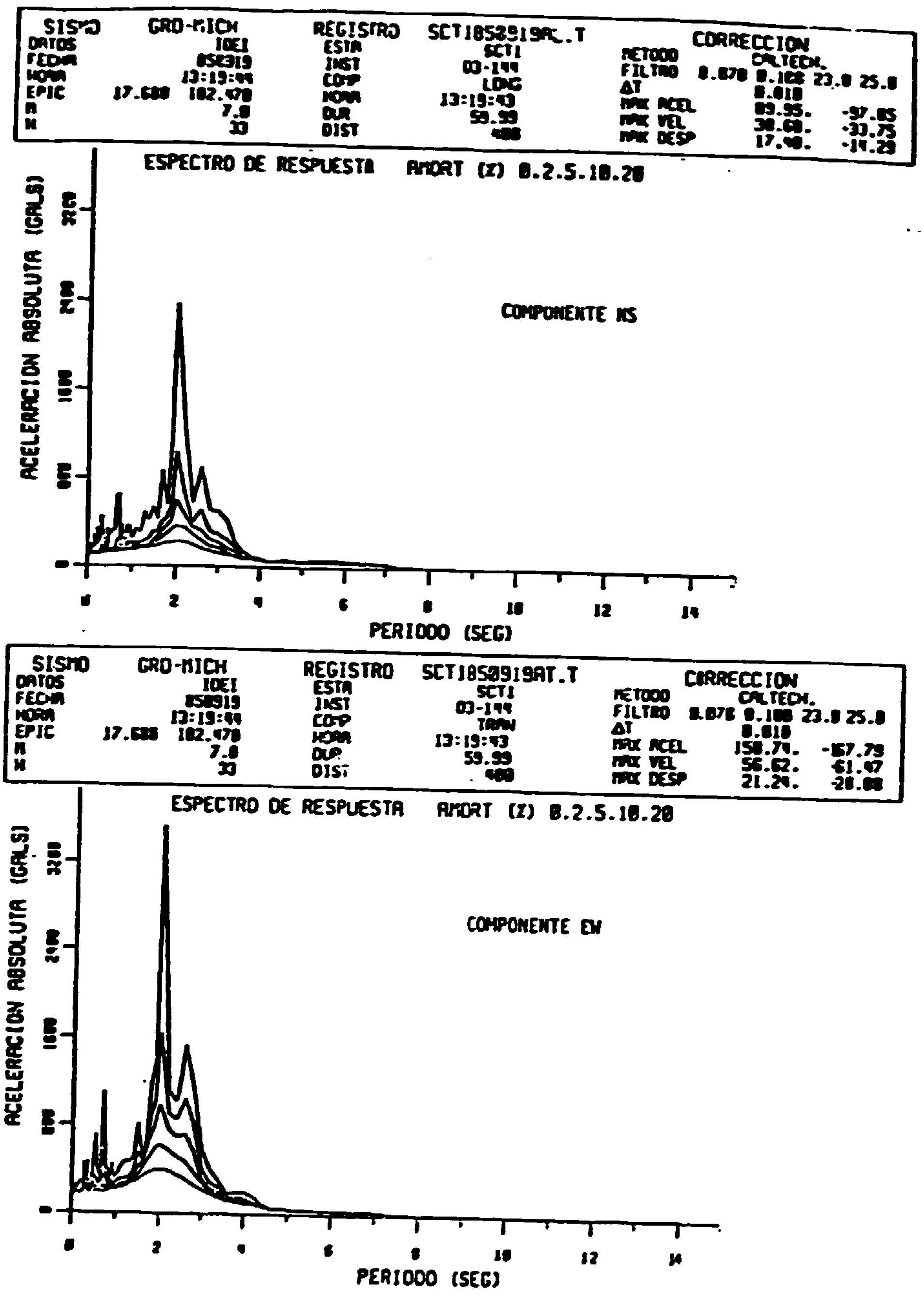


Figure 7. Linear Response Spectra for the NS and EW acceleration components recorded at the SCT Location.

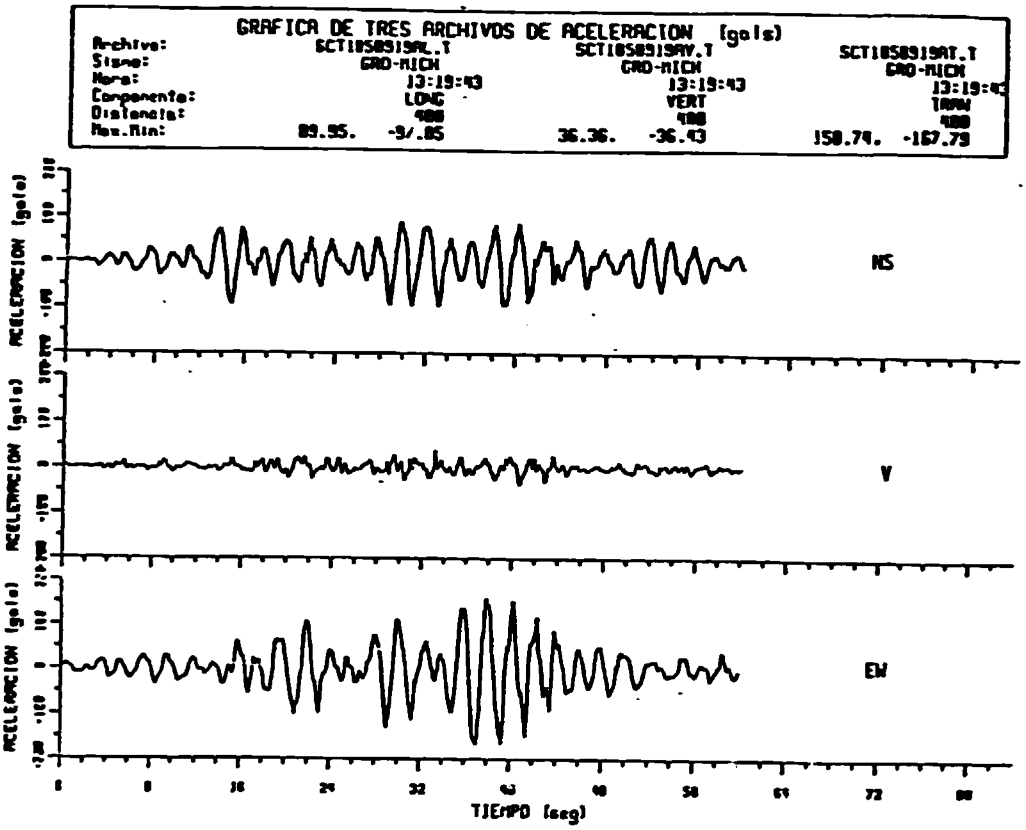


Fig. 8. Three-Directional Acceleration Records of the September 19, 1985 Earthquake recoreded at the SCT Location

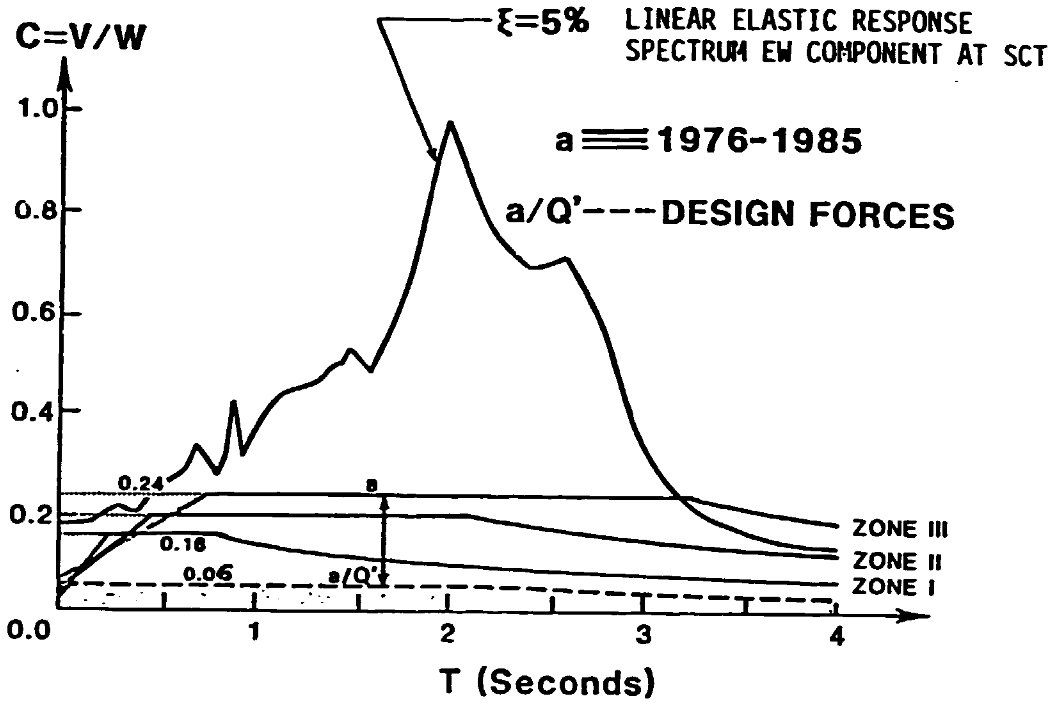


Fig. 9. Comparison of the 5% Damped Linear Elastic Response Spectrum of the EW Component of the Acceleration Record recorded at SCT with the 5% Damped Linear Elastic Spectra used by the Mexican Code (1976) and with the Design Forces for a Ductility Factor of Q = 4.

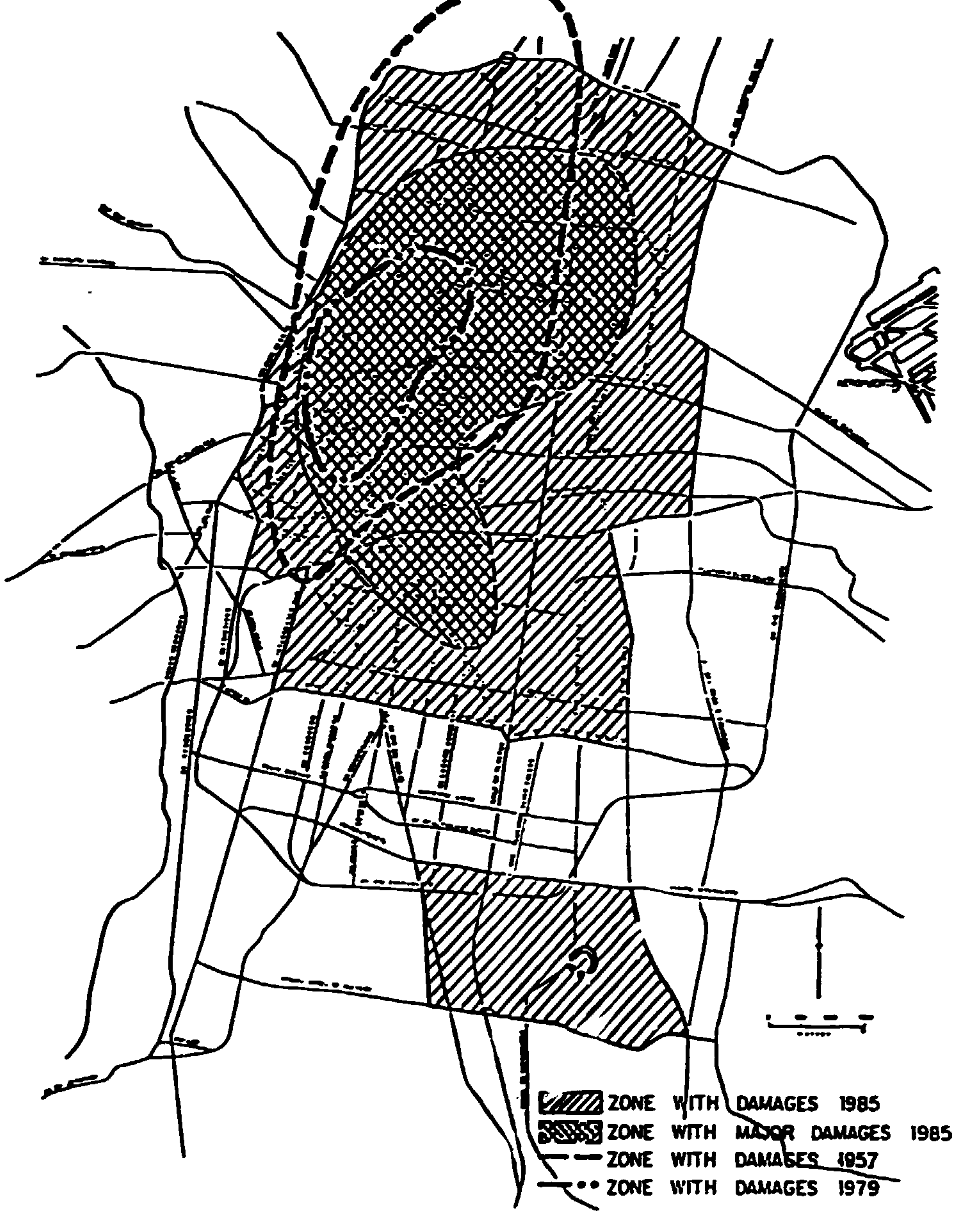


Fig. 10. Zones of Earthquake Damages in the 1985, 1957 and 1979 Earthquakes.

Figure 11. Curves indicating Depth to First Layer of Firm Soil

Figure 12. Curves indicating Depth to Second Layer of Firm Soils (Deep Deposits)

11. FINDINGS AND RECOMMENDATIONS ON THE MICHOACAN-GUERRERO EARTHQUAKE OF 19 SEPTEMBER 1985, WITH SPECIAL REFERENCE TO MONUMENTAL BUILDINGS AND ADMINISTRATIVE PROCEDURES

BY

G. G. PENELIS

CONTENTS

Annexes

1. INTRODUCTION

This report submitted by the subscriber who visited Mexico City from 2 to 15 of December 1985 as a member of the UNIDO'S mission of consultants after the earthquake of 19 and 20 September 1985 for technical advise in aseismic construction strengthening and repair of buildings.

In the frame work of the job description SI/MEX/85/11·54/32.1.K of the above mentioned mission the reporter together with the other members of the group has participated in the follc ng activities:

- Study of the behaviour and situations of various types of buildings.
- Study of the soil conditions and types of foundations of various buildings and assess to what extend the soil and foundations caused the recent earthquake disaster.
- Study of the research conditions in Mexico.
- Reviewing of the administrative procedures established for the damage evaluation repair and strengthening.
- Reviewing of the existing Code and its modification recently adopted.
- Transfer of experience in design of earthquake resistant structures, from Balkan region to Mexico.
- Transfer of experience in damage evaluation, repair and strengthening from Balkan region to Mexico.

The above mentioned activities were carried out by means of visiting fully and less damaged buildings in the city and outside, having meetings, discussions, visiting the research institutes, giving lectures and having exchange of technical information with members of the team and local specialists. In Annex I a detailed shedule of the reporter's stay in Mexico City is given.

It was decided by the members of the consultant's group that for a better coordination of the mission each member of the group in parallel to the general overview would focus his interests to some special areas and the results of his activities in these areas would be presented in his report in detail. So it was decided the following job distribution:

1. Prof. J. Bouwkamp :a) General information
   b) Steel Structures
2. Prof. Dulatscha : Masonry Buildings
3. Prof. M. Erdik : Soil-structure interaction
4. Prof. G.Penelis :a) Monumental Buildings
   b) Administrative procedures for the rehabilitation of the City.
5. Prof. M.Velkov : R/C structures

In the frame of this decision in the preceding chapters the interest will be focused on

a) Monumental Building-Repair and Strengthening.

b) Administrative procedures for the rehabilitation of the City.

2. EARTHQUAKE CHARACTERISTICS

The Mexico earthquake of September 19, 1985 had a magnitude M_s=8.1(NEIS) and the epicenter was located about 30km off the Pacific coast, near the mouth of the Balsas river and about 400km from Mexico City.

The earthquake intesity had a regular deminuation law in a zone of 30-40km from the epicenter and then due to the special soil conditions in the valey of Mexico City heavy damages appeared 400km away from the epicenter.

Eight digital strong motion instruments installed at various sites of the city recorded the accelerograms of the earthquake. The peak ground acceleration recorded varied from 40gals(cm/sec^2) in "firm" ground to 200gals(cm/sec^2) is "soft" ground. The predominant period of the earthquake was appr. 2,0sec and the duration appr. 120sec in some instances. In Annex II more detailed information on the earthquake characteristics may be found [1], [2], [3], [4], [5].

3. SOIL CONDITION AND MICROZONATION

Mexico City is founded in a valley at an altitude of 2.240m. The major part of the city was founded on a drained lake. Three main zones may be defined: the hill zone of firm soil, the lake zone of deep deposits of a very soft clay and in between a transition zone where the clay deposits are shallow.

The amplification of the max ground acceleration from 40 to 200 gals is a very good correlation to the subsoil condition at the hills and in the lake area (see Annex III) [6].

There is a clear correlation between the geographical distribution of damage and the type of subsoil. Areas of significant damage are located in the lake zone; outside this zone only minor damage is reported. The area of highest damage is located at the west of the lake zone where the firm deposit is between 26 and 46m (see Annex III) [6].

The lack of damage in the zone of firm soil and in the transition zone it is explained by the small intensity of the ground movement that was not amplified by layers of soft soil. The low damage in the areas of thicker soft soil layers is due partially to the fact that the natural period of vibration of these layers is significantly higher than the dominant periods of vibration of the movements that arrived from the subjacent firm deposits, thus giving rise to a less severe amplification; it is also due to the lower density of tall buildings in these areas [6].

4. CHARACTERISTICS OF THE DAMAGED BUILDINGS

The highest incidence of damage in the most defected area was in buildings from 6 to 15 stories (161 cases). The number of damaged buildings lower than five stories was rather small (101 cases) in comparison with the number of buildings of this type which are the majority in this area, Very few buildings of more than 15 stories were damaged (3 cases).

An approximate census of all existing buildings in the most affected area was made. The data were used to estimate the percentage of damaged buildings with different heights. The percentage are as follows:

Buildings up to 2 storeys	: 2%
" between 3 and 5 storeys	: 3%
" " 6 and 8 "	: 16%
" " 9 and 12 "	: 23%
" higher than 12 storeys	: 22%
Total damaged buildings	: 3%

Taking into account that the predominant period of the earthquake was 1.5 to 2.5sec it is quite rational that buildings with number of storeys between 6 and 15 and with low stiffness as they are mainly frame resistant or waffle slab building with no R/C shear walls appear the highest percentage of damage as the fundamental period of these buildings lies in the same range with the predominant period of the earthquake. In Annex III [6] a classification of the damaged buildings is given according to the number of storeys the year of construction which is related to the followed successive Code modifications and to the type of structure.

The classification regarding structural systems included: R/C frame , waffle slab and steel frame, masonry and others.

In the three first cases the main structural system was commonly stiffened by masonry partition walls and only seldom by R/C shear walls.

In the case of masonry buildings the masonry constitutes the bearing structural system and this system was the most common for low rise construction. The small number of damaged building of this last case reflects the more favorable response of low, stiff buldings. Regarding the first three type of buildings it could be said that those with waffle slab appear the worst behaviour due to the punch effect in the column area.

5. TYPES OF STRUCTURAL FAILURE

The main reason of the large number of failures is the extraordinary severity that the ground motion reached in an area of the city where the characteristics of the subsoil were such that the movements transmitted by the firm subsoil were greatly amplified. The structures were submitted to a very large number of cycles of ground movements of large amplitudes and approximately constant period. The effects of this ground movements were significantly larger than those contemplated by the present code especially for the medium flexible buildings.

However it can not be overseen the fact that the damaged buildings in the most defected area constitute a small percentage for each category of buildings, which means that also other factors contributed to make more severe the effects of the ground movement and they gave rise to some prevailing modes of failure that are mentioned below [6].

a) Brittle behaviour due to column failure.
b) Non symmetric array of partition masonry walls.
c) Soft ground floor.
d) Damages due to previous earthquakes.
e) Short columns.
f) Pounding of adjacent buildings.
g) Overload.
e) P-Δ effect.
h) Punching of waffle slabs.

It is very important also to mention that it is the first time that the failure appeared to such an extend at the upper storeys of the buildings (38% of the damaged buildings). This phenomenon must be studied in depth before conclusions could be drawn. It is also important to mention that in Mexico City a clear correlation of corner buildings and failures appeared too (42% of the damaged buildings are located at the corners of the blocks).

6. FOUNDATION FAILURE

Only seldom the collapse of the building could be totally attributed to a foundation failure; the few collapses of this kind are related with very slender buildings where large overturning moments were produced on raft foundations or friction piles.

It is assumed however that in several cases the settlements and rotations at the building basis due to incipient foundation failure significantly contributed to the collapse of the building amplyfying the P-Δ effect.

In many cases classified as severe damage, the incipient foundation failure was the dominant cause as in tilting of slender buildings [6].

7. ASEISMIC CODE

Before 1957 Seismic Regulations were very simplistic and unconservative. A horizontal load of 2.5% of vertical load was taken into consideration to simulate the earthquake exitation while an increase of 30% of the allowable stresses was introduced for the combination of horizontal and vertical loading.

At the end of 1957 a rather comprehensive set of seismic design recommendations was enforced. From 1976 the present code governs the structural design. It can be said that this code is a modern code that has taken into account the updated knowledge in earthquake engineering at the time it was issued. However it can be said that this Code has not taken into account the special problem of Mexico City which is the soft soil amplification of the ground motion of the stiff layers for long period (1.5-2.5sec) movements. This could be achieved either by regulations imposing the construction of stiffer buildings (lower buildings or buildings of the same height, but with a stiffening shear wall bearing system) or by increasing of the horizontal base shear for the analysis. This second alternative is adopted in the modifications of the code of 1976 introduced after the earthquake of September 19, 1985 for the repair and strengthening of the buildings together with some extra provisions especially for the column design.

It should be noted here that a new updated Code is now elaborated to be in force in due time [6], [7], [8].

8. DAMAGE EVALUATION REPAIR AND STRENGTHENING

For the better information on the procedure followed in Mexico City for damage evaluation repair and strengthening the reporter visited the "DEPARTAMENTO DEL DISTRITO FEDERAL , SECRETERIA GENERAL DE OBRAS (S.G.O)" where he had a long discussion with Ing.Alejandro Rivas Vidal, active technical coordinator. The conclusions of this discussion may be summarized as follows:

a) Just after the earthquake a political counsil was formed with the participation of the army, the police, e.t.c. to take care of the earthquake relief of the city.
b) The general coordination of the activities was entrusted to DDF.
c) The first efforts were oriented to the rescue of lives and to the relief of injured citizens.
d) Then the efforts were concentrated to the gathering of deads.
e) Afterwards, working groups with the necessary equipment began with th demolition and shoring activities.
f) In parallel various institutions as S.G.O., COVITUE, FI UNAM,

C.I.C.M. e.t.c. began to work on damage evaluation.
For this activity there was not a central coordination neither an in advance adopted plan of action. Up to 2-12-1985, 11.802 buildings had been inspected and sometimes one building had been inspeced twice or three times by different intitutions.

g) After October 31 the "Secreteria General de Obras"(S.G.O.) has been in charge for the inspection of buildings with more than 4 storeys while the other institutions continued the inspections in buildings with 4 storeys or less.
   According to the inspections carried out up to December 2 the following results were announced [9].
   - o Buildings with damage in the struct.system with 4 storeys or less 2.299
   - o Buildings with damage in the struct.system with more than 4 storeys 907

- Total partial collapse	:	309
- Severe damage	:	267
- Local damage	:	331
TOTAL	:	907

h) According to the Modification of the Aseismic Code being now in force the owners of the buildings are responsible to notify to the "S.G.O." possible damage and ask for inspection, which means that the inspection of the buildings is limited only to those for which such a notification has been made [8].

i) The damage evaluation especially in the buildings with more than 4 storeys is carried out the high qualified engineers approved by the S.G.O. and according to a detailed format prepared by this service (see Annex IV).

j) The repair and strengthening should be carried out according to a series of new regulations put in force after the earthquake. According to these regulations the damaged buildings are divided into two categories:
   i) Buildings with local damage.
   ii) Buildings with damage which affects the overall structural behaviour.

   For the buildings of the second category a structural reanalysis must be carried out according to these new regulations.It should be noted that according to these regulations much higher horizontal forces are to be taken into account for the earthquake loading and much less ultimate resistance should be adopted for the columns than in the up to now Code in force [8].

k) In parallel an updated new Code is going to be elaborated that will be in force in due time.

9. DAMAGE EVALUATION REPAIR AND STRENGTHENING OF HISTORICAL MONUMENTS

For a better information on the procedure followed in Mexico City for damage evaluation repair and strengthening of historical monuments, the reporter visited the "INSTITUTO NATIONAL ANTROPOLOGIA E HISTORIA, S.E.P., DIRECCION DE MONUMENTOS HISTORICOS" where he had a long discussion with the Architect Restaurer Virginia Isaak, responsible for the restauration of the historical monuments of the City. Afterwards the subscriber was guided by two architects of the Institute to some of the most important monuments to form a personal idea of the situation. The conclusions of the discussions and the visit at site may be summarized as follows:

a) There are in Mexico City and in the surroundings 1425 historical monuments. These monuments belong to three major periods
   - i) Precollonial period (prehistoric period).
   - ii) Hispanic period (historical period).
   - iii) Artistic period (20th century).

b) The prehistoric monuments as they are massive and rigid enough did not appear any damage.

c) For each historical monument a visual inspection is carried out and the findings are recorded on a special format prepared by the Institute (see Annex V).

d) Up to now 500 historical monuments have been inspected.

e) From the inspection carried out so far 100 monuments have been found to have major damage from which24 are churches.

f) The most important reasons of this damage according to Mrs. Virginia's Isaak opinion are the following:
   - Abandonment and lack of preservation .
   - Modification of the original form for alternative uses.
   - Overloading and setbacks.
   - Pounding of adjacent buildings.

g) It is the reporter's personal estimate that the historical buildings having a nassive form with high stiffness were out of fase of the recent earthquake so they did not suffer much. They

had the same behaviour with the majority of the masonry buildings.
The author of this report has inspected the buildings around the Constitution square (Zocalo square), the Cathedral, the Santo Domingo church, the Inglesias de Encarnation and some collonial smaller buildings as well. He has inspected also the most important prehistoric monuments of the city as the Pyramids of Teotihuacan, and the "Templo Mayor". His findings agree with the reasons of damage mentioned above. In the important monuments which are preserved well only the reappearance of old cracks could be observed and in some cases new cracks due to soil settlements.

h) The damage evaluation repair and strengthening procedure is carried out so far without any cooperation with structural engineers. Only restaurer architects are engaged. This has as a result the use of traditional technics in the restauration procedure without any laboratory tests or analytical procedures for the justification of the interventions decided.

10. RESEARCH CONDITIONS IN MEXICO

The members of the consultant's group have visited the "Instituto de Ingeneria de la UNAM" many times and they have got there many discussions for the research activities in progress. They have visited also the laboratories of the institute.

It is the author's opinion that the research in earthquake engineering in Mexico is in a very high level.

It should be noted that within some days after the earthquake of September 19, they were in position to report a series of accelerograms and the first conclusions on the microzonation and the behaviour of the buildings to the earthquake .

However it should be noted also that their research activities would be accelerated if they have got updated laboratory equipment and especially displacement controlled double acting jacks equiped with servovalves and data aquisition system for large scale quasistatic cyclic loading tests.

It is also important to note that the author of this report can not estimate the degree of knowledge transfer from the research institute to the practicing engineers and the degree of usage of the existing laboratory facilities for testing and controll of Construction Industry.

11. TRANSFER OF EXPERIENCE IN EARTHQUAKE ENGINEERING FROM BALKAN REGION TO MEXICO

This task was accomplished with meetings, discussions and primarily with the lectures that the members of the group gave in the University and in the Chamber of Construction Industry. These lectures have covered a very broad range of subjects in earthquake engineering. As far as the author of this report is concerned he presented two subjects.

i) "Repair and Strengthening of historical Monuments", Lecture in the University.

ii) "Post-earthquake rehabilitation activities following the June 20-1978, Thessaloniki's earthquake", Lecture in the Chamber of Construction Industry.

In Annex VI copies of the used diatransparencies for these two lectures are given just for information.

12. CONCLUSIONS

From the above report the following conclusions may be drawn.

a) The main reason of the large number of failures seems to be soil condition in Mexico City which has amplified the stiff subsoil's movement to a large extend for a predominant period of 1.5-2.5 sec. However it should not be overseen that the whole situation worsened by the low stiffness of a large number of buildings and by some typical structural mistakes in many buildings with damage.

b) Only seldom the collapse of the building could be totally attributed to a foundation failure.

c) The aseismic Code in force although modern did not protected enough the City as this Code had not taken into account the special problem of Mexico City which is the soft soil. This is the reason that after the earthquake additional regulations had to be put in force for the repair and strengthening of the buildings.

d) The damage evaluation procedure was not centrally coordinated. It is the author's opinion that a preparedness plan should be elaborated for future use. However the repair and strengthening procedure is well organized.

f) For the monumental buildings a close cooperation should be established among Archeologists, Architects and Structural Engineers so that a justified repair and strengthening methodology would be used, as it happens allover Europe.

g) The research activities in earthquake engineering which are at a high level in Mexico would be accelerated with updated laboratory equipment.

h) The transfer of experience in earthquake engineering from Balkan region to Mexico was accomplished to the maximum possible degree in the framework of a fifteen days visit.

Prof. Dr. Engineer

G. G. PENELIS

REFERENCES

[1] J.PRINCE et al., "Accelerogramas en Ciudad Universitaria del sismo del 19 de Septiembre de 1985", Inf.IPS-10A.Instrumentatión Sismica, Instituto de Ingenieria, UNAM.

[2] E.MENA et al., "Acelerograma en el centro SCOP de la Secretaria de Comunicaciones y Transportes sismo del 19 Septiembre de 1985" Inf.IPS-10B. Instrimentatión Sismica, Instituto de Ingeniería, UNAM.

[3] R.QUAAS et al., "Los dos acelerogramas del sismo de Septiembre 19 de 1985, obtenidos en la central de Abastos en Mexico D.F.", Inf.IPS-10C. Instrumentatión Sismica, Instituto de Ingeniería, UNAM.

[4] J.PRINCE et al., "Espectros de las componentes horizontales registradas por los acelerografos digitales de Mexico D.F. Sismo del 19 de Septiembre de 1985. Acelerogramas en viveros y en Tacubaya", Inf.IPS-10D. Instrumentatión Sismica, Instituto de Ingeniería, UNAM.

[5] E.MENA et al., +Analisis del acelerograma "Zacatula" del sismo del 19 de Septiembre de 1985", Inf.IPS-10E. Instrumentatión Sismica, Instituto de Ingeniería, UNAM.

[6] "Effects of the September 19, 1985 Earthquake in the Buildings of Mexico City". Preliminary report of the "Instituto de Ingeniería, Unam.

[7] "Provisions for Earthquake Resistant Design in Mexico", Al Reglamento de Construcciones para el Distrito Federal 1976.

[8] "Emergency Code for Materials and Construction in Mexico", Diario Oficial, 18-10-1985.

[9] "Control de Inspecciones", Resumen 28-11-1985, Secreteria General de Obras del Departamento del Distrito Federal, Mexico.

A N N E X I

Detailed schedule of stay in Mexico City

Monday, Dec.2 : Arrival in Mexico City.

Tuesday, Dec.3 : Visit of the group of consultants at fully and less damaged buildings guided by scientists of the "Instituto de Ingeniería de la UNAM".

Wednesday, Dec.4: Visit of the group of consultants at the "Instituto de Ingeniería de la UNAM".

- o Presentation there by Prof.MELI of recent available information on:
  - Statistical evaluation of damage .
  - Strong motion measurements and analysis.
  - Correlations on damage distribution and soil conditions (soil predominant period).
  - The main points of the existing and recently modified Code.
- o Discussions on the above presented material.
- o Visit at the laboratories of the Institute and discussion on research programs in progress.

Thursday, Dec.5 : Visit of the group at the Chamber of Construction Industry to attend a video film on the damage in the City.
Visit afterwards of the reporter at the "DEPARTAMENTO DEL DISTRITO FEDERAL" to get information on the administrative scheme that is followed for the damage evaluation repair and strengthening.

Friday, Dec.6 : Presentation of a series of lectures by the members of the consultant's group in the "Instituto de Ingeniería de la UNAM". Discussion.

Saturday, Dec.7 : Additional visit of the consultants at damaged buildings of interest at TLATELOLCO.

Sunday, Dec. 8 : Free for visit at places of cultural interest.

Monday, Dec. 9 : Visit of the reporter at the "INSTITUTO NATIONAL ANTROPOLOGIA E HISTORIA" for getting information on damage evaluation of Monuments and the procedures followed for repair and strengthening.

Visit to some monuments of interest guided by two architects of the Institute.

Tuesday,Dec.10 : Presentation of a series of lectures by the members of the group in the Chamber of Construction Industry. Discussion.

Wednesday,Dec.11:Attendance of the presentation of a series of lectures on new materials and on the base isolation of structures to earthquakes by UNIDO's experts. Participation in the discussion followed.

Thursday,Dec.12: Classification of the collected material ,preparation of the report.

Friday,Dec. 13 : Classification of the collected material, preparation of the report.

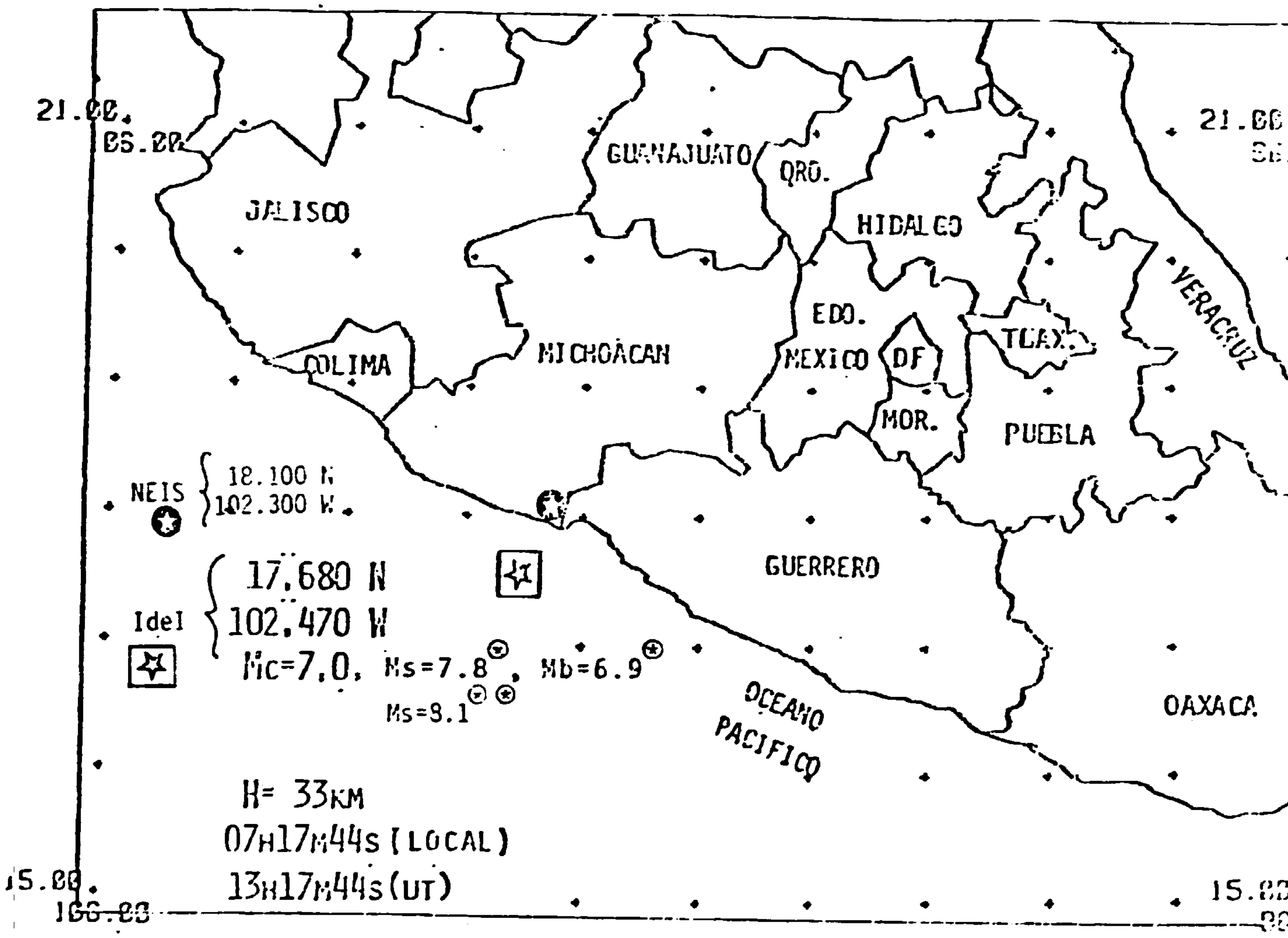
Saturday,Dec.14: Visit of places with cultural interest.

Sunday, Dec. 15: Departure.

A N N E X II

INFORMATION ON EARTHQUAKE CHARACTERISTICS AND STRONG MOTION RECORDS

LOCALIZACION PRELIMINAR DEL SISMO DEL 19 DE SEPTIEMBRE DE 1985.



Datos del National Earthquake Information Service. Paul Bodin, comunicación personal.

Dato corregido por NEIS el 260985.

Fig. 1. Preliminary localization of the epicenter and principal seismological data (Info IPS-10D, fig.1)

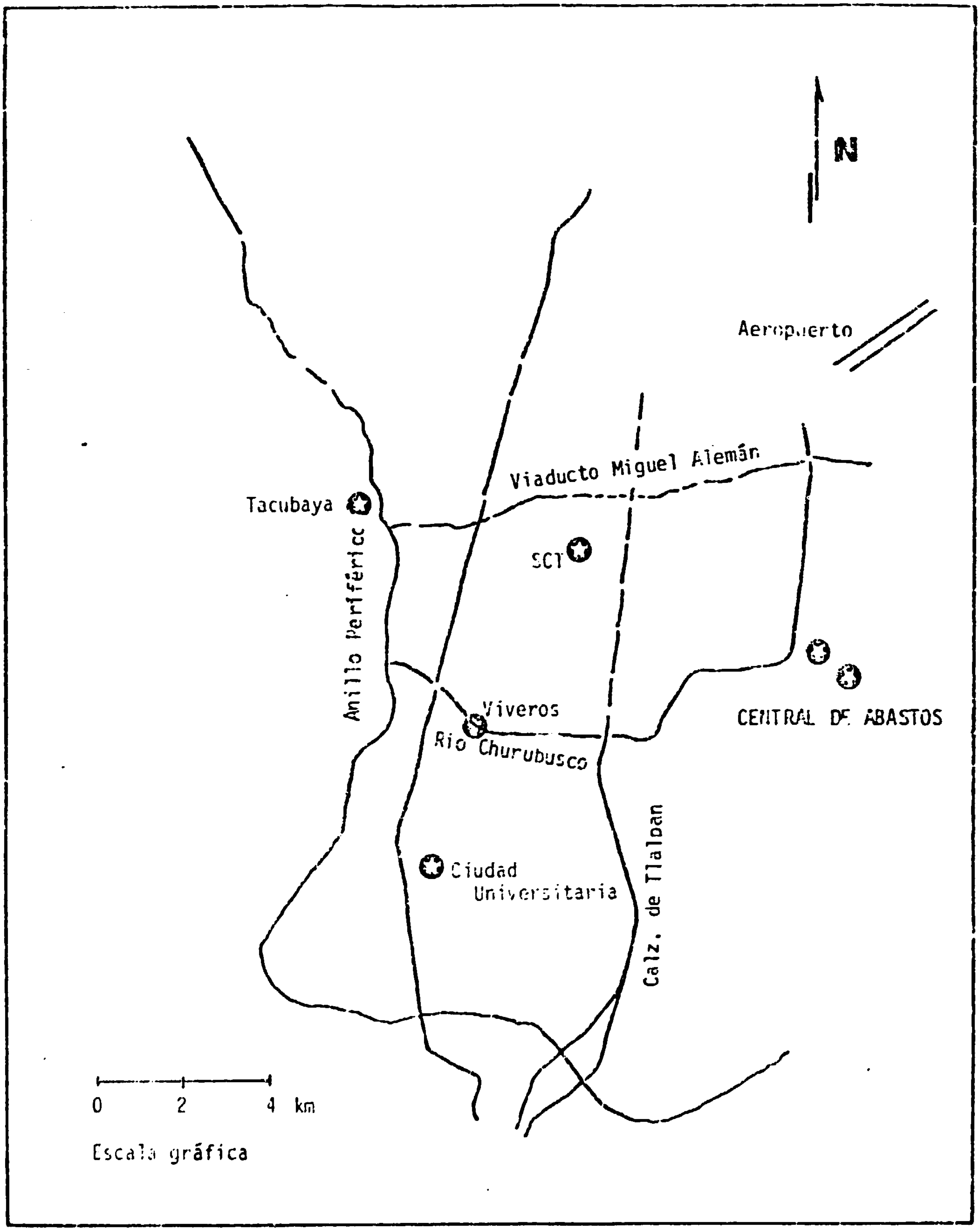


Fig. 2. Location of the digital accelerographs in Mexico City D.F. (Info IPS-10D, fig.2)

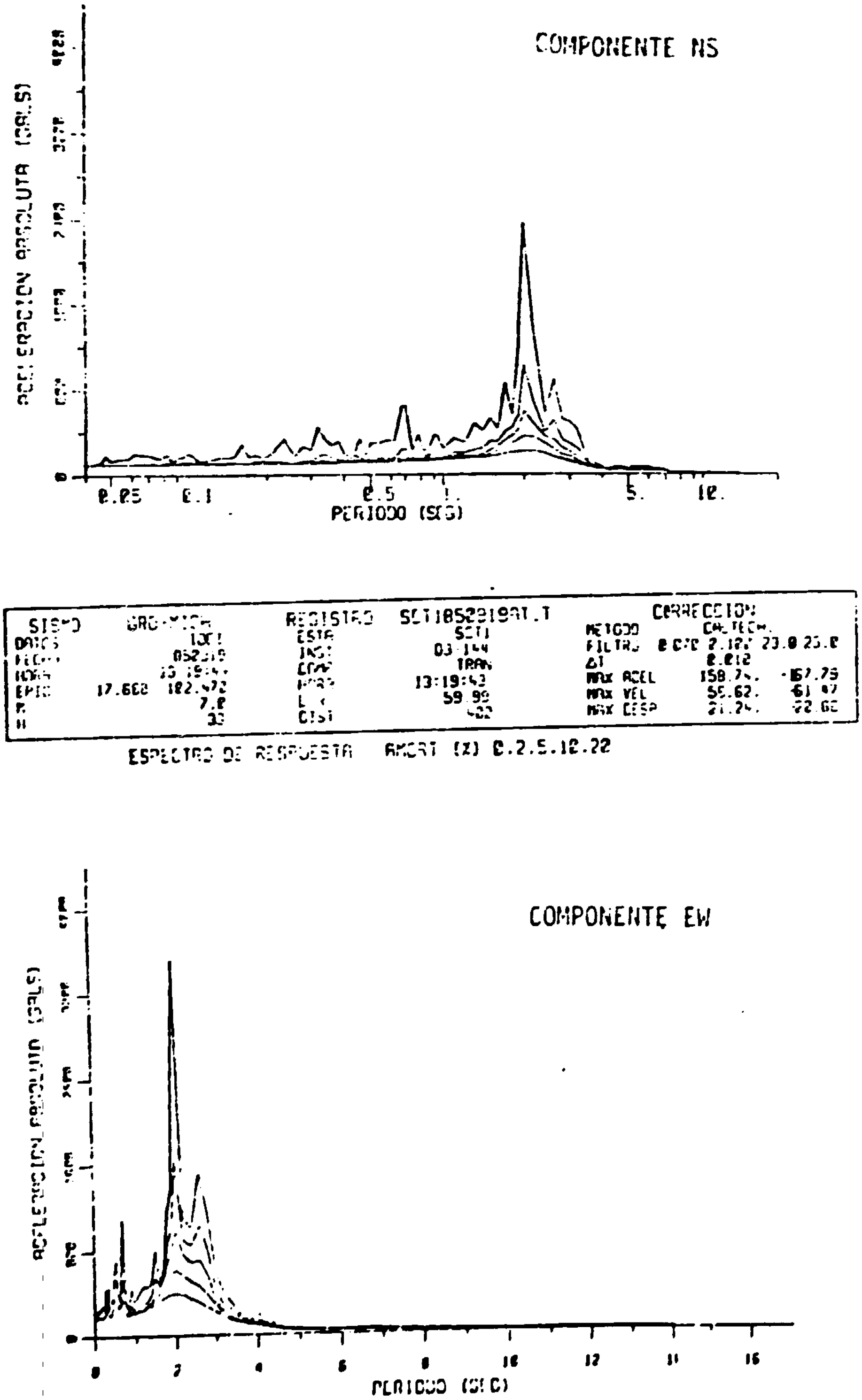


Fig. 3. Acceleration spectra of the accelerogram recorded at the SCOP Center of S.C.T. (Info IPS-10D, fig.10).

VARIABLE	DIRECCION	CIUDAD UNIVERSITARIA			SCT	C. ABASTOS		VIVEROS	TACUBAYA
		CUMV	CUIP	CU01		CDAO	CDAF		
Aceleración gals (cm/seg^2)	NS	37	32	28	98	69	81	44	34
	EW	39	35	33	168	80	95	42	33
	V	20	22	22	36	36	27	18	19
Velocidad (cm/seg)	NS	9	10	10	39	35	25	11	14
	EW	11	9	9	61	42	38	12	10
	V	8	8	8	9	11	9	6	8
Desplazamiento (cm)	NS	6	6	6	17	25	15	9	12
	EW	4	8	7	21	25	19	7	9
	V	5	7	7	7	9	8	7	8

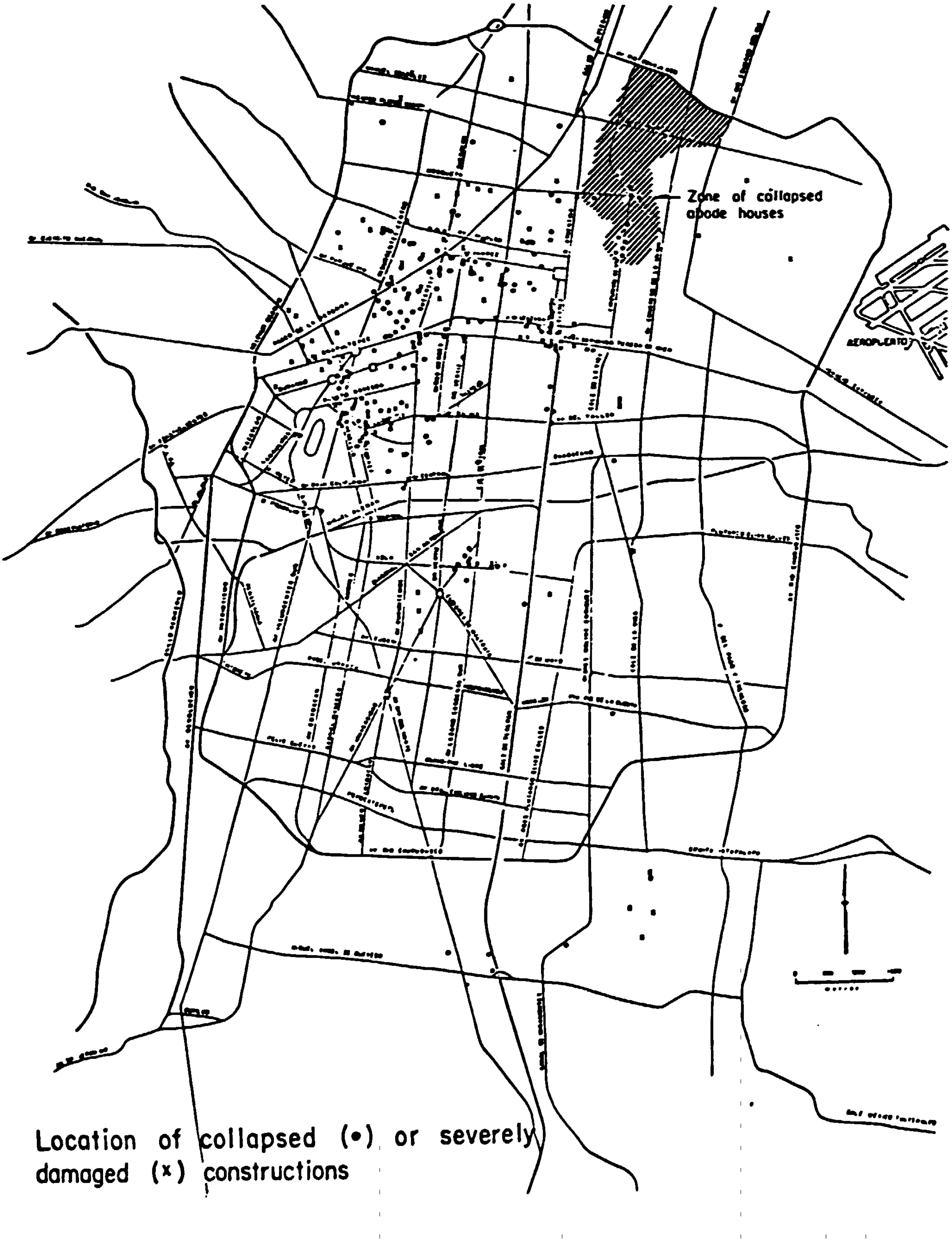
Table 1.
Max ground Acceleration, Velocity and Displacement values recorded in Mexico City. (Info IPS-10D, Table 2).

DIRECCION	PORCENTAJE DE AMORTIGUAMIENTO CRITICO	CIUDAD UNIVERSITARIA			SCT	C. ABASTOS		VIVEROS	TACUBAYA
		CUMV	CUIP	CU01		CDAO	CDAF		
NS	5	109	107	118	598	415	326	167	114
	10	81	66	76	360	232	217	114	84
EW	5	120	133	126	983	340	421	159	99
	10	92	93	89	625	204	284	123	72
V	5	81	79	76	129	107	90	65	72
	10	53	54	52	83	82	63	40	48

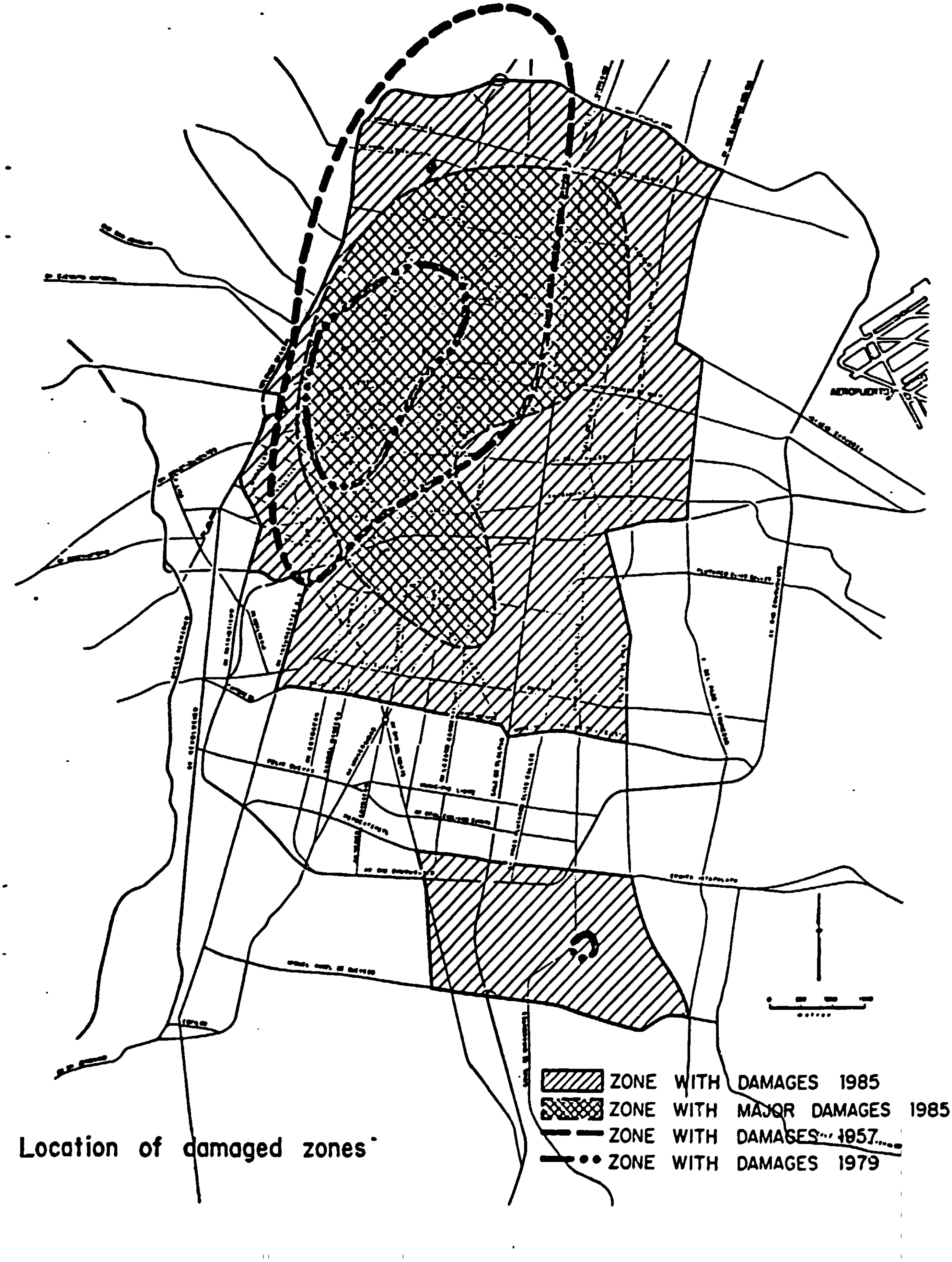
Table 2.
Max spectral Accelerations(gals) of the recorded accelerograms.
(Info IPS-10D, Table 3).

A N N E X III

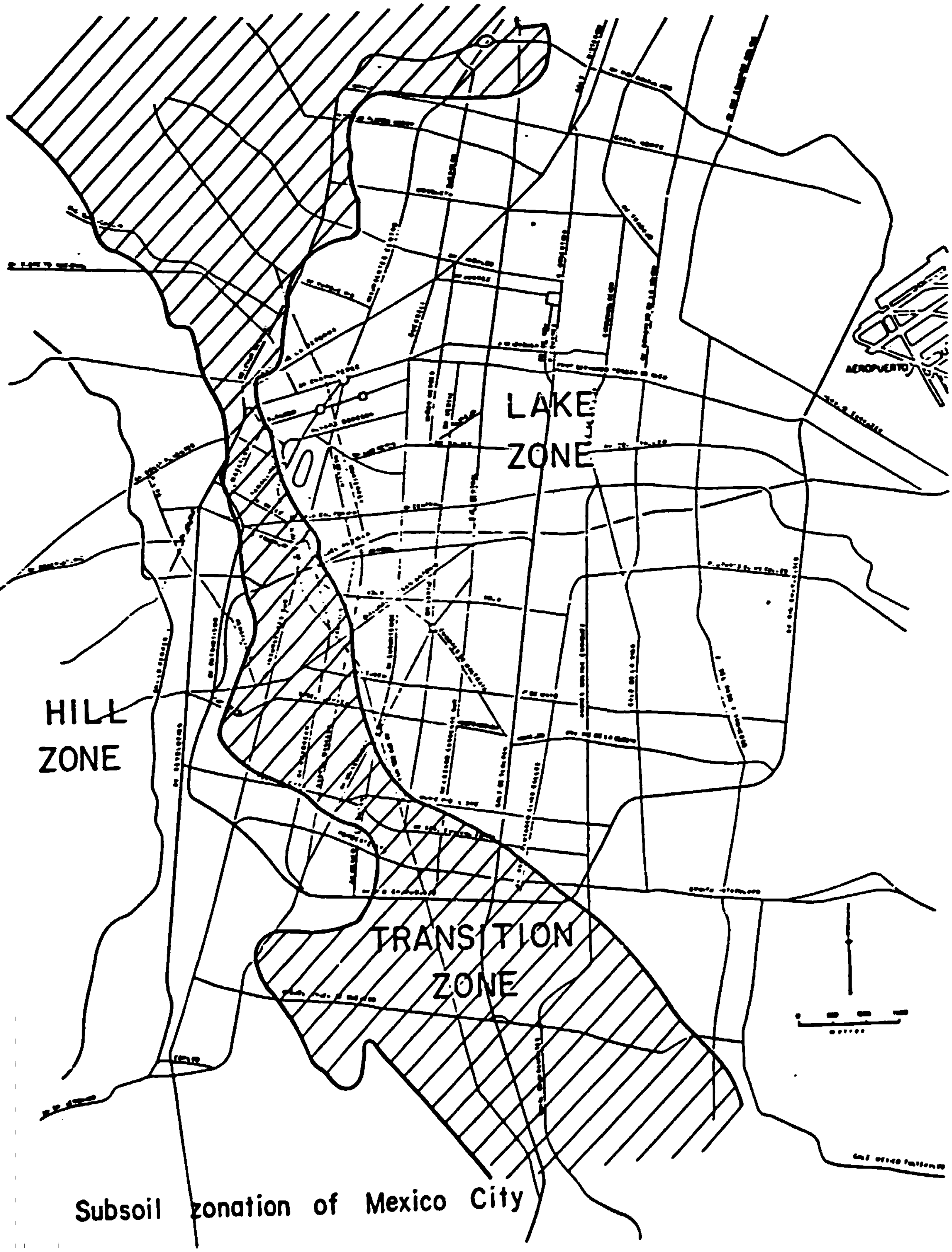
MEXICO CITY MICROZONATION-BEHAVIOUR OF THE BUILDINGS TO THE EARTHQUAKE



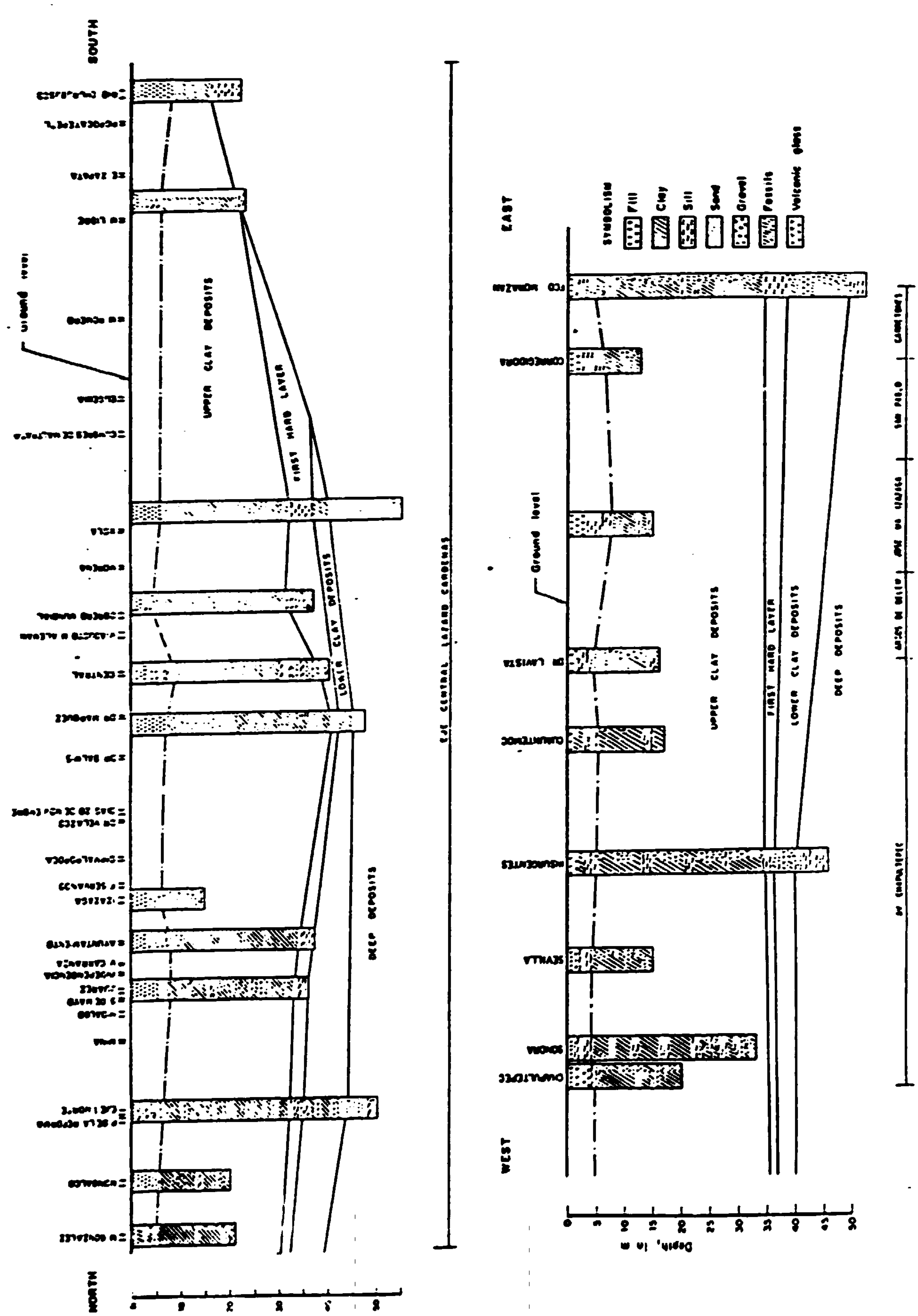
Location of collapsed (•) or severely damaged (x) constructions



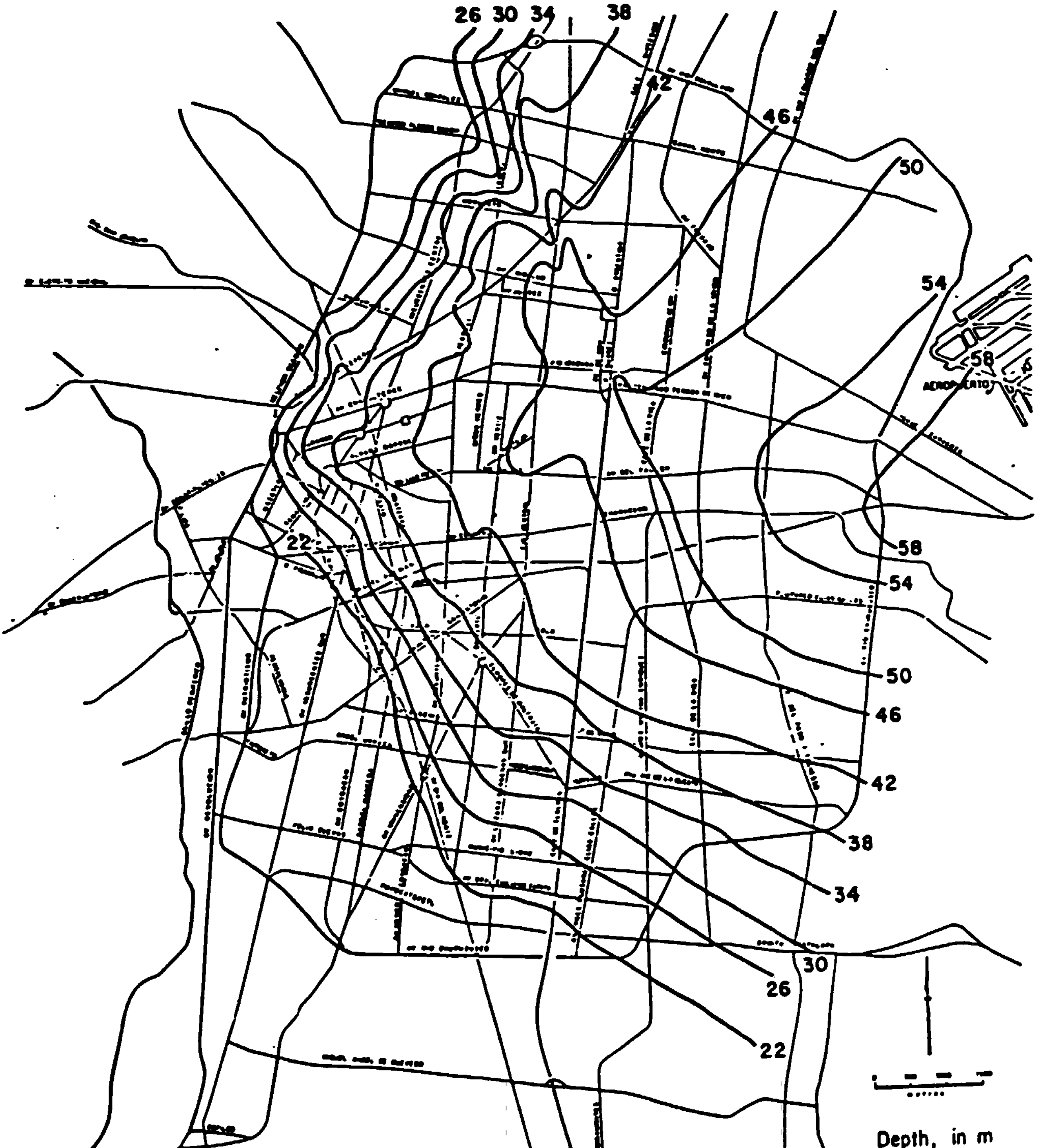
Location of damaged zones



Subsoil zonation of Mexico City



Subsoil profiles



Equal depth curves of the deep deposits

A N N E X IV

LECTURES PRESENTED BY THE AUTHOR DURING HIS STAY IN MEXICO CITY

(Copies of diatransparancies used for
the lectures together with slides)

REPAIR AND STRENGTHENING OF HISTORICAL MONUMENTS

PRESENTED BY PROF G. PENELIS, GREECE.

I. INTRODUCTION

o It must be emphasized from the begining that the attempt of restoring and strengthening a monument is a complicated procedure which demands the collaboration and the coordination of high qualified scientists of various fields. Because, the significance of these works which consist a very inheritance for the World, but also a historical document for the abilities of the Nations that have built them for creation of worldwide significant cultural monuments does not allow improvisations. Any decision for intervention requires a complete justification, as the uniqueness of such structures and the perspective of preserving them for an unlimited period of time do not allow for unjustified decisions, which may prove, in the near or far future, harmful for the monument and its genuiness.

- The monumental buildings as far as their vulnerability, the assessment of their earthquake resistance and their strengthening are concerned may be grouped in two categories.
  - Hinged (without mortar) structures
  - Masonry structures •

In the preceeding presentation only masonry monuments will be dealt with.

2. CLASSIFICATION OF THE MASONRY MONUMENTS BY ARCHITECTURAL FORM

Classification in accordance with their function

- Buildings of worship (churches, mosques e.t.c)
- Public buildings (palaces, mausoleums, markets e.t.c)
- Fortifications (Walls, towers, e.t.c)

3. MATERIALS AND METHODS OF CONSTRUCTION

3.1. Materials

- Mortars
- Bricks
- Timber
- Steel ties

3.2. Methods of construction

- Masonry
- Vaults - Domes
- Barrel vault
- Groin vault - domical - vault
- Drum dome
- Dome on pendetives
- Pendentive dome
- Dome on Squinch
- Half domes
- Timber roofing - Timber ties.

4. STRUCTURAL TYPES - VULNERABILITY

4.1. Classification by structural form

- Free standing walls
- Timber - roofed Basilicas
- Multistory buildings
- Domed circular buildings

- Vaulted and domed Basilicas
- Cross domed central nucleum
- Towers and Minarets

4.2. Structural elements

4.2.I. Free walls

Structural characteristics

- d/h : I:2.5 ÷ I:3
- T : = 0.I sec
- ε^{o}_{max} : 0.30 ÷ 0.33g

Vulnerability

- differential settlements (diagonal cracks)
- Lack of homogeneity (buckling of outer skin)

4.2.2. The Arch and the barrel vault.

Structural characteristics

- $\beta \leq 60^{o}$
- external baking at the base to form a pier

Vulnerability

- cracks at the crown for loads less than the service loads
- Arch collapse is due to failure of the supporting system
- The failure of the supporting system is usualy a shear compression failure.
- cracks and spliting at the crown
- Ties at the base improve the behaviour

4.2.3. Groin - Vaults

- Necessity of "Flying - buttress" or timber ties for the stabilization of diagonal ribs.
- In case, those ties are destroyed havy damage appears.

4.2.4. Domes

Structural characteristics

- spherical dome
- membrane behaviour to large extend
- Ring forces positive for $\varphi \leqslant 50.82^{\circ}$
- Main problem the supporting system
- It is formed as a ring capable in carrying N_{φ} acting as a series of independent buttresses as its tensile strength is not reliable.
- Case study : The ring of Rotonda

  Cross - domed church

  Quincunx Type (Cross - in - square)

Vulnerability

- Existance of cracks in the directios of meridian even for service loads
- The drums of the domes are sensitive at their base.

4.3. Structural systems

4.3.I. Free walls

Already examined as struct.element.

4.3.2. Timber roofed Basilicas

Description of the structural system

- An external rectangle.
- 2 or 4 internal colonades with arcades carried by the columns
- Above the columns and on the arcades rest the galleries
- The timber roof over the nave is raised usualy above the gallery

Structural behaviour

- Because of lack of horizontal stiff connection each element functions indepedently.
- For vertical loads all the elements are free of horizontal thrusts.
- For horizontal seismic loads the system vulnerable.
- Walls $^h/t \approx 15$ Eigenperiod : $T \cong 1.5$-2 sec.
  Out of fase usually.

Vulnerability

- permanent declinations of walls and colonades
- dislocations of vertical masonry walls at their joining line
- Dislocations of the colonades
- Cracks at the crowns of the arches of the arcade.

4.3.3. Multistorey Buildings

Structural description

- It includes internal and external bearing walls
- Floors are usually of timber or shallow masonry vaults.
- Roofs are almost invariably timber.

Structur... ehaviour

- For vertical loads each element functions independently.
- For seismic loading the behaviour of the system depends on the degree of the cooperation of the various elements to transfere the loading to walls parallel to it.

Vulnerability

- It depends on the degree of integral response of all walls to the horizontal shear.

4.3.4. Domed circular buildings

Structural behaviour

- A large spherical dome
- A ring
- A drum with ratio of $h/d \cong 2.2$

Structural behaviour

Already described. Eigenperiod $T \approx 0.02$ sec.

Vulnerability

- Very good behaviour for vertical and for seismic loading as well.

4.3.5. Vaulted and domed Basilicas

Structural description

- Instead of timber roof vaults and domes are used for covering the church.

Structural behavior

The vaults cause strong horizontal thrusts which must be carried either with ties or with flying buttresses.

Vulnerability

Very sensitive to earthquakes even in the form of transept Basilica.

4.3.6. Crossed domed central nucleum

Structural description

- Four strong piers mark the corners of the centre square.
- The piers carry the barrel vaults of four short cross arms
- A dome on pendentives.

Structural behaviour

- The loads of the dome flow through the ring of the drum as vertical and horizontal thrusts to the pendentives and through the barrel vaults smoothly to the piers.

- The system is very raubust and compact
- It has very short eigenperiod and does not amplify the ground acceleration.
- The existance of the shear walls in plan consists a very good layout for seismic loading.

Vulnerability

- Radial cracks in the ring and in the lower part of the dome.
- Horizontal cracks in the base of the drum.
- Diagonal cracks in the spandrels
- Diagonal cracks at the piers

4.3.7. Minarets

Structural description

- It has tubular form, tall and slender and with a stair in the centre. Some meters from the top it has one slightly projecting platform.

Structural behaviour.-Vulnerability

- It is characterized by long natural period.
- The higher harmonics cause usualy damage at the top.

5 REPAIR AND STRENGTHENING

5.I. Introductory remarks

5.2. Initial damage evaluation and temporary shoring.

- Generalities
- Methodology of first inspection
- Preliminary structural analysis for shoring decisions
- Shoring material equipment and arrangements
- Supports for carrying dead loads

- Buttresses for carrying horizontal thrusts.
- Transversal confinements
- Prestressed rings
- Prestressed tendons
- Prestressed ties

5.3. In situ investigations

- Geometrical and constructional survey
- Surveing of the damage
  - cracking.
    - Indication for the future condition of the monument
    - Study of the methodology of repair of each crack.
    - Rough estimate of the grouting work
    - Visual indication of the causes of cracking
  - Tilting.
- In situ destructive and non destructive tests
  - coretaking
  - hammertesting
  - ultrasonic measurements.
- Determination of dynamic properties - In situ tests
- Soil investigations

5.4. Laboratory Tests

- Chemical properties of building materials
  - proportion between paste and aggregates
  - existance of hydraulic agents in the mortar.
- Mechanical properties of the materials
- Soil tests
- Model tests.

5.5. Analytical research of the damaged strutrure

- Determination of the mech. characteristics of the monument
  - $\tau - \sigma$ strength curves

- Modulus of elasticity of the masonry
- Poisson's ratio
- Dynamic charecteristics of the monument.
- Ductility factor
- Seismic base shear.

- Reanalysis of the Monument. Principles and Methods
  - Static loading

    Linear analysis

    - Shell theory
    - Theory of linear structures
    - F.E.M.
- dynamic analysis

  linear analysis for special cases only (Minarets e.t.c.)
- Static loading

  non linear analysis

  - application of upper and lower bound theorems for the collapse mechanism
  - Step by step methods
  - F.E.M. methods
- dynamic analysis

  non linear analysis ⇒ not yet.

5.6. Damage evaluation and assessment of Earthquake resistance

- Load combinations
  - Dead Loads : $\nu_g G$
  - Dead + Seism. Load : $\nu_g G + \nu_e E$

$$\nu = \nu\ (\nu_{f1}\ ,\ \nu_{f3}\ ,\ \nu_{fm})$$

ν_{f1} = for loads

ν_{f3} = for M,N,Q

ν_{m} = for Materials

$v_\varepsilon = 2.0 - 3.0$

$v_\varepsilon = I.I$

$v_E = I.4 - I.5$

Local damage

General type damage

5.7. Materials and methods of repair and strengthening

5.7.I. Generalities

- Materials

  mortars, mortar groutings, epoxy groutings steel reinforcement, steel of prestress, concrete timber.

- Methods

  partial reconstruction

  injections in cracks

  gunite concrete

  cast in place concrete

  prestressed rings

  prestressed ties

  prestressed stitching nails

  transversal confinements

5.7.2. Non ferrous materials

5.7.3. Ferrous materials

5.8. Repair and strengthening proposals

- General repairs
- Local repairs

Principles

- The repairs have to be visible
- The repairs have to be reversible
- Repairs non reversible have to be caried out with materials compatible to those of the monument.

5.9. Final redesign, repair and strengthening

5.9.I. Safety coefficients of actions

a) Masonry walls, piers

- Dead Loads : $S_{act,d} = (2.0 \div 2.3)G$
- Dead + Seism : $S_{act,d} = I.IG + (I.4 \div I.5)E$

b) Steel rings, ties, junctions

- Dead Loads : $(I.6 \div I.75)G$
- Dead + Seism : $I.IG + (I.4 \div I.5)E$

5.9.2. Strength of materials

a) Masonry walls, piers

- Compression : Prismatic Strength (5% fract.)
- Flexure : Flexural strength (" ")
- Bond : 0.5 of Bond strength (" ")
- Shear - Compression : - curves (" ")

b) Steel

- Tensile yield strength

POST EARTHQUAKE REHABILITATION ACTIVITIES FOLLOWING THE JUNE 20-1978 THESSALONIKI'S EARTHQUAKE

BY PROF. G.PENELIS, DR. ENGINEER

1. INTRODUCTION

In this short presentation an effort is made outline the problems raised after the earthquake of June 20-1978 in Thessaloniki and the procedured followed for the rehabilitation of the City and the district in general.

Acting at that period as one of the main consultants of the State for the rehabilitation program I had the opportunity to live from very near the problems in a global way and to contribute to the decision making in all steps of the procedure to the level I could. So, I thought, it would be worthwhile to present to you in short my experience from Greece.

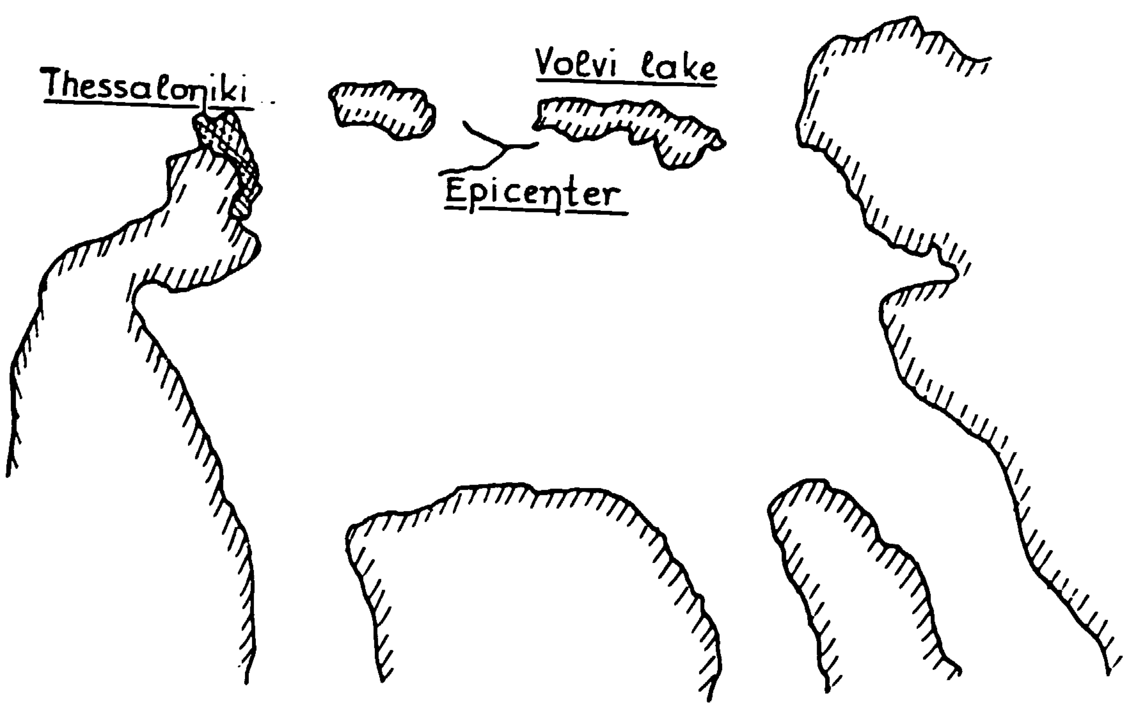
2. CITY CHARACTERISTICS

- Population
  Approx. 1.000.000 inhabitants.
- Layout of the City
  The City is developed like a band around Thermaikos Gulf. The upper part is founded on the bedrock while the lower part on aluvium deposits.
- Number of buildings
  Appr. 60.000 buildings.
- Main type of construction
  - R/C buildings with a structural system consisting of frames and shear walls.
  - High degree of strong partition walls.
  - The predominant number of storeys is 5÷9 storeys.
  - The majority of the buildings were built after 1950.

3. EARTHQUAKE CHARACTERISTICS

- Epicenter of the earthquake
  - The epicenter was near to Volvi lake, appr.20km from Thessaloniki in the Serbomakedonic Zone.
  - The depth of the focus was estimated to 15-20km.

Balkan region and Thessaloniki



Epicenter of the earthquake June 20-78

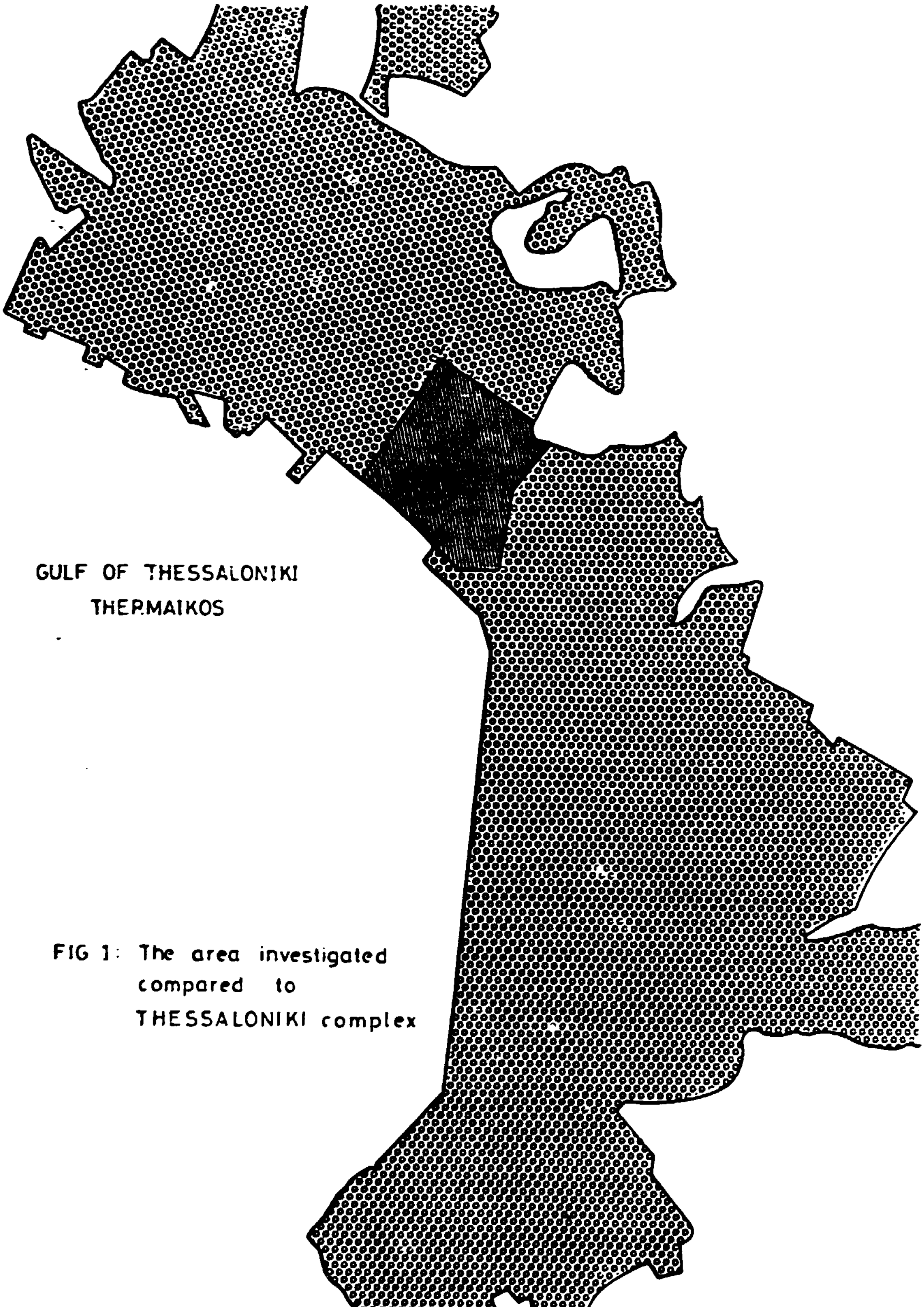
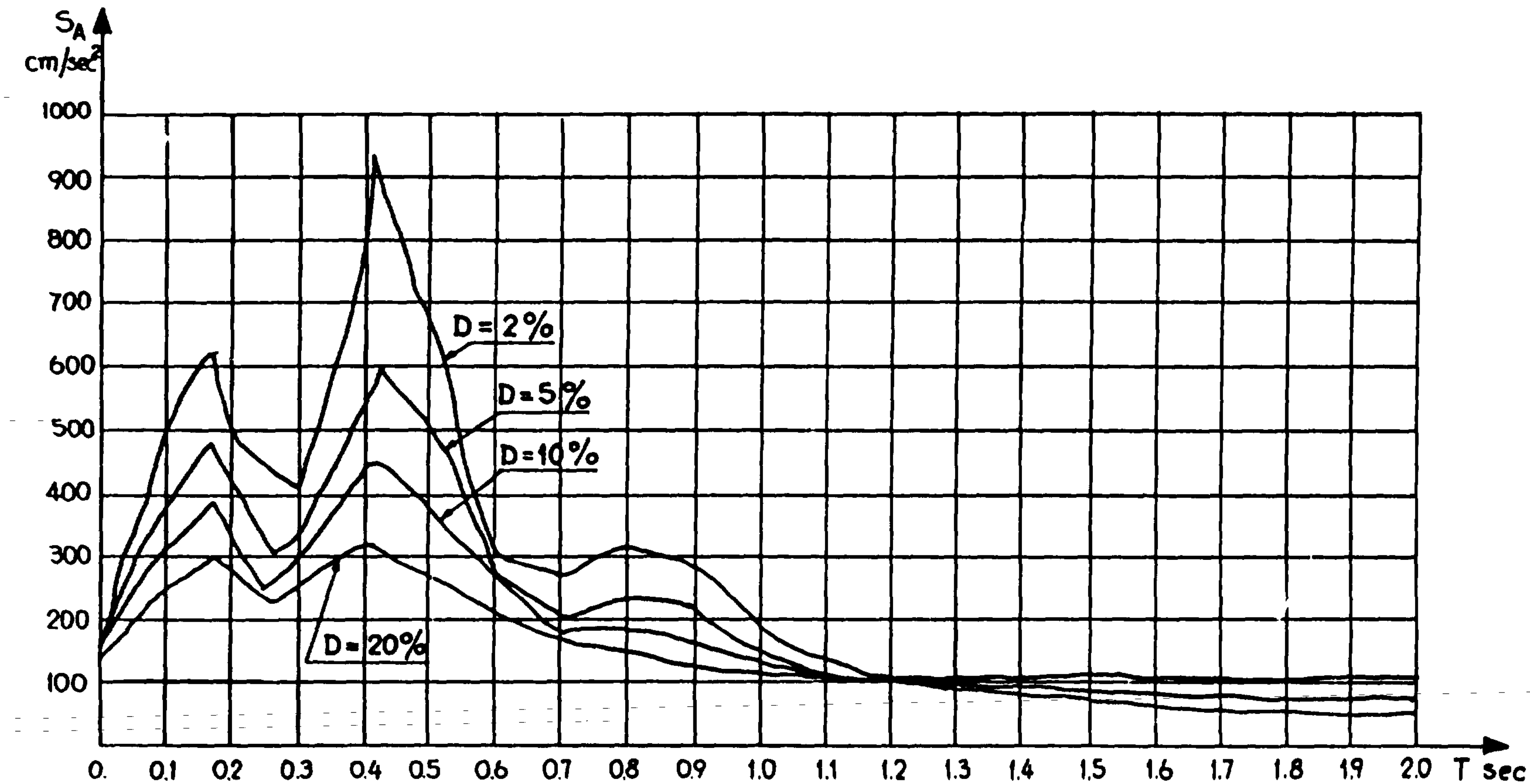


FIG 1: The area investigated compared to THESSALONIKI complex



JUNE THESSALONIKI EARTHQUAKE, S_A-SPECTRUM LONG.COMPONENT

- Magnitude of the earthquake
  - 6.5 Grades in Richter scale.
  - Duration 10sec.
- Intensity of the earthquake in the City.
  - VII÷VIII degrees of M.Mercalli scale.
  - Accerelogram :One in the basement of the City Hotel
  - Spectral analysis of the accelerogram.
    - Max ground accel. : 15-13% g.
    - Predom. period : 0.4-0.5 sec.

4. RESPONSE TO THE EARTHQUAKE

- Buildings
  - One 9-storey building collapsed.
  - 2.5% had severe structural damege.
  - 23.5% had limited structural damage.
  - 74% without visual struct. damage.
- Life lines
  - Limited damage
- Population
  - 50 citizens dead
  - Chaotic situation for the first 24 hours
  - Paralysis of the economic life of the City for many days.
- State
  - It was the first time after fifty years that an earthquake of that intensity had stricken the second big city of the Country.
  - On the other hand, after the earthqukaes of 1954-57 that had stricken a large area of Greece no serious problem had appeared.

  So

  - The State was unprepared to face the problems.
  - The existing general plans for extra notes did not function well in the chaotic situation of the first days.
  - New plans of action had to be decided upon in these first difficult days and nights.

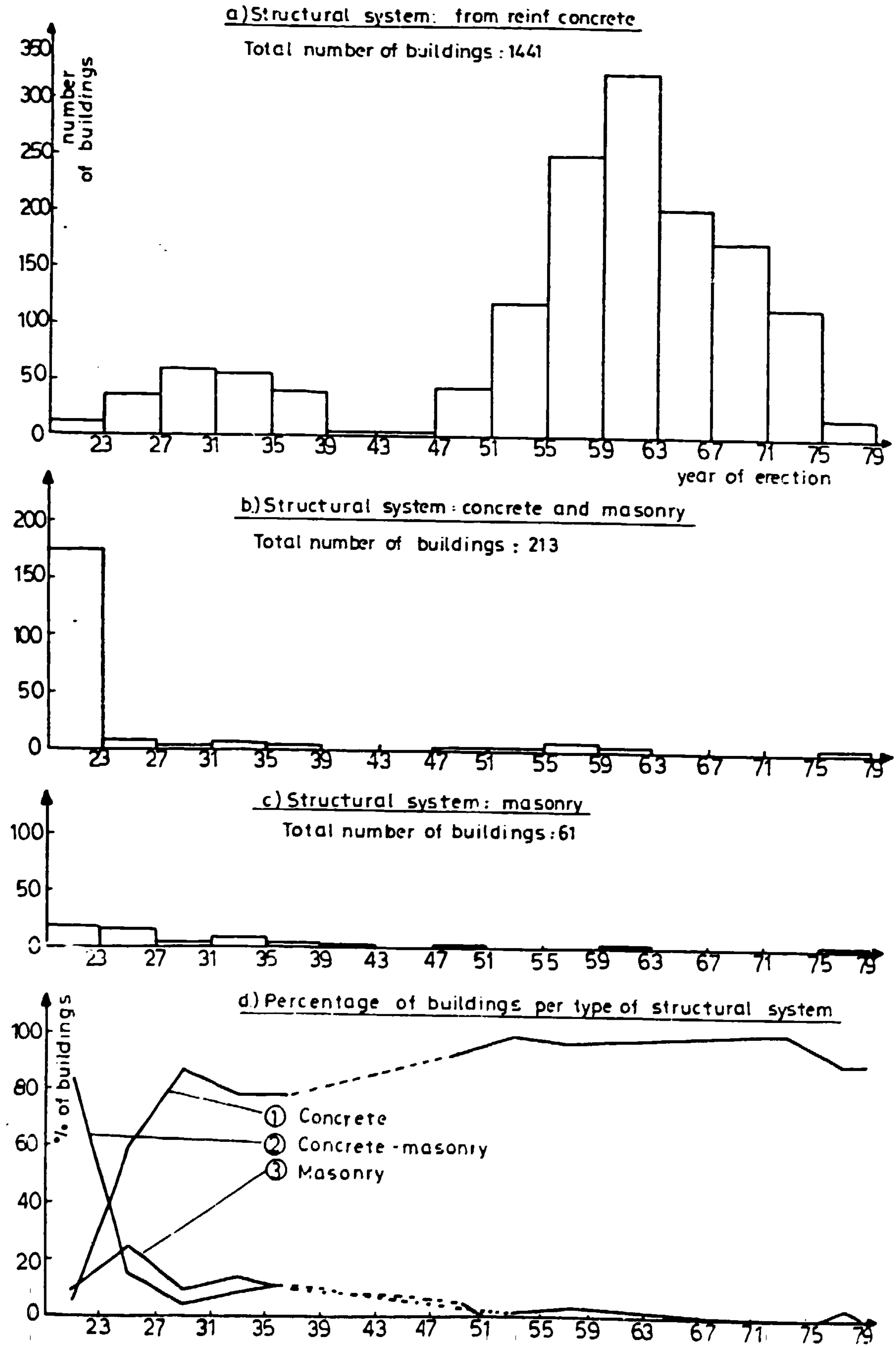


FIG. 3. The evolution of construction practices in the area.

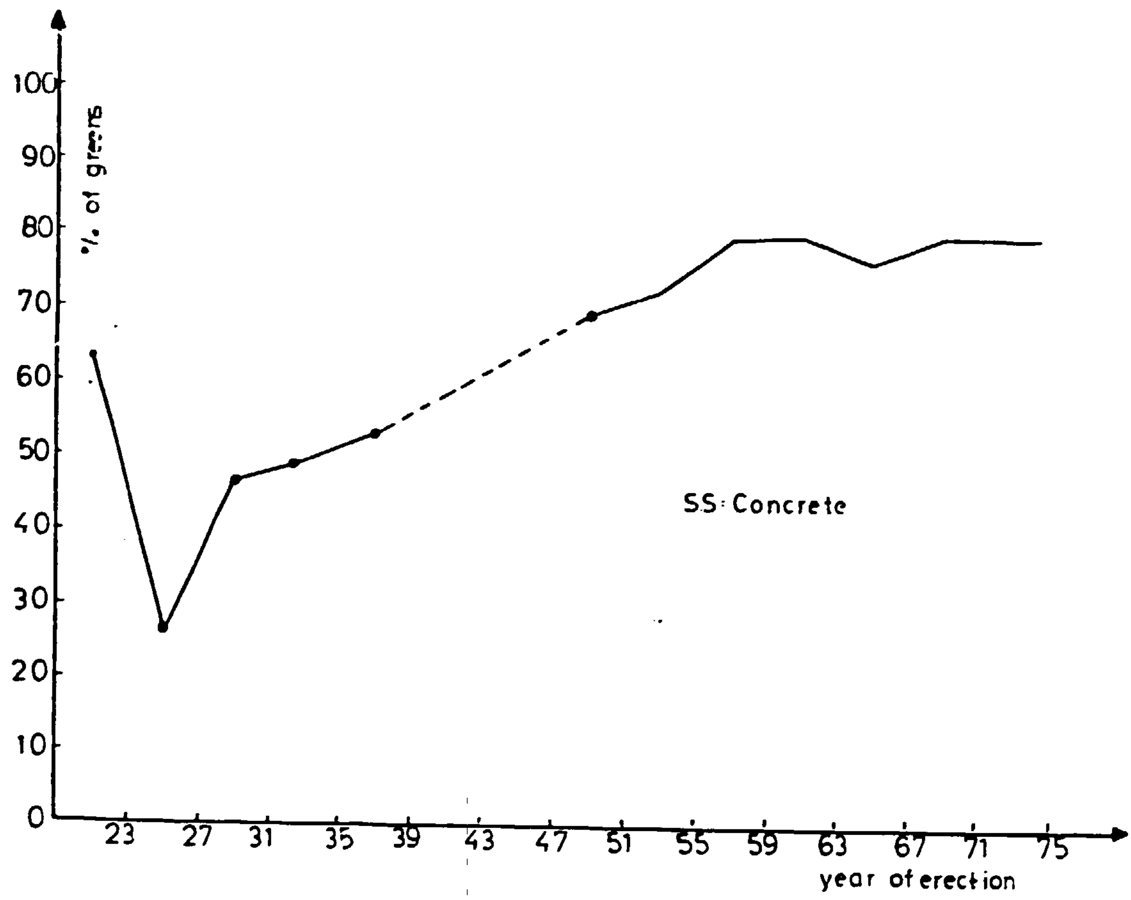


FIG. / Percentage of green buildings in the whole area, as a function of the year of evection.

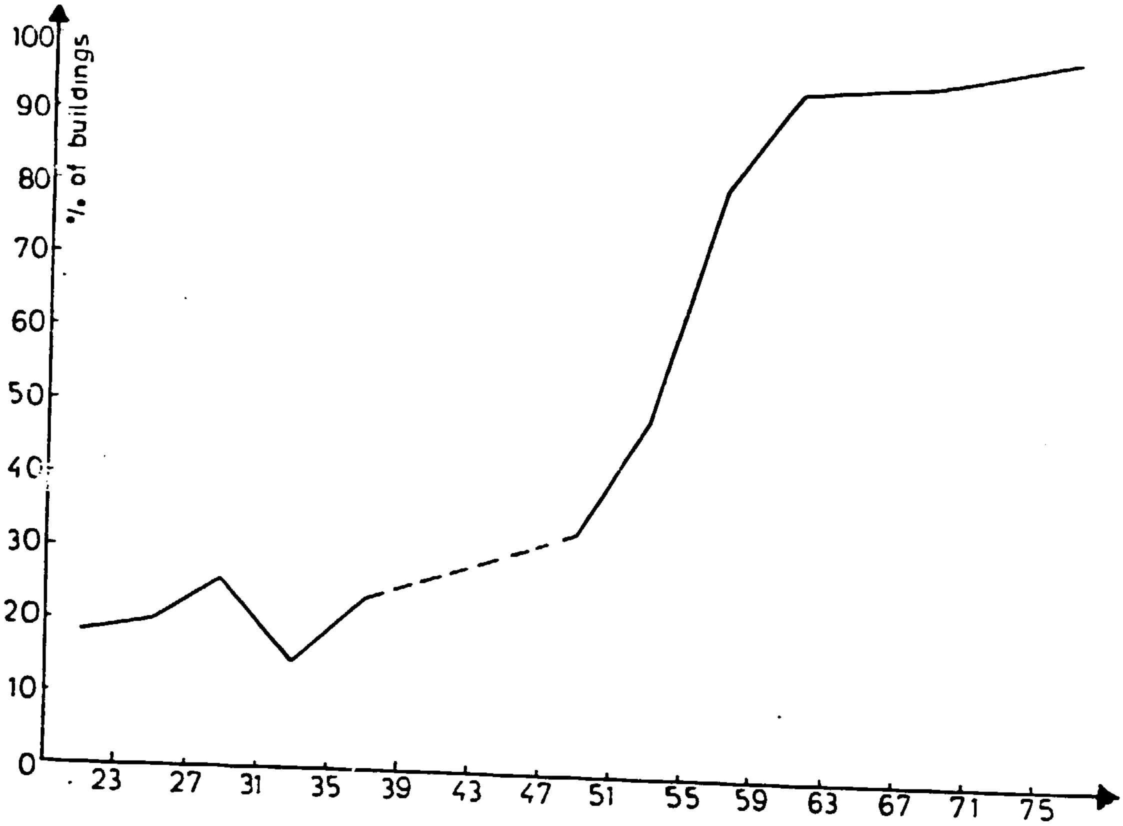


FIG. 15. Percentage of buildings incorporating shear walls at the staircase.

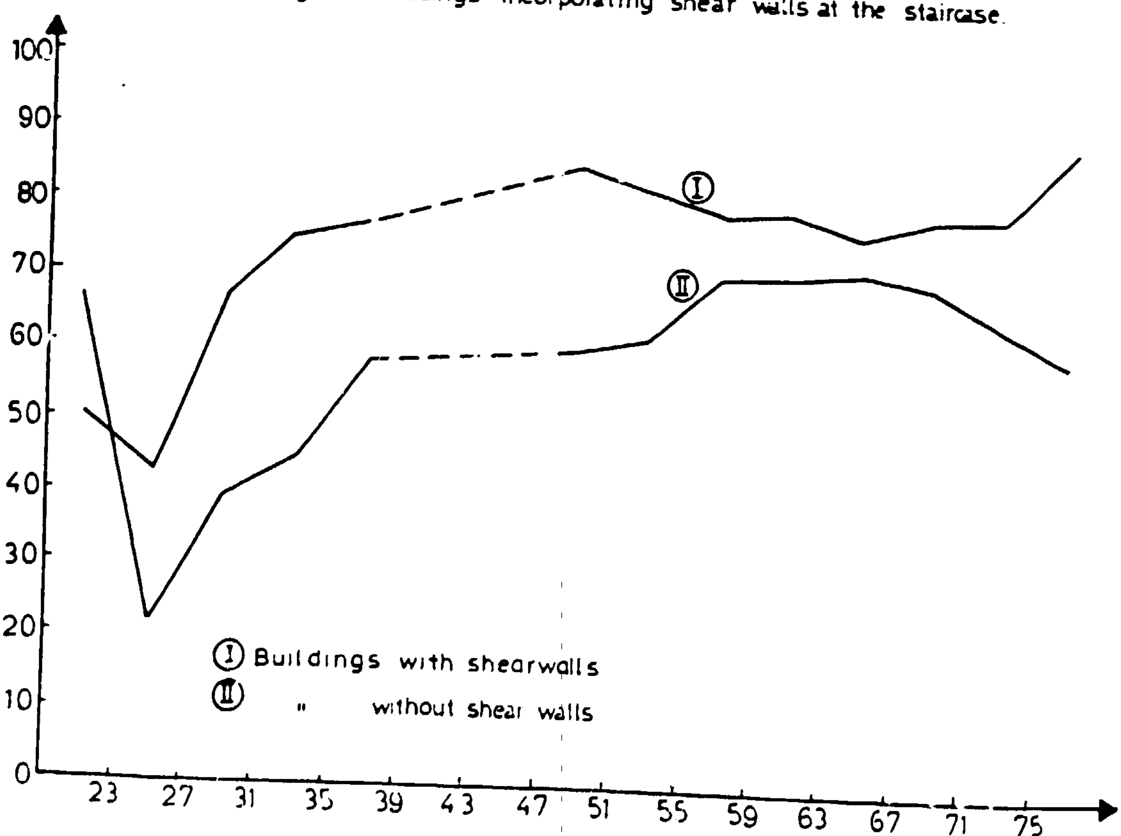


FIG 16 Percentage of green buildings as a function of the year of erection and the use of shear walls

5. DAMAGE EVALUATION AND ASSESSMENT

• Buildings to be inspected

by indiv.engineers | by a State Service

After Call | All Buildings

After Call

fors	againsts
•limited work	•limited control
•no Law problems	•unreliable final estimation

All Buildings

fors	againsts
•high degree control	•high cost
•good losses estim.	•long term deval. of the real estate
•feeling of safety	•high degree of coordination

It was decided that all buildings should be inspected and officially characterized

• Type of inspection

Qualitative visual inspection of the structural system of the buildings

• Degrees of damage

• No visual damage in the structural system
Unlimited use of the building (G R E E N)

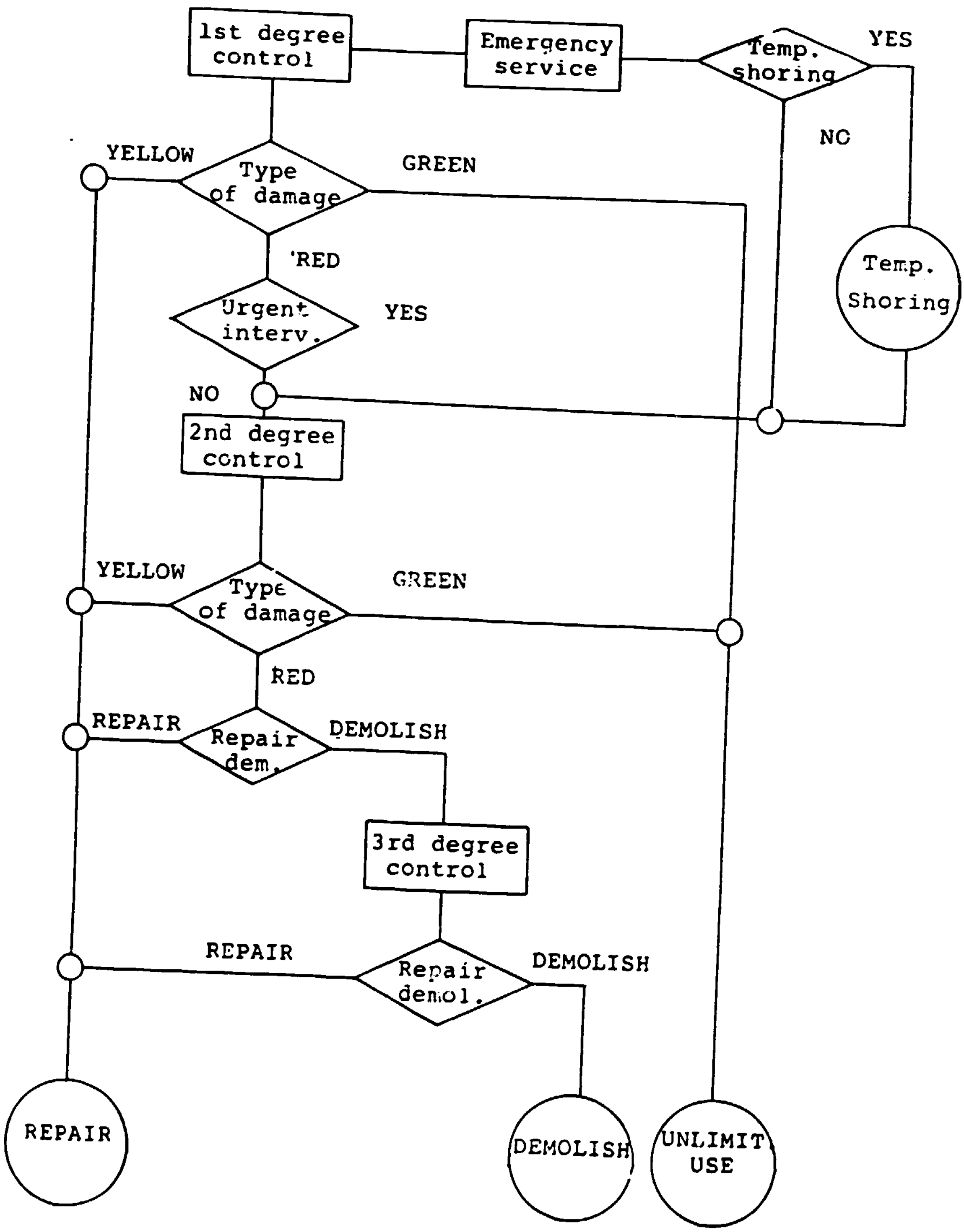
• Limited not severe damage in the structural system
Limited entrance in the building on users responsibility (Y E L L O W)

• Severe damage in the structural system
Entrance in the building Prohibited (R E D)

• Administrative scheme

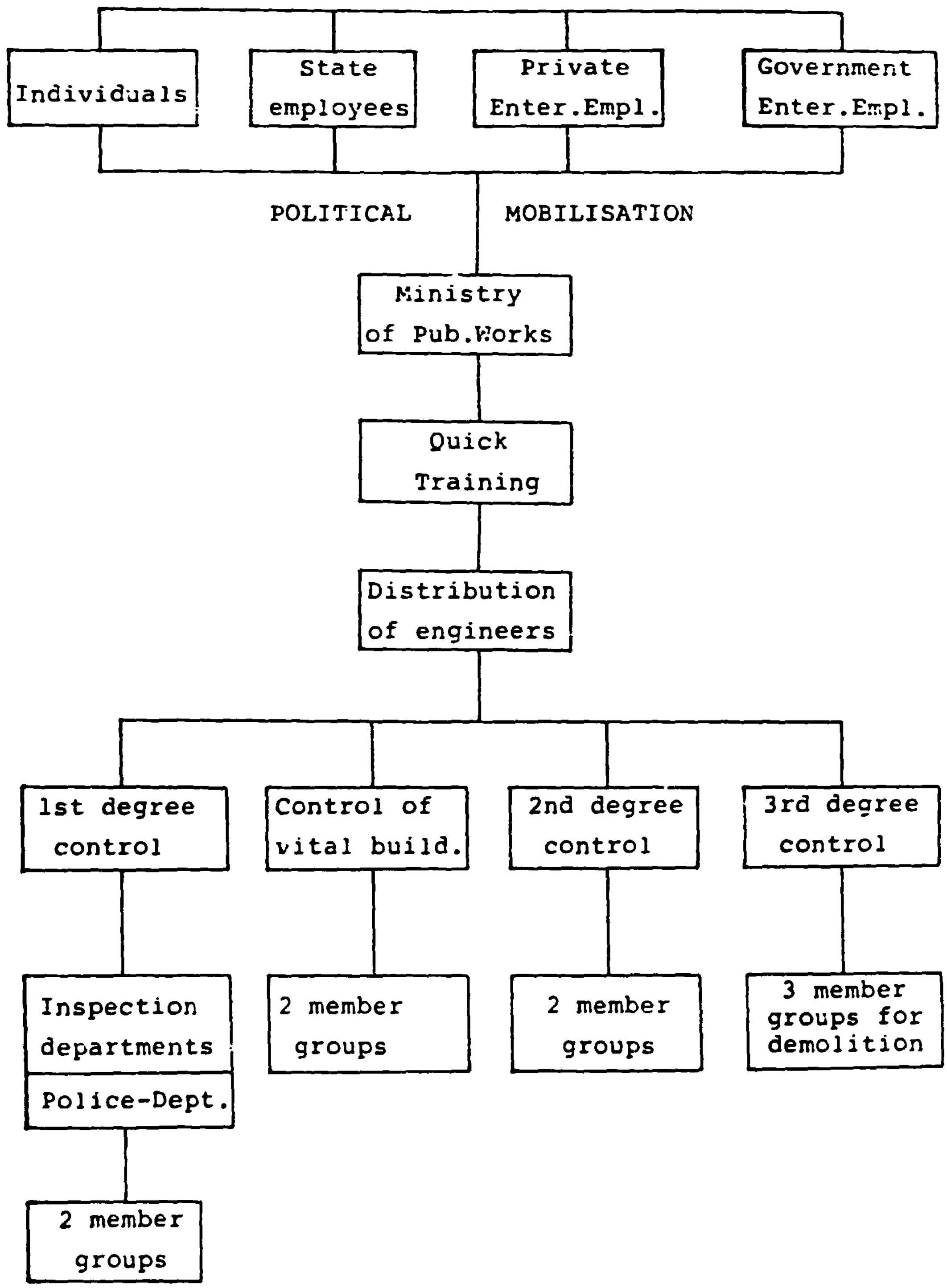
As a damage evaluation after an earthquake is a job that

a) Requires a large number of engineers
b) Includes a degree of subjectiveness



Damage evaluation procedure

CIVIL ENGINEERS



<u>Administrative scheme of the Service charged with the building inspections</u>

So
an administrative scheme was necessary to secure:

- Quick termination of the whole procedure
- Uniform and objective if possible assessment
- Ability for tracing of mistakes of the first inspection
- Quick information of the shoring and demolishing service in case of urgent cases.

The basic points of the elaborated plan were the following:

- Inspection by two member groups of Civil Engineers
- Preparation of proper formats
- Provision of a second degree inspection
- Provision of quick information of the shoring and demolition service
- Inspection of the buildings of vital importance with priority

- For the whole campaign a number of appr.1000 Civil Eng. was required that was covered through political mobilisation for 45 days.
- This staff was distributed properly in departments and was centrally controlled by the Ministry of Public Works.
- The whole campaign terminated in 40 days.

5. REDESIGN AND REPAIR

- Problems raised
- Financing problems
- Administrative problems
  i) Degree of State interference
  ii) Necessary State Services for the campaign.
- Bureaucratic arrangements
- Scientific and Technical problems
  i) Level of strengthening
  ii) Specifications for repair and strengthening
  iii) Cost analysis and standarisation
  iv) Training of Engineers and Technicians
  v) Provision of the required labourers.
- Law arrangements

- <u>Financing of the repair and strengthening</u>
  - The whole activity was supported by the State with 70% of the cost as a loan for a 20 year period with an interest of 3% and the rest 30% as Social aid.
  - Extra taxes had to be added.

- <u>Administrative problems</u>
  - It was decided that the State through the Ministry of Public Works would interfere for the private buildings only in the control of the design to give the permission of the works, the supervision of the repair activities and the control of money consumption. For the public buildings it was decided the works to be carried out with the procedure of the public works construction.
  - For the whole campaign a new service was established in the Ministry of Public Works where some of the most high qualified civ.engineers were moved from other services and many new were employed.

- <u>Bureaucratic arrengements</u>
  - It was decided and a new Law came in force to simplify the whole procedure for the approval of the designs and the issue of the permission for repair so that the whole procedure could run quickly.
  - The required time for a usual case was about 15-20 days.

- <u>Scientific and technical problems</u>

Accepted level of strengthening

According to a new updated Code

According to the Code with which were designed

Division of the damaged buildings into two categories

i) Buildings with local damage
ii) Buildings with damage that affects the overall struct. behaviour

<u>First category</u>

Local repair following newly issued specifications

<u>Second category</u>

Reanalysed and redesigned according to the Code in force taking into account for the elements under repair and the new ones the requirements for strength and ductility established by the specifications

- The Ministry of Public Works in cooperation with the Technical Universities of Athens and Thessaloniki issued in a three months period after the earthquake, specifications for the redesign repair and strengthening.
- In the same period unit prices were elaborated for the works of repair and strengthening necessary for the objective determination of the financing height in each case.
- As the most engineers were not experienced in the reanalysis, redesign, repair and strengthening procedures, seminars had to be organized. The whole procedure was carried out by the Ministry of Public Works in cooperation with the University of Thessaloniki and the Technical Chamber of Greece.
- In order that labour and meaterials be secured for the repair and strengthening the issueof permissions for new building was suspended for a period of six months.

• Law arrangements
  - For all these activities a series of new laws had to be approved or modified by the Parliament and especially:
    - The Law of horizontal ownership.
    - The Law of building demolition.
    - Overcoming of the General building Code for strengthening reasons or for reasons of reconstruction.

7. CONCLUSIONS

The State should be organized in advance in detail and in visible way to be in position to face the problems that raise after a strong earthquake in a big city.

III. FINDINGS AND RECOMMENDATIONS ON THE MICHOACAN-GUERRERO EARTHQUAKE OF 19 SEPTEMBER 1985, WITH SPECIAL REFERENCE TO SOIL-STRUCTURE INTERACTION

BY

M. Ö. ERDIK

LIST OF CONTENTS

PREFACE

This mission undertaken under the initiation, guidance and the contractual agreement with UNIDO has contributed to the exchange of information gained in different regions of the world on the earthquake performance of structures and has paved the path for better understanding of the earthouake phenomena through scientific and technical interaction. In this respect my sincere thanks are due to Mr. Petrossian of UNIDO, Mr. Silva-Aranda, UNDP Resident Representative and to Ms. Schubert, UNDP Program Officer.

Prof. L.Esteva, Prof.R.Meli, Prof.J.Prince and Prof.M.Romo of Instituto de Ingenieria, UNAM have generously shared their information and vast experience on this earthquake, and these contributions are gratefully acknowledged.

1. INTRODUCTION

Following the earthquakes of 19 and 20 September 1985 in Mexico with disasterous consequences in the Mexico City and on the basis of the request of the Government of Mexico from UNIDO to benefit from the experience gained in UNIDO's previous projects on relevant subjects, A team of Earthquake engineering experts, including myself have been contracted by UNIDO for the first two weeks of December, 1985 to provide technical advice on earthquake resistant construction, strengthening and repair of buildings in the Mexico City.

The team of experts recruited on the basis of their involvement in the RER/79/015 UNDP-UNIDO Balkan Regional Project consisted of Prof.J. Bauwkamp (team leader), Prof.M.Erdik, Prof.G.Penelis, Prof. M. Velkov, Dr E.Csorba and Prof. E.Dulacska. The duties of this team, as elaborated in the UNIDO Job Description No: SI/MEX/85/804/11-55/32.1.K, encompases the assessments of the earthquake performance of various type of structures, study of the strong ground motion, soil-structure interaction and site response as well as the exchange of information with the Instituto de Ingeneria, UNAM and the consultation on the repair and strengthening of damaged structures.

Although most of the activities pertinent to these duties have been undertaken as a team, it has been decided to have each member of the team to concentrate on a specific area and to prepare the individual report on the same area so as to form a specific entity of the final report to be synthesized by UNIDO for presentation to the appropriate authorities. In this context, I have been assigned the assessments and suggestions involving (1) the strong ground motion, (2) site response (soil amplification) and (3) the soil-structure interaction (foundation response) aspects of the duties and of the final report.

After this introductory chapter, the Chapter 2. of this report covers my program in this mission. Chapter 3 provides a general assessment of the earthquake disaster, followed by the Chapters Strong Ground Motion, Site Effects and Soil-Structure Interaction. The last Chapter of the report is devoted to the suggestions for future undertakings.

2. PROGRAM OF THE MISSION

The brief program of the mission can be given as follows

December 2, 1985 -	A.M. -	Visit to UNDP, Briefing
	P.M. -	Visit to UNAM (Prof.Esteva) Visit to Chamber of Construction
December 3, 1985 -	A.M.,P.M. -	Visit to Damaged Areas in Mexico City
December 4, 1985 -	A.M. -	Meeting with Prof.Meli, UNAM
	P.M. -	Meeting with International Association for Earthquake Engineering (IAEA) team on earthquake performance of rural structures (Prof.Arya, Prof.Ichikawa, Dr.Scawthorn, Prof.Neuman, Dr. Boen)
December 5, 1985 -	A.M. -	Meeting with Prof.Prince, UNAM
	P.M. -	Meeting with Chamber of Construction (Watching Video Movie on Earthquake)
December 6, 1985 -	A.M.,P.M. -	Presentations on various relevant subjects in UNAM by Prof.Bouwkamp, Prof.Erdik, Prof.Velkov, Prof.Penelis.
December 7, 1985 -	A.M.,P.M. -	Visits to damaged structures in the Mexico City. (Masonry)
December 8, 1985 -	A.M.,P.M. -	Visits to damaged structures in the Mexico City (Nanoalco-Tlaltelelco appartment complex)
December 9, 1985 -	A.M. -	Meeting with Prof.Romo, UNAM
	P.M. ·	Visit to damaged structures in the Mexico City.
December 10, 1985 -	Presentations at the Chamber of Construction.	
December 11, 1985 -	Presentations at the Chamber of Construction.	
December 12,13,14 -	Other Businness and Return of the Consultants.	

3. GENERAL ASSESSMENT

The main shock of earthquake occurred at 13:17:47.8 UTC (07:17:47:8 Mexico City time) on September 9, 1985 with a Surface Wave Magnitude of 8.1 as reported by the U.S. Geological Survey. The epicenter was located ($18^{o}.18$ N - $102^{o}.58$ W) near Lazaro Cardenas, a small town on the Pacific coast of Mexico in the State of Guerra. The epicentral region of the earthquake is located in the so-called Michoacan Gap, previously identified by Singh et.al.[1] to be capable of producing a large earthquake in the next few decades.

More than 500 structures in Mexico City, located at about 400 km from the earthquake epicenter, collapsed or severally damaged causing an estimated 5,000 - 10,000 deaths, at least 11,000 injuries, and economic losses of US $ 5 - 10 billion. The extraordinarily high degree of damage at this large epicentral distance was mostly due to specific site amplification of ground motion by the alluvial sediments in Mexico City. Past [illegible]ant earthquakes also originating from the Pacific Coast (e.g. 28 July [illegible], MS = 7.5) had also caused similar damage attributed to the same reason

The peak accelerations measured in the epicentral (near-field) area ranged between 0.10 - 0.20 g with almost white spectra These are considerable lower than those that would be expected from a shallow earthquake of such a high magnitude. The accelerations measured in the Mexico City ranged from few percent of the acceleration due to gravity on the "firm ground" zone, to 0.20 g with a coherent time history at 0.5 Hz. on the "compressible soil" zone.

Most of the collapsed structures were in the 5-15 story range, with relatively less damage in lower and taller buildings, indicative of the pseudo-resonance phenomena between the vibration these structures and that of the underlying sediment. Complete foundation failure seems to be involved in about 10 percent of the collapses. Most of the damaged reinforced concrete buildings tended to have waffle slab - flat plate type construction with long span beams and relatively slender columns. The damaging interaction between the closely spaced buildings were also found to be contributory to the damage.

---

(1) Singh, S.K., L.Astiz and J.Havskov (1981), Seismic Gaps and Recurrence Periods of Large Earthquakes along the Mexican Subduction Zone: A Reexamination, Bull.Seism.Soc.Am., 71, pp.827-843.

For example the failure of a 21-story steel building also caused the failure of a 14-story nearby building. Critical effects from plastic foundation movements seems to be an important factor for the overall building performance. There has been at least one instance of complete shear slip-circle failure of the foundation of a 10-story reinforced concrete building and several instances of buildings leaning out of plumb for at least 10 degrees. Government buildings,as a group, sustained considerable damage compared to the privately owned buildings.

The Constructions Regulations for the Federal District of Mexico-1977 (Manual de Diseno por Sismo, Segun el Reglemento de Construcciones para el Distrito Federal - Universidad Nacional Autonama de Mexico) used in the earthquake resistant design of the post-1977 buildings has been found to be in conformity with the modern concepts and developments is the Earthquake Engineering. The code provides the seismic coefficients of 0.16, 0.20, 0.24 respectively for the three microzones of Mexico City involving firm, transition and compressible soil grounds with a maximum allowable ductility of 6 for unbraced frame structures. A set of emergency modifications imposed on this code for the post-earthquake repair-strengthening and construction purposes (Modificaciones de Emergencia al Reglamento de Construcciones Para el Distrito Federal) have increased the seismic coefficients to 0.27 and 0.40 respectively for the "transition ground" and the "compressible soil" zones and have reduced the maximum ductility factor to 4.

The post-earthquake reconstruction in the Mexico City have already started. It fill be a slow and expensive process worthy of due care and attention to avoid future disasters.

4. STRONG GROUND MOTION

The Guerrero accelerograph array, installed as a joint project of the UNAM (National Autonomous University of Mexico) and the University of California at San Diego (UCSD) to record strong ground motions on bedrock from large subduction earthquakes, had 20 of the 30 planned accelerographs in place. Seventeen of these stations produced strong motion records as reported in the Preliminary Report GAA-1A(2). Figure 1 shows the locations of the accelerometers with peak accelerations in each of the three axes. Measured peak accelerations in the epicentral region are about 0.16 g with about 25 seconds of strong shaking with accelerations about 0.10 g. At an individual strong motion station operated by UNAM at Zacatula in the epicentral area have recorded peak accelerations reaching 0.27 g. Accelerations reaching 0.2 g are measured with the same array in the epicentral region during the September 21, 1985 major aftershock (MS = 7.5) of the earthquake.

These recorded accelerations on the bedrock produce pseudo velocity response spectra which are almost flat in the medium frequency ranges and are similar to those obtained in the near field and on bedrock from earthquakes of similar mechanism. What is unusual, however, is the unexpectedly low values of the peak near-field accelerations. Such low accelerations can not have been predicted on the basis of available strong motion information, especially noting that the 3 March 1985 Chilean earthquake (MS = 7.8) of similar mechanism have produced peak ground accelerations reaching 0.85 g.

The Institute of Engineering at UNAM maintains a network of about a dozen digital strong motion accelerographs in Mexico City. The Institute has adequate, state-of-the art equipment and menpower to process the instrumental data and have produced excellent reports on the strong ground motion in a matter of days after the earthquake. Figure 2 indicates the location of the accelerograph stations in Mexico City and the peak accelerations obtained. The stations Tacubaya, Viveros and Ciudad Universitaria are located on the firm ground within the zone called "Zona de Lomas", whereas the stations of

(2) UCSD and UNAM (1985), Preliminary Presentation of Accelerogram Data from the Guerrero Strong Motion Accelerograph Array. Mechoacan - Guerrero, Mexico, Earthquakes of 19 and 21 September, 1985, Preliminary Report GAA-1A, October 10, 1985.

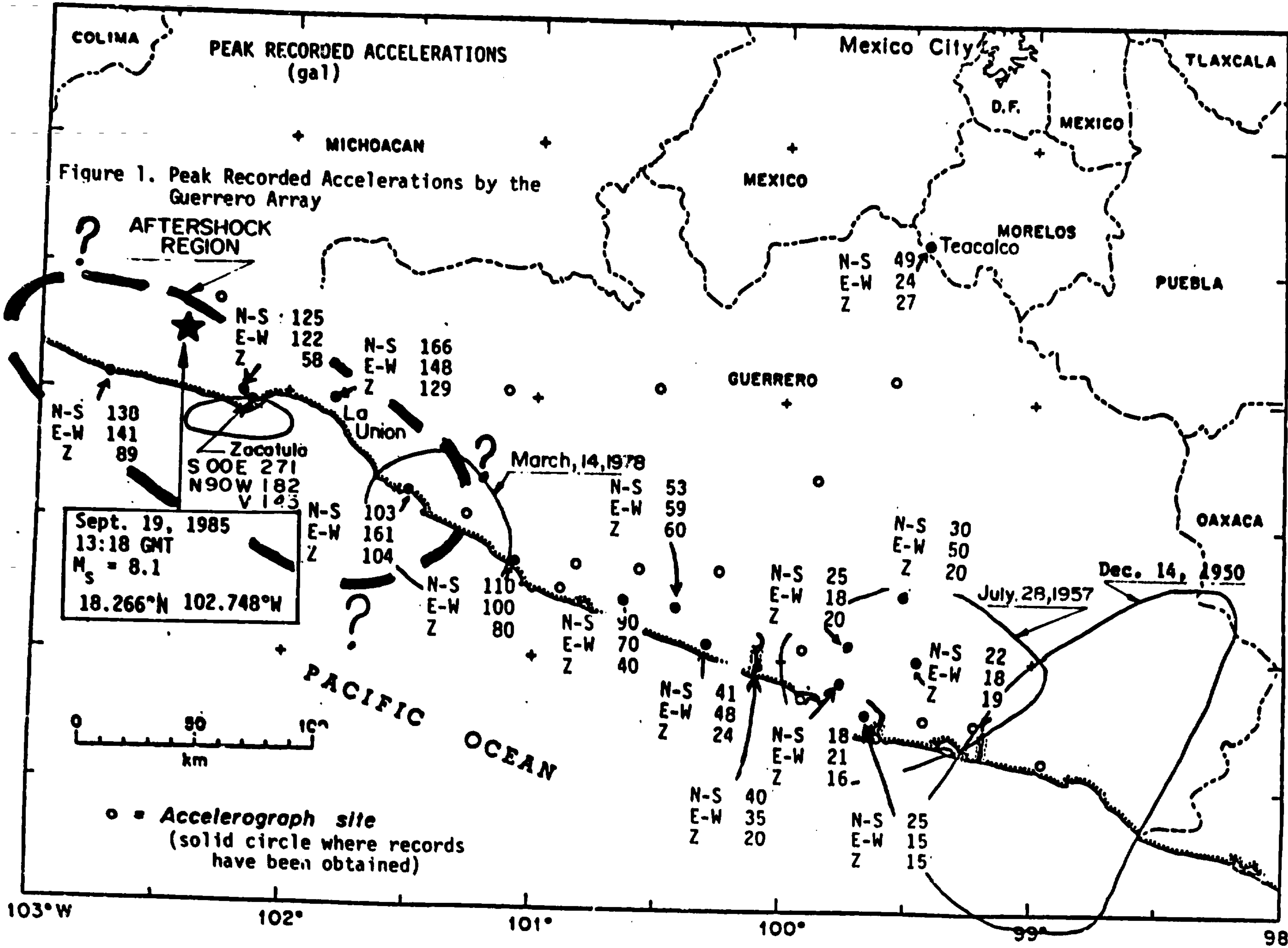


Figure 1. Peak Recorded Accelerations by the Guerrero Array

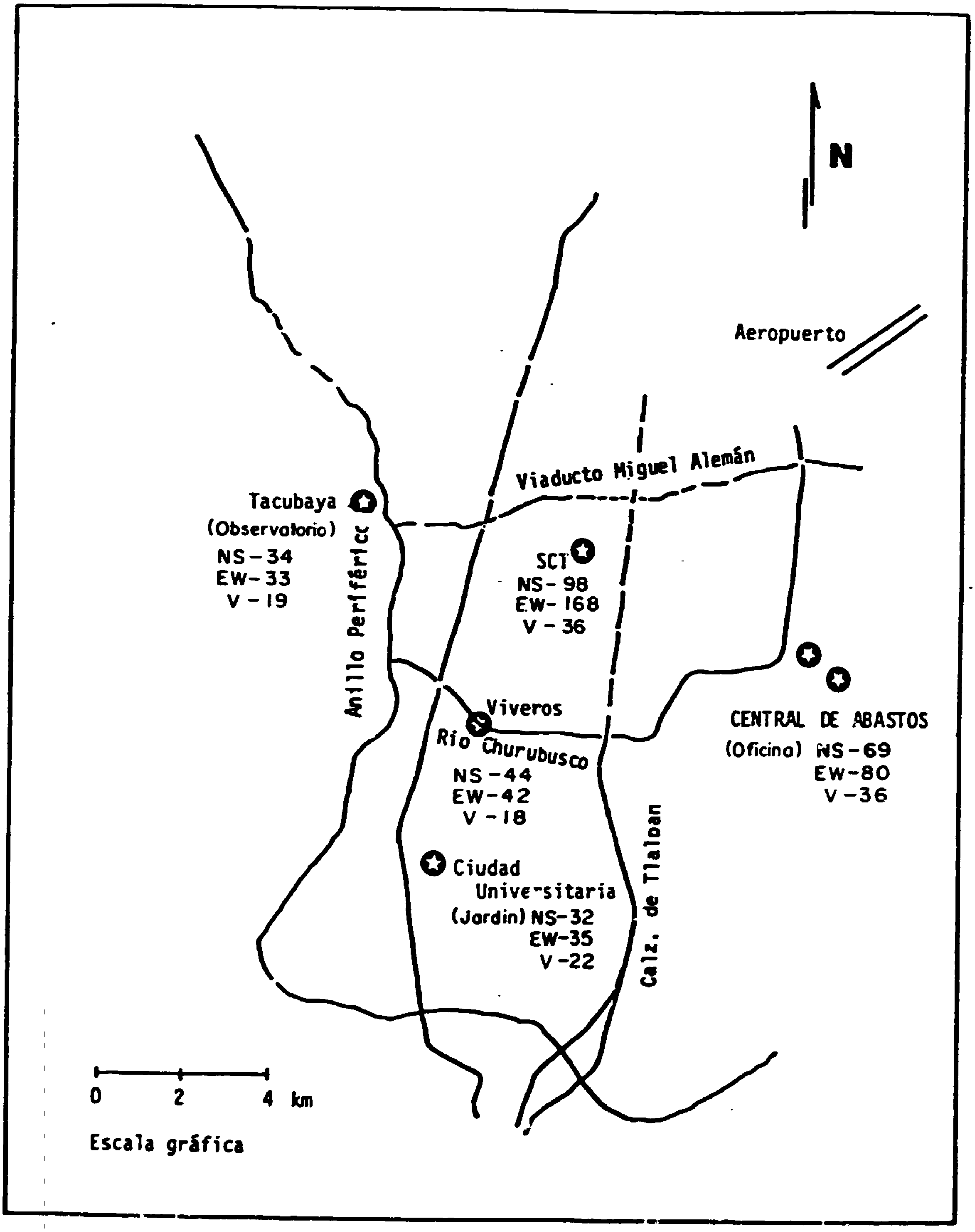


Figure 2. Peak Accelerations in Gals (cm/s^2(Measured by the Accelerometers in Mexico City.

SCT and Central de Abastos are located on compressible lacustrine clay sediments in the zone called "Zona del Lago". The time history and the absolute acceleration response spectrum of the significant free-field motions recorded on firm ground (Ciudad Universitaria) and on compressible soil ground (SCT and Central de Abastos) are provided respectively in Figures 3, 4 and 5. These figures are taken from a series of reports published by the Institute of Engineering, UNAM after the earthquake. It can be seen that the energy contained in the Ciudad Universitaria record are spread over a period band between 0.2 to 3.5 seconds (or within the frequency range of 0.3 to 5 Hz), whereas the accelerograms recorded at the compressible ground show distinct periods where the energies are concentrated (2 sec. for the SCT record and 3.5 sec. for the Central de Abastos record).

The peak accelerations recorded on the firm soil in the Mexico City could have been predicted on the basis of the attenuation relationships based on previously recorded worldwide data. For example for an earthquake of magnitude MS = 8.1 and at an epicentral distance of 400 km's Campbell[3] relationships provide a mean peak acceleration of 25 gal and mean-plus-one-standard deviation peak acceleration of 36 gal, well within the range of peak accelerations recorded in the Mexico City on firm ground.

In Figure 6 the velocity response spectrum of the SOOE component of the ground motion recorded at SCT is compared with the same of the El-Centro NS record of the 1940 Imperial Valley earthquake, widely used for the earthquake resistant design of structures throughout the world. It can be seen that after 1.5 sec. period the motion at SCT have up to 7-8 times higher spectral velocity than the El-Centro record. Similar comparisons also hold for the U.S. Nuclear Regulator Agency Site-Independent response spectrum.

(3) Campbell, K.W. (1981), Near Source Attenuation of Peak Horizontal Acceleration, Bull.Seism. Soc.Am., 69, pp.2039-2070.

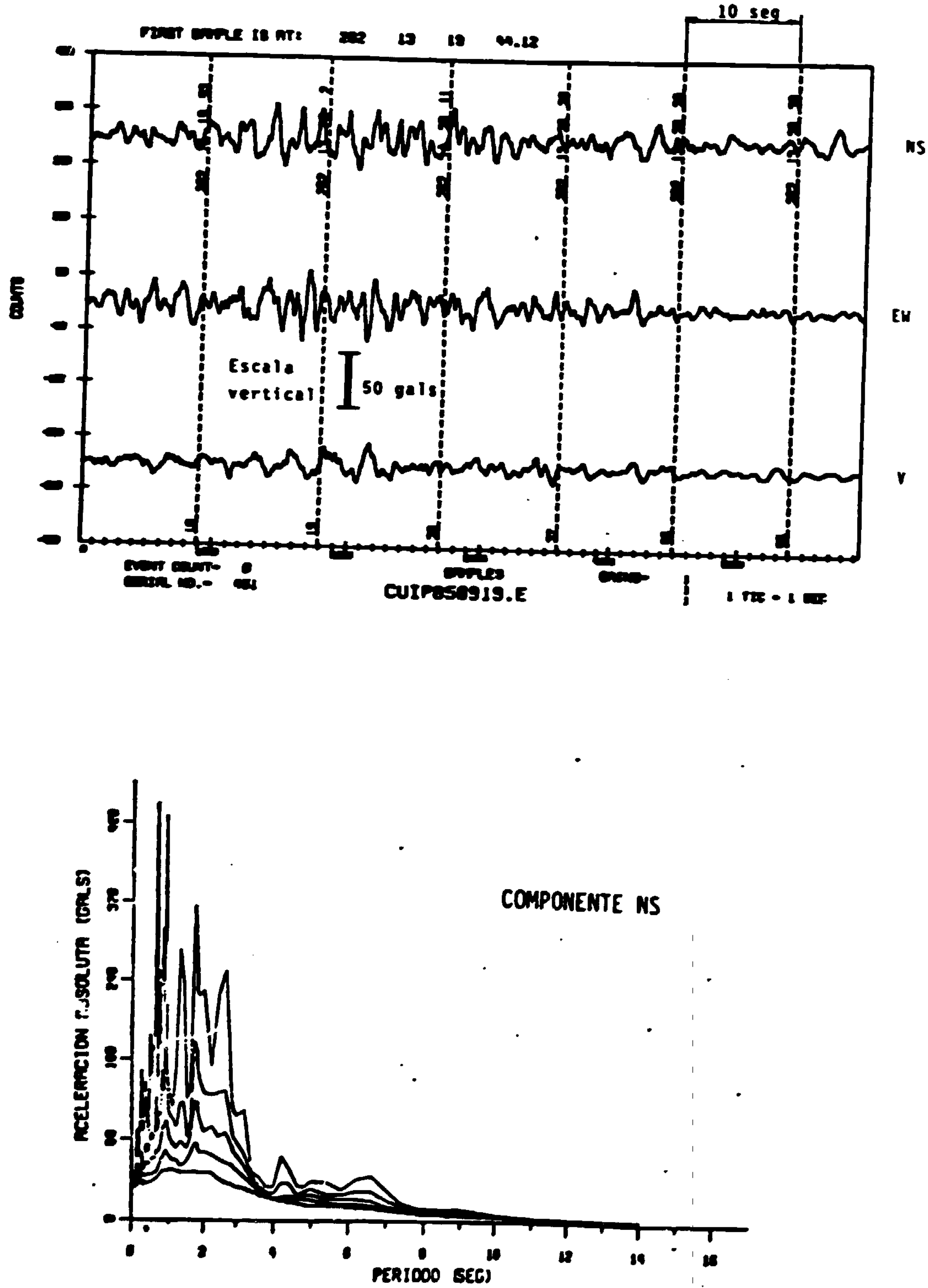
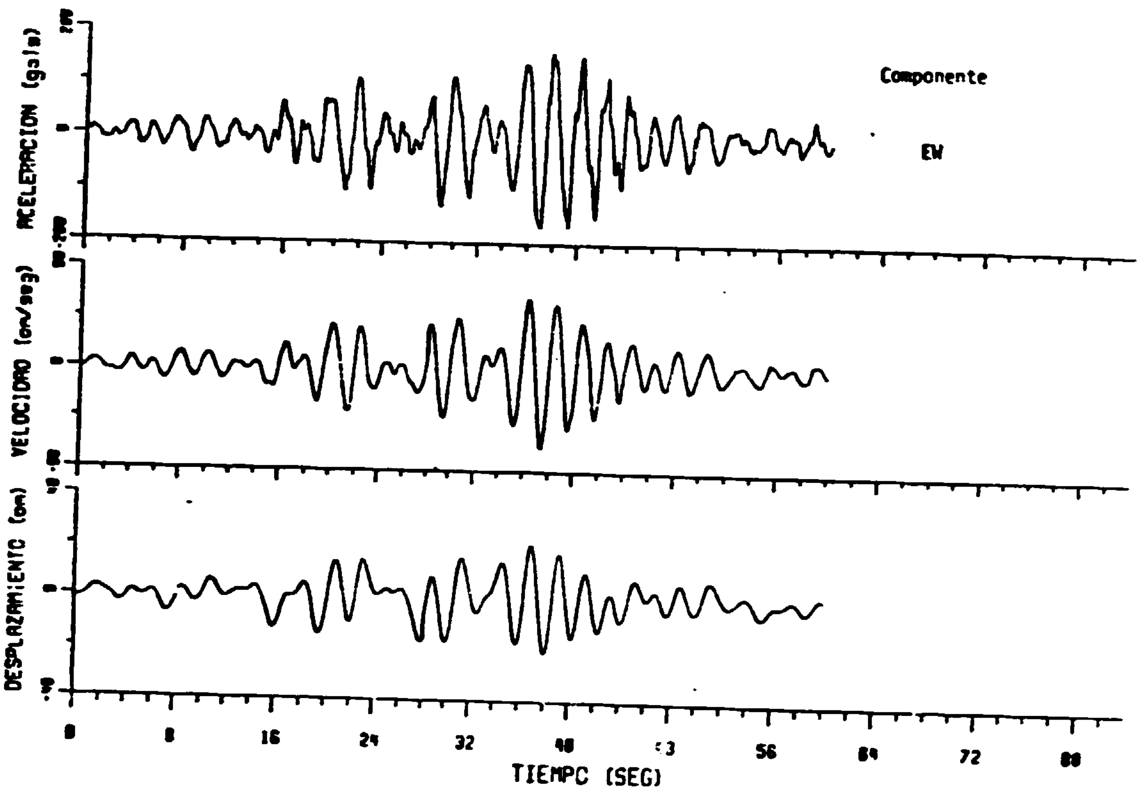


Figure 3. Acceleration Time History and the Absolute Acceleration Response Spectrum of the NS Component of the Ground Motion Recorded at the Ciudad Universitaria (Jardin)



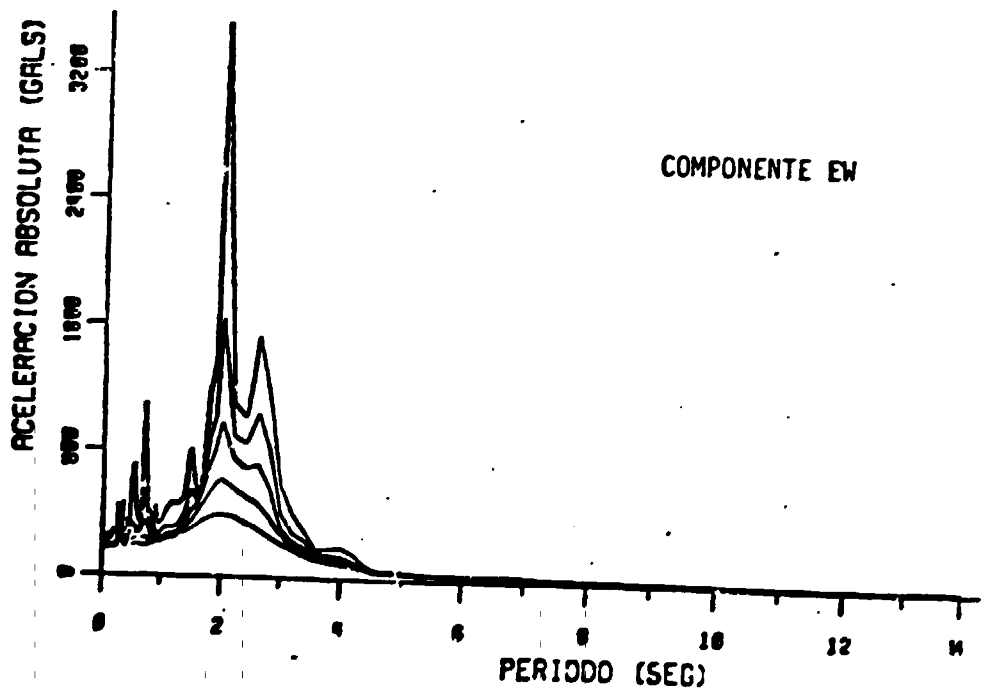


Figure 4. Acceleration Time History and the Absolute Acceleration Response Spectrum of the EW Component of the Ground Motion Recorded at SCT.

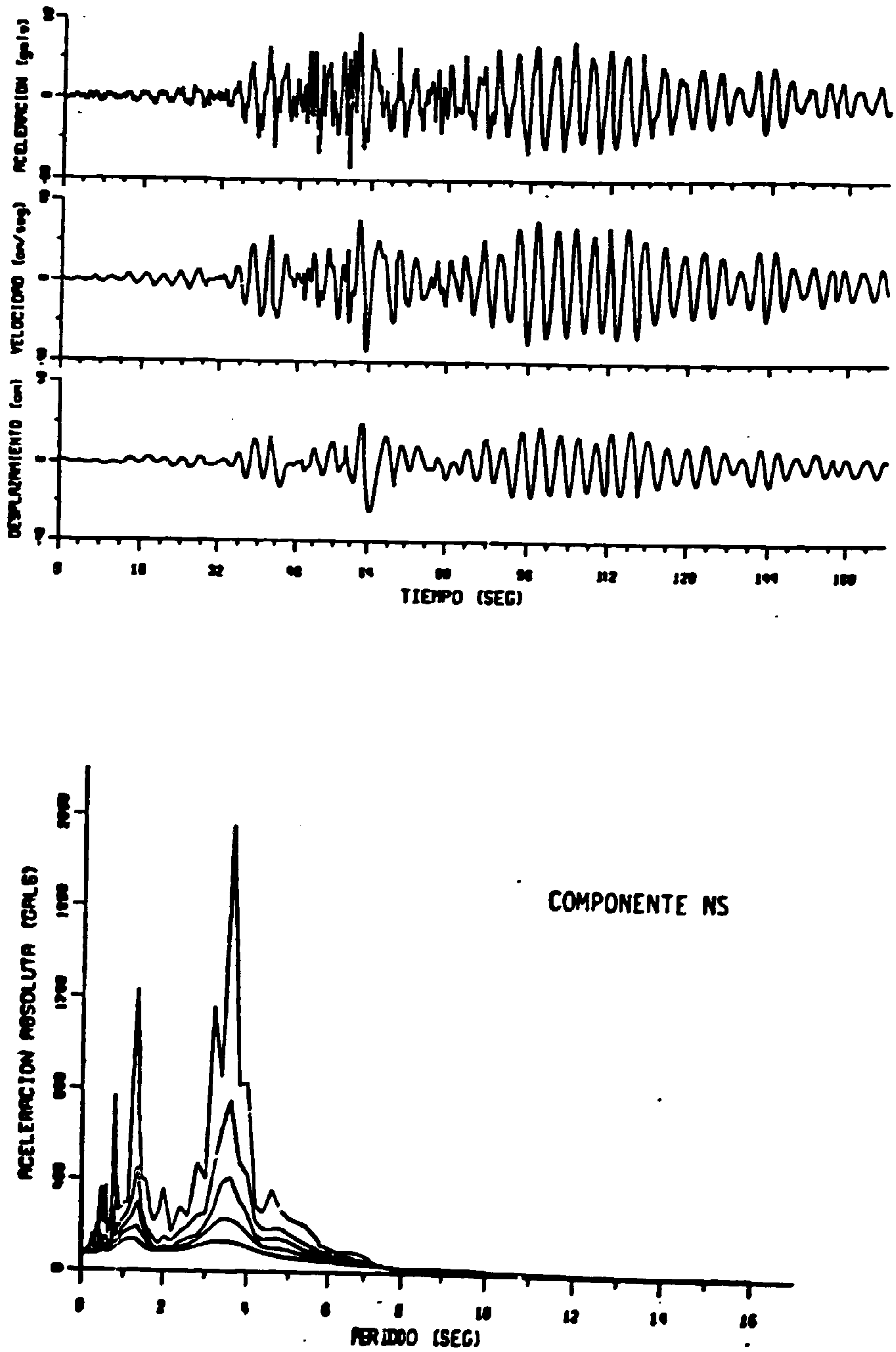


Figure 5. Acceleration Time History and the Absolute Acceleration Response Spectrum of the NS Component of the Ground Motion Recorded at Central de Abastos (Officina)

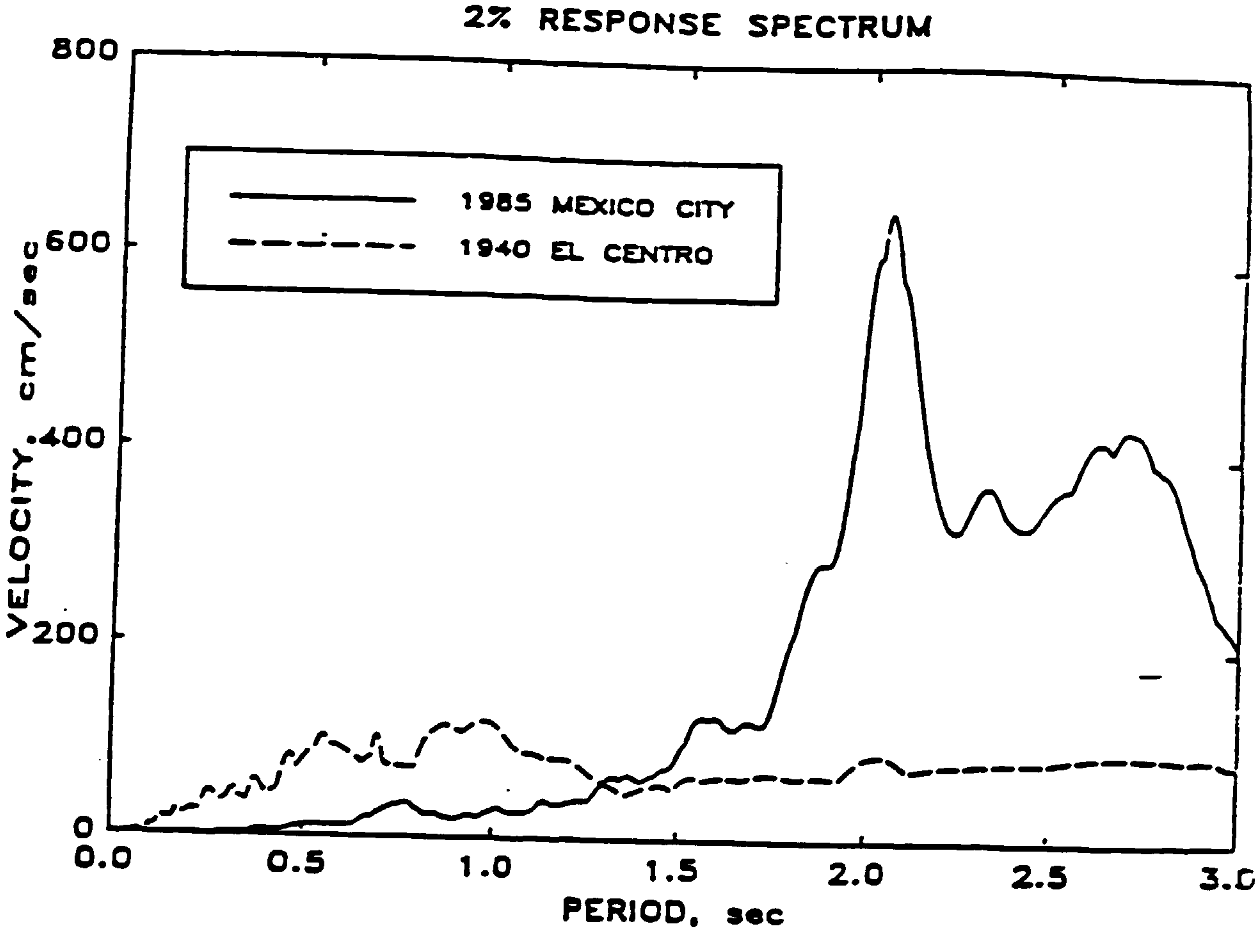


Figure 6. Comparison of El Centro 1940 N velocity response with the record obtained at station SCT, Mexico City, S60E component. (After NRC-EERI Preliminary Reconnaissance Report titled "Impressions of the Guerrero-Michoacan, Mexico Earthquake of 19 September 1985", October 1985).

5. SITE EFFECTS

Mexico City has a long history of being particularly sensitive to earthquake shaking and was more heavily damaged than cities and towns closer to the recent earthquake, local geology and soil conditions must have played an important role in the distribution of recent damage as they have in the past. Mexico City lies in a broad basin formed by block faulting of a uplifted plateau about 30 million years ago. About 3 million years ago volcanism in the region was renewed and lava flows formed a dam across this valley just south of Mexico City. This dam resulted in the formation of a large lake that slowly began to fill with silt, clay, and ash from nearby volcanoes. Changes in the climate during this period led to oscillations in the level of the lake. The Aztecs built their capital on an island in the lake connected to the shore by causeways. During the Spanish era, to solve the flood problem, canals were built to drain the lake to the north. As the water level dropped, several small lakes were formed from the single lake and portions of the old lake bed were exposed. This lake bed has been used for the expansion of Mexico City. Today, much of the City rests on lake deposits made up of silty, volcanic clays and sands with thicknesses of 50 meters or greater. These lake deposits overlay other, older sedimentary sequences with thicknesses up to 2,000 meters. The lake clays are very compressible and have a high-water content.

Today in the vast metropolitan area of the Mexico City there exist a vide spectrum of soil conditions ranging from basaltic lavas or very compact soils, as is the case of foothills, to the highly compressible soft soils of the Lake, with a transition zone where clayey layers of the lacustrine origin exists. These zones are marked by the stratigraphic changes, especially by the depth of the shallow compressible clay layers. Figure 7 depicts the excellent correlation of the earthquake damage with the thickness of compressible soil deposits. Figure 8 shows the seismic microzoning adopted for the Mexico City based on the stratigraphic data. Typical soil stratigraphies along the North-South and East-West Profiles of the Mexico City are provided in Figure 9. As it can be seen there exists very soft "Upper Clay Deposits" of about 30 m depth followed by a 3-4 m deep "First Hard Layer" and then the deeper deposits. The "Upper Clay Deposits" have shear wave propagation velocities between 50-100 m/s and unconfined compressive strengths of between 0.7-0.9 kg/cm^2. In the "First Hard Layer" the unconfined compressive strengths are between

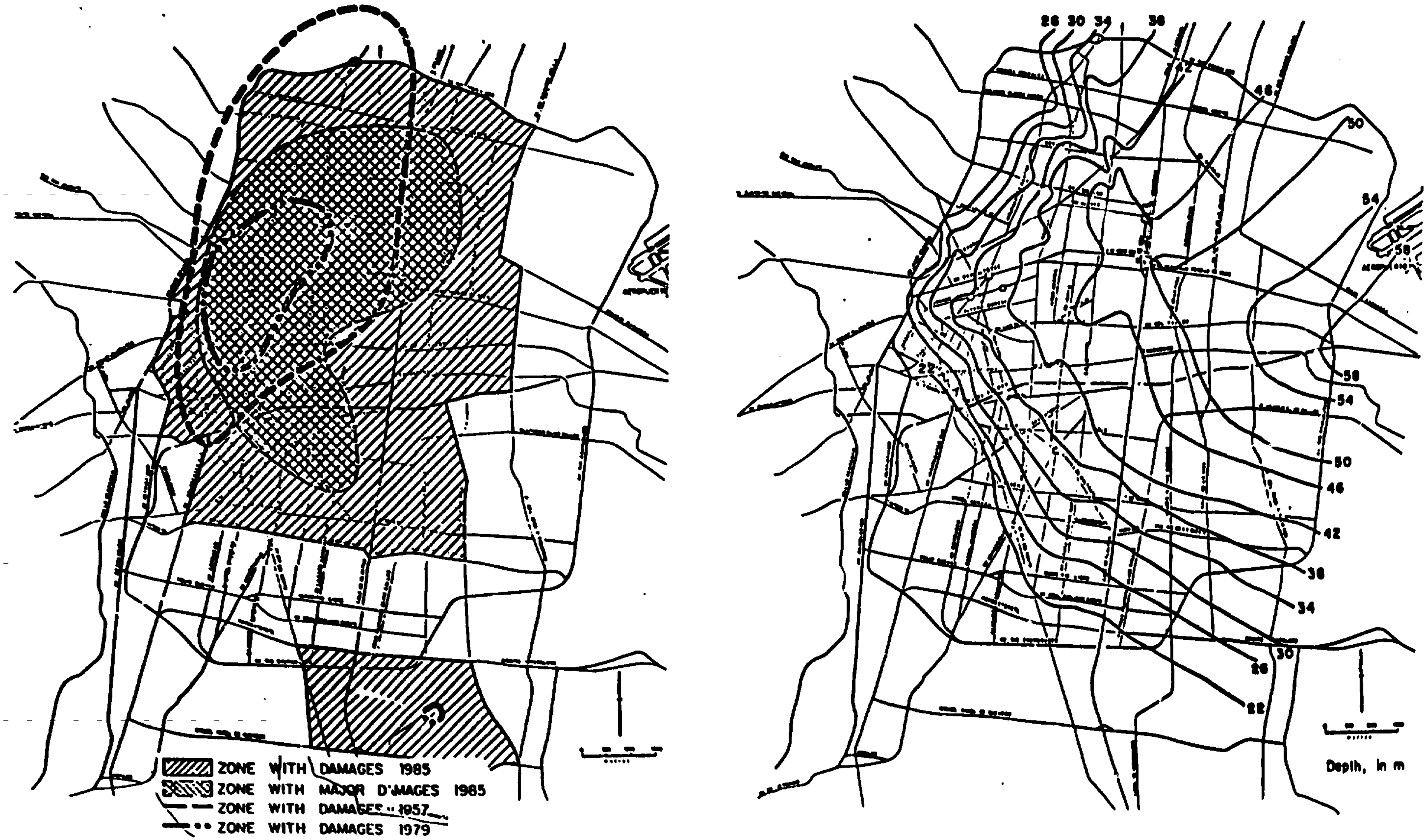


Figure 7. Correlation of the Earthquake Damage Zones with the Depth of the Compressible Soil Deposits.

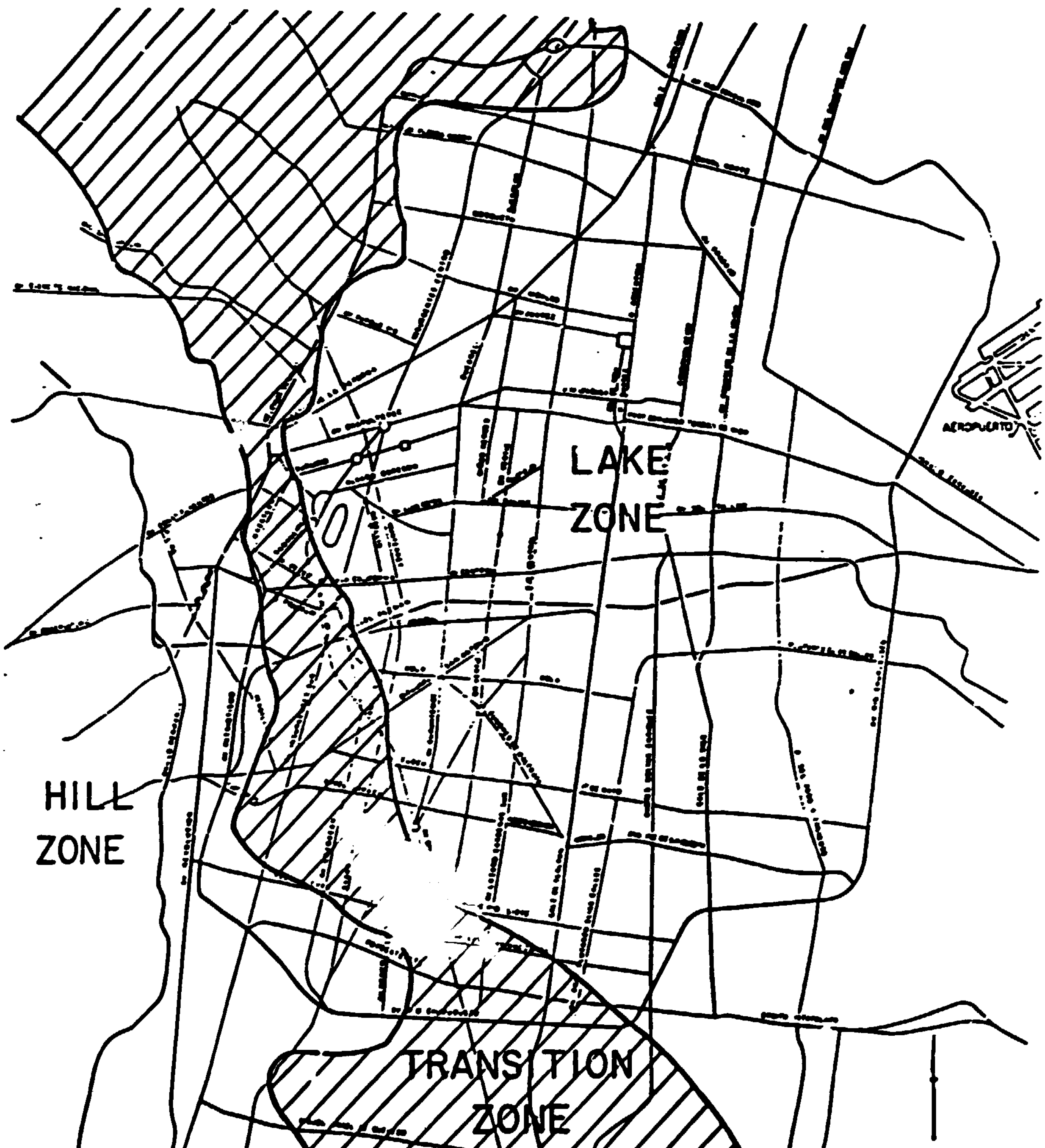


Figure 8. Seismic Microzoning Map for Mexico City

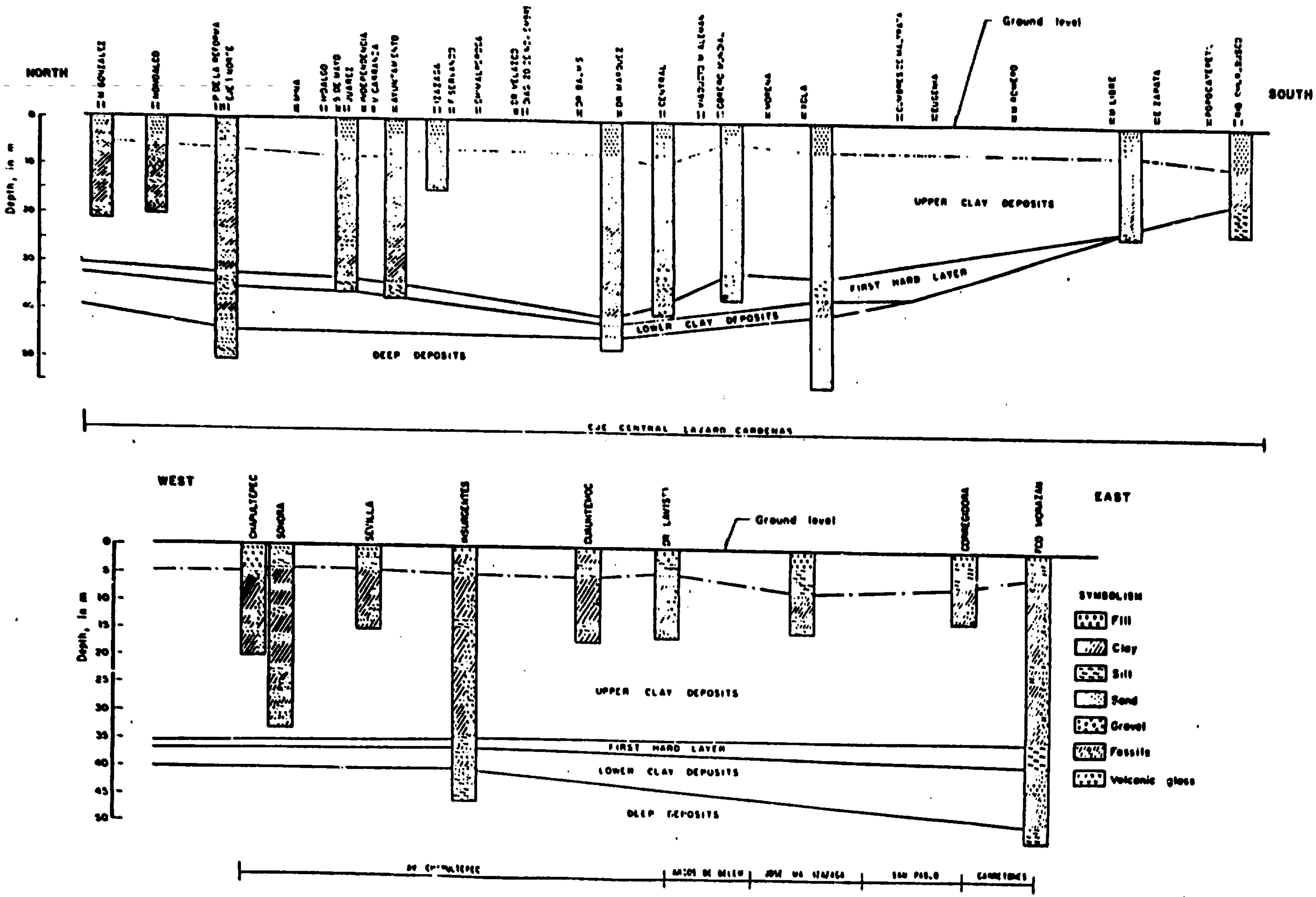


Figure 9. Stratigraphic Sections Along the North-South (Eje Central - Lazaro Cardenas) and Along the East-West (Av. Chapultepec - Carretones) Profiles in Mexico City.

2 - 2.5 kg/cm^2 (4,5). Under such adverse soil conditions the engineers have made extensive use of pile foundations for supporting structures. Some of the typical pile foundations used are provided in Figure 10.

It is well known that when unconsolidated soft material underlies a site, the seismic motions are modified from what it would have been on hard soil. The amplitude of the motion may change depending on the frequencey region, the degree of this change (amplification) being a function of the physical properties of the media. This amplification is evident upon comparison of the Figures 3, 4 and 5. A theoretical assessment(6) of this amplification phenomena can be made on the basis of the transfer functions between motions on the hard and soft soil zones. Figure 11 provides the theoretical SH-wave transfer function for the soil profile at Tlatelolco, which is quite similar to the SCI accelerograph site. The soil profile and the characteristics are taken from (5) as provided by Prof.M.Romo, UNAM. It can be assessed that there exist an amplification of 10 at 0,55 Hz (about 2 sec. period) even for 5% material damping. This figure correlates excellently with the response spectra provided in Figures 4 and 6.

The direct influence of the soft soil deposits on the ground motion can be assessed through the analysis of the non-linear propagation of S-H waves. For such an analysis, lets consider the soil profile provided by Facioli and Ramirez(5) for the Tlatelolco area shown in Table 1.

Table 1 - Soil Profile at Tlatelolco

Layer	Thickness(m)	Density(kg/m^3)	Shear Wave Propagation Velocity(m/s)
1 (FAS)	15	1300	47
2 (FAS)	15	1200	75
3 (Capa dura)	5	1770	105
4 (FAZ)	10	1270	138
5 (DP)	10	1770	245
6 (DP)	10	1800	265
7 (DP)	10	1900	275
8 (DP)	10	2000	287
Halfspace Base	∞	2100	1040

The stress-strain characteristics of the soil layers are assumed to

(4) Marsal, R.J. (1975), The Lacustrine Clays of the Valley of Mexico, UNAM, Report No.I.16.

(5) Faccioli, E. and J.Ramirez (1975), Repuestas Sismicas Maximas Probables en Las Arcillas de le Ciudad de Mexico, UNAM, Report No.359.

(6) Erdik, M. (1985), Site Response Analysis, Proc. NATO Advanced Study Institute on Strong Ground Motion Seismology, Reidel, Holland.

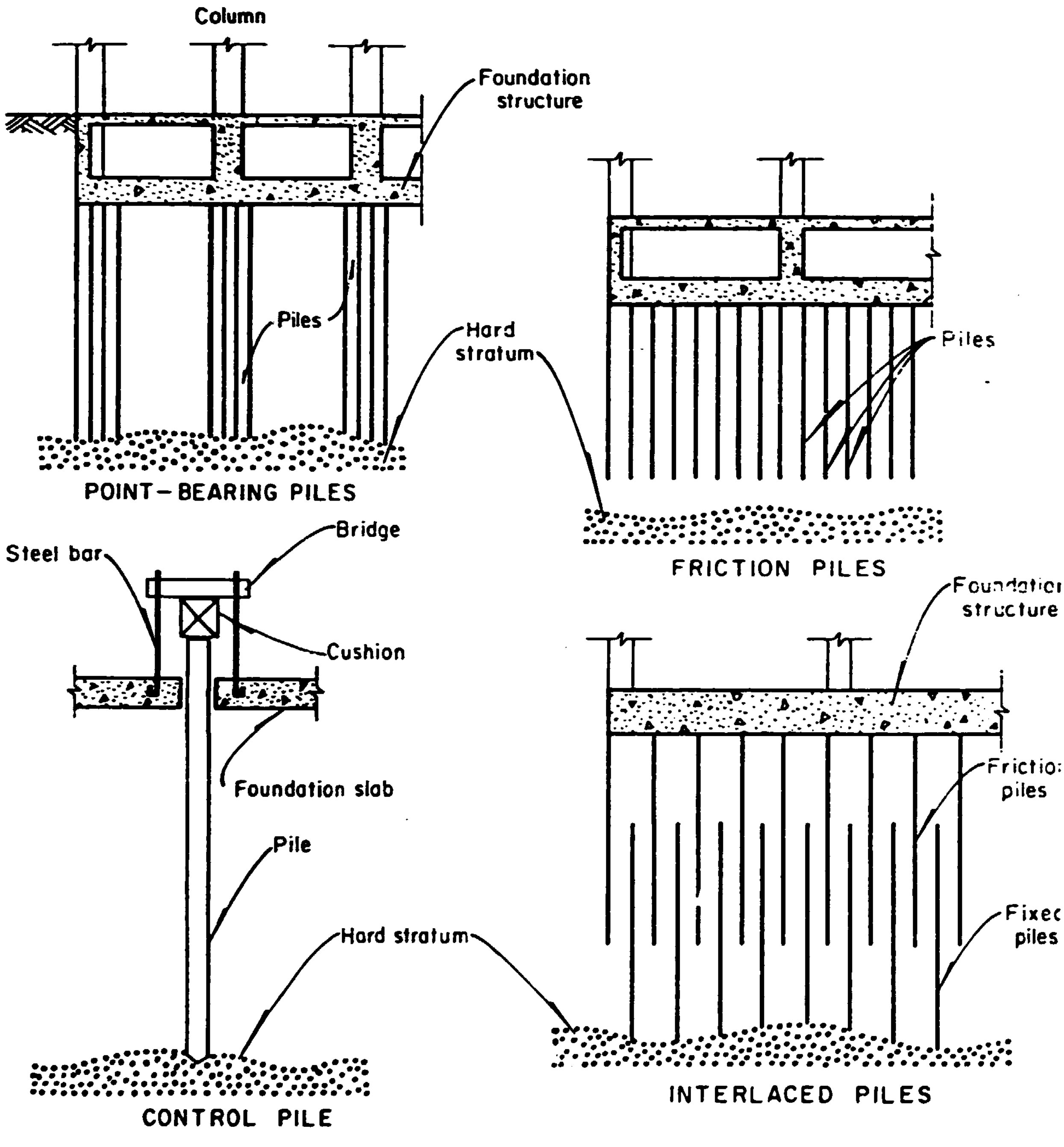
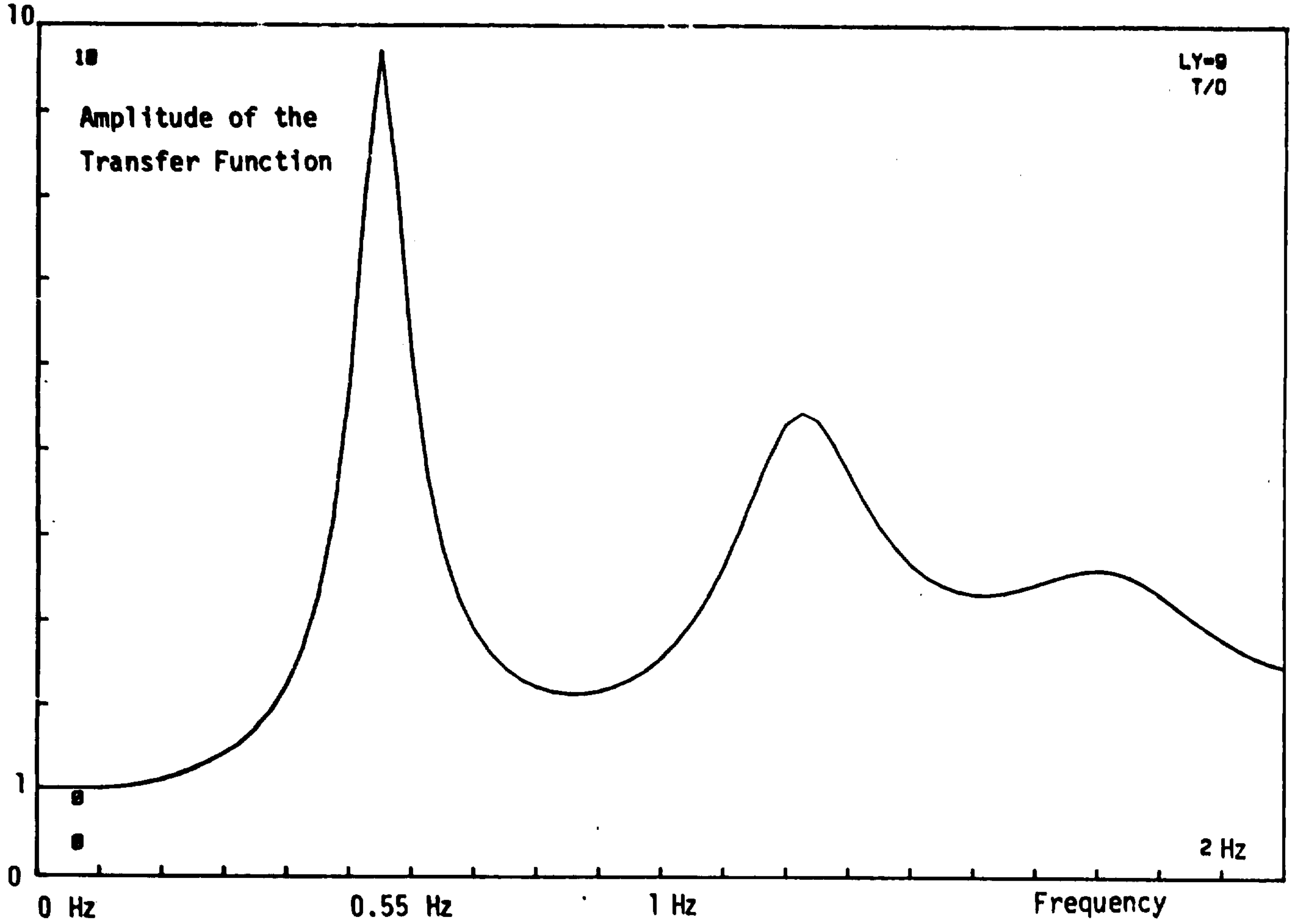


Figure 10. Typical Pile Foundations Used in Mexico City
(After Marsal, 1975)

TLATELOLCO




```
# of LAYERS= 9
REFERANCE LAYER No= 1
TYPE TOF TO OUTCROP
------------------------------------
MASS DENSTY 1= 1300  (Kg/cu.mt)
MASS DENSTY 2= 1200  (Kg/cu.mt)
MASS DENSTY 3= 1770  (Kg/cu.mt)
MASS DENSTY 4= 1270  (Kg/cu.mt)
MASS DENSTY 5= 1770  (Kg/cu.mt)
MASS DENSTY 6= 1800  (Kg/cu.mt)
MASS DENSTY 7= 1900  (Kg/cu.mt)
MASS DENSTY 8= 2000  (Kg/cu.mt)
MASS DENSTY 9= 2100  (Kg/cu.mt)
------------------------------------
SHEAR VELO 1=47(Mt/sec)
SHEAR MDLS 1=2871700(Nt/sq.mt)
SHEAR VELO 2=75(Mt/sec)
SHEAR MDLS 2=6750000(Nt/sq.mt)
SHEAR VELO 3=105(Mt/sec)
SHEAR MDLS 3=19514250(Nt/sq.mt)
SHEAR VELO 4=138(Mt/sec)
SHEAR MDLS 4=24185880(Nt/sq.mt)
SHEAR VELO 5=245(Mt/sec)
SHEAR MDLS 5=106244250(Nt/sq.mt)
SHEAR VELO 6=265(Mt/sec)
SHEAR MDLS 6=126405000(Nt/sq.mt)
SHEAR VELO 7=275(Mt/sec)
SHEAR MDLS 7=143687500(Nt/sq.mt)
SHEAR VELO 8=287(Mt/sec)
SHEAR MDLS 8=164738000(Nt/sq.mt)
SHEAR VELO 9=1040(Mt/sec)
SHEAR MDLS 9=2271360000
(Nt/sq.mt)
------------------------------------
DAMPING FACTOR 1= .05
DAMPING FACTOR 2= .05
DAMPING FACTOR 3= .05
DAMPING FACTOR 4= .05
DAMPING FACTOR 5= .05
DAMPING FACTOR 6= .05
DAMPING FACTOR 7= .05
DAMPING FACTOR 8= .05
DAMPING FACTOR 9= .02
------------------------------------
LAYER THICKNESS 1= 15 (mt.)
LAYER THICKNESS 2= 15 (mt.)
LAYER THICKNESS 3= 5 (mt.)
LAYER THICKNESS 4= 5 (mt.)
LAYER THICKNESS 5= 10 (mt.)
LAYER THICKNESS 6= 10 (mt.)
LAYER THICKNESS 7= 10 (mt.)
LAYER THICKNESS 8= 10 (mt.)
```

Figure 11. Theoretical SH-Wave Transfer Function of the Tlatelolco Soil Profile

follow the hyperbolic hysteretic relationship with the ultimate shear stress equal to 10^{-3} times the low amplitude shear modulus.

The reference motion, assumed to act at the outcrop of the halfspace base is taken to be similar to the ground motion recorded at the Ciudad Universitaire (Jardin) and is simulated by a band pass filtered white noise as indicated in Figure 12. This simulated reference motion matches the amplitude and the frequency characteristics of the motion recorded at the firm ground free-field at Ciudat Universitaire (Jardin). The true non-linear vertical SH-wave propagation analysis is carried out as described in Erdik(6). The resulting surface motion is shown on the same figure. As it can be assessed both the amplitude and the frequency characteristics of this synthetic motion matches closely to that of the actually recorded motion at SCT with similar soil characteristics to the Tlatelolco site.

The non-linear actions taking place at the 15 m depth of the soil are indicated in Figure 13 as the hysteresis curves. It can be seen that the shear strains up to 5×10^{-3} have resulted in the soil media, resulting possibly in plastic permanent deformations.

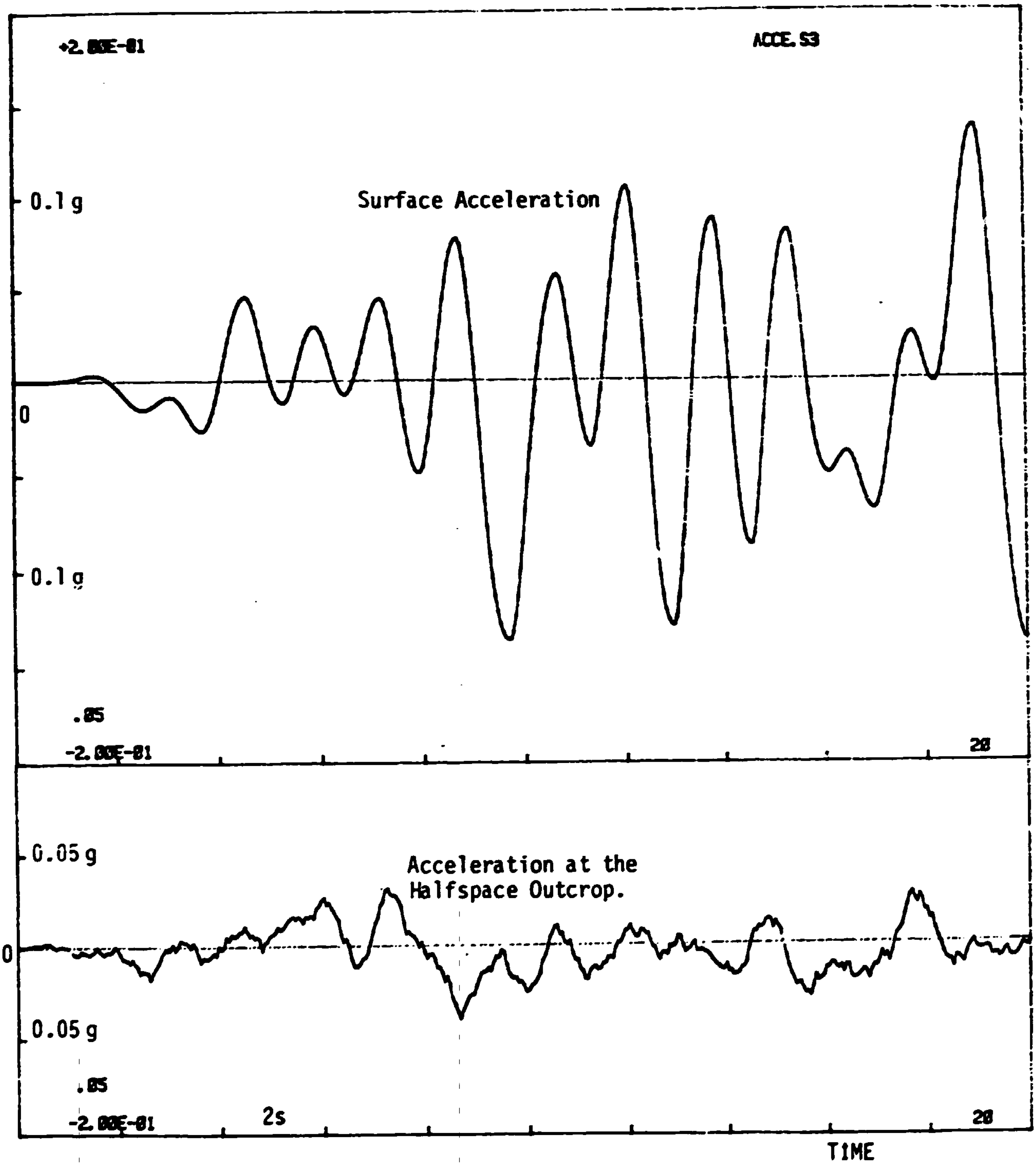


Figure 12. Nonlinear Site Response Analysis for the Tlatelolco Site

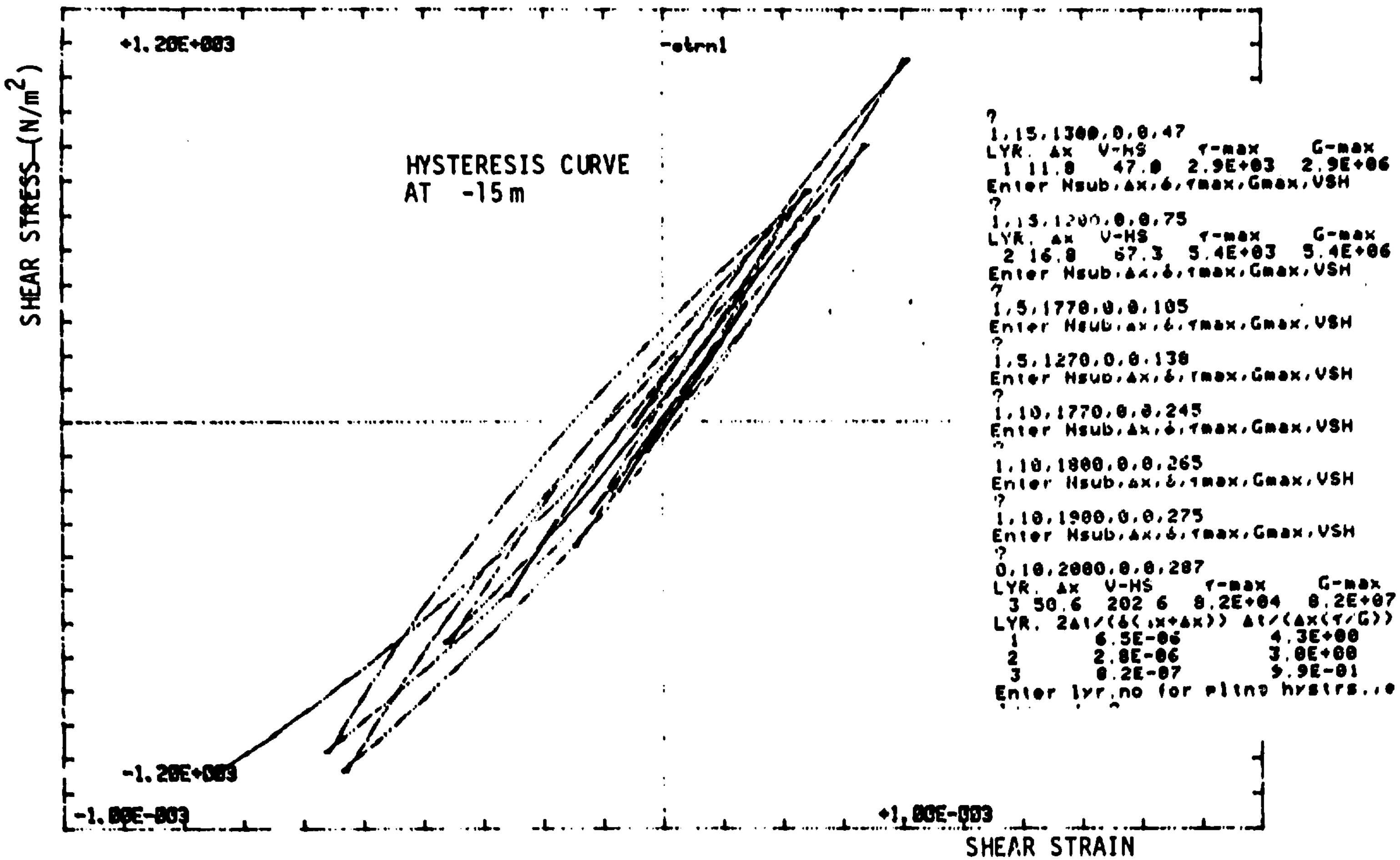


Figure 13. Shear Stress-Strain Hysteresis Curve at -15m depth.

6. SOIL-STRUCTURE INTERACTION

The extensive soil-structure interaction displacements experienced by the foundations of the slender structures resting on soft soil deposits resulted in the following typical failures commonly observed in Mexico City.

Compaction or consolidation occured under the foundations of braced frame and/or shear wall structures because of the overturning forces and moments generated. This results in the extensive "rocking" of the building with residual out-of-plumbness tilt. Several buildings in Mexico City have been observed to be tilted from the plumb line as much as few degrees. In one instance, the whole foundation of the building failed in slip-circle mode due to extensive shearing stresses in the soil.

Lateral compression or sliding of the materials surrounding the foundations of braced frame and/or shear wall structures may result from the translational movement of the foundation. Such lateral displacements may cause ruptures in the walls, floors and/or foundation ties of the structure. However such damage are not easily observable.

Lateral and flexural failure of the foundation piles due to the bending and shear forces caused by the movements of the soil surrounding the piling may occur in soil formations such as those found in Mexico City. However such failures can only be detected on the basis of examination trenches.

Open frame type structural systems can usually yield at the prescribed design forces and limit forces and moments transferred to the foundation medium. However in the case of braced frame and shear wall structures this yielding phenomena cannot occur, unless special precautions have been taken, and the structure can overload the foundation above and beyond those forces and moments used in the design of the foundation. This seems to be the major reason of the permanant tilts observed in most of the slender shear wall structures in the effected areas of the Mexico City. Although no attempt have been made to discriminate between the pre-earthquake differential settlements and the co-earthquake plastic deformations it is believed that the post-earthquake tilts reflect the combination of the both causes in varying degrees.

As further investigation of the soil-structure interaction phenomena the response of the Type-M buildings of the Tlatelelco Appartment Complex will be analysed. These are 22 story shear wall and braced frame reinforced concrete buildings supported by piles. A typical plan section and the elevation section of the building is shown in Figure 14. These buildings are associated with tilts of about (1/50) or 0,02 radian. A substantial part of these tilts seemed to be so-seismic as evidenced by the fresh sections of the mozaic facade unearthed (about 30 cm) after the earthquake due to the permanent rocking deformation of the foundation.

For the soil-structure response analysis of this structure a simple three-degree-of-freedom model shown in Figure 15 is used. The use of such a model is also sanctioned by the ATC-03[(7)] earthquake code. In this model the super structure is modeled by its rigid base first mode behavior and the foundation is assumed to have one lateral and one rocking degree-of-freedom. The soil and the foundation impedance parameters are deduced from the soil profile provided in Table 1. As the free-field ground motion a 0,5 Hz (2 sec) monochromatic acceleration with a peak amplitude of 0.17 g is used. This motion is believed to be the representative of the actual motion at the site during the earthquake.

In Figure 16 the results of the response analysis in the strong direction of the structure are presented. It can be seen that the total relative first mode absolute acceleration experienced of the building is about 0.43g and the total absolute acceleration is about 0.60g. Since the super structure receives very small intrinsic accelerations (about 0.054g) the super structure remains in the linear ranges of deformation. However the foundation receive shears and overturning moments that would correspond to a seismic coefficient of about 0.60. This value is probably several times above that was used in the design of the foundations. Thus the foundation material are stressed in the plastic ranges of deformation resulting in permanent settlements.

---

(7) ATC-03 (1978), Applied Technology Council, Tentative Provisions for the Development of Seismic Regulations for Buildings, NB25, Washington DC.

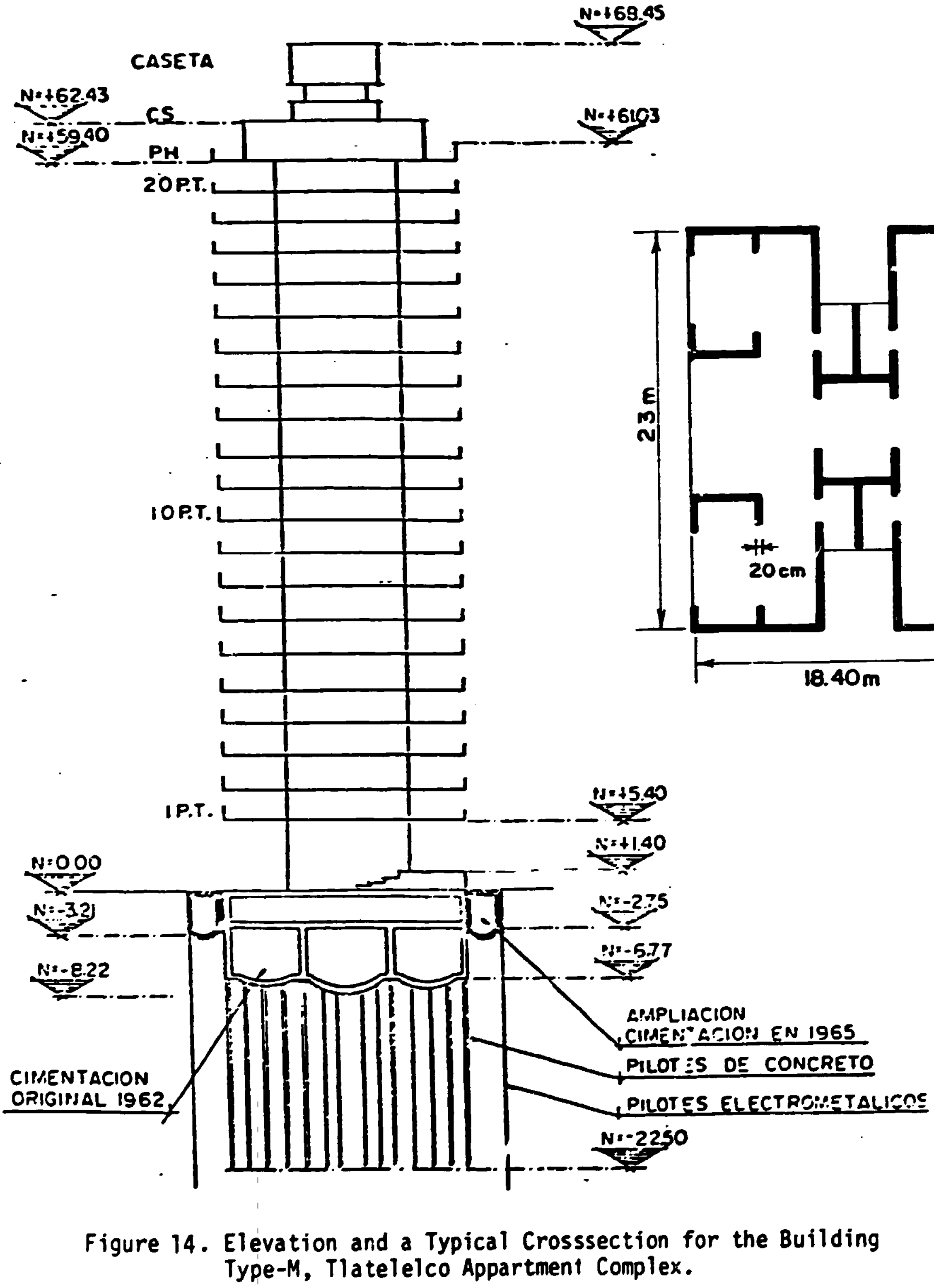


Figure 14. Elevation and a Typical Crosssection for the Building Type-M, Tlatelelco Appartment Complex.

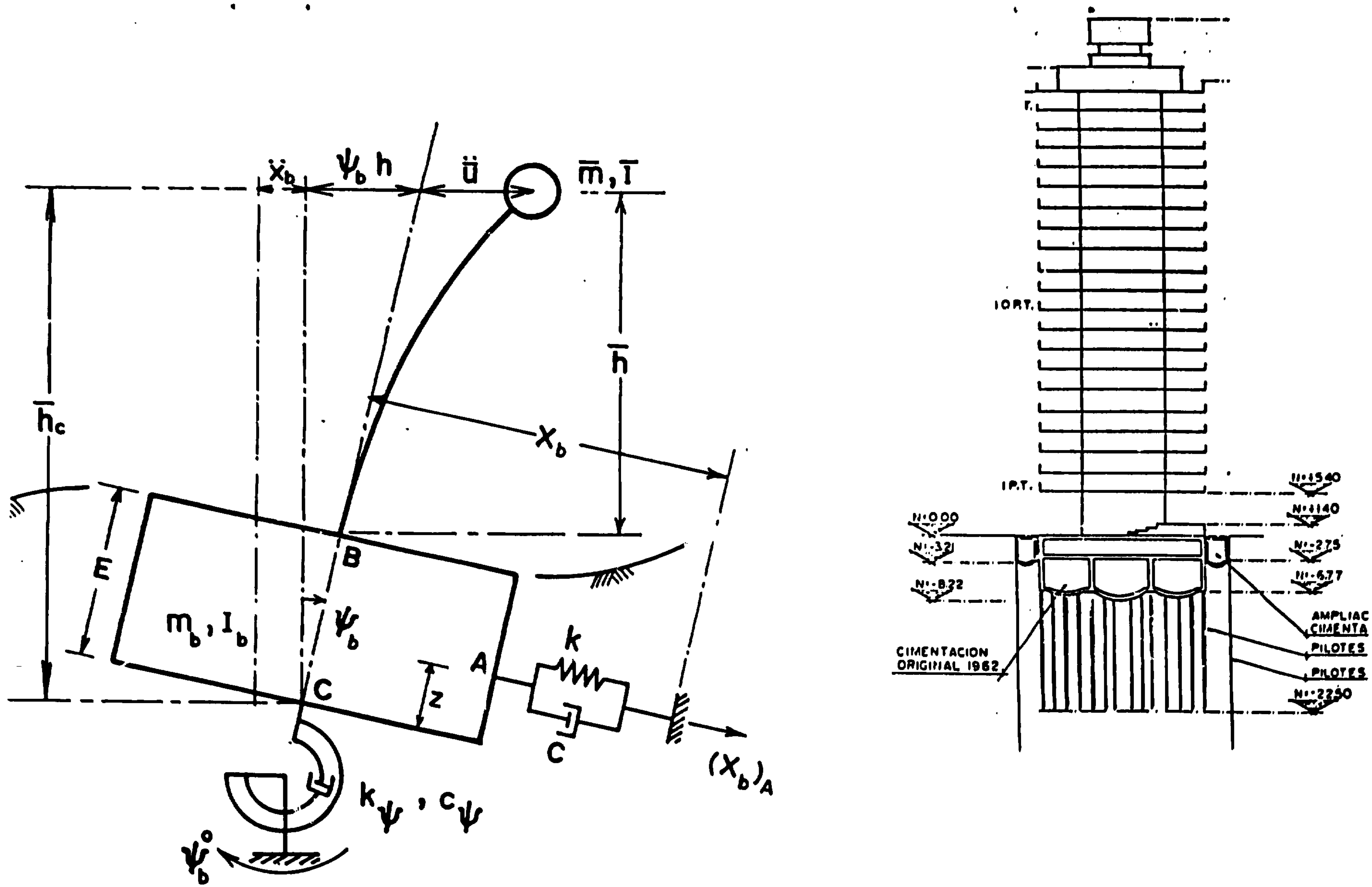


Figure 15. Three-Degree-of-Freedom Soil Structure Interaction Model Used in the Analysis.

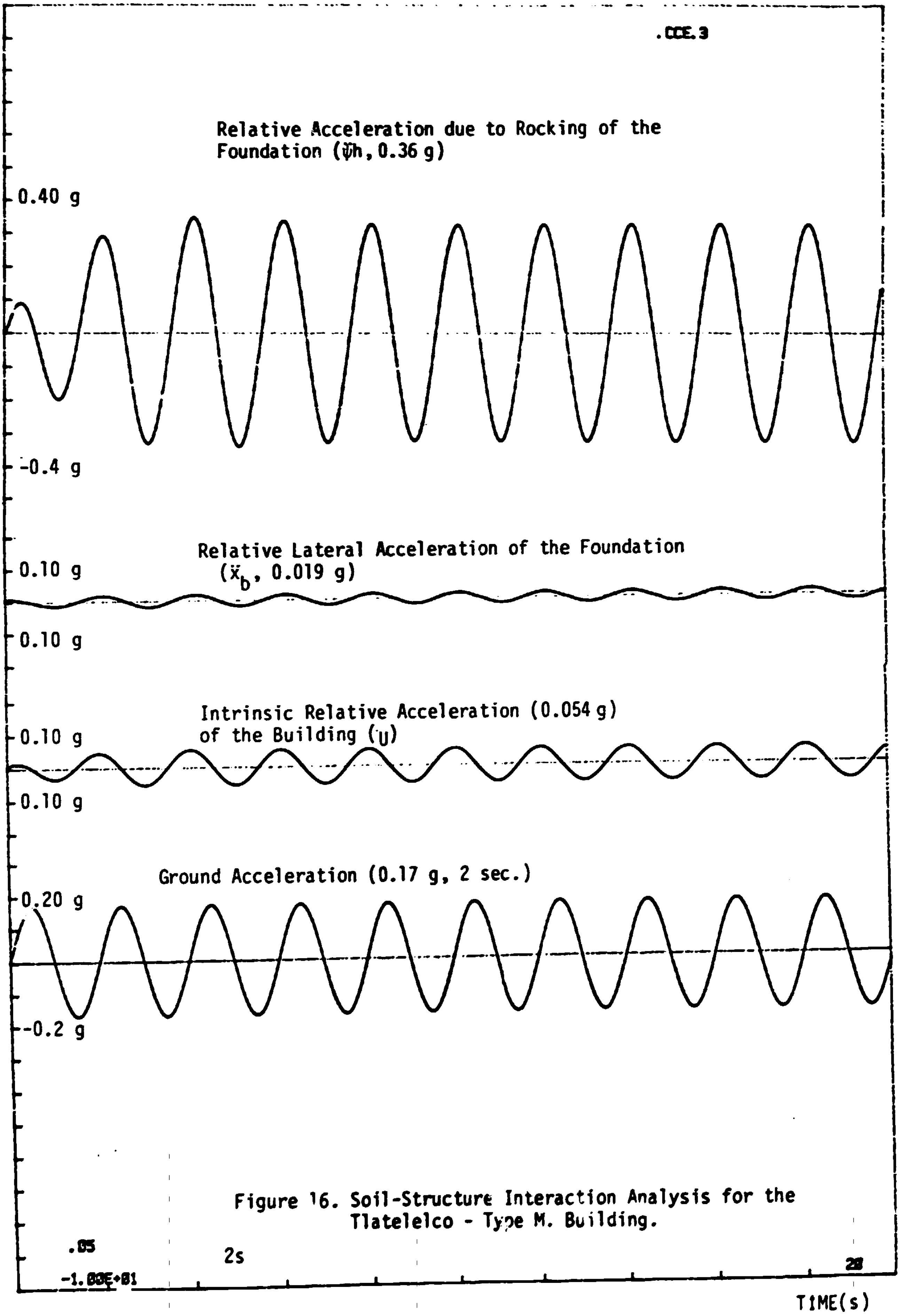


Figure 16. Soil-Structure Interaction Analysis for the Tlatelelco - Type M. Building.

7. SUGGESTIONS

The suggestions will be limited to those assessments presented in the sections 4, 5 and 6, and are intended to provide insight for the post-earthquake repair and strengthening efforts and reconstruction as well as future research and development.

1. The repair, strengthening and the re-leveling of distressed foundations, especially pile foundations, is a highly site-structure and-soil specific technique and only general comments can be made. The following suggestions are adopted from ATC-03 (7) :

   Exploration below grade can be made to determine the condition of footings and piles. In the case of pile footings, if significant lateral movement has occurred the piles may have been damaged due to excessive bending. This can be verified by uncovering a length of pile. However, removal of earth from the pile surface will reduce the load-bearing capacity of friction piles. In some cases, damage may be so great or soil properties so poor that repair or strengthening is not economically feasible.

   Foundation elements may be strengthened by adding cross-sectional area reinforcement as described in these sections. Where epoxy injection is used to repair cracks in exterior basement walls, the injection may be limited to one side. In such cases it may be difficult to obtain full penetration, and special care must be exercised in determining adequacy of such repair.

   Correcting settlement distress involves both preventing future settlement and releveling. The method used to correct settlement distress will generally depend on the supporting soil characteristics and a thorough soils investigation should therefore be made and evaluated prior to selecting a method.

   Pile footings that have settled due to earthquake forces are difficult to relevel and strengthen. Soil stabilization rather than underpinning should be considered. Where possible, such procedures obviate the need for additional, difficult-to-install piles. Where calculations show that multiple pile footings do not meet appropriate code criteria, it may be necessary to remove the existing pile cap, place additional piles, and provide a new pile cap. The existing column or other load-bearing vertical element must be cut loose and jacked back to its required position. Such vertical components must be temporarily supported before being cut loose.

Space for adding new piles must be available, including vertical clearance for a pile driver if driven piles are used or space for a drill rig where drilled and cast-in place concrete piles are used.

Some methods of soil stabilization and compaction, such as vibration, preloading, and blasting, increase surface settlement of the area and therefore cannot be used to repair foundations. Other methods, such as pressure grouting or intrusion grouting with cement grout or chemical grouting, do not settle the ground surface and are frequenctly used to stabilize soils under existing buildings in that the bearing capacity of the soil is increased. Pressure grouting may also be used to raise settled footings or floor slabs.

Soil stabilization should never be used unless a thorough investigation of underlying soils has been made. Selection of a soil stabilization technique is a function of specific soil characteristics, such as size of granules, moisture content, and soil chemistry. Some soils are not adaptable to any type of soil stabilization technique. In such cases, abandonment and demolition of the building may be necessary.

2. Strengthening of super-structure without attention to foundation problems may aggravate the foundation problems in the future earthquakes since a stronger (i.e. not easily yielding) structure can overload the foundation above its design values. For proper performance of foundation the ductility of the super structure should be increased rather than its strength.

3. For slender and rigid buildings (i.e. shear wall, braced frame and heavy panel-prefabricated) resting on compressible soils special provisions should be incorporated in the earthquake resistant design codes to avoid overstressing of the foundations. These provisions may include the checking of the foundation stresses that corresponds to the ultimate overturning moment capacity of the superstructure.

4. The soil-structure interaction analysis should be made mandatory for slender and rigid buildings founded on compressible soils for proper assessment of the stresses in the foundation, structural displacements, vibration frequencies and the P-Δ effects.

5. Limits should be imposed on the overal allowable tilt of the buildings. Several codes provide 0.002 radians for such distortions (i.e. Indian Standards IS - 904 - 1978).

6. To provide much valuable data for the future developments in the soil structure analysis, an appropriate structure, foundation and the soil layer should be instrumented with a three dimensional strong motion array. Such an array can have about 10 triaxial force-balance acceleration transducers (four of which are down-hole units) connected to a control signal conditioner, trigger and digital recorder unit at an approximate cost of US $ 70,000. The data obtained from such a system will especially enchange the design techniques for soil - structure interaction problems.

IV. FINDINGS AND RECOMMENDATIONS ON THE MICHOACAN-GUERRERO EARTHQUAKE OF 19 SEPTEMBER 1985, WITH SPECIAL REFERENCE TO REINFORCED CONCRETE STRUCTURES

BY

M. VELKOV

This Report is prepared by Prof. Miodrag Velkov, UNIDO Consultant, who in accordance with the agreed tasks accomplished a consultant mission in Mexico City in the period 2 to 15 December 1985.

Within the scope of the tasks accomplished by the Consultants group discussions were held at the Institute of Engineering, National University of Mexico, Camara Nacioanl de la Industria de la Construction aimed at getting introduced to the global causes, records and consequences of the disastrous Mexico earthquake. Also, the basic concept and achievements of the research project on the Balkan region, building construction under Seismic Conditions in the Balkan Region, have been presented. It was proposed and accepted to hold lectures related to the research results obtained within the project. Therefore, a series of lectures were given at the Institute of Engineering N.U.M. as well as at the Camara Nacional de la Industria de la Construction.

My lectures considered problems associated to the study and design of R/C structures in seismic regions in relation to the problems of structural behaviour of reinforced concrete elements during the Mexico earthquake. In addition, a review was given on the prefabricated structures which have been subjects of study and which have been constructed in the European and Balkan countries.

The enclosed Appendix presents the basic problems included in the lectures given.

On the basis of the general insight of the state of the structures and the observed damage caused by the September 19, 1985 earthquake in Mexico, the following can be concluded :

1. It is inevitable to carry out a deeper analysis of the design practice for new and strengthened of existing R/C structures and structural elements, aimed at ensuring ductile behaviour without brittle failure, reinforcement anchorage).

2. Analysis of the optimal level of equivalent seismic forces, in compliance with the design Code, in order to provide technically optimal and economically rational buildings.

3. Analysis of the depth and the way of foundation based on soil condition analysis which should ensure minimum deformations due to dead and additional seismic loads with the required level of structural safety.

For accomplishment and long-term resolving of all the problems and earthquake consequences, it is necessary to consider the possibility for initiation of a large-scale project on international and regional basis with participation of related United Nations agencies.

APPENDIX

CONCEPT FOR DESIGN AND STUDY OF HIGH-RISE BUILDING STRUCTURES

It is known that during the seismic effect only a limited number of structures remain in elastic - linear range of behaviour. It means that in case of infrequent violent earthquakes a large number of structures behave beyond the elastic range, suffering, thus, non-linear deformations. Nonlinear deformations cause certain level of structural vulnerability, which requires a specific design treatment, particularly for determination of the safety criteria of the basic Code concept for design and construction in seismic regions.

Buildings to be constructed in large series are of particular importance, either because of their typified character or when prefabricated elements or whole structures are applied, which imperatively imposes the need for seismic risk assessment.

The concept for determination of the non-linear behaviour as a design criterion, as well as the determination of the maximum seismic effect is carried out in accordance to experimental and analytical studies of structural elements and whole buildings.

From the structural viewpoint, depending on the built-in material, structural type, construction technology, i.e., production in case of industrially produced structural elements, when analyzing the structural stability it is very important to determine the following effects to which buildings could be subjected :

- local blast (inside or outside the building);
- Larger-scale soil settlement due to different effects;
- wind effect
- seismic effect.

The experience with structures have shown that the risk of failure and particularly the risk of progressive failure, in case of local primary damage, is directly associated with one of the following structural systems :

- the building is constructed without the risk for local failure - damage, which means that the structure remains, always, in elastic range;
- to allow for local damage to the system or to design the system allowing neither complete nor partial failure.

In case of high-rise building, structural systems which can sustain partial damage under a strong earthquake are regularly used, which means that the structure shall experience nonlinear deformations. This concept of construction of high-rise buildings is nowadays considered as the basis for design and construction. Assessment of the behaviour of the structural elements of the main structural system in both linear and non-linear range by determination of the stress and deformation level depending on the vulnerability level is the immediate task of contemporary structural engineers designing high-rise buildings.

Design Concept

Construction of modern high-rise buildings in seismic areas requires a particular design approach with experimental and theoretical study of the stability of structural elements and the structural system as a whole. Depending on the used construction materials and the type of structural elements, the following basic problems can be distinguished, as they require particular investigation:

1. Basic structural elements (frame or structural walls) rigidity, deformability, strength capacity, reinforcement of R/C structural systems, failure mechanism).

2. Floor structures (strength capacity, connection with the main structural system, particularly in the case of prefabricated and steel structures).

3. Joints between horizontal and vertical elements of the main structural system (particularly in the case of prefabricated structures)

   - load bearing capacity and transmission of normal stresses,
   - transmission of shear stresses
   - energy dissipation mechanism,
   - bond effect between concrete and reinforcing bar in case of R/C structures,
   - buckling of panel zone in case of steel structures.

4. Construction of details, structural elements depending on the main structural type and failure mechanism.

5. Formulation of the mathematical model of the structural system.

6. Analysis : static and dynamic, linear and nonlinear.

In some cases, when structures are designed and constructed in large series (typified structures, prefabricated structures) seismic risk evaluation is required. They require additional investigation of site seismicity, and determination of the seismic design parameters applying a non-deterministic approach.

Safety Criteria

Fig. 1 presents a block-diagram showing the design criteria for design of high-rise structures. The seismic intensity effect is classified into the following three levels :

Level I - for seismic effects of lower intensity, earthquakes which occur more frequently, structural vibrations should be controlled so that story drift does not cause disturbance in structural functioning.

Both structural and nonstructural elements remain strictly in elastic range of performance.

Level II - For stronger earthquake, so called design earthquake level, the main structural system behaves mainly in linear range, however, limited nonlinear deformations that strictly control the stress and deformation level (relative storey drift) are allowed.

Level III - In case of disastrous, very infrequent earthquakes, structures suffer considerable damage - nonlinear deformations characterized by high energy dissipation and yielding of material, however no even local structural failure are allowed.

Concept of Experimental and Theoretical Studies

When designing modern high-rise buildings it is necessary to determine the points of potential plastic hinges and plastic zones of the system, in order to define the corresponding mathematical model of the structural system as the basis for theoretical study of structural behaviour under seismic conditions.

In the most cases, in order to simplify the analysis, the following assumptions are made in formulation of the mathematical model of the system :

- horizontal floor structures are of doubled stiffness, i.e., high rigidity,

- plastic deformation mechanism, in case of frame structural systems, develops, usually, in the beams and then in the columns. In case of structural wall systems, plastic zones appear at one or two levels with previous occurrence of nonlinear deformations in the lintals above wall openings. In case of prefabricated large panel structures, vertical panels remain usually in linear range, while non-linear deformations occur in the joints.

- the first level of multi-storey structures (basement or ground floor) remain permanently rigid, with high level of rigidity,

These principal assumptions ensure a rational analysis, giving almost always reliable results on the structural response to seismic excitations.

In case of uncertainty regarding some assumptions defining the physical properties of the mathematical model, it is possible that even a subtle analysis (like three-dimensional static and/or dynamic analysis) show the behaviour of a completely different structural system, which is not the subject of study. That being the case, it is necessary to use available experimental investigations for formulation of the considered structural system model, its strength and deformability characteristics as well as the failure mechanism itself.

The experimental studies, generally, include :

- Structural elements of the main structural system (bending effect, shear forces, anchorage and bond effect of reinforcement, for R/C structural elements, buckling, for steel elements, failure mechanism.

- Joint zone: joints and weldings in joint zones of steel structures. Joints of prefabricated structures, etc.

- Structural fragments: single or multi-storey, etc. Tests can be carried out by quasi-static cyclic loading or by applying dynamic loads, usually, full-scale model testing on a shaking table.

- Forced vibration tests on constructed buildings in linear range of behaviour, in order to determine dynamic characteristics.

- Forced vibration tests on structural elements or fragments in corresponding scale up to total collapse.

By the experimental technique it is possible to formulate the realistic mathematical model of the structural system, as well as to determine the nonlinear behaviour mechanism depending on the earthquake effect: intensity, frequency content for design and maximum expected earthquake level.

Such extensive experimental and theoretical studies may be rather costly, however scheduling of experimental studies, that are necessary for certain types of high-rise building structures is carried out depending the level of the seismic risk, the use of the building, the structural type, etc., which should provide the optimal conditions for the selection of the most appropriate structural system, which in turns shall justify these studies.

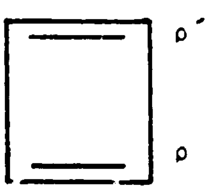
BEHAVIOUR OF R/C STRUCTURAL ELEMENTS

1. Structural Elements Subjected to Bending and Shear Forces

The behaviour of R/C elements exposed to bending combined with shear forces is directly affected by the geometrical proportions of the structural elements, the quality of the built-in concrete and reinforcement, as well as the percentage and distribution of section reinforcement.

The ductile behaviour of these elements is related to the following boundary conditions:

- One Way Reinforcement



$$\rho'\% \leq 0.2\ \rho\%$$

$$\rho\% \leq 0.4\ \frac{R_c}{f_y}$$

where, Rc - compressive cube strength of concrete,

f_y - specified yield strength of reinforcement.

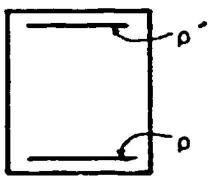
Example:

Steel S400 ; Concrete Mc = 30 N/mm^2

$$\rho\% \leq 0.4 \frac{30}{400} \leq 3.00\%$$

It, practically, means that the section of this example should not have more than 3% of reinforcement, because otherwise brittle failure may occur.

- Double Way Reinforcement



$$\rho' \geq 0.5\ \rho\%$$

$$\rho\% \leq 0.8 \frac{R_c}{f_y}$$

Example:

Steel S400 ; Concrete Mc = 30 N/mm^2

$$\rho\% \leq 0.8 \frac{30}{400} = 6.00\%$$

In case of double way reinforcement, when the minimum reinforcement criteria for compressive reinforcement are met (according to the above criteria), the reinforcement percentage can be doubled, since, practically, in case of symmetrically reinforced sections bending brittle failure does not occur due to plain flexure.

- Shear Force

$$Q_i = \frac{|M_u^L| + |M_u^R|}{L_b} + Q_g$$

where: $|Mu^{L}|, |Mu$ ultimate capacity moments at the two ends of the beam

Lb - the beam clear spain

Qg - shear force due to dead and live load

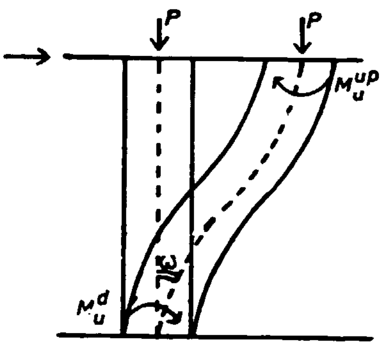
b, d_o - dimensions of the active beam cross-section

$$b \cdot d_o \geq \frac{Q_i}{0.17 \cdot R_c}$$

By analyzing the above expressions it is clear that the shear force value is directly related to the ultimate capacity moments, meaning that strong (properly reinforced) cross- sections, with the same concrete quality, may easier suffer brittle failure before reaching the ultimate bending strength capacity.

2. Elements Exposed to Bending, Compression and Shear

- Axial Force Effect $A_c \geq \frac{P}{\Phi_e \cdot R_c}$; $0.15 \leq \Phi_e \leq 0.25$



where, Ac - cross-section area

P -total axial force

Rc - compressive cube strength of concrete

The load bearing capacity of the column in ultimate state, when plastic hinges are formed at its ends, and ultimate capacity moments develop, is directly influenced by the axial force in the column and its change due to the seismic effect, which depends on the main structural system and the position of the considered column in it.

The above expression with the coefficient of $0.15 \leq \Phi c \leq 0.25$ should, practically, provide ductile behaviour of the column in the structural system.

- Shear Force Effect

Basic criterion

$$1.5\, Q_i^s \leq Q_i = \frac{\left|M_u^{up}\right| + \left|M_u^d\right|}{h_i} \leq 3\, Q_i^s$$

$\left|Mu^{up}\right|$; $\left|Mu^{d}\right|$ - ultimate capacity moments with absolute values

hi - effective net height of i-th column

Qi^s - seismic shear force of i-th column

- Short columns

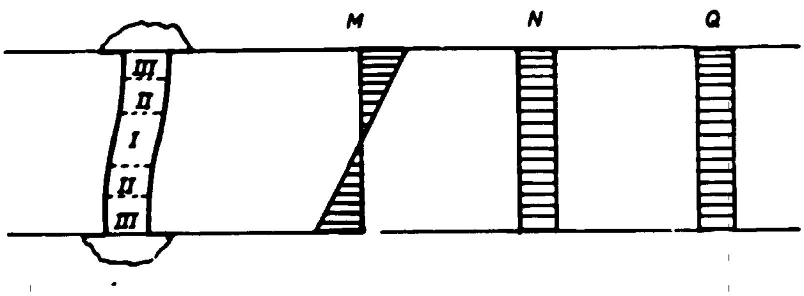
$$1.5\, Q_i^s \leq Q_i \leq 4\, Q_i^s \; ; \; \frac{h_i}{b} = 3\text{-}4$$

- Short columns

with $\frac{h_i}{b} = 2 - 3$ must be avoided

$\frac{h_i}{b}$ - ratio of effective net height of column "hi" to the largest dimension of the cross-sections "b" of the column.

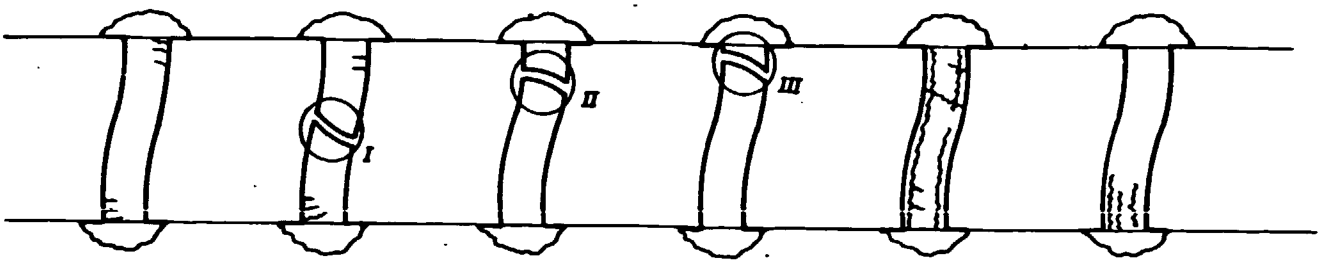
3. <u>Failure Mechanisms of Elements Exposed to Bending Shear and Compression</u>



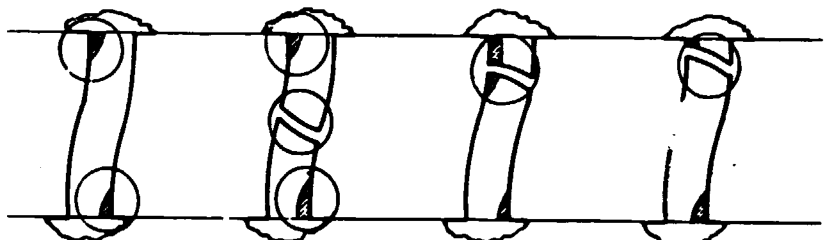
Three zones of development of failure mechanism (I, II, III) can be distinguished depending on the geometrical parameters, the percentage of reinforcement, the quality of built-in concrete and the axial force.

Modes of Failure of Columns

- Single mechanisms of failure



- Combined modes of failure



4. Confinement

Confinement reinforcement is required only in the case when the design axial load exceeds 40% of balanced load.

$\frac{\sigma_0}{R_i} \geq 0.2$ - confinement is required

$\frac{\sigma_0}{R_i} \leq 0.4$ - upper limit, which is not, practically, exceeded

$\frac{\sigma_0}{R_0} \geq 0.6$ - the transverse confinement steel requirement usually becomes excessive, and for practical reasons it will be necessary to choose large column sections.

PREFABRICATED STRUCTURAL SYSTEMS

The basic prefabricated structural systems can be identified as :

- large panel systems,
- frame systems,
- slab-column systems with walls,
- mixed systems.

The structural system of a particular building can consist of one basic system or of combinations of basic systems. The system can also be composed either wholly of prefabricated elements, or of prefabricated elements in combination with cast in situ concrete.

The configuration of the basic prefabricated structural systems, and the use of non-structural prefabricated elements are discussed below.

- large Panel Systems

Structural Components and Configurations

When considering a structural system of precast systems, the designer almost always think of a monolythic reinforced concrete structure. For construction of such a system, the structure and the structural elements should be designed to include structural walls capable of withstanding both vertical and horizontal loads. It means that the cast-in-place structures of panels acting as structural walls can also be constructed as large panel systems. This enables construction of the system according to alternative technological, prefabrication and architectural programs.

The basic concept for design of a precast system, including transformation and design of assemblages of a cast-in-place structure into a large panel system, is being developed according to the following criteria: prefabrication technology, transportation, capacity of transportation facilities and connection concept. According to the European practice, the connecting points between panels are the intersecting edges between the walls and the intersection between the horizontal panels and walls.

Seismic considerations are given in correlation with the position and the general configuration concept of connections, as follows :

(a) In order to provide sufficient strength of the system against earthquake effects, it is necessary to incorporate earthquake resistant structural members in both orthogonal directions of the structure. Currently, in all seismic regions of Europe, especially in Eastern Europe, a two-way system which mobilizes all the walls to withstand the seismic effects is adopted. This arrangement of vertical structural panels is rather efficient for small apartments. In future, a thought should be given to the concept that only a part of the existing walls should act as structural walls during an earthquake. In this way, the external facade walls and a part of the internal walls will act as non-bearing walls. So, the total length of connections will be decreased and a possibility given for application of light weight material for the nonstructural members in the system. The system designed in this way will have a much simpler configuration and at the same time more clearly defined response to earthquake effects.

(b) In general, the walls may have various sections, rectangular and flanged. Even the possibility for barbell section should not be excluded although panel walls of such section have not been used so far. Concerning reinforcement, the reinforcing bars may be longitudinal, transverse, and additional "confinement" reinforcement. Their combination can improve the system as a whole, resulting in better performance during earthquakes.

(c) It is desirable to define in advance the zones where development of nonlinear deformations is allowed, avoiding the performance in plastic range, particularly, if it is due to sliding along the vertical and horizontal connections. It is necessary that th nonlinear deformations of the structural panels be realized through reinforcement yielding due to overall bending caused by overturning moment.

(d) In some cases the possibility should be considered to design a structure with one part cast in situ, where nonlinear deformations are expected.

- Connections

Depending on the direction of the joint, two main types of connections may be identified :

- "vertical joints", which connect the vertical edges of adjoining wall panels and primarily resist vertical shear force due to seismic loading.

- "horizontal joints", which connect the the horizontal edges of adjoining wall and floor panels and primarily resist vertical normal forces due to gravity loads from the upper panels and floors, horizontal shear force due to seismic loads, and bending moments in two directions due to seismic loading action on the upper panels and gravity loading acting on the adjoining floor panels.

A wide range of details for joints are possible. In general, the joint may be either "wet" or "dry". Wet joints are constructed with case-in-situ concrete in the joint regions between prefabricated panels. If structural continuity is required through the joint, protruding reinforcing bars from the panels are welded, looped or otherwise connected in the joint zone before casting-in.situ the concrete. Dry joints are constructed by welding or bolting together steel plates or other steel inserts which have been cast into the ends of the prefabricated panels for this purpose. In dry joints the actions between the panels are transferred at discrete points at the panel edges where tne steel inserts are connected, and hence stress concentrations occur. Wet joints result in a structure more closely approaching monolithic construction, but dry joints result in faster erection.

- <u>Frame System</u>

Prefabricated multi-storey frames are used for both residential and industrial buildings. They have been more frequently used for industrial buildings than for residential buildings, because fewer partition walls are required in industrial buildings. In those positions where partition walls are needed, they can be appropriately separated and detailed so as not to interfere significantly with the deformability of the frame during an earthquake.

Prefabricated frames can be constructed of linear elements or of spatial beam-column subassemblages comprised of linear elements. Prefabricated beam-column subassemblages have the advantage that the connecting faces between the subassemblages can be placed away from the critical regions of the frames. For example, the connecting faces can be placed at the mid-height of storeys and within the spans of the beams, away from the regions of maximum moment caused by earthquake loading. However, linear elements are generally preferred because of the difficulties associated with forming, handling and erecting spatial elements. The use of linear elements generally means placing the connecting faces at the beam-column junctions. That is, the beams are generally prefabricated in lengths to occupy the clear spans between the columns, and the columns are either prefabricated, or precast on site, or case in situ, so a s to pass through the junction. The beams are normally seated on corbels at the columns, for ease of construction and to aid the transfer of the vertical shear from the beam reaction due to gravity load.

- <u>Rigid Beam-Column Joints</u>

The connections between prefabricated members can be designed to provide the frame with rigid joints when subjected to live load and seismic forces. Continuity of longitudinal reinforcement through the beam-column joint is obtained either by welding the bars together on the steel plates, or by the use of mechanical connectors, or by anchoring the bars in a sufficient length of cast in situ concrete. Cast in situ concrete is required between the ends of the beams and the columns. The columns may be cast in situ. An example of the beam-column connection of such a frame is shown in Fig. 2.

- Hinged Beam-Column Joints

The connection between the prefabricated beams and columns can be designed to be hinged. This is normally achieved by seating the beams on column corbels and by holding the beam ends in place by welded steel shoes, or by tue use of vertical dowels or bolts, so that shear can be transferred between the beam and column but not bending moment.

- Floors

Floors for frame systems can be prefabricated as panels occupying the area bounded by the clear spans of the beam grid and acting as two-way slabs, or semi-panels made into continuous two-way spanning elements placed site by side. A topping of cast in situ concrete may be used. Wholly cast in situ concrete slabs are a possible alternative.

The floors need to be designed to act as diaphragms to transfer the seismic forces to the horizontal load resisting elements, as well as to be capable of carrying the gravity loading.

- Slab-Column Systems with Walls

Prefabricated slab-column systems with walls have been devised which have as their special feature the method of construction. Two such systems are a lift-slab system involving cast in situ reinforced concrete flat plates and prefabricated reinforced concrete columns, and a system consisting of prefabricated reinforced concrete slabs and columns which are prestressed together after erection to form a continuous structure. Both systems rely on structural walls, either of cast in situ or prefabricated concrete to resist the horizontal seismic loads.

- Lift-Slab System with Walls

A lift-slab system which has had extensive application is shown in Fig. 4. The system is used for multi-storey residential, office and industrial buildings. The reinforced concrete slabs are cast in situ at ground level,, and above the other, and are continuous over the whole area of the building. The reinforced concrete columns are prefabricated in lengths of one clear storey height. The columns are designed to carry only gravity loads. The horizontal seismic loads are resisted by cast in situ concrete structural walls and stair-well cores. The other walls of the building have only partition function. The significant feature of the system is that all slabs are lifted simultaneously as a package. When the required storey level is attained the one storey high prefabricated columns are positioned under the slabs, the bottom slab is then left bearing on the columns, and the lifting procedure continues for the remainder of the slabs. Usually the stair-well cores are constructed before lifting the slabs and act as the guiding and bracing system. The column to slab connection is designed as hinged and the slab to stair-well core connection is rigid. The connections between the slabs, structural walls, and stair-well cores are designed to permit satisfactory transfer of horizontal seismic loading.

- Slab-Column System Prestressed for Continuity

A prefabricated slab-column system uses horizontal prestressing to achieve continuity. The system has had extensive application in the construction of buildings for a wide range of uses. The reinforced concrete columns are prefabricated in lengths of 1 to 3 storey heights, depending on the building. The reinforced concrete floor slabs are prefabricated generally of a size to fit the clear spans between columns. The slab soffit is either cofferred of flat with the slab voided. After erecting the slabs and columns of a storey the columns and floor slabs are prestressed into a monolithic whole, by prestressing tendons which pass through ducts in the columns at floor slab level and along the gaps left between adjacent slabs. After prestressing, the gaps between the slabs are filled with in situ concrete and the tendons then become bonded within the spans. Horizontal seismic loads are resisted mainly by special prefabricated structural concrete walls which are positioned between columns at appropriate locations.

- Mixed Systems

There remain some structural systems which do not fit specifically into large panel systems or frame systems or slab-column systems with walls. For example, one particular system in used consists of cast in situ structural walls and prefabricated floor slabs. Such systems can be referred to as mixed systems.

The further presentation was followed by the demonstration of the procedure and realization of the testing program at IZIIS, Yugoslavia.

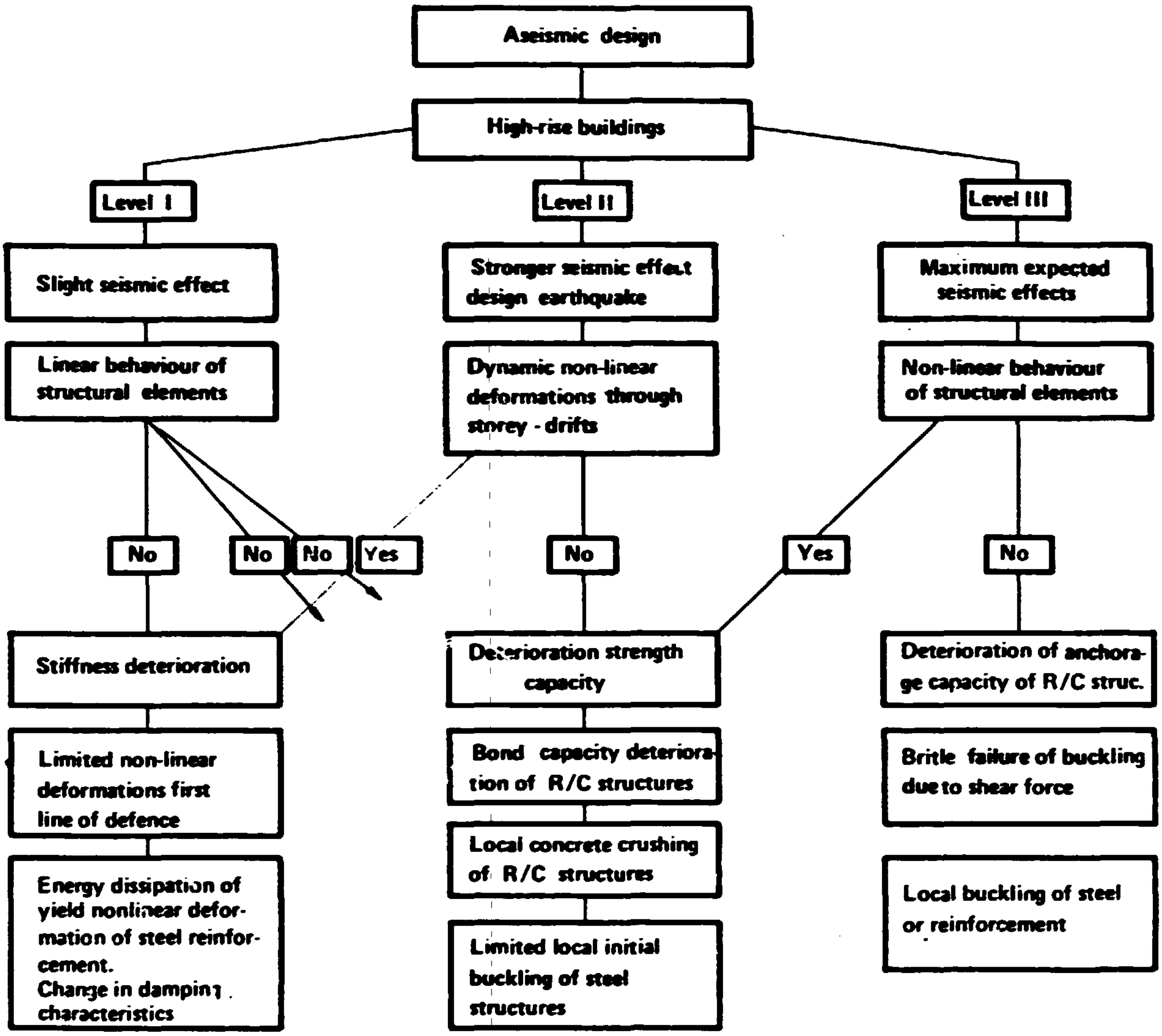

Aseismic design
High-rise buildings
Level I
Level II
Level III
Slight seismic effect
Stronger seismic effect design earthquake
Maximum expected seismic effects
Linear behaviour of structural elements
Dynamic non-linear deformations through storey - drifts
Non-linear behaviour of structural elements
No
No
No
Yes
No
Yes
No
Stiffness deterioration
Deterioration strength capacity
Deterioration of anchorage capacity of R/C struc.
Limited non-linear deformations first line of defence
Bond capacity deterioration of R/C structures
Britle failure of buckling due to shear force
Energy dissipation of yield nonlinear deformation of steel reinforcement.
Change in damping characteristics
Local concrete crushing of R/C structures
Local buckling of steel or reinforcement
Limited local initial buckling of steel structures

Fig. 1

V. FINDINGS AND RECOMMENDATIONS ON THE MICHOACAN-GUERRERO EARTHQUAKE OF 19 SEPTEMBER 1985, WITH SPECIAL REFERENCE TO MASONRY BUILDINGS

BY

E. DULACSKA

CONTENTS

1. Instroduction

2 to 15 December, 1985. the Author had an opportunity to study damages in Mexico City caused by an earthquake of 8.1 Magnitudes intensity. By then part of the damaged buildings have been demolished, only ruined buildings subsisting to that time could be studied.
The earthquake had a period of 1.5 to 2.5 sec, to what it may be ascribed that of the 330 buildings seriously damaged and collapsed, only 36, that is, about 10 % had a masonry structure. Namely, masonry structures having periods from 0.02 to 0.3 sec did not resonate to the earthquake period.

The well-known relationship

$$\log \frac{1}{T} = 7.7 - 0.9\,M$$

is an empirical correlation between period T /sec/ and magnitude M. From it is obvious that period of earthquakes of magnitude M = 6.0 to 7.5 equals that of masonry structures, hence this is the earthquake magnitude the most dangerous to masonry structures.
Nevertheless, in Mexico City, 10 percent of collapsed buildings had masonry structures, imposing to find structural causes responsible for it. To this aim, let us investigate damages typical of masonry structures.

2. Damages to masonry structures classified according to causes

2.1. Typical crack patterns in masonry structures

Masonry structures may develop four kinds of typical cracks

a) Horizontal cracks may develop by crumbling of a poor-quality mortar layer, or at tilting of a masonry pillar /middle pillar in Fig. 3/.

b) Vertical cracks may develop upon cracking of poor quality bricks due to faulty masonry bonds: or in walls with vertical layers /see left side of Fig.1, top of Fig.4, or Figs 7 and 8/.

c) Skew cracks may develop upon crumbling of poor mortar, as a "stepped" crack /Figs 1 and 5/.

d) Skew cracks may also develop so that relatively weak bricks crack. In this case wall parts separated by the crack may be relatively displaced, causing progressive collapse /Figs 2, 5, 6/.

No such slide arises in case c); after cracking, motion of the masonry absorbs much energy by friction, thus, the masonry shows a more ductile behaviour than in case of a crack of type d).

Vertical cracks under b) may entrain failure of the wall body by dividing it to slender vertical elements that may fail by deflection.

Shear walls in framework usually develop crack patterns c) or d) at a loss of bracing ability.
A broken wall tends to fall out, even if braced by r.c. ribs /Fig.11/.

2.2. Relatively thin structural walls

Walls of thicknesses of half or one brick have an insufficient resistance to lateral seismic forces, and tumble down.
If these are structural walls, tumbling entrains collapse of the building /Figs 12 and 13/.

2.3. Missing ties or r.c. tie beams

Adequately constructed masonry buildings comprise ties or r.c. tie beams embracing the walls and making them to interact. Otherwise the walls are prone to tumbling, entraining collapse of the building. The actually collapsed buildings show no trace of a tie or tie beam /Figs 12, 13, 14, 15/.

2.4. Missing connection between floor beams and walls

If floor beams are connected to the walls then the two-way wall system can interact by letting the floor transfer horizontal forces to the wall plane. The collapsed buildings were exempt from such connections /Figs 13, 14, 15/.

2.5. A wall of superposed but in sufficiently coherent layers

A masonry of superposed but poorly coherent layers gets disconnected by shear forces due to seismic effects. Thereby the wall is transformed to a structure of reduced load capacity where slender layers can fail piecewise. Such a damage may arise also if there are too many chimneys in the wall. Such a damaging is shown in Figs 7 and 8.

2.6. Shallow arches spanning openings

Relative displacements between wall bodies entrain arch support displacements. There upon steep arches behave as three-hinged arches to a degree indifferent to support displacement. Shallow arches, however, slip off the support, initiating thereby collapse of the supported floor. Such an incipient arch slip is seen in Figs 9 and 10.

2.7. Application poor quality of bricks

Bricks applied in Mexico city have strengths ranging from 30 to 50 kp/sq.cm. This brick strength is insufficient in seismic areas, and through letting the bricks crack across, it may entrain damages of types 2.1.b. or 2.1.d. /Figs 5 and 6/.

3. Suggestions for masonry structures less prone to damaging

Most of damages in buildings with masonry structures may be avoided by applying correct constructional rules, such as:

3.1. At every floor level of buildings with masonry structures, r.c. tie beams or steel ties have to be applied, and the floors connected to them. This constructional rule provides for the interaction between all the walls of the building with masonry structure.

3.2. Only reinforced concrete or steel beams lather than arches have to be used for lintels.

3.3. Masonry structures should have a wall thickness of at least one brick length /25 to 30 cm/ since thinner walls than that have no sufficient lateral resistance.

3.4. Load-bearing masonry structures have to be made with masonry bricks of min. 100 kp/sq.m compressive strength while the mortar strength must not exced 10 % of the brick strength.

Namely a higher mortar strength hardly adds to the compressive strength of the wall, while inducing it to crack patterns 2.1/b or 2.1/d depriving it from ductility. For a mortar weak compared to the brick, failure develops without brick cracking, by mortar layer crombling, by ductile deformation, peferable for the building behaviour.

3.5. In masonry buildings, it is advisable to apply a r.c. tie beam over the top level of foundation, with the intermediary of a sheet 1,5 to 2 cm thick of poor mortar /of a compressive strength of 3 to 4 kp/sq.cm/ so that the top level of foundation is wider by 10 cm each side than the wall. This weak, thin mortar layer is irrelavant to the vertical load capacity of the wall, while it is able to crumbling upon horizontal seismic forces, and to horizontal displacement with friction. Thereby much of the seismic energy is absorben. The system of r.c. tie beams over the crumbling layer provides for the displacement of the integer building on the weak mortar sheeet.

3.6. The dimensional ratio of masonry pillars has to be such that no skew shear crack arises in the wall body. This can be achieved by having the wall body width less than half its height. In this way, however, the stability of the masonry is garanteed only to a seismic coefficient a/g = 0.2 /intensity X of MKS skale/. Therefore in areas of intensities XI or XII, reinforced concrete shear walls are advisable also in masonry structures.

4. Problems of infill walls

Buildings with reinforced concrete framework are often made with brick shear walls. These are built between r.c. columns, and if adequately of a sufficient number they often prevent the r.c. framework from collapsing. Though, in several occasions, masonry walls caused collapse themselves, such as:

Exposed to a lateral force, the masonry shear wall reckoned with in design falls out from between the columns, rather than to act as expected.

Failure of part of the masonry shear walls causes eccentricity of the bracing system, and arising torsions destroy the r.c. framework.

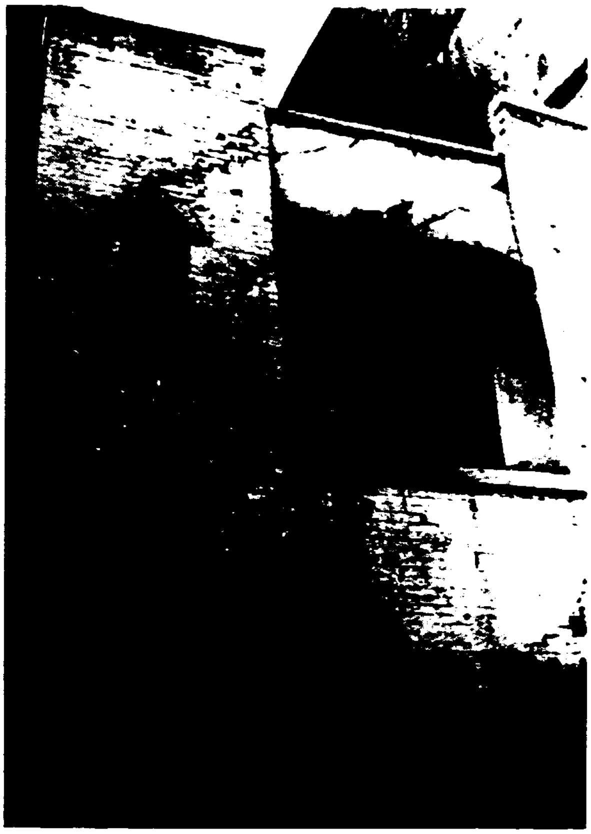
Infill walls, parapets out of design add to forces acting on columns in their plane, and transmit lateral forces to columns accelerating their failure.

Eccentric bracing of corner buildings due to infill walls causes early failure of the r.c. framework.

To escape the outlined effects, it seems advisable to develop a system having masonry walls before, rather than between, the r.c. columns, connected to them by flexible connections against tumbling.

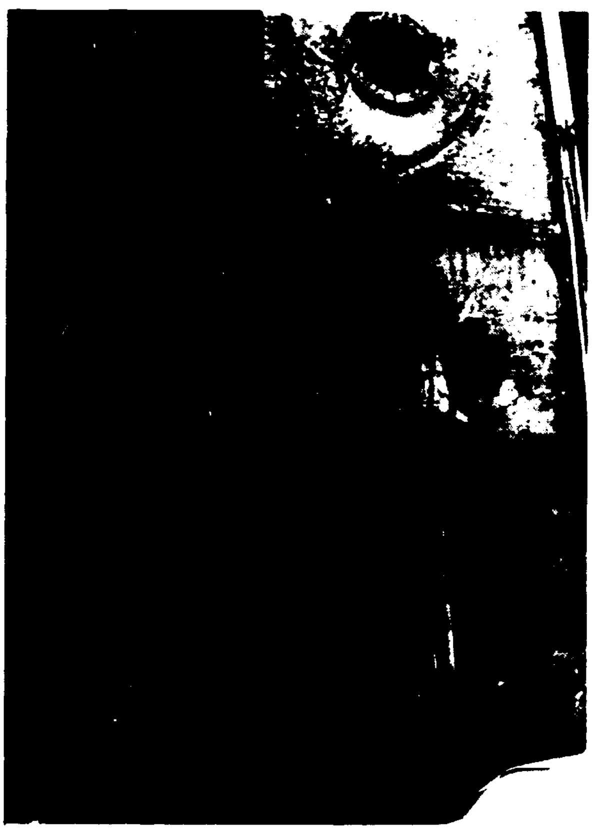
This solution would of course exclude the bracing effect of the masonry wall, requiring adequately designed and constructed r.c. walls to this aim.

Thereby building collapses due to accidental masonry effects would be excluded.



1.

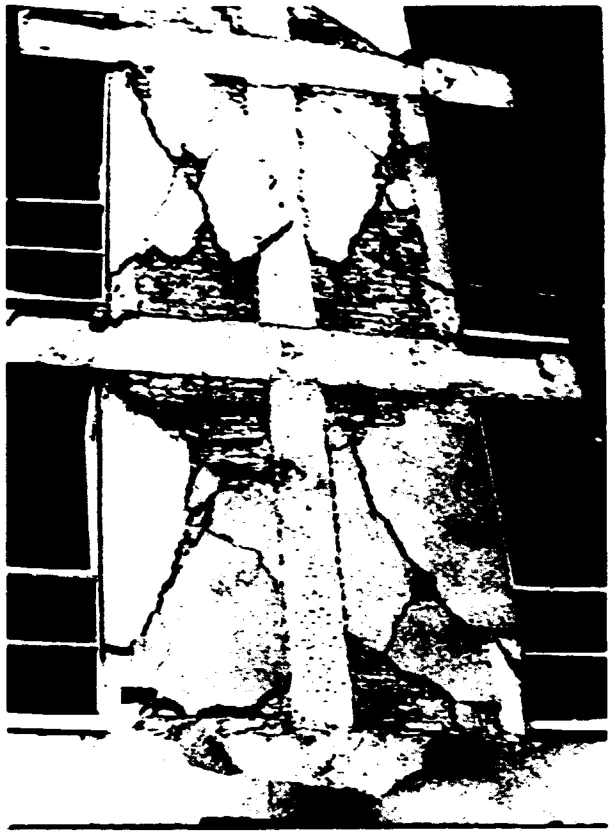
2.



3.

4.

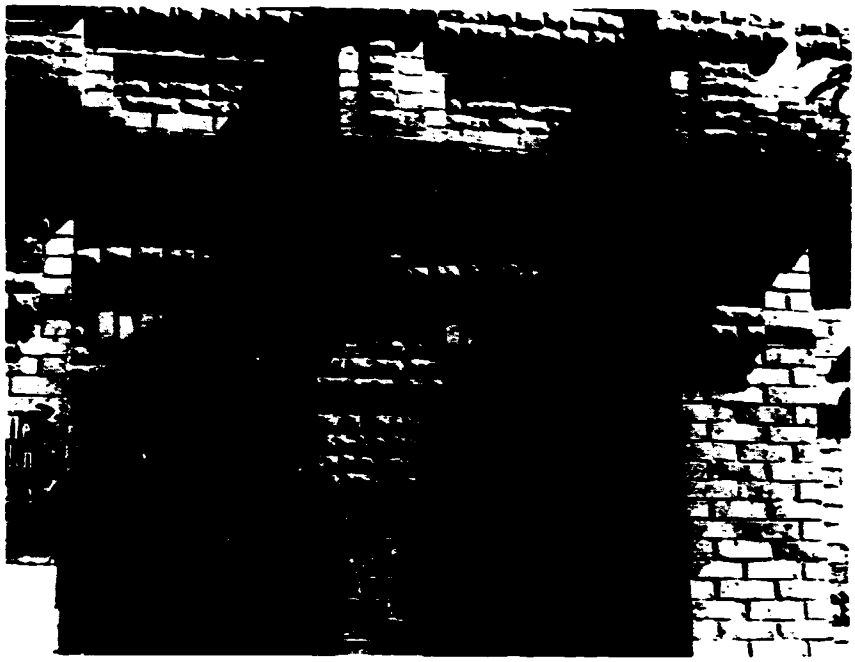
5.



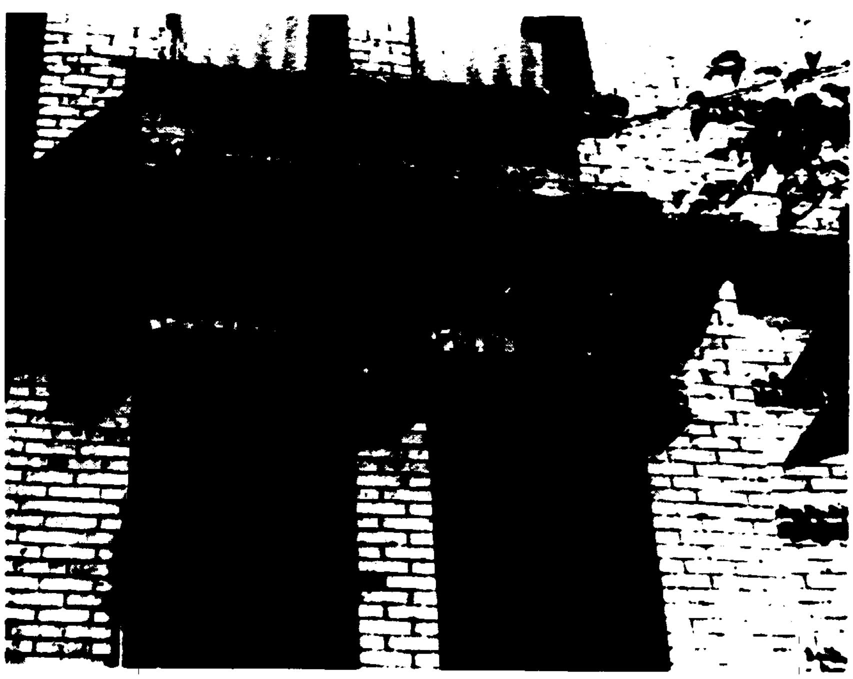
6.

7.

8.



9.



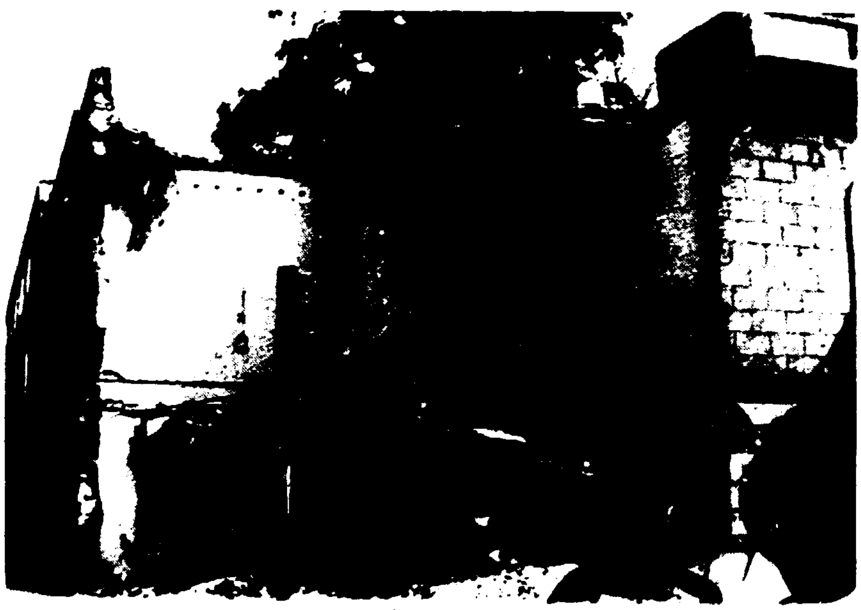
10.

11.

12.

13.

14.



15.

VI. FINDINGS AND RECOMMENDATIONS ON THE MICHOACAN-GUERRERO EARTHQUAKE OF 19 SEPTEMBER 1985, WITH SPECIAL REFERENCE TO MITIGATION OF SEISMIC RISK AND DEVELOPMENT OF A PROGRAMME FOR EARTHQUAKE PROTECTION

BY

H. SANDI

TABLE OF CONTENTS

1. INTRODUCTION

1.1. Frame of the Mission

The mission to Mexico was undertaken at the invitation of UNIDO, addressed to some of the participants in the UNDP/UNIDO Balkan Project RER/79/015, "Building Construction under Seismic Conditions in the Balkan Region".

The job description, as transmitted by UNIDO, is reproduced in Annex I of this report. The majority of the team nominated by UNIDO undertook the trip to Mexico for a two-week period from 2 to 13 December 1985. Since it was not possible for the author to obtain the required travel visas in due time, the trip was undertaken separately, with the agreement of the UNIDO Headquarters and of UNDP/Mexico, from 6 to 18 January 1986.

It is suggested to UNIDO to put together this report with the report(s) : of the other members of the project, who undertook the trip to Mexico in last December.

1.2. Activities

The main activities that took place during the two-week mission were:

a) initial contacts with the Embassy of Romania, UNDP/Mexico, UNIDO/Mexico and UNAM/Instituto de Ingenieria, aimed to detail the schedule;

b) field visits in the most affected zones of Mexico City;

c) technical discussions with specialists of UNAM/ Instituto de Ingenieria and of Camara Nacional de la Industria de la Construccion;

d) examination of the technical documentation provided by UNIDO/Mexico, UNAM/Instituto de Ingenieria and CNIC;

e) presentation of two confereces at UNAM/Instituto de Ingenieria;

f) presentation of preliminary comments and suggestions to a group of specialists of UNAM/Instituto de Ingenieria during a final meeting.

The activities related to the mission took place exclusively in Mexico City, according to the schedule adopted during the initial contacts with UNIDO/Mexico and UNAM/Instituto de Ingenieria. The findings and recommendations presented in the report are therefore related essentially to that city. Some aspects dealt with are nevertheless of wider interest.

The list of persons contacted during the mission is given in Annex II of the report. The chronological sequence of activities is given in Annex III of the report.

The bulk of activities was carried out with the kind support of UNAM/I de I, that arranged the field visits, the meetings with highly qualified experts and the lectures presented by the author and made available most of the information asked for.

1.3. Importance of the Seismic Event of 19 September 1985

The importance of the seismic event having occurred on 19 September 1985 in Mexico is obvious in case one considers:

a) its high magnitude and the extent of the areas affected by high seismic intensities;

b) the paramount importance of local ground conditions, as put to evidence by the geographic distribution of intensities;

c) the large number of buildings (out of which many designed to resist earthquakes) that have been subjected to high seismic intensities (and out of which many suffered severe damage or even collapse);

d) the number of lost lives and injured persons and the overall economic losses;

e) the high likelihood of occurrence, in the not too remote future, of events of comparable size and importance, given the seismic activity of the source zone that has generated the events referred to;

f) the high qualification of Mexican specialists, involved in the investigation of earthquake effects, which creates premisses for efficient learning from this experience as well as for dissemination of highly valuable knowledge at an international level.

1.4. Addenda on the Report

The mission report has two main sections: section 2, on findings provided by the mission activities, and section 3, on recommendations for future activities. Some final remarks on the place of the seismic events referred to and on the suitable future activities are presented thereafter in section 4. The content of Annexes I, II and III was mentioned in previous sections. The Annexes IV and V present lists of titles of technical documentation overhanded to Mexican institutions and received from them. The final Annex , VI, gives a list of some additional references, found to be useful during the drafting of the report.

In reading the report, a reference /IV.I.A.6/ means that it is necessary to look in Annex IV, part I, for list A, item 6.

The terminology adopted in the report in relation to the analysis of earthquake disasters, vulnerability and risk, is basically the same as that agreed upon during the experts meeting convened by UNDRO in 1979 /VI.6/ and used afterwards in the UNDP/UNESCO/UNDRO Balkan Project /VI.7/, considering also the subsequent developments proposed in /IV.I.A.8/.

Some figures were reproduced in the report from the reports of UNAM/I de I overhanded to the author during the mission. The figures are referred to as fig. RP.1, fig.RP.2 etc.(see figure captions too).

2. FINDINGS

2.1. On the Earthquake of 19 September 1985 and on the Seismic Hazard

The source of the earthquake of 19 September 1985 was, according to /V.II.A.1./, in the zone of contact of the Cocos Plate and of the North American Plate (fig.RP.1). The earthquake mechanism was of subduction. The source was at a depth estimated as not more than 35 Km and at a distance of some 400 Km. South-West from Mexico City. The magnitude estimates were from 7.8 to 8.1. The earthquake was followed by numerous aftershocks, of which the most important was that of 20 September, with magnitude estimates from 6.8 to 7.5.

The intensities generated on 19 September 1985 were in the range of IX (MM) for some zones of the ocean shore. The intensities estimated for Mexico City were in the range of V to VI for fi[illegible] soil, but much higher, in the range of VIII to IX, in the lake zone. The strong motion instrumental data presented in the reports /V .II.A.1/ to /V.II.A.8/ of UNAM/I de I refer to accelerations in the range of 0.4 g in Acapulco with dominant periods shorter than 0.15. and without important destructive effects, of 0.27 g in Zacatula (close to the epicenter), of 0.03 to 0.04 g in Mexico City - Zone I (firm ground), 0.04 to 0.05 g in Mexico City - Zone II, at Viveros de Coyoacan (transition) and 0.08 g to 0.2 g in Mexico City - Zone III (lake). Intensity estimates on the basis of instrumental data (response spectra), applying the methodology described in /IV.I.A.6/ to the response spectra of fig.RP.8 through RP.14, lead to intensity estimates of VIII for Zacatula, VI for Ciudad Universitaria,Tacubaya and Viveros de Coyoacan, VIII for Central de Abastos and IX for the place of the most important record, Secretaria de Comunicaciones y Transportes.

The analysis of the record obtained in the epicentral zone as well as in the capital zone put to evidence the existence of significant low frequency components of motion, which is less usual for crustal earthquakes. Note that in the Romania earthquake, that was generated also by a subduction mechanism (in that case of intermediate depth) significant low frequency components were present too. The attenuation for firm soil conditions in Mexico City was somewhat lower than the average according to data of Chapter VI, by L.Esteva, of /VI.1/. This might be due to the location of source, that was deeper than the average for crustal earthquakes. Note that in Romania the magnitude 7.2 earthquake of 1977 generated intensities of VII or VII$^+$ at places located as far as 250 to 300 Km from the source (cities of ``aiova, towards WSW, and of Iasi, towards NNE), which is about t! grades higher than predicted according to the reference given.

The local ground conditions must be considered, in general,from two main view points: the role of dynamic filter played by the system of upper soft layers, that modifies (under free field conditions) the basic rock motion on one hand, and the role of deformable support of buildings, in connection with the soil-structure interaction phenomenon. The first aspect is considered in this section only, leaving the interaction aspect for section 2.2.

The most striking feature of the ground motion was represented by the extremely strong influence of local ground conditions, as put to evidence by the records of Zone III of Mexico City (Secretaria de Comunicaciones y Transportes - SCT and Central de Abastos - C de A). The very strong filtering effect of the soft clay layer is present at both places, and creates particularly high amplifications. One can remark at both places strong peaks of the acceleration response spectra, though for different periods (some 2.0 seconds at SCT, some 3.5 seconds at C de A). These dominant periods correspond to particularly soft upper layers. For example, in the case of SCT, the period of 2.0 seconds, for a soft surface layer of some 35 m., corresponds to a propagation velocity of S-waves around 70 m/s (according to verbal information obtained, the S-wave propagation velocities tend to be located generally in the range of 50 to 100 m/s in the lake zone). The peak shear stresses transmitted by the clay at the lower boundary (contact with base rock) were under such conditions close to 0.1 MPa. To compare again with dates from Romania, the record of INCERC - Bucharest (1977) at a place with a 125 m. thick upper layer of clay and sand, with an average propagation velocity of S-waves of some 400 m/s.(determined directly), led to a dominant period of some 1.5 seconds (which is somewhat longer than the period of some 1.2 seconds which should be predicted on the basis of calculations and is put to evidence by records of microtremors, due to the decrease of stiffness for higher strain oscillation amplitudes). The record of INCERC corresponds to peak shear stresses of some 0.35 MPa. transmitted by the upper layer at the base rock level.

An attempt of evaluation of the seismic conditions (free field) in Mexico City must consider the sources affecting the zone and their activity, the attenuation phenomenon and the local conditions.

Examining the seismicity that affects Mexico, more specifically Mexico City, it turns out that the seismic hazard is dominated by the activity of the zone of contact of the Cocos Plate and of the North American Plate. The number of events recorded during 86 years(from 1900 to 1985 inclusively) is, according to /VI.3/, that relies on /VI.2/, of 34 for events of magnitudes 7 or more and of 8 for events of magnitudes 8 or more, with a maximum of $M = 8.4$ in 1932. It may be noted also that the activity was considerably higher for the time interval from 1900 to 1932 inclusively, when 19 events of magnitudes of 7 or more and 7 events of magnitudes 8 or more, out of the total mentioned previously, were recorded. It may be stated thus, that less, and less strong, events affected t : modern building stock, built during last five decades, then what should correspond to the average for this century. In case a sequence comparable to that of the period from 1900 to 1932 is considered, the return period of events of magnitude 8 is lower than five years.

The attenuation phenomenon, considered in the light of the direct experience of the 1985 event and of the sequence of intensities estimated for this century /V.II.A.1/, seams not to raise unusual problems in case of local condition. of firm soil. It would be interesting, nevertheless, to check more directly the attenuation parameters for the sequence of major earthquakes of this century, referred to previously.

The local ground conditions in Zone III (lake) of Mexico City appear to be a factor of paramount importance to deal with. The instrumental strong motion data referred to must be considered in connection with data on the features of microtremors, obtained verbally during the last discussions at UNAM / I de I. The records of microtremors, obtained in late 1985 at almost 100 points in Mexico City put to evidence a high diversity, or non-homogeneity, of conditions (dominant periods that tend to be in Zone III generally long and well correlated with the thickness of the soft clay). The dominant periods are of some 5 seconds for the crossing Constituyentes / Reforma, 2 seconds for Roma (hardest hit), 4 seconds for the way to Puebla, 2 seconds for SCT, short for Alameda. It may be noticed here also that there appears practically no shift of dominant periods from strong motion to microtremor record and this permits to state that the upper layers were not overstressed during the strong motion, such that they could be able to transmit eventually even higher stresses and accelerations during a future earthquake, provided a corresponding disturbance occurs at the base rock interface. A belief that the soft clay would not be able to transmit disturbances that are stronger than those of 19 September 1985 would thus not be justified. The agreement between the instrumental data provided by the monitoring of microtremors and strong motion data obtained in 1985 (and earlier) creates most encouraging conditions for the prediction of the spectral content of future events. It may be stated thus that, assuming source mechanisms generating at source spectral contents not very different from that of 19 September 1985, the spectral content of ground motion is in principle quite accurately predictable in Mexico City, but the significant differences between not too remote surface points must be accounted for.

Returning now to the expected sequence of events, as considered for Mexico City, it may be recalled that, accounting to the estimates of /V.II.A.1/, which deserve the whole confidence , three major events affected the capital, in 1911, 1941 and 1985, with magnitudes, according to /VI.3/ of 7.9, 7.9 and 8.1 respectively. The strong earthquake of 1932, of magnitude 8.1 according to /V.II.A.1/ and of 8.4 according to /VI.3/, would have generated smaller intensities in the capital. It may be stated on this basis that, assuming stationarity of seismic activity, two to four events of intensities comparable to those of 19 September 1985 are likely to occur every century in Mexico City. There is no reason to believe that the intensities of 1985 represent an upper bound of possible intensities, since higher magnitudes than 8.1 are likely to occur in the source zone (according to /VI.3/ the magnitude 8.1 was exceeded three times since 1900), the radiation pattern could lead to higher intensities under firm ground conditions and the soft layers of Zone III of Mexico City appear to be able to transmit higher accelerations than those recorded in 1985.

Normal buildings, built for use for about one century, must be expected to undergo several events of intensities comparable to those of 1985 (possibly even higher than those).

In case one compares the intensities observed in 1985 with the provisions of the regulatory basis /VI.4/, it turns out that, in Mexico City, the intensities having occurred in 1985 are considerably higher at some places of Zone III, but that the regulatory basis appears to have been at least sufficiently conservative for Zones I and II. Buildings built in Zones I and II according to the regulatory basis in force, should have been subjected only to moderate post - elastic deformation (if any) in 1985. This statement is supported by the absence of apparent damage under the conditions referred to.

Unless there exist significant seismic sources of a different category than those of the contact zone of the Cocos and North-American plates, which could lead to a different picture of the intensity distribution and, perhaps, of the spectral content of ground motion too, it turns out that the regulatory basis for Mexico City is conservative for Zones I and II, but unsufficiently conservative for Zone III. More discussion on this subject is presented in sections 2.3 and 2.4 of the report.

2.2. On the Earthquake Effects (Building Performance)

The information available during the preparation of this report does not permit to make any statement about the overall effects of the earthquakes expressed in terms of losses. Only aspects of the physical performance of buildings, based on the effects observed during the field visits may be tackled, under these conditions. The relatively late date on which the inspection of affected areas could be undertaken, made impossible the direct sight of most of the buildings collapsed. It was nevertheless possible to examine, in cases of buildings where partial collapse or only damage of different nature and level represented the effects of the earthquake, some relevant mechanisms and causes of damage.

The artifacts examined were buildings (residential buildings, office buildings, hotels etc.), of different heights, ranging from one or two, to some twenty stories. Most of them had reinforced concrete load bearing structures, combined frequently with masonry walls that contributed to, or influenced significantly, the mechanism of transmitting seismic inertia forces to the ground. In a few cases steel structures were inspected too.

Before going into an examination of some technical aspects related to the earthquake performance, it must be clearly stated that the proportion of buildings of Zone III (in the center of Mexico City) nominally designed to resist earthquakes, but having collapsed or having been heavily damaged during the earthquake, was unacceptably high, by any standard.

The most severe form of damage observed (except for cases of total collapse) was that of failure mechanisms extended over complete stories. It may be remarked here that the buildings examined were characterized by the failure of upper stories in most cases, what is rather unusual if the experience of other earthquakes is considered. It was possible to see, in such cases, lower stories standing and bearing a package of slabs of collapsed upper stories. The relatively frequent incidence of this severe failure mechanism must be noticed (about one quarter of the approximately eighty buildings inspected). One could observe also some cases of failure of lower or of intermediate stories, but the incidence of such cases was relatively low at the moment of inspection. This frequent collapse of upper stories may be considered as a significant feature of the performance of buildings in Mexico City, that is rather unusual for other destructuve earthquakes and deserves in depth analysis and explanations.

A structural solution used extensively in Mexico City, namely flat slabs or waffle slabs supported by reinforced concrete columns, was involved in most of the full story failure mechanisms mentioned. Another type of dramatic full collapse of such structures, that was still observable in a few cases was represented by the sliding of slabs downwards the columns, after punching-type failure. One could observe also cases when upper stories collapsed only partially (e.g. some corner zones of the buildings involved). That might be related e.g. to the effects of oscillations where overall torsion was relatively important.

Another kind of severe damage frequently observed (in many cases beyond repair) was represented by brittle type failure of columns. Brittle failure could be observed mainly as a result of excessive shear force in cases where the layout adopted led to short columns. Cases of buildings standing, but practically condemned, may be mentioned in this category. Another type of brittle failure of columns, that occurred relatively frequently, was that of failure of the ends of columns, characterized by relatively small areas of concrete sections, by low web reinforcement and by excessive longitudinal reinforcement. The zones close to the nodes were destroyed in such cases by crushing of unconfined concrete followed by buckling of unsufficiently tied longitudinal reinforcement bars. Such damage resulted in some cases analyzed in general collapse and in more cases in damage beyond repair.

It was possible, in another category, to observe cases of failure of rigid members that worked in parallel with more flexible ones. This failure pattern occurred frequently for façade structures of buildings with relatively flexible frame type main loadbearing structures. Such types of failure may have not directly endangered structures as a whole, but they may have produced, on one hand, life threatening damage and, on the other hand, considerable economic losses, The brittle failure of façade elements or components was observed again in case of about one fifth or one quarter of the buildings inspected.

Another significant aspect to be mentioned was that of the unfavourable influence of eccentric layout (i.e of eccentricities between the inertia, and stiffness, axes respectively). This kind of layout resulted in important amplitudes of the oscillations of overall torsion, that led to overstressing of the lateral load-bearing substructures and, in several cases, to important remanent deformations of overall torsion. The unfavourable torsional effects were evident in the general damage pattern not only for structures characterized by non-symmetric layout, but also for some Y-shaped structures, for which the stiffness is considerably lower for overall torsion than for overall bending-shear deformation.

The unfavourable effects of relatively large deformations and displacements were present not only in the extensive damage of non-structural components (as external, or partition, walls), but also in other kinds of damage, that were sometimes more severe and hazardous. The large deformations that exceeded the permissible limits as determined by the ductility of r.c. members led in some cases to unfavourable stress redistributions (in several cases, the exaggerated flexion deformation led to a considerable decrease of the area of cross sections of columns able to transmit shear forces in columns, producing thus brittle shear-type failure). On the other hand, the large deflections led to pounding between neighbouring structures and, thus, to uncontrolled damage degree. The cases where severe pounding effects could be observed represented 10 % or more of the buildings inspected.

A last type of severe damage to structures to be mentioned here was that of the steel structures of the 23-stories tall buildings of Conjunto Suarez. One of them collapsed, while the other ones were very severely damaged, such that several upper stories had to be removed. This case put again to evidence the fact that, in spite of the potentially high ductility of construction steel, as a building material, steel structures may not perform as ductile structures in case of unappropriate design. In fact the rows of external columns were severely damaged by local buckling due to excessive compressive forces generated by overall bending of the structures represented as vertical cantilevers.

A final aspect, that was of wide importance for the conditions of Mexico City, was represented by the behaviour of ground considered as a supporting system (the second aspect of ground conditions, as mentioned in section 2.1). The unfavourable ground conditions could be observed in many cases, with different unfavourable aspects. There was, first, the unfavourable influence of differential settlements having occurred before the earthquake, that exhausted, partially or fully, the ductility of some structures. There were, on the other hand, several cases of important differential settlements inflicted by the earthquake to relatively resistant structures for which the observable damage was limited, but the post-earthquake important differential settlements raised the question of the possibility of future use. The observed post-earthquake settlements at some parts of tall buildings reached in some cases the order of 1.m or even 1.5 m., putting some parts of buildings that pertained initially to ground floors (sometimes provided for access of vehicles) under conditions of basement floors. One may consider in this category first of all the performance of friction piles foundation systems. The ability to transmit vertical forces through friction was lost during the earthquake, resulting in overall settlement of some (in most cases high-rise) buildings. It seems likely , on the other hand, that the very important differential settlements observed at the basement level of several buildings were due to the high non-homogeneity of ground, which put to evidence, among other, by the important differences in the spectral content of microtremors recorded at different places of Zone III.

The most significant damage patterns, presented in this section, deserve some immediate discussions at this place. Some further analysis is presented in next section, devoted to findings in relation to the pre-earthquake activities.

The first preliminary report /V.II.A.1/ , in spite of having been drafted very shortly after the earthquake, presents a comprehensive, in-depth, analysis of the earthquake inflicted damage. The main features of behaviour and types of damage mentioned are:

a) brittle behaviour of columns;
b) influence of partition walls;
    b1) lack of symmetry in a horizontal plane;
    b2) soft first storey;
    b3) damage-induced lack of symmetry;
c) damage due to previous earthquakes;
d) short columns;
e) pounding between neighbouring structures;
f) excessive gravity overload;
g) $P - \Delta$ effects;
h) punching of waffle slabs.

Attention is given also to the non-satisfactory performance of foundation systems, which could be observed quite often, as well as to the negative influence of differential settlements having occurred prior to the earthquake and having more or less exhausted the ductility of structures. Some statistical data given in the report put to evidence some features of the damage distribution, putting to evidence the high incidence of heavy damage or collapse in buildings with eight to fifteen stories.

The second preliminary report /V.II.A.2/, which is devoted exclusively to structural aspects, goes into more details and gives more accurate statistical data. The damage quantification methodology used is described. Data on the geographic distribution of damage incidence are given. A typological analysis of damaged buildings is presented. The categorization of features of behaviour and types of damage reproduced previously is reconsidered and extended, adding:

e') failure of upper stories;
j) problems of foundation displacements;
k) damage in secondary members.

The preliminary conclusions presented put to evidence the need of revision of design regulations and in design practice. The main aspects referred to are:

1. Revision of design spectra.
2. Revision of ductility factors.
3. Revision of detailing provisions.
4. Promotion of more resistant structural systems.
5. Restrictions in the use of flat slabs.
6. Problems raised by masonry partition walls in flexible structures.
7. Problems of soil-structure interaction.
8. Control of application of regulations.

The results of the field inspection mentioned above, the data and comments of the preliminary reports referred to and some additional analyses, are at the basis of the attempt of summary presented at this place.

1. The ground motion was extremely severe in Zone III of Mexico City in case one considers the records of SCT (Secretaria de Communicaciones y Transportes) and C de A (Central de Abastos). The peak ground accelerations were not particularly high, yet the high number of cycles of comparable amplitudes led to highly unusual peaks of the absolute acceleration response spectra. In case one considers as a

reference, as usual, the spectral accelerations for 0.05 critical damping, the spectral peaks were of more than 1.0 g at SCT for periods of some 2 seconds and of some 0.5 g at C de A for periods of some 3.5 seconds. It is practically impossible to build tall buildings, with fundamental natural periods in the range of spectral peaks, to withstand such accelerations with controlled post-elastic deformation. In case one considers stiffnesses of framed structures in the elastic range, a period of 2 seconds (site of SCT) is a fundamental period of buildings of some 20 stories, while a period of 3.5 seconds (site of C de A) is a fundamental period of buildings of some 35 stories. In case one considers an apparent four fold decrease of stiffness, due to seismic overstressing, the fundamental periods will double , i.e. a period of 2.0 seconds will be a fundamental period for a building of some 10 stories, while a period of 3.5 seconds will be a fundamental period for a building of some 17 stories. Post-elastic deformation results, as known, in stiffness decrease, as far as non-ductile performance does not lead to brittle failure. In spite of the relative narrowness of spectral peaks, a wide range of buildings were affected by them. Thus, the spectral peak of SCT should have affected buildings of twenty stories (or somewhat more), but a slight stiffness decrease of these should put them in a better position, in the range of longer periods, where the spectral accelerations were relatively moderate. The ductility requirements for them may have been thus moderate. On the other hand, lower-rise buildings, of some ten stories, were overstressed initially even for the lower values of response spectra, corresponding to, say, 1.0 second oscillation period. The stiffness decrease due to overstressing led such buildings into a much worse position, close to the spectral peak. The already overstressed members had to face the spectral peak and, thus, ductility requirements that were beyond their physical limits. So, it is easy to understand why, in a zone where the response spectra were not much different from those of SCT, an important share of the buildings of 8 to 15 stories collapsed or underwent heavy damage, while the higher rise buildings (i.e. 20 stories or more), or the lower rise buildings (i.e. 5 stories or less) were not affected that severely. In other terms, considering the base shear coefficients that may have ranged, given the code provisions, between some 4% and 10%, the gap between these values and the spectral peak implied unrealistically high ductility requirements, corresponding to a factor "Q" of the Mexican Code /VI.4/ of 10, or 20, or even more.

On the other hand, in case one considers the records of UNAM, Tacubaya and Viveros de Coyoacan, the ground motion was moderate in Zones I and II of Mexico City, what is in good agreement with the absence of significant damage.

The seismic inertia forces generated in buildings during an earthquake must be transmitted to the ground by a chain of members of which the last is represented by the foundation system. Seismic overloading, like that which was due to the 1985 earthquake, will lead to yielding (or full failure) of the weakest link of this chain and the weakest link may lie in some of the structural members, or in the foundation and ground system. A relatively strong structure will lead in many cases to failure of the foundation system (as this actually happened in several cases).

2. A characteristic feature of building performance in Mexico City was represented by the highly unusual frequent incidence of collapse of upper stories. This feature of performance might be tentatively explained by the effects of oscillations along higher modes, or whipping effects, but more in depth analyses, based on non-linear time-history computations are required in this connection.

3. The lack of appropriate ductility of structural members could be observed at many places. The columns were often too slender, what may have led to exaggerated compressive stresses. The exaggerated longitudinal reinforcement ratios, visible at several places in damaged columns, contributed to the lack of ductility as well. A negative influence in the same sense was due to the insufficient web reinforcement, observable at several places too. Another important source of lack of ductility was represented by the presence of short columns, that could be observed in some cases also.

4. Building structures consisting of reinforced concrete columns and flat slabs (waffle slabs) were particularly common in buildings severely damaged. This solution may present advantages from the architectural-functional view point, but it is seldom appropriate for earthquake protection, since it does not lead to a proper clear frame as a loadbearing structure. The unfavourable features of performance of such solutions were often evident in the form of severe damage in the node regions (the most dramatic cases were those of punching and sliding of slabs downwards, along the columns).

5. The presence of rigid members obliged to undergo the same deformation as more flexible members of the main loadbearing structure resulted in many cases in severe damage of the rigid, but unsufficiently resistant, members. Such phenomena occurred particularly frequently, affecting partition walls, façade finishing, glazing etc. The presence of rigid members may have been beneficial in most cases for the performance of the main loadbearing structures designed to resist too low earthquake loads (unless they led to shear failure of columns or they generated eccentricities), yet their failure must have led to heavy economic losses and, in some cases, to life-threatening damage.

6. The lack of dynamic symmetry led in several cases to poor structural performance. The adoption of a layout characterized by eccentric bracing could be observed in reinforced concrete structures or in steel structures, with significant negative effects in all cases.

7. The unsufficient separations between neighbouring structures were at the origin of heavy damage due to pounding . This damage may have affected in some cases only locally the structures involved, but it led, in other cases, to collapses of important parts of some stories.

8. Steel is recognized as the most ductile building material. Steel structures are nevertheless not automatically ductile. Unappropriate layout and/or detailing may lead to phenomena like buckling of dominantly axially loaded columns, to rupture in some connections etc., as this could be observed at some places.

9. The poor foundation conditions generated by the soft clay of Zone III were at the origin of many cases of poor performance of buildings. The friction piles became unable to transmit friction during the shaking, while, in some cases, piles transmitting forces through their peaks were put in bad condition by the settlements having occurred prior to the earthquake, leaving their upper parts in conditions of short columns. The failure of the systems of piles (in the mildest cases in the form of important unequal settlements) represented an important source of highly costly damage.

10. The differential settlements having occurred in some cases prior to the earthquake were also at the origin of poor performance

in some cases, since the ductility reserves of structures were more or less exhausted by them.

2.3. On the Pre-Earthquake Activities

Before tackling specific technical aspects, some remarks of general interest must be made at this place. The ability of structures to withstand earthquakes is determined by a chain of human activities, ranging from general planning and design to manufacturing and construction activities, as well as to use, survey of performance and intervention during service. The causes of damage or of non-satisfactory performance must be looked for in principle in each of the links of this chain, and this is to be done considering on one hand the regulatory basis in force during the pre-earthquake period, and , on the other hand, the way in which the building regulations were applied in practice.

An attempt is made at this place to identify, on the basis of information available and of comments of previous sections of the report, some main aspects of pre-earthquake activities that led to unsufficient earthquake resistance, according to the lessons of the earthquake.

1. The Mexican code /VI.4/ is generally an advanced and refined code, if compared with the codes in force in various countries. The earthquake experience put to evidence nevertheless some shortcomings of it, of which the most important are the underestimate of seismic loading for Zone III of Mexico City and the lack of concern for formulation of precise general ductility requirements. The factor "c", which represents the fraction of gravity acceleration corresponding to the maximum ordinate of the design acceleration spectrum, was 0.24 for Zone III, while the record of SCT led(for 0.05 critical damping) to a homologous value 4 to 5 times higher. This gap cannot be covered by ductility of normal structures, such as to control the earthquake inflicted damage degree. On the other hand, the lack of concern for explicit, precise, guidelines on how to provide suitable ductility by means of appropriate layout and detailing, contributed to adoption of solutions that were in many cases unable to perform adequately under the earthquake induced overstresses.

In relation to the urban planning activities it may be stated that streets were in most cases sufficiently wide. The main shortcoming of pre-earthquake urban planning activities was connected with the unsufficient protection degree adopted for Zone III. In case the features of actual earthquake spectra would have been predicted, the urban planning should have practically prohibited buildings with critical heights (say 8 to 15 stories) from being built in Zone III.

2. Some shortcomings of design activities, that are partially connected with the code, but represent also a result of technical tradition, must be emphasized here again. The numerous cases of buildings with non-symmetric (eccentric) bracing members, the use of flat (waffle) slab structures, the adoption of unappropriate detailing of r.c. members (especially columns), the lack of concern for the behaviour of rigid members (e.g. partition walls or façade components) obliged to follow the deformation of relatively flexible loadbearing structures, the unsufficiently wide and unappropriately detailed separations, the adoption of unappropriate foundation solutions, were all factors with significant negative influence on building performance.

3. The information made available with respect to the construction activities was very limited and the time devoted to on - site inspection was limited too. The quality of building materials appeared to be in general satisfactory if not good. It is likely that the unappropriate detailing of some members was due rather to design than to construction mistakes.

4. The use of buildings was in some cases unappropriate. The information available on this subject permits to warn on the hazard increase genetated by overloading of floors.

5. The survey and maintenance of buildings was unsufficiently systematic. According to information available, damage due to previous earthquakes or other sources of overloading was not inspected systematically, in order to rehabilitate and, eventually, upgrade buildings affected.

6. It seems that some interventions on buildings, carried out during service without appropriate engineering and design were also at the basis of unsatisfactory performance.

Some comments on the pre-earthquake research activities may be of interest too. The information available is related to the activities of Instituto de Ingenieria of UNAM. During the visits to UNAM/I de I the high level of research activities, which made this institute one of the leading institutes of the world, was obvious. The successful installation and maintenance activities which made possible an excellent work of the strong motion accelerographs during the earthquake must be acknowledged. On the other hand the unsufficient number of accelerographs installed in the country as a whole and in Mexico City as well, must be strongly emphasized. There should have been allocated considerably more financial resources, as required by a much more extended strong motion network. In relation to the profile of activities of UNAM/I de I, it may be stated that too few resources were allocated for applied research on structural systems and that the cooperation with engineers involved in developing new structural systems was not sufficiently extensive. Some further comments and suggestions on these activities are given in subsequent sections of the report.

2.4. On the Post-Earthquake Activities

Some comments are presented at this place on post-earthquake activities on which information was obtained during the mission.

1.The emergency modifications of the Mexican code /V.II.A.3/ were necessary and are welcome. The seismic factors were modified in the right sense and more conservative conditions for the ductility factors were set. It must be stated on the other hand that, if a ground motion like that of 19 September 1985 is likely to occur again during the next decades (what is the belief of the author), the revised value c = 0.4 is yet unsufficient if compared with the spectral acceleration determined for SCT. On the other hand, the more conservative and careful adoption of the values of the ductility factor "Q" cannot compensate for the lack of general recommendations on how to provide ductile behaviour (to avoid brittle failure in r.c. structures, loss of local stability like that of the columns of the steel structures of Conjunto Suarez etc.).

2. It was not possible to obtain sufficient information on the general strategy of post-earthquake recovery, in the sense of rehabilitation and upgrading. It may be remarked, according to what could

be seen on site, that the works proceed at a relatively slow pace and that the bulk of work is still devoted to demolition.

3. The analyses of the earthquake information and lessons, as seen in the reports of UNAM/I de I/V.II.A.1/,/V.II.A.2/ etc. were generally excellent, but it may be stated that this work is yet to be completed. Some suggestions in this sense are given in next sections of the report.

4. Without denying the positive contribution of UNAM/I de I in the post-earthquake recovery effort, it seemed, during the visit, that not sufficient forces were devoted to consulting activities, to checking of structural solutions by means of engineering analyses and physical testing etc.

3. RECOMMENDATIONS ON FUTURE ACTIVITIES

3.1. On the Development of Research Facilities and Activities

The high qualifications of researchers and the fact that UNAM/I de I is one of the leading institutes in the world in several areas related to earthquake engineering were already mentioned in the report. It was also mentioned that, from the view point of practical needs, there is some imbalance, since too few resources are devoted to applied research on structural systems.

It is recommended, in this view, to develop more powerful experimental facilities, especially one or more new shaking tables able to reproduce (or model), with a sufficient safety factor, ground motions like those recorded in Zone III of Mexico City for sufficiently large structural components or models, as well as a strong reaction wall with a system of electronically controlled powerful hydraulic actuators able to load up to failure full scale components of structures. The development of such facilities should be accompanied by a corresponding development of the staff concerned with structural analysis. Perhaps a most convenient model in this sense is represented by the Earthquake Engineering Research Center of the University of California, Berkeley.

It is also recommended to find more efficient ways of cooperation of UNAM/I de I with other staffs. As an example, some field studies suggested further on in Section 3.3 will require large staffs, that could operate under the scientific coordination of UNAM/I de I.

Some further recommendations on the development of strong motion instrumentation are given in section 3.2.

3.2. On the Investigation of Seismic Conditions

As it was already mentioned in section 2 of the report, the experience of the earthquake put to evidence the considerable underestimate of ground motion severity by the Mexican code. It is of obvious importance to correct this situation,reaching a realistic characterization of the seismic hazard that affects especially Zone III of Mexico City.

Realistic hazard estimates should be compatible with the available information on source activity (8 events of magnitudes 8.0 or more from 1900 to 1985) , on macroseismic effects (3 highly destructive events for Mexico City in this century, in 1911, 1941 and 1985) and on spectral content (strong motion records of 1985 and of previous events, correlation with the spectral content of microtremors).

A useful exercice would be an attempt to assess (or retrodict) the response spectra of 19 September 1985 for various subzones (believed to be relatively homogeneous) of Zone III of Mexico City. The available information on microtremors and on damage distribution, as well as an analytical approach, could be considered for this goal.

It is also recommended to install a dense network (at surface level and at bedrock level both) in Zone III, perhaps mainly along the profiles of figure RP.4. The high frequency of cases of occurrence of strong motions makes it practically sure that, within ten years, highly valuable information will be made available. It is suggested, in relation to the strong motion instrumentation, to considerably extend the conventional strong motion network (which consists of individual instruments) and to equip some representative buildings with central recording systems.

The outcome of studies on seismic conditions, especially for Zone III, should consist of a sequence of (smoothed) response spectra (for , say, 0.05 critical damping) with some definite non-exceedance probability (say 60 % or 80 %) for various durations (say 5, 10, 20, 50, 100, 200, 500 ... years). This outcome would represent a suitable basis for a realistic microzoning, which is obviously needed in the nearest future.

The necessary support for analytical work, for recording and processing microtremors, for carrying out geophysical prospections etc. should be provided in this view.

3.3. On the Investigation of Earthquake Effects and of Vulnerability

The statistical data on the damaged buildings, given in /V.II.A.2/ are of obvious value. It may be stated, nevertheless, that these data are far from exhausting the possibilities offered by the exceptional amount of basic information at hand after the earthquake. A more complete statistical inbestigation of the damage degree inflicted to various classes of buildings would be must beneficial for the immediate tasks faced in Mexico as well as fot the international technical community.

An in depth statistical investigation requires the use of appropriate inspection forms (computer compatible) to be used for randomly selected samples of the populations of buildings (buildings more or or less affected) and afterwards storage of information in a comprehensive data bank . This would make possible a statistical treatement of subsamples believed to have been subjected to homogeneous loading conditions. Information on some in depth analyses carried out in Europe subsequently to destructive earthquakes are given in several publications, like /IV.I.B.1/, /IV.I.A.8/, or volume B of /VI.7/. The information on the damage distribution for various classes of buildings, presented at the beginning separately for various subzones of Zone III of Mexico City, would make it possible first to derive conclusions on the features of ground shaking, in spectral terms (statistical damage spectra and conclusions on the geographic distribution of intensities in Bucharest, as related to various spectral intervals, derived in this way in Romania, are presented in /IV.I.B.1/). Further on, the data on the damage distribution may be used to derive vulnerability matrices for various classes of buildings and, hence,to dispose of basic data required for full risk analysis, as well as for basic information that could greatly contribute to the refinement of the macroseismic intensity scale (some technical aspects on this subject are dealt with in /IV.I.A.6/). This latter category of information would be of especially high interest for the instrumental community.

Finally, it is of paramount importance to carry out comprehensive and realistic estimates of the earthquake inflicted losses. It is necessary to consider separately the various loss components (lives, property, activities disrupted etc.) and it is most desirable to collect information such as to derive conclusions for the proneness to losses as conditional upon the severity of physical damage or upon some parameters of the oscillations of buildings. This information could be organized in the form of "secondary vulnerability" matrices /IV.I.A.8/.

Some of the information given verbally during the Perugia, Italy, International Seminar "Learning from Earthquakes" (April, 1985),may be mentioned here, in relation to the earthquakes of Friuli (1976) and off the Adriatic Coast (1979). Both earthquakes struck with high intensities (IX or more) some populated zones, producing locally heavy losses. In each of the cases the population living in areas affected by intensities VIII or more was less than 100,000 inhabitants and the losses of lives were definitely smaller than in Mexico City. The complex loss estimates mentioned during the seminar were of 5 billion US dollars for Friuli and of 4 billion US dollars for Yugoslavia.

Note again that realistic loss estimates are indispensable for carrying out comparative cost-benefit analyses of various possible protection or intervention policies, aimed to justify the most reasonable of them.

3.4. On the Development of Design Regulations

Only some emergency problems are considered at this place in relation to the design regulations. Some aspects related to the emergency modifications /V.II.A.3/, that should be corrected were mentioned already in section 2.4.

It was mentioned there first that even the provisionally increased factor "c" (c = 0.4) adopted, is unsufficient, since this would imply, in case of occurrence of a new similar event, exaggerated ductility demands and consequently lack of control of building performance (the occurrence of a new motion with a response spectrum peak of some 1 g, which is, according to the author's belief, rather likely within the next half century, would lead to ductility demands of some 2.5 times the design values Q of the code). A first recommendation is thus to bring the value of "c" into better agreement with the peak spectral values, calculated on the basis of instrumental data, or inferred, on the basis of other information available (as suggested to be done in section 3.2).A significant increase of the value "c" for Zone III may make some classes of buildings prohibitive, but hazardous buildings must be prevented any way from being built any more. Some recommendations in this sense are given in section 3.7 of the reports in connection with the future strategy of development of the building stock.

It is, according to the author's belief, very important to introduce in the design code provisions on the layout and detailing of buildings and other structures, considering the structural and non-structural components both. As an example, the Romanian technical community is strongly convinced that the provisions of this nature,given in the Romanian code /IV.I.B.4/ are at least as important as the provisions of quantitative nature, concerning the design loads, the design stresses of materials etc. Given the experience provided by the on-site inspection during the mission, such provisions should concern the location of structures, the distribution of masses and stiffnesses along the height and in the horizontal plane (emphasis on dynamic symmetry!) etc. The ductility requirements must be expressed in quantitative terms. For reinforced concrete members this will mean limiting the compressive forces (average compressive stresses) in columns, limiting of the average compressive stresses at the most loaded zones of shear walls, limiting of shear stresses in horizontal sections, limiting in spacing of web reinforcement in the zones where post - elastic behaviour is expected to occur (in order to provide confinement of the concrete), avoiding of short columns, avoiding of shear failures in beams, avoiding of failure mechanisms to develop in nodes of frames and in columns etc. General recommendations on the layout and detailing of steel structures are necessary too, because steel structures are in no way automatically ductile (consider the performance of Conjunto Suarez). Loss of overall or local stability must be avoided, the zones where plastic strain is to develop must be sufficiently extensive, the connections must not permit brittle failure etc. Similar provisions are necessary for masonry buildings or components of composite structures. Even in case there exist more detailed design provisions for some particular classes of structures, the general guidelines must be present in the general earthquake resistant design regulation and a consistent philosophy must lie at the basis of this category of provisions and must be also explicitely presented. This philosophy must be consistent with that of limitation of displacements (absolute and relative, both), of deformation or strain, such as to avoid dangerous P - Δ effects, severe damage to non-structural components and even alteration of the schemes of transmission of internal forces due to exaggerated deformation (curvature of reinforced concrete members etc.).

More strict provisions in relation to separations between neighbouring buildings or structures must be considered too, regarding the required distance and the finishing elements.

Attention should be paid also to the 3 D oscillations of buildings (note, as an example of practical approach, Annex II of the Romanian code /IV.I.B.4/).

Given the relatively short time interval between destructive earthquakes, which is in the range of a few decades, it is necessary to develop a philosophy of protection of buildings against several strong earthquake actions to occur during a normal life time. Minimal conditions in this sense would lead to consistent avoiding of non-ductile post-elastic performance and to limitation of post-elastic strain, i.e. to sufficiently conservative values of the factor "Q" of the design regulations.

It is not possible to formulate here more specific recommendations, given the lack of detailed information on the structure of the system of building regulations of Mexico and on the general technical environment. What can be mentioned in this view is the need for a consistent treatement of the regulatory basis as a whole and, more specifically, particular attention to be paid to the coordination between the regulatory basis concerning the design of new works and the regulatory basis concerning the repair and strengthening of damaged structures and the evaluation of the existing building stock in relation to the mitigation of seismic risk related to it (see section 3.6 of the report in this relation too).

3.5. On the Development of New Structural Systems

The examination of structures adversely affected by the earthquake put to evidence several shortcomings of the structural solutions, as mentioned in section 2 of the report. Among other, the direct experience of the 19 September earthquake has confirmed the fact that flat (waffle) slabs on columns are not an appropriate solution for zones of high seismic loading and this system shall not be used in future in its present form, at least in Zone III. One condition to adopt better suited structural systems is represented by an improved regulatory basis, but this necessary condition is not sufficient too. The development of appropariate earthquake resistant solutions for large structures represents a demanding activity, which requires in many cases special studies to be carried out by research staffs, with approoriate means of engineering analysis and of physical testing. It is thus desirable to organize a close cooperation of design and research staffs, in order to develop some pilot, or standard , solutions, for some types of buildings, such as to present a controlled safety degree for the specific conditions of Mexico (especially for Zone III of the capital). The positive experience of development of standardized solutions in some countries on the basis of extensive teams work should be considered in this connection.

From another view point, without denying the importance of architectural and functional aspects, it is not permissible to leave the architects the upper hand in developing the layout of buildings, since this may lead (as it actually has during the pre-war period in Romania and more recently in Mexico City) to various shortcomings like eccentric bracings, discontinuities of main load bearing members, unbalanced distributions of stiffnesses and masses etc.).

3.6. On the Mitigation of Seismic Risk Affecting the Existing Building Stock

The direct experience of Mexico, as that of Romania and of numerous other countries has put to evidence the high risk connected with the existing building stock, due either to the lack of appropriate protection measures adopted in design, or to the adverse effects of previous earthquakes or of other overloading sources. The existing building stock deserves special attention in order to mitigate the risk of severe future losses. The problem of mitigation of seismic risk has two main sides: development of appropriate repair and strengthening techniques and development of an appropriate methodology of adopting and implementing the decisions on intervention.

The design of repair and strengthening works for existing buildings is much more an art than the design of new buildings and appropriate education and experience are necessary in this view. It is necessary to consider, at the same time, the local aspects, of introducing new material which is to be well connected with the existing one, and the system aspects, of providing finally a structure with an appropriate layout and with a satisfactory earthquake performance. It is highly desirable to develop a comprehensive regulatory basis, covering both aspects referred to. A useful guide in this sense may be represented by volume 5 of /VI.8/, which is the outcome of an international effort, in which experts of the Balkan region were assisted also by most competent experts of other regions of the world. The repair and strengthening works require also the development of appropriate construction techniques and equipment Given the dimension of the problem facing Mexico City and perhaps other regions of the country, it is suggested to consider organizing of some special enterprises, to carry out pilot work, to check and endorse work carried out by other enterprises. This could be a concern for the Camara Nacional de la Industria de la Construccion.

In relation to the repair and strengthening works the special case of some buildings that were damaged only slightly, but underwent strong uneven settlements, must be considered. It may be reasonable to consider, for such cases, the possibilities of relocating them, in relation to the modification of the foundation system. Such buildings could be moved by some tens of meters in connection with some possible changes in the urban pattern. Some suggestions on this subject are given in section 4.2. of the report.

The adoption of a reasonable decision on the intervention on the existing building stock must rely, in principle, on a more or less explicit cost-benefit analysis, that must rely, on its turn, on an appropriate risk assessment for the actual state of the existing buildings and for the alternative solutions on intervention considered to be feasible. The basic data to be considered in the risk assessment are related to the seismic hazard to the vulnerability of the building stock and to the specific elements at risk. The evaluation of the existing stock is an activity dealt with especially in Japanese and American literature and the Japanese report /V.II.A.9/

presents extensive developments in this sense. It must be mentioned, in this respect, that the methodology presented at that place includes the evaluation and the adoption of decision. It may be recommended to consider in this context a different philosophy too, namely to use directly, from the methodology referred to, only the part that makes possible an assessment of the resistance (or, conversely, of the vulnerability) of the existing buildings, such as to leave more freedom in the adoption of decisions, considering other factors too(primarily, the subsequent lifetime considered as reasonable for some existing buildings, given the general pattern and strategy of urban development and, also, the role of a building in the urban fabric). One can consider, for methodological aspects, the excellent work coordinated by Mexican specialists /VI.1/, developments of volume 4 of the UNDP/UNIDO Balkan Project referred to /VI.8/, or the debate of the symposium /IV.I.B.3/. It is suggested to use, in order to determine the input for risk analyses, also the data on vulnerability (damage probability matrices) reproduced in /VI.1/, and those obtained more recently in Europe in the frame of Working Group B of the UNDP/UNESCO/UNDRO Balkan Project /VI.7/ and in other countries too /IV.I.A.8/. Further on, it is desirable to develop practical criteria, suited for the Mexican conditions of differentiation of the required degree of upgrading as a function of functional importance (including the consideration of elements at risk) and of subsequent lifetime. Finally, the decision on the intervention must be correlated with the general policy of urban renewal and development, considering the position of individual structures in the urban fabric.

3.7. On the Strategy of Future Urban Development

The extent of damage and losses inflicted in 19 September 1985, especially in Zone III (center) of Mexico City, as well as the high likelihood of new seismic events of comparable severity in the not too remote future, makes necessary the development of a consistent strategy for the future urban development. It is not possible to express here ideas about other regions of Mexico, due to lack of appropriate information on hazard and on the features of the building stock.

The information on the seismic conditions of Zone III is convergent in demonstrating the repeatability of ground motions with response spectra similar to those of 19 September. On the other hand, it is uneconomical if not practically impossible to build buildings 10 or 15 stories high to resist with sufficiently high probability ground motions with such spectra, using design ductility factors "Q" with values of 2 to 4. It turns out that a building strategy must be developed such, that only low-rise buildings (say up to five or six stories) or tall buildings (with fundamental periods somewhat larger than the period of the response spectra peaks) will be built in future. The design regulations should be sincere, in penalizing buildings of hazardous heights (more precisely of hazardous fundamental natural periods) with values of the design spectra that are consistent with a response spectrum with appropriate non-exceedance probability.

The strategy of urban development must consider in a consistent way the new developments and the existing building stock. A comprehensive and consistent assessment of the risk affecting the center of Mexico City should be carried out in the nearest future and intervention priorities must be set on this basis. The task of mitigating the risk is urgent, since the return period of ground motion with spectral peaks equaling 50% of the spectral peaks of 19 September may be in the range of 20 to 30 years and many of the damaged structures would not resist such a loading if they are not properly rehabilitated.

Finally, it must be stated that the urban planners and architects must closely cooperate with earthquake engineering specialists in order to set up an appropriate strategy of development.

3.8. On the Development of a National Program of Earthquake Protection and Earthquake Preparedness

The high seismicity affecting Mexico and in particular Mexico City makes obvious the importance of the development of a comprehensive national program of earthquake protection and earthquake preparedness. Such a program must encompass the pragmatic aspects related to the mitigation of risk and to the preparedness to efficiently react under earthquake conditions, as well as the scientific aspects related mainly to the ability to learn as much as possible in the event of future strong earthquakes.

The pragmatic aspects dealt with must cover all the aspects of risk mitigation in new developments and in the existing ones, considering the possible direct effects as well as the possible secondary effects generated by chains of events. They must cover also the various facets of preparedness, related to the function of automated devices, to the reaction of humans, or to the availability of resources and know-how required by emergency interventions. The scientific aspects dealt with must cover the installation and maintenance of automated recording equipment, as well as the human preparedness for prompt and efficient post-eartnquake inspection (including inspection forms aimed to cover various specific aspects, prepared such as to permit the storage of information in appropriate data banks).

A primary pre-requisite of successful preparedness is represented by appropriate education measures at all levels, from the large public to the specialists asked to react in order to limit the earthquake effects and to scientists.

It may be suggested to consider as a useful model the Japanese preparedness activities presented in /VI.5/.

4. FINAL REMARKS

4.1. On the Event of 19 September 1985

The earthquake of 19 September 1985 was evidently a major event, given its seismological characteristics, as well as its impact. It is important to derive from this event all lessons, which will surely benefit not only Mexico, but also the international community, from several scientific and pragmatic view points. It is therefore necessary to provide to the highly qualified community of specialists of Mexico the whole necessary support in order to complete under best conditions the work under way.

There may be mentioned some significant similarities between the events of Mexico (19 September 1985) and of Romania (4 March 1977). The earthquake mechanisms were similar and the earthquakes originated in sources zones that generate destructive earthquakes several times in a century. High intensities occurred in both cases at large distances from the source and the most affected areas are in both cases the central zones of capitals. The ground motion in the capitals was characterized by relatively long dominant periods in both cases, such that tall buildings were mainly affected. There existed, in both cases,a large building stock built to resist earthquakes that was subjected to high seismic intensities. Given the high probabilities of occurrence of similar events within the next few decades in both countries, an exchange of information and of experience is of obvious mutual interest.

4.2. On Some Suitable Subsequent Activities and Projects

The proposals to organize UNDP assistance projects, at the national and regional levels both, presented in /V.I.1/,are most welcome and deserve all available support. It may be mentioned in this respect that the experience provided by the recent destructive earthquakes of the Balkan region (especially those of 1977 in Romania and of 1979 off the Adriatic Coast) should be transmitted to Mexico, using the services of interdisciplinary teams of specialists. Among other, it may be mentioned that the experience acumulated during the last years in Romania in relation to the relocation of several buildings (especially historical monuments), could b rticularly useful for Mexico City, where relocation could represent a unique solution for rehabilitating some buildings relativel . ghtly damaged, but having undergone important post-earthquake diff-ential settlements (note that buildings with masses of hundreds ousands of tons were displaced in Romania at distances of tens, or even hundreds, of meters and that, in residential buildings aving been moved the inhabitants continued their normal life, dipposing of all services, like power, gas, water etc.). The positive experience of Romania in developing, repair and strengthening solutions for r.c. members using epoxy injections and plating with fiberglass fabric embedded in epoxy resins can be mentioned here too and recommended for application under the conditions of Mexico City. A visit of Mexican specialists to Romania and /or conversely, of Romanian specialists to Mexico, to provide detailed information on the actual possibilities, would be suitable in this relation.

ACKNOWLEDGEMENTS

The author is glad to express his gratitude to the UNIDO HeadQuarters for having been selected as a consultant to UNDP/ Mexico, to UNIDO/Mexico and to the Embassy of Romania for the Support provided during the mission. He is highly indebted to Instituto de Ingenieria of UNAM for the warm hospitality as well as for the excellent opportunities of gathering information and of learning during the mission. Finally, he wants to express his thanks to INCERC - Bucharest for the support provided for participation in this mission and for preparing this report.

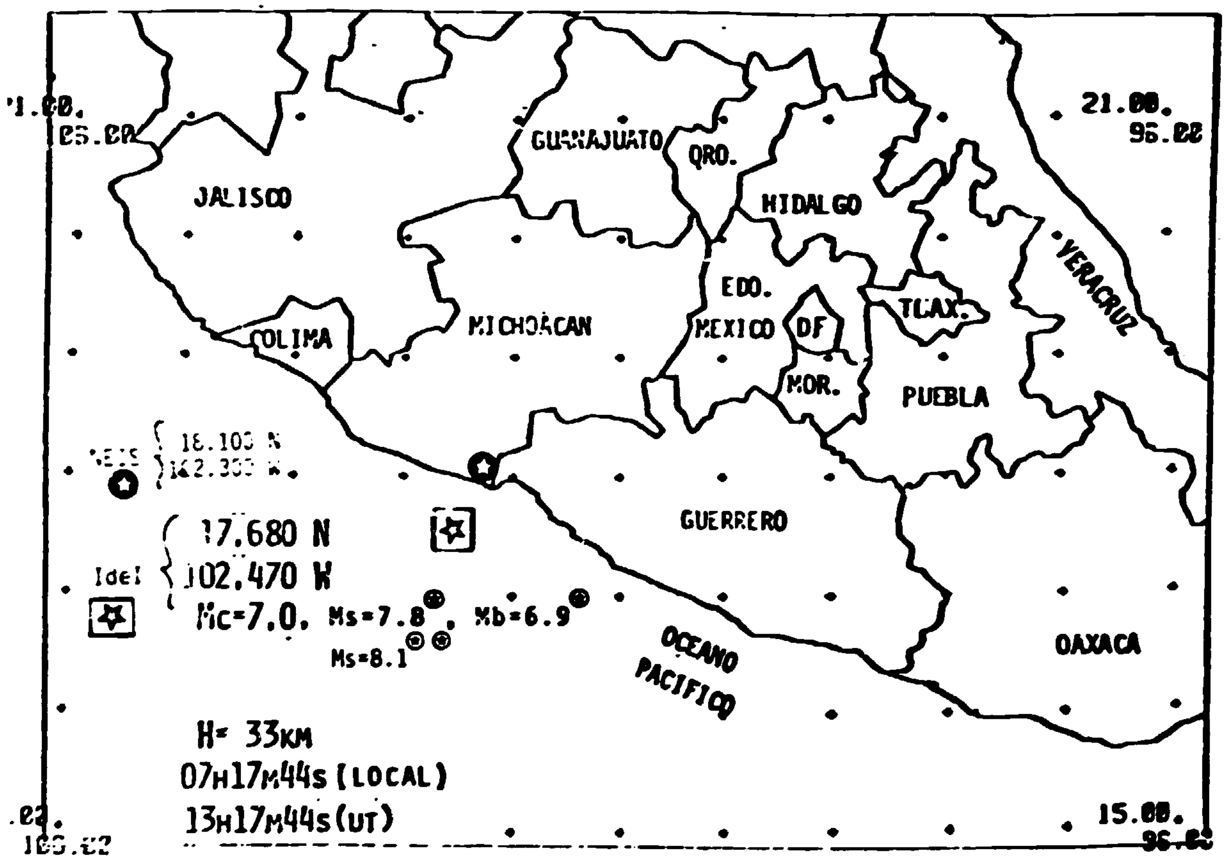
The author is glad to mention also that the information gathered during the mission is of highest interest for the community of Romanian specialists involved in earthquake protection activities and to express to UNIDO and to UNAM/I de I, on behalf of the Romanian national committee on earthquake engineering, the gratitude for the opportunities provided in this connection.

FIGURE CAPTIONS

No.	Title	Source
RP.1	Preliminary epicenter location	Fig.1 of /V.II.A.7/
RP.2	Location of digital accelerographs in Mexico City	Fig.2 of /V.II.A.7/
RP.3	Subsoil zonation of Mexico City	Fig.3 of /V.II.A.1/
RP.4	Subsoil profiles in Mexico City	Fig.4 of /V.II.A.1/
RP.5	Equal depth curves of the deep deposits	Fig.5 of /V.II.A.1/
RP.6	Location of collapsed or severely damaged buildings	Fig.1 of /V.II.A.1/
RP.7	Location of damaged zones	Fig.2 of /V.II.A.1/
RP.8	Response spectra, Mexico City,CU,outdoor	Fig.8 of /V.II.A.7
RP.9	Response spectra,Mexico City,Tacubaya	Fig.14 of/V.II.A.7/
RP.10	Response spectra, Mexico City, Viveros de Coyoacan	Fig.13 of/V.II.A.7/
RP.11	Response spectra, Mexico City, SCT	Fig.10 of /V.II.A.7/
RP.12	Response spectra, Mexico City, Cde A , front of refrigerator	Fig.12 of/V.II.A.7/
RP.13	Response spectra, Zacatula, NS	Fig.10 of/V.II.A.8/
RP.14	Response spectra, Zacatula, EW	Fig.11 of/V.II.A.8/

9.

LOCALIZACION PRELIMINAR DEL SISMO DEL 19 DE SEPTIEMBRE DE 1985.



Datos del National Earthquake Information Service. Paul Bodin, comunicación personal.

Dato corregido por NEIS el 260985.

Fig 1. Localización del epicentro preliminar y principales datos sismológicos (todos sujetos a revisión)

Fig.RP.1

10.

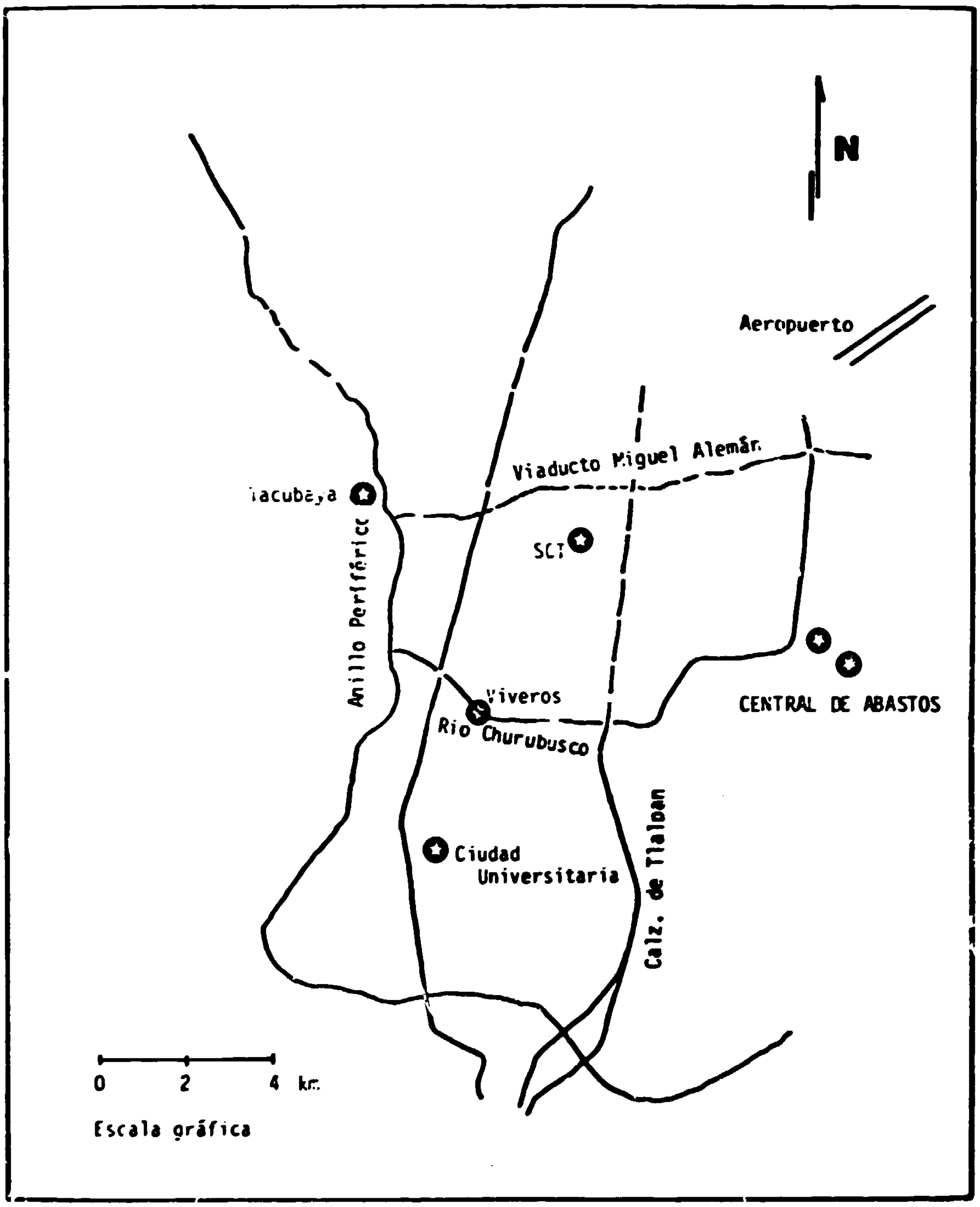
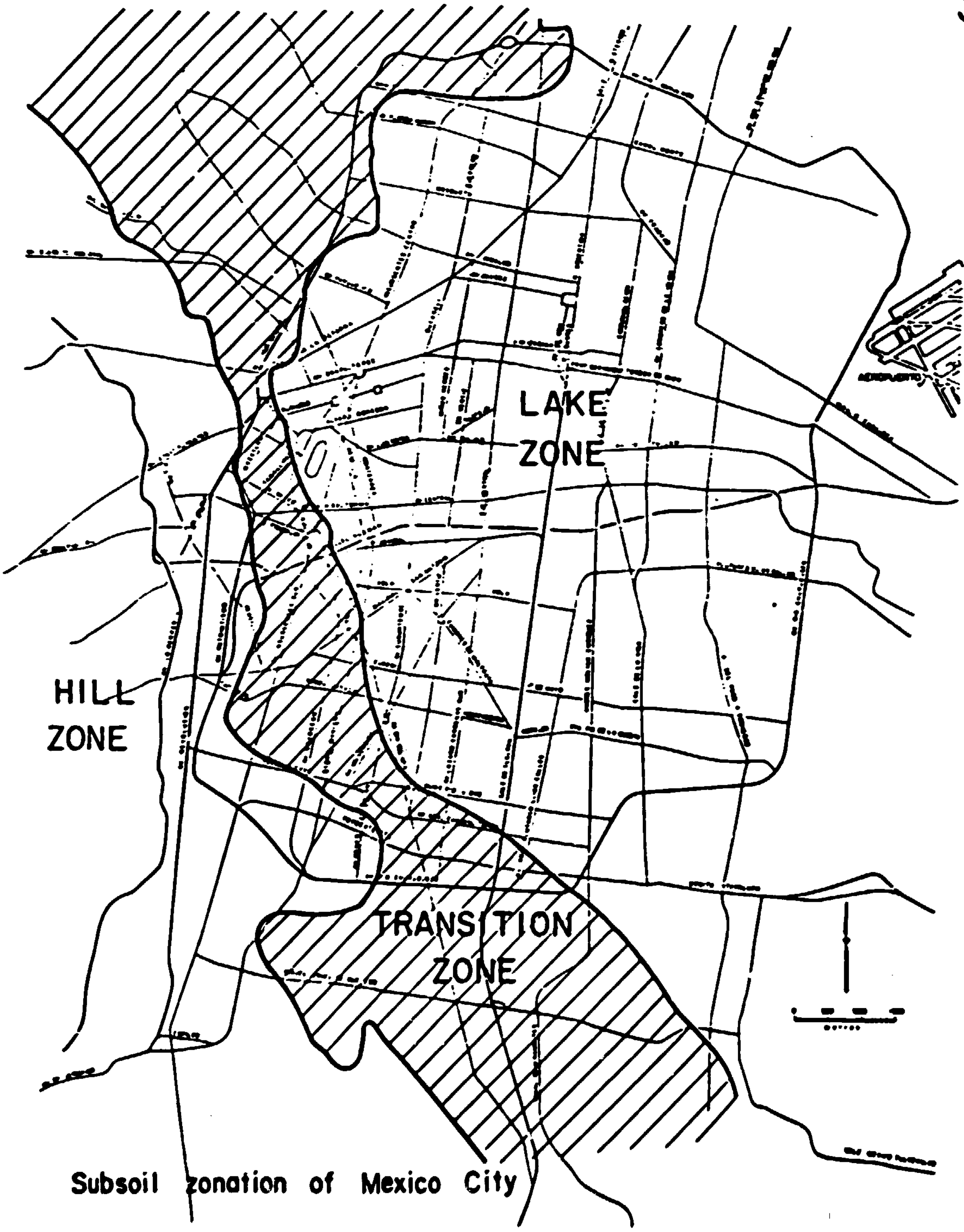


Fig 2. Localización de acelerógrafos digitales en México D.F.

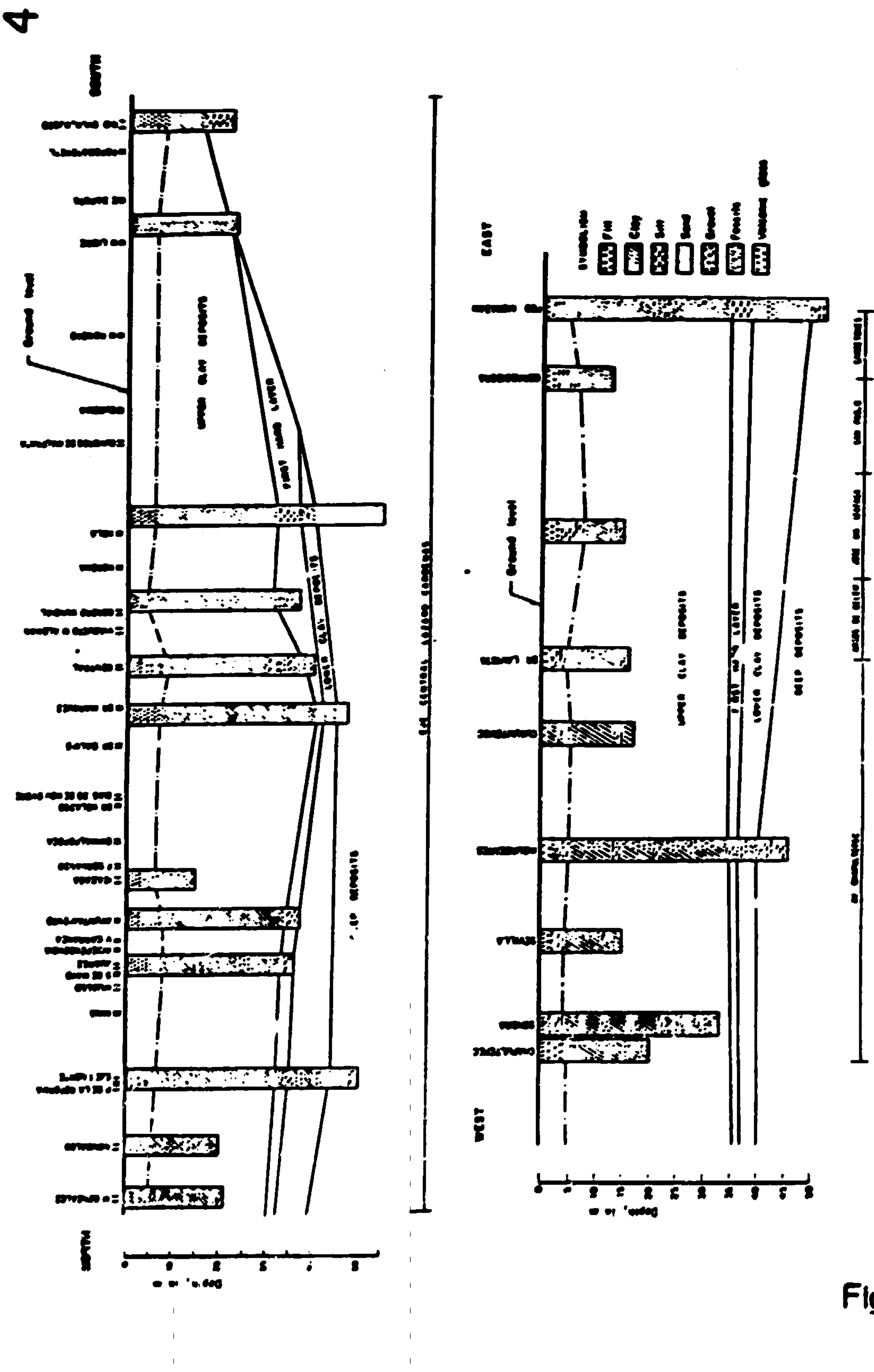
Fig.RP.2

3



Subsoil zonation of Mexico City

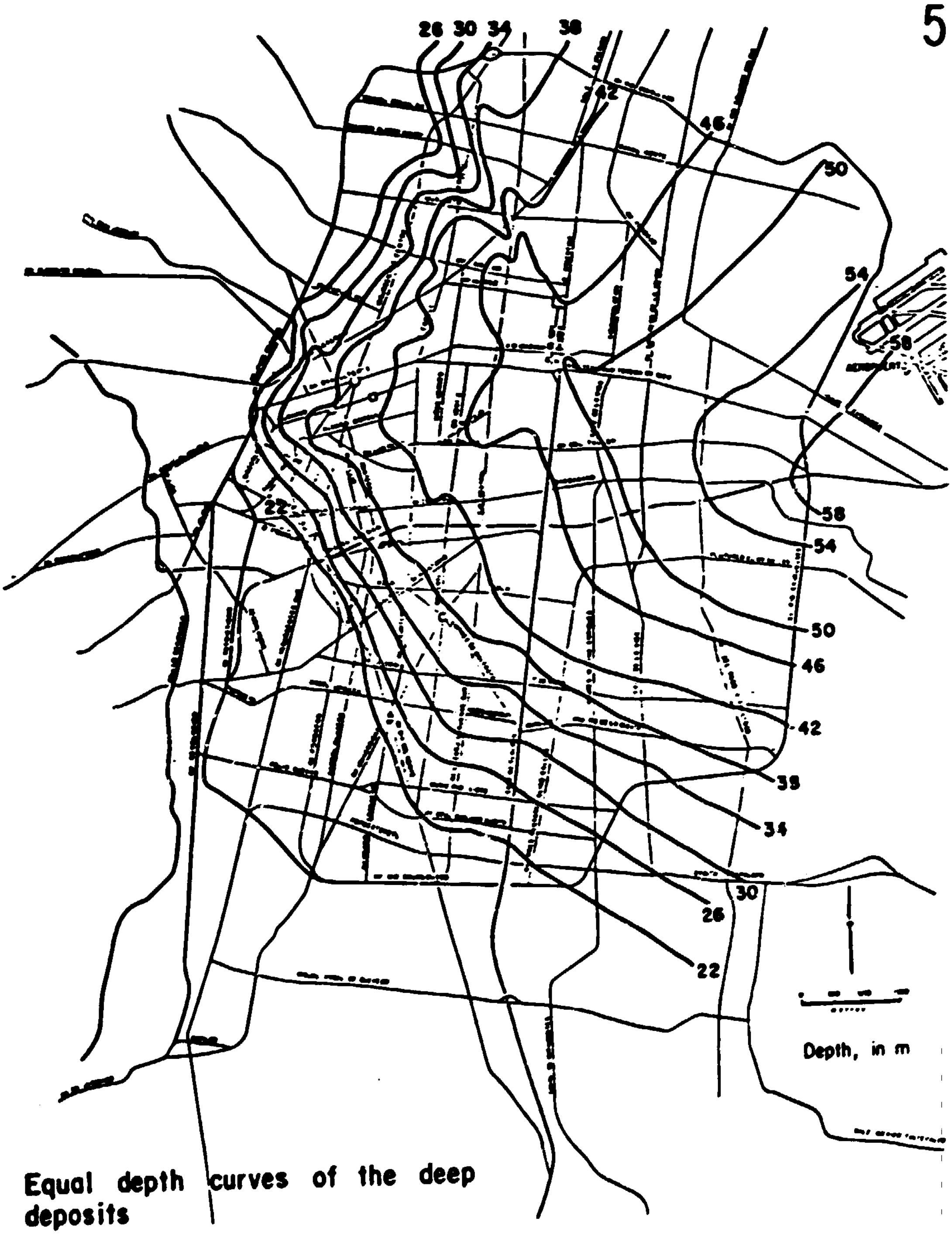
Fig.RP.3



Subsoil profiles

Fig.RP. 4

5



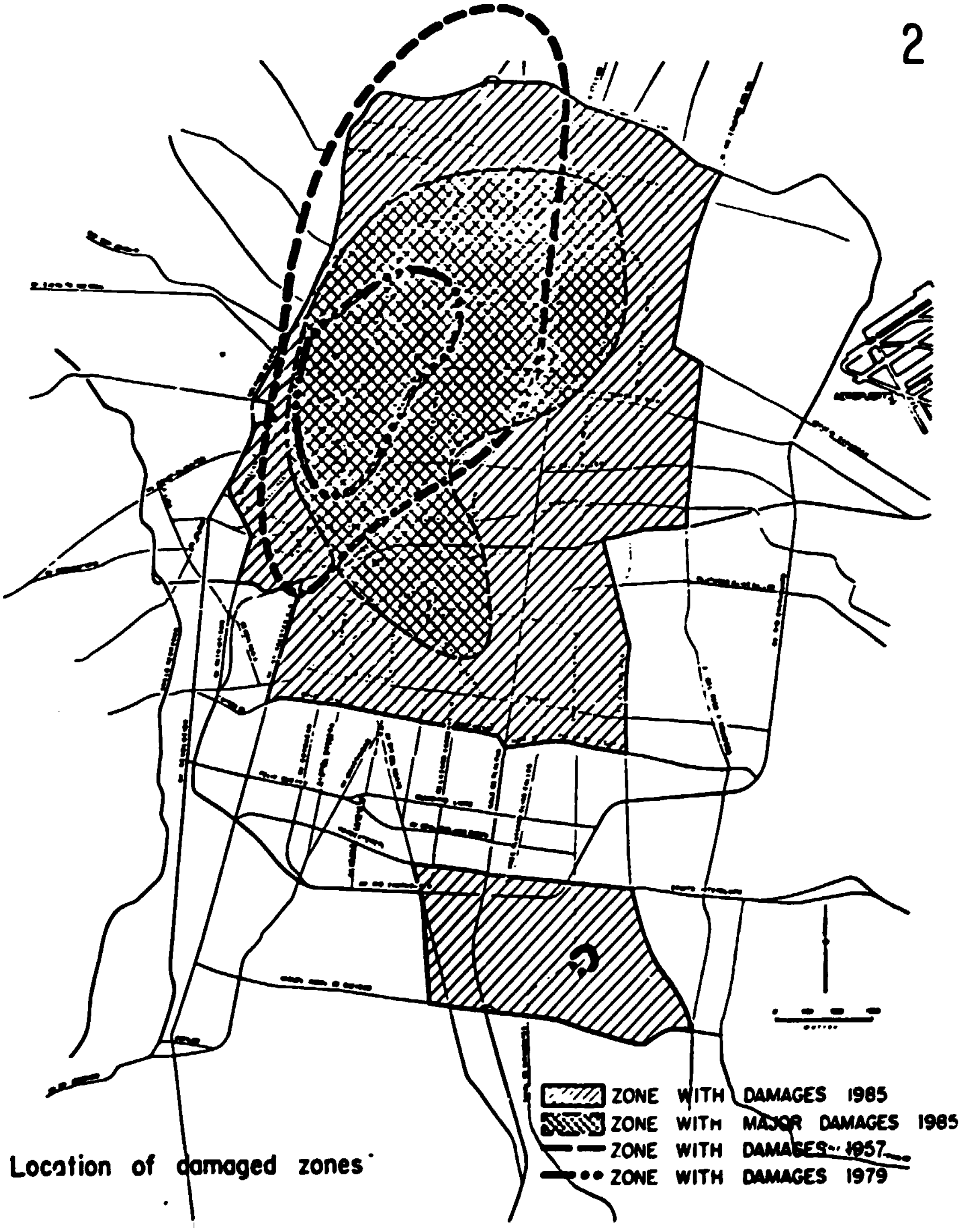
Equal depth curves of the deep deposits

Fig. RP.5

Location of collapsed (•) or severely damaged (x) constructions

Fig.RP.6

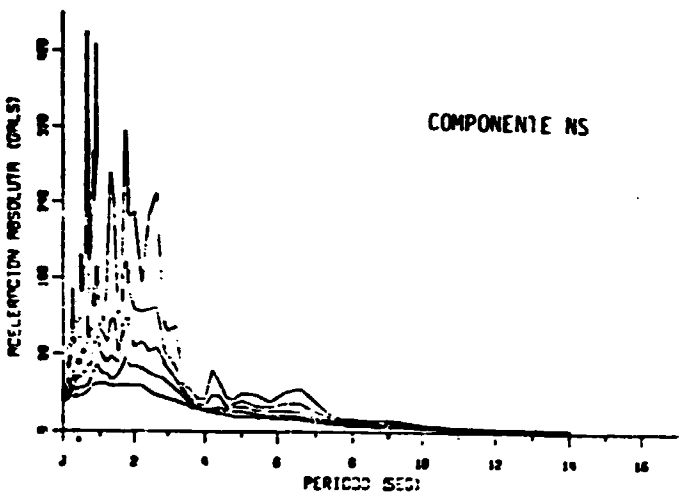
2



Location of damaged zones

Fig.RP.7

20.



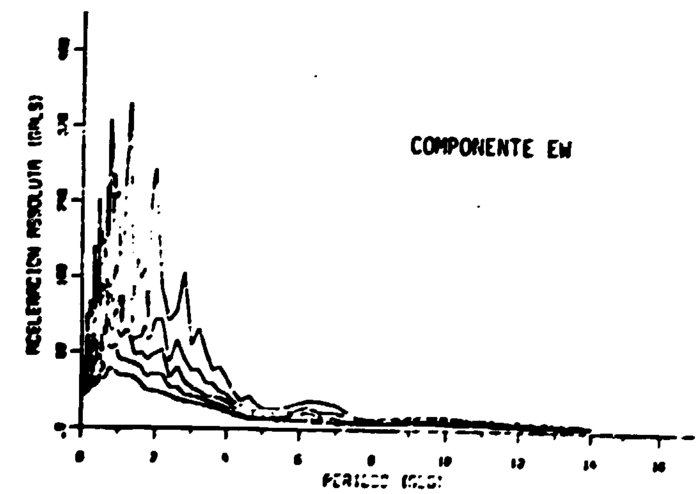
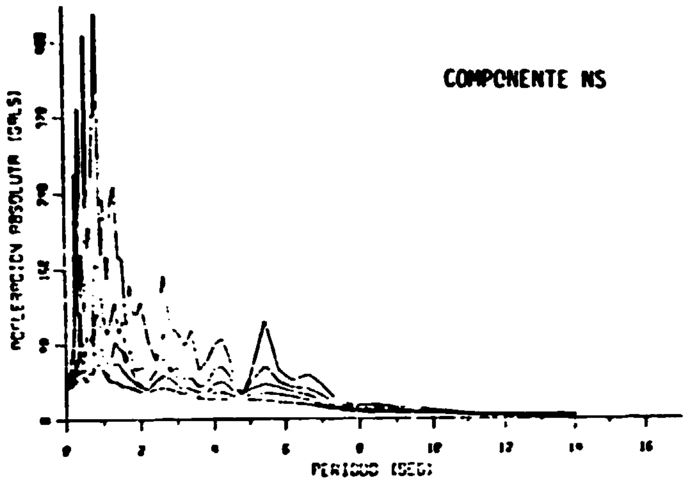


Fig 8. Espectros de respuesta para las componentes NS y EW del registro obtenido en el jardín del Idei en CU, en México DF el 190985.

Fig.RP.8



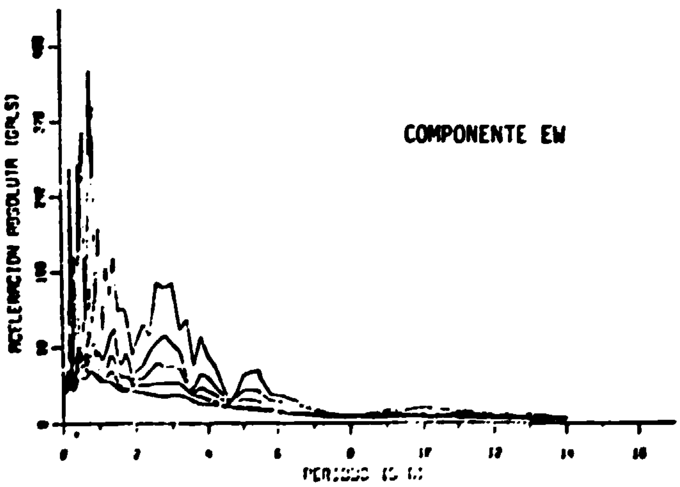
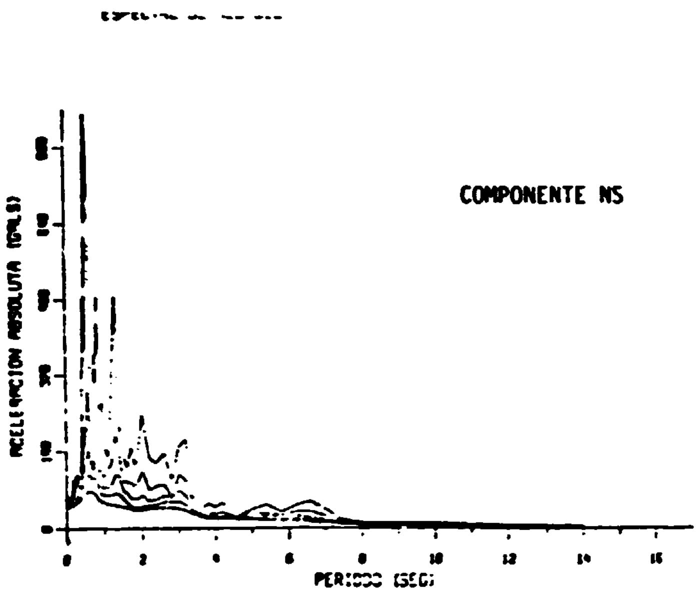


Fig 14. Espectros de respuesta para las componentes NS y EW del registro obtenido en el Observatorio Sismológico de Tacubaya en México DF el 190985.

Fig.RP.9

25.



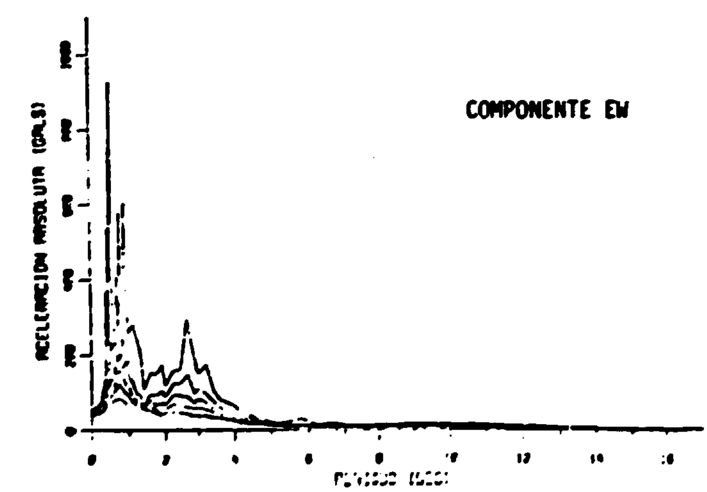
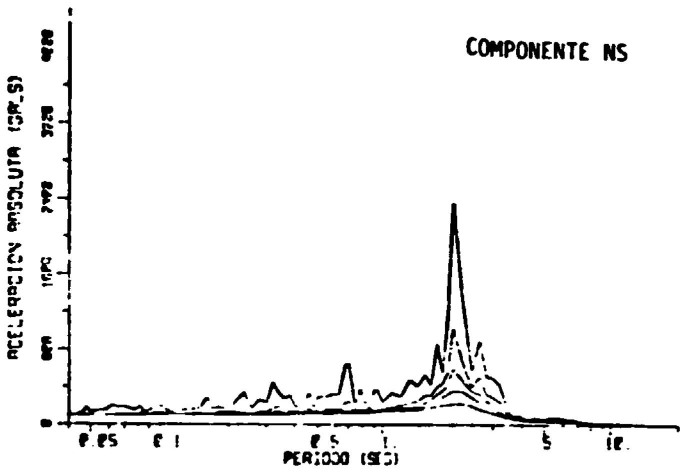


Fig RP 10

Fig 13. Espectros de respuesta para las componentes NS y EW del registro obtenido en los Viveros de Coyoacán en México DF el 190985.



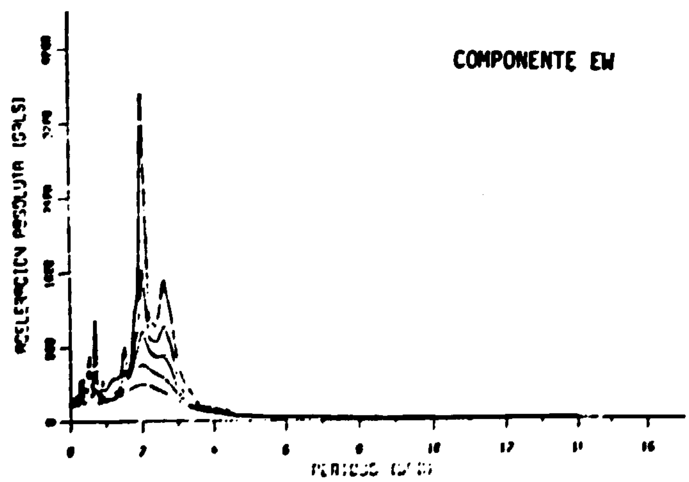


Fig.RP. 11

Fig 10. Espectros de respuesta para las componentes NS y EW del registro obtenido en el Centro SCOP de la SCT en México DF el 190985.

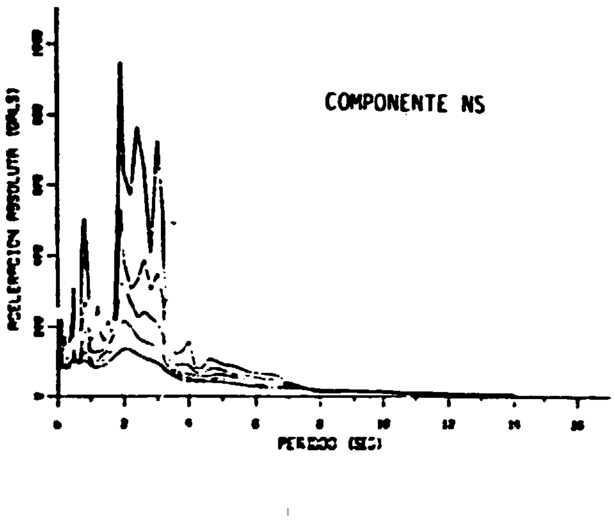


Fig 12. Espectros de respuesta para las componentes NS y EW del registro obtenido frente al frigorífico de la Central de Abastos en México DF el 190985.

FigRP.12

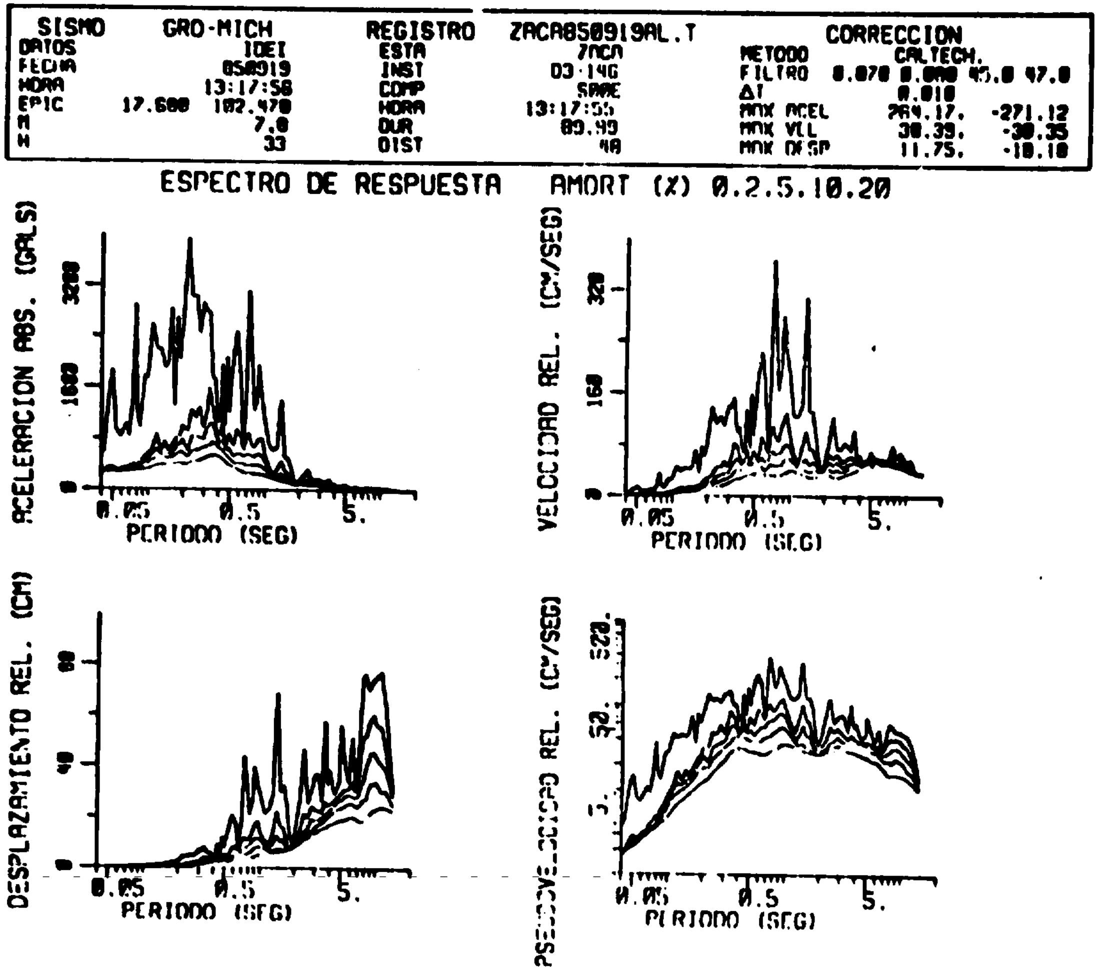


Fig 10. Espectros de respuesta dela componente N-S del acelerograma ZACATULA-19 sept 85.

Fig. R.P.13

22

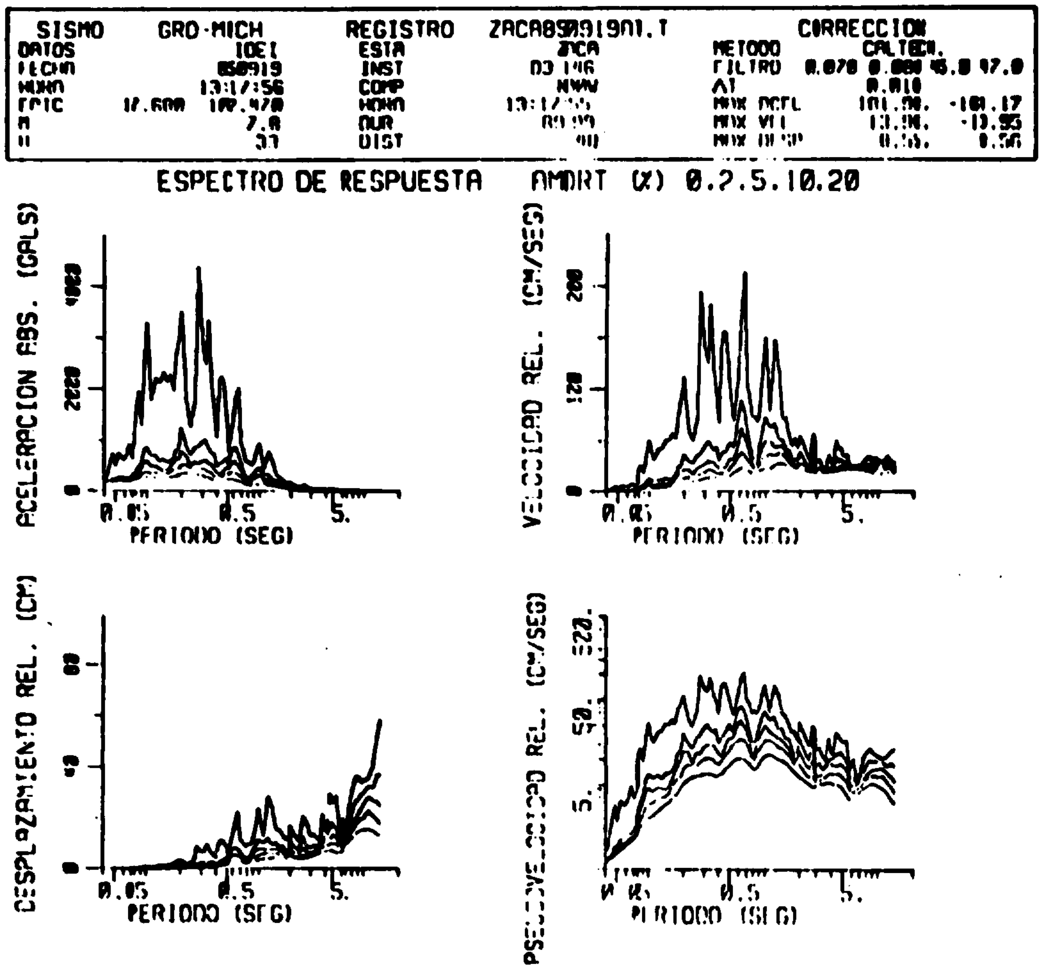


Fig 11. Espectros de respuesta de la componente E-W del acelerograma ZACATULA-19 sept 85.

Fig.RP.14

ANNEX I

JOB DESCRIPTION

SI/MEX/85/804/11-52/32.1.K.

Post title Consultant in earthquake engineering

Duration Two weeks

Date required Late November/early December 1985

Duty station Mexico City with travel outside as required

Purpose of project Technical advise in aseismic contruction strengthening and repair of buildings

Duties

The consultant will be a member of an international team consisting of earthquake specialists who are assigend to Mexico City to assist the Government and construction industry in strengthening their capacity and capability in earthquake engineering. The duties of the group and individual members are summarized as follows:

- Study the behaviour and situation of various types of buildings (fully damaged, less damaged, and well maintained), classify the types of buildings such as brick masonry, reinforced concrete steel structures, prefabricated buildings, framed or wall type buildings, etc.

- Study the soil conditions and types of foundations of various buildings and assess to what extent the soil and foundations caused the recent earthquake catastrophe.

- Advise the Mexican engineering institute (Instituto de Ingeniera de la UNAM) on the ways and means of design, methodology and construction of buildings resistant against earthquakes.

- Consult and advise in developing methods and techniques for strengthening and repair of the damaged buildings, also demolishing unstable buildings.

..../..

- consult and advise for strengthening of small undamaged buildings in rural areas for protection against future earthquakes.

- Review the existing regulations and standards for design and construction of buildings resistant against seismic forces, advise if improving the existing regulations is found necessary.

Above mentioned duties will be carried out by means of visiting fully and less damaged buildings in the city and also outside, having meetings, discussions, visiting the research institutes, inspecting the research activities , giving lectures and having exchange of technical information with members of the team and local specialists, etc.

The expert will also be expected to prepare a final report, setting out the findings of his consultancy mission and also a set of recommendations to the Government on further action which might be taken.

Qualifications: Civil/structural engineer with high university degree and a minimum of 15 years' experience in earthquake engineering, research, design and construction of buildings.

Language: English

Background information: Earthquakes have had catastrophic consequences because structures have not been properly designed and many safety parameters have not been taken into consideration both during the design and the construction stages.

It is essential for countries situated in earthquake zones to consider this facts very seriously and improve their construction industry in order to minimize the natural disaster consequences.

Following the earthquake of 19 and 20 September 1985 in Mexico which destroyed a large number of buildings in Mexico City and many other provinces, the Government has urgently requested UNIDO to share the experience gained in UNIDO's previous projects dealing with earthquakes in other countries and regions.

Since the early 1970s' UNIDO has experience in assisting the Balkan states in projects dealing with the construction and rehabilitation of buildings after earthquakes. It is considered highly desirable for UNIDO to introduce and exchange information with the Mexican Authorities on UNIDO's assistance in this area.

PERSONS CONTACTED DURING MISSION

UNDP :	Mr.G.Silva A., Resident Representative
UNIDO :	Mr.J.Ayza, Alto Asesor Industrial Extrasede Ms.E.Schumacher.
UNAM/I de I:	Mr. L.Esteva M, Director Mr. R.Meli P., Subdirector Mr. E.Rosenblueth D. Mr. E.Miranda M. Mr. M.Chavez G. Mr. F.Sanchez - Selma Mr. G.Ayala Mr. M.Romo O. Mr. J.Prince Ms. S.Ruiz Mr. M.Mendoza Mr. R.Quaas W. Mr. E.Mena S. Mr. X.Chen
CNIC:	Mr.H.Esqueda H., Technical Director
Univ.of California, Bekeley :	Mr. V.Bertero Mr.R.Clough
Embassy of Romania :	Mr. C.Babalau, Ambassador.

DIARY

Saturday, 4 January :	Departure from Bucharest, flight RO 237 to Madrid.
Sunday, 5 January :	Arrival to Mexico City, flight IB 971.
Monday, 6 January :	Meeting at the Embassy of Romania, with Mr.C.Babalau. Meeting at UNIDO/Mexico with Mr.J.Ayza and Mrs. E.Schumacher. Meetings at UNAM /II with Mr.L.Esteva and Mr.R.Meli,drafting of mission program.
Tuesday, 7 January :	Meeting at UNIDO /Mexico with Mr.J. Ayza and Mrs.E.Schumacher, Field visit with Mr.E.Miranda at the area with damaged modern buildings (Paseo de la Reforma - Av.Chapultepec - C. Durango - C.Zacatecas).
Wednesday, 8 January :	Meeting at UNIDO/Mexico with Mr.G.Silva A., Mr.J.Ayza and Mrs.E. Schumacher. Field visit at the area with damaged older and modern buildings (C.Donceles - Av.Hidalgo - Av.Juarez - V.Carranza).
Thursday, 9 January :	Field visit with Mr.E.Miranda (Av.Rio de la Loza - Fray Servando Teresa de Mier, C.Chimalpopoca - C.Xocongo).
Friday. 10 January :	Meetings at UNAM with Mr.E.Rosenblueth Mr.X.Chen and Mr.M.Chavez. First lecture ("Probability Based Criteria for Checking Earthquake Resistant Structures").

Saturday, 11 January, Sunday, 12 January :	Free. Sightseeing, Mexico City, Teotihuacan.
Monday, 13 January :	Meeting at UNIDO /Mexico with Mr.J.Ayza. Meeting at UNAM/II with Mr.J.Ayza and Mr. L.Esteva. Meetings at UNAM/II with : Messrs.V.Bertero andR.Clough;Mr.Meli. Visit to Laboratory of Dynamics. Meetings at UNAM/II with: Mr.F.Sanchez - Selma ;Mr. G. Ayala.
Tuesday, 14 January :	Meetings at UNAM /II with :Mr.M.Romo; Mr.J.Prince ;Ms. S.Ruiz. Meeting at CNIC with Mr.H.Esqueda.
Wednesday, 15 January :	Second lecture ("Vulnerability and Risk Analysis for Individual Structures and Systems"). Meeting at UNAM/II with Mr.M.Mendoza.
Thursday, 16 January :	Meeting at UNAM/II with Mr.R.Quaas; Mr.E.Mena. Closing meeting with Mr.L.Esteva, Mr.E.Rosenblueth, Mr.R.Meli, Mr.M. Chavez, Mr.M.Romo, Ms.S.Ruiz. Presentation of first findings and suggestions in relation to the mission. Meetings with :Mr. E. Ronseblueth; Mr.M.Chavez.
Friday, 17 January :	Final Meeting at UNDP/Mexico with Mr.G.Silva and Mr.J.Ayza. Summary of mission. Meeting at the Embassy of Romania with Mr.C.Babalau.
Saturday, 18 January :	Free. Sightseeing. Departure from Mexico City, flight LH 481 , to Frankfurt.
Sunday, 19 January :	Departure from Frankfurt, flight LH 354 to Budapest.
Monday, 20 January :	Arrival at Bucharest, flight RO. 234

ANNEX IV

TECHNICAL DOCUMENTATION TRANSMITTED TO UNAM/INSTITUTO DE INGENIERIA AND TO CAMARA NACIONAL DE LA INDUSTRIA DE LA CONSTRUCCION (List)

I. UNAM/INSTITUTO DE INGENIERIA

A. In English

1. N.Ignatiev :Repair and Strengthening of Reinforced Concrete Structures. 12-th EAEE Regional Seminar on Earthquake Engineering. Halkidiki, Greece, 1985.

2. V.Kalevras :Seismic Evaluation of Existing Reinforced Concrete Structures. Ibid.

3. H.Sandi :The Decision on the Intervention on Existing Structures: Alternatives and Cost-Benefit Analysis. Proc.4-th International Conf. of Applications of Statistics and Probabilities in Soil and Structural Engineering (ICASP-4), Firenze,1983.

4. H.Sandi: Analysis of Seismic Risk for Geographically Spread Systems. Proc. International Conf.on Structural Safety and Reliability (ICOSSAR' 85), Kobe, 1985.

5. H.Sandi :Some Technical Aspects of Technical Preparedness. Notes on the Romanian Experience. Proc. International Seminar "Learning from Earthquakes"(to be published by UNDRO, Geneva). Perugia, 1985.

6. H.Sandi : Engineering Aspects and Possible Refinements of the Concept of Seismic Intensity, 12-th EAEE Regional Seminar on Earthquake Engineering, Halkidiki, Greece, 1985.

(with Appendix in Romanian concerning the processing of some instrumental data obtained in Mexico City during the 19 September 1985 earthquake).

7. H.Sandi : Probability Based Criteria for Checking Earthquake Resistant Structures. Ibid.

8. H. Sandi (coordinator) : Report to the 8-th European Conference on Earthquake Engineering (Lisbon,1986) on "Vulnerability and Risk Analysis for Individual Structures and Systems" (draft). EAEE Working Group 5, 1985.

B. In Romanian

1. St.Balan, V.Cristescu, I.Cornea (editors) : The Romania Earthquake of 4 March 1977. Editura Academiei. Bucharest, 1983.
2. H.Sandi :Elements of Structural Dynamics. Editura Tehnica, Bucharest, 1983.
3. Romanian National Committee on Earthquake Engineering :Proc. Symp. on Structural Safety of Residential Buildings ;Evaluation of the Level of Earthquake Protection and Proposals for Mitigation of Risk Affecting the Existing Building Stock, Ploiesti, 1984. Published in Constructii 3, 1985.
4. ICCPDC (Central Institute for Research, Design and Guidance in Civil Engineering) : Code for Earthquake Resistant Design of Residential, Social, Agricultural Buildings and Industrial Structures, P-100-81. Collection of Codes and Instructions, Bucharest, 1982.

II. CNIC

The same as I.A.5 above

ANNEX V

TECHNICAL DOCUMENTATION RECEIVED FROM
UNIDO/MEXICO, UNAM/INSTITUTO DE INGENIERIA AND
CAMARA NACIONAL DE INGENIERIA DE LA CONSTRUCCION (List)

I. UNIDO/MEXICO

1. E.Csorba, H.O.Landa (UNDP Consultants)	Report on the Mission to Mexico 14 November - 15 December 1985

II. UNAM/INSTITUTO DE INGENIERIA

A. Reports

1. UNAM/II	El temblor del 19 de septiembre de 1985 y sus efectos en las construcciones de la Ciudad de Mexico. Informe preliminar del Instituto de Ingenieria de la Universidad Nacional Autonoma de Mexico, el 30 de Septiembre de 1985.
2. UNAM/II	Efectos de los sismos de septiembre de 1985 en las construcciones de la Ciudad de Mexico. Segundo informe del Instituto de Ingenieria de la Universidad Nacional Autonoma de Mexico. Noviembre de 1985.
3. x x x	Modificaciones de emergencia al reglamento de construcciones para el Distrito Federal.
4. J.Prince et al.	Acelerogramas en Ciudad Universitaria del sismo del 19 de septiembre de 1985. Informe IPC-10 A, Septiembre 20, 1985. Instrumentacion Seismica, Instituto de Ingenieria, UNAM.
5. E.Mena et al.	Acelerograma en el centro SCOP de la Secretaria de comunicaciones y transportes. Sismo del 19 de septiembre de 1985. Informe IPS-10 B, Septiembre 21, 1985. Instrumentacion ... (etc).
6. R.Quaas et al.	Los dos acelerogramas del sismo del 19 de septiembre de 1985, obtenidos en la Central de abastos en Mexico, D.F. Informe IPS-10 C.Septiembre 23, 1985. Instrumentacion ... (etc).

7. J.Prince et al. — Espectros de las componentes horizontales registradas por los dos acelerografos digitales de Mexico D.F.Sismo del 19 de septiembre de 1985. Acelerogramas en Viveros y en Tacubaya. Informe IPS-10 D. Octubre 1,1985 Instrumentacion ... (etc.).

3. E.Mena et al. — Analisis del acelerograma "Zacatula" del sismo del 19 de septiembre de 1985. Proyecto 5741. Informe IPS-lo E.Octubre, 1985. Instrumentacion ... (etc.).

9. The Japan International Cooperation Agency Mission — Recommendations on Damage Evaluation, Repair and Strengthening for Buildings Damaged by the September 19-2o, 1985 Mexico Earthquakes. Report Submitted to the Department of Federal District of Mexico by the Japan International Cooperation Agency Mission Dispatched to Mexico from October 19 to November 22, 1985.

B. Papers

1. E.Rosenblueth — On the Processing of Doubtful Information :Part 1 :General Theory. Paper No.5, Aug.1983, Inst.for Risk Research, Univ. of Waterloo, Ontario, Canada.

2. E.Rosenblueth, C.Ferregut — On the Processing of Doubtful Information Part. 2 :Gaussian Distributions Paper No.6, Dec.1983, Inst. for Risk Research, Univ. of Waterloo, Ontario, Canada.

3. E.Rosenblueth — Use of Statistical Data in Assessing Local Seismicity. Inst. de Ingenieria, UNAM .

4. E.Rosenblueth Karmeshu Han P.H. — Maximum Entropy and Discretization of Probability Distributions. Inst. de Ingenieria, UNAM.

5. F.J.Sanchez-Sesma, J.A.Esquivel — Ground Motion on Alluvial Valleys under Incident Plane SH Waves Bull SSA, Vol.69, No.4, Aug.1979.

6. F.J.Sanchez-Sesma — Diffraction of Elastic Waves by 3-D Surface Irregularities. Bull.SSA,Vol.73,No.6,Dec.1985.

7. F.J.Sanchez-Sesma — Diffraction of Elastic SH Waves by Wedges. Bull. SSA, Vol.75, No.5, Oct.1985.

8. M.Chavez, D. de Leon — Reliability of Nonlinear Systems with Uncertain Parameters and Random Seismic Excitation. Proc. 8WCEE, San Francisco,1984.

9. M.Chavez, M.P.Romo, M.Arroyo — Probabilistic Site Dependent Design Spectra for an NPP. Inst. de Ingenieria, UNAM.

C. Maps

1. UNAM /I de I — Maps of the Ex-Lakes of Mexico, D.F.

D. Records

1. UNAM/I de I — Digital Strong Motion Records of the 19 September 1985 Event.

III. CAMARA NACIONAL DE LA INDUSTRIA DE LA CONSTRUCCION

1. CNIC — Construccion. No.364, Febrero de 1985.

2. ICIC — Que es ICIC.

ANNEX VI

ADDITIONAL REFERENCES

1.	C.Lomnitz, E.Rosenblueth (editors)	Seismic Risk and Engineering Decision. Elsevier, Amsterdam-Oxford-New York, 1976.
2.	S.K.Singh, M.Rodriguez, J.M.Espinola	A Catalog of Shallow Earthquakes from 1900 to 1981. Inst. de Geofisica, UNAM, Mexico City.
3.	J.Tonda	Articles on the earthquake of 19 September 1985. El Dia, Mexico City, 9,10 and 11 December 1985.
4.	EERI Committee on the Anticipated Tokai Earthquake	The Anticipated Tokai Earthquake :Japanese Prediction and Preparedness Activities. EERI Publication No.84-05, June 1984.
5.	IAEE	Earthquake Resistant Regulations of the World. Tokyo, 1984.
6.	UNDRO	Natural Disasters and Vulnerability Analysis. Report of Expert Group Meeting UNDRO, Geneva, 1979.
7.	UNDP/UNESCO/UNDRO Project RER/79/014 "Earthquake Risk Reduction in the Balkan Region"	Final Reports of Working Groups A,B,C,D,E. UNESCO,Paris,1984.
8.	UNDP/UNIDO Project RER/79/015 "Building Construction under Seismic Conditions in the Balkan Region"	Manuals, Volumes 1,2,3,4,5,6,7. UNIDO,Vienna,1984.