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A WORKSHOP ON "REDUCING POTENTIAL LOSSES FROM EARTHQUAKE HAZARDS IN PUERTO RICO"

May 30-31, 1985

Dorado, Puerto Rico

SPONSORED BY

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FOREWARD

The September 19, 1985, Mexico earthquake reminds us of the potential damage, injury, and loss of life which can occur in major population centers located in active seismic zones throughout the world. Due of the large increase in population and investment in capital stock, Puerto Rico today could experience devastating human and economic impacts if a major earthquake were to occur.

The San Juan Metropolitan Area, which is the capital city and the major urban settlement of Puerto Rico, generates 50% of the Island's output. In 1983 the estimated total output was about 18 billion dollars of which nine billion dollars originated directly or indirectly in San Juan.

The sectors with the largest proportion of San Juan operations tend to be those related to service sectors. Manufacturing activity is more evenly distributed all over the Island of which the most capital intensive sectors are chemicals and cement. Obviously the sector with the largest capital investment is the real estate sector.

According to preliminary results of a study by Fernando Zalacain (University of Puerto Rico at Rio Piedras) of the estimation of economic damage in the event of a large earthquake (Modified Mercalli VIII), the sectors in which the damage could be greatest will be the following:

Capital Stock Affected (Millions of $)

Housing	$980.00
Power Plants	240.00
Government	205.00
Business/trade	145.00
Telephone service	118.00
Ports and Facilities	89.00

Total economic impact to capital stock only could be in the range of 2.15 billion dollars. This level of destruction represents 13% of total capital stock calculated for San Juan. Taking in consideration that total construction investments in the Puerto Rico economy accounted to 1.2 billion dollars in 1981 and that machinery and equipment investment implied another 729 million, the estimated damage would be much larger than the total investment by the private and public sector in a typical year like 1981.

The level of damage to capital stock will reduce losses in output, income and employment. The level of losses will also depend on the length of the reconstruction period. The preliminary estimates for two different scenarios, project output losses in the range of 345 to 525 million dollars. Employment losses could be in the order of 10,000 to 15,000 man-year lost as result of the earthquake.

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BACKGROUND AND SUMMARY OF THE WORKSHOP ON "REDUCING POTENTIAL LOSSES FROM EARTHQUAKE HAZARDS IN PUERTO RICO"

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INTRODUCTION

One hundred earth scientists, social scientists, engineers, architects, urban planners, and emergency management specialists met in Dorado, Puerto Rico, on May 30-31, 1985, to update their knowledge of earthquake hazards and potential risk in the Puerto Rico region, to review progress of current studies, and to formulate action plans to reduce potential losses from future earthquakes that will recur in the region. The workshop continued the work of a similar workshop on geologic hazards which was held in San Juan, Puerto Rico, in the spring of 1984 (Hays and Gori, 1984) and accelerated the dissemination of knowledge gained from the March 1985 Chile earthquake.

This workshop, the 30th in a series of workshops and conferences sponsored since 1977 by the U.S. Geological Survey (USGS) under the auspices of the Natural Earthquake Hazards Reduction Program (NEHRP), was cosponsored by the Federal Emergency Management Agency (FEMA), the Department of Natural Resources of Puerto Rico, the Puerto Rico Planning Board, and the Geological Survey of Puerto Rico. The workshop was timed to coincide with the completion of the early phases of an earthquake preparedness and planning program sponsored by the Federal Emergency Management Agency (FEMA) and managed by the Department of Natural Resources of Puerto Rico. The opportunity to contribute to vulnerability studies as a part of this important program had been identified earlier in the 1984 workshop held in San Juan, Puerto Rico, and had been advocated throughout the year. The 1985 workshop focused on ways to enhance earthquake vulnerability studies (see Figure 1).

THE 1985 CHILE EARTHQUAKE

The large earthquake (M_s = 7.8) that occurred near Valparaiso, Chile, on March 3, 1985, provided an unique opportunity to increase the public°s awareness of earthquake hazards in Puerto Rico and to transfer relevant information to

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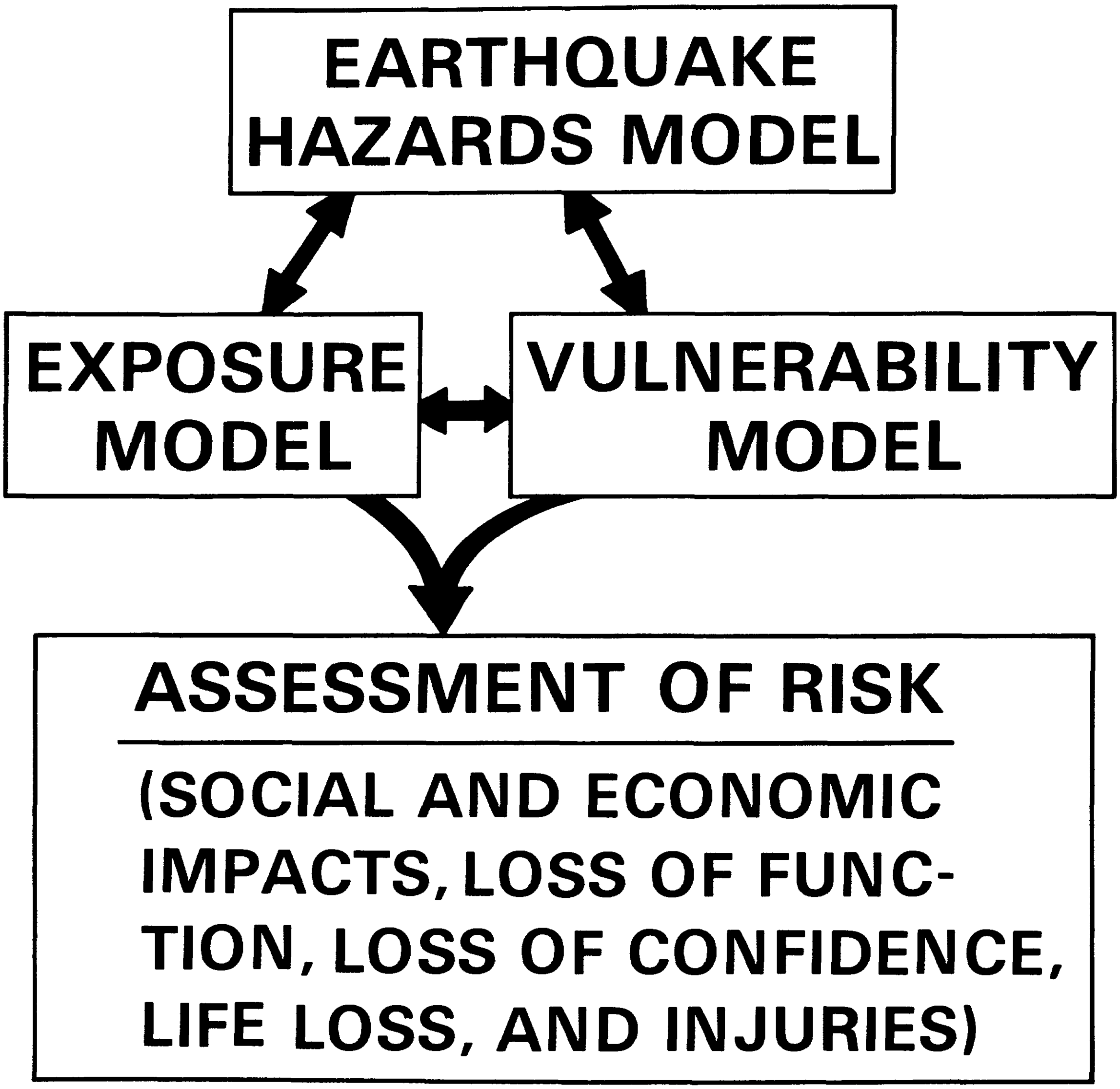

Figure 1.--Schematic illustration of the earthquake hazards, exposure, and vulnerability models which must be developed in order to assess the risk in an urban area.

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those conducting the Puerto Rican earthquake preparedness and planning program.

Consequently, two strategies were devised to enhance the value of the workshop to Puerto Rican engineers, scientists, planners, emergency managers, and public officials. They were:

1) Scheduling the workshop to follow immediately after the seminar, "Fundamentals of Earthquake Engineeing in Puerto Rico," conducted by the Earthquake Engineering Research Institute (EERI) under the sponsorship of the National Science Foundation. The seminar was held in Mayaguez on May 27-29. More than 100 engineers, scientists, and planners participated in the seminar. Technical information on the 1985 Chile earthquake was integrated on the third day into a special session on the state-of-the-art of earthquake engineering in Latin America and presented by experts from Chile, Costa Rica, Columbia, Peru, Guatemala, Ecuador, Dominican Republic, and El Salvador.

2. Inviting two Chilean engineers, Dr. Rodolfo Saragoni and Dr. Mauricio Sarazin, to participate in both the seminar and workshop, presenting information on the 1985 Chile earthquake. (Their participation was sponsored by the National Science Foundation and the U.S. Office of Foreign Disaster Assistance.)

The experience and information provided by the 1985 Chile earthquake are very relevant to three regions of the United States: Puerto Rico, the Puget Sound area, Washington, and Southern Alaska. Similar effects as those in the Chile earthquake could happen in each of these three regions. All four regions have a similar tectonic setting, namely a subduction zone where one tectonic plate is sliding at the rate of several inches per year beneath another tectonic plate (see Figure 2). The world's greatest earthquakes (e.g., 1960 Chile earthquake ($M_w = 9.5$) and 1964 Prince William Sound, Alaska, earthquake ($M_w = 9.2$)) have occurred in subduction zones. The 1960 and 1985 Chile earthquakes were caused by subduction of the Nazca tectonic plate beneath the South American plate. The 1985 earthquake caused 176 deaths, 2500 injuries, and economic losses from architectural and structural damage to buildings and

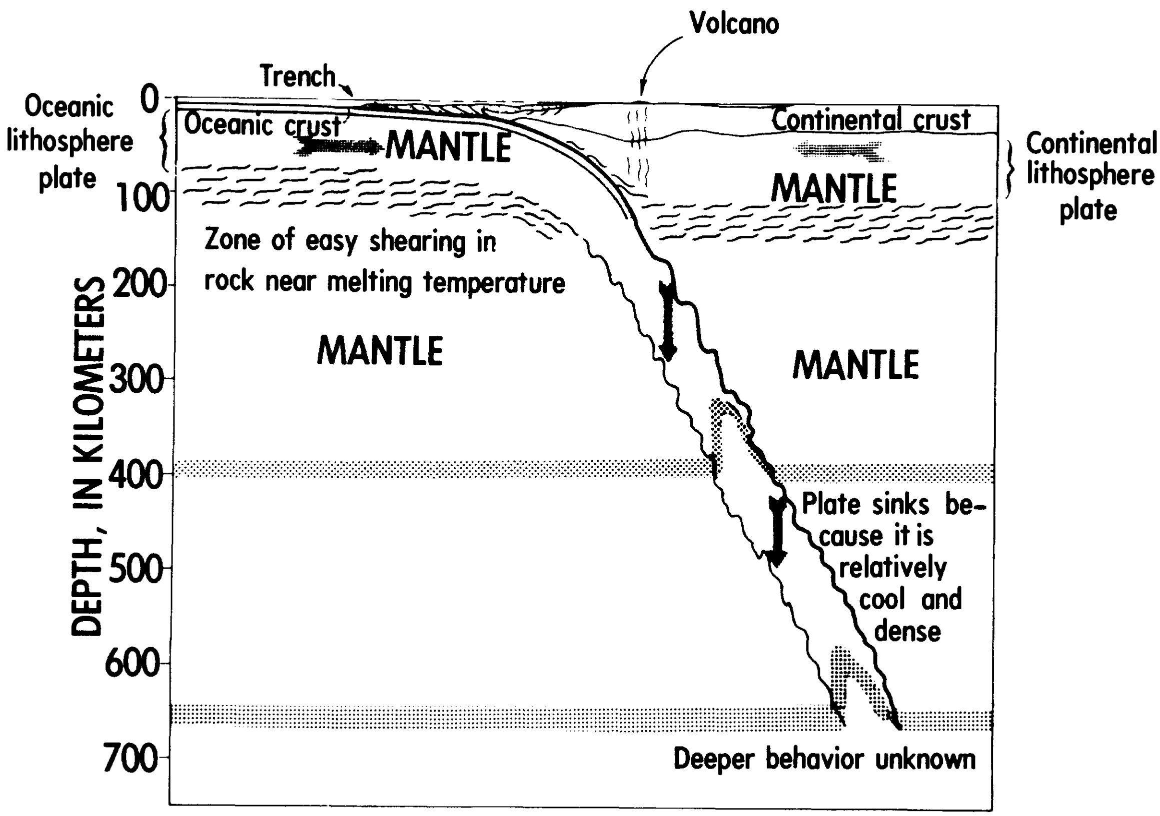

Figure 2.--Schematic illustration of a subduction zone. A subduction zone is a dipping planar zone descending away from a trench that is typically marked by high seismicity. The sinking oceanic place may be strongly coupled along part of its boundary with the overriding continental plate. The rate of movement typically ranges from a fraction of an inch to about 5 inches per year. Earthquakes occur when one plate slips relative to the other. The worlds greatest earthquakes have occurred in subduction zones (e.g., 1960 Chile, 1964 Prince William Sound, Alaska, 1985 Mexico).

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lifelines adding to about $2 billion. Unreinforced masonry and adobe buildings sustained the greatest damage from ground shaking. Although, well-engineered buildings generally performed well, a hospital suffered extensive damage, indicating the need for stringent earthquake-resistant design criteria for critical facilities and tough inspection standards and enforcement procedures.

An unpresidented set of 30 strong motion accelerograms (each having 3 components) documented the ground shaking in the 1985 Chile earthquake. The significant facts were: 1) ground shaking reached levels of 0.85 g. (horizontal) and 0.65 g (vertical), 2) both high and low ground-shaking frequencies were recorded, and 3) the duration of shaking was long (60-80 seconds). Other than in Japan, these ground motion data are the first comprehensive sample from a subduction zone earthquake; they are essential for probabilistic ground shaking hazard assessments and other applications that require a seismic wave attenuation function with specification of the dispersion.

The 1985 Chile earthquake also caused physical effects such as the following:

1. Numerous landslides occurred in the coastal mountains, locally blocking roads.
2. Liquefaction occurred in saturated beach sands.
3. Ground cracks were common in the epicentral area.
4. Part of the coastline subsided.
5. A small local tsunami having wave heights of 3.6 feet at Valparaiso, Chile, was generated. This tsunami caused wave runups of 1.7 feet in Hilo, Hawaii, and 0.2 feet in Seward, Alaska.
6. The extensive aftershock sequence that followed the mainshock included a M_s 6.6 earthquake on March 17, and a M_s 6.3 earthquake on March 19.

THE 1985 MEXICO EARTHQUAKE

Just before this report went to press, a great earthquake occurred in Mexico on September 19, 1985. This earthquake was the most devastating earthquake of the past decade in North America. Because it was also a subduction zone earthquake having relevance for Puerto Rico (as well as Puget Sound and Alaska), its effects are summarized below for completeness.

The great 1985 Mexico earthquake, initially rated as $M_s = 7.8$ but later upgraded to $M_s = 8.1$, occurred in the Mexico trench subduction zone where the Cocos tectonic plate is being subducted beneath the North American plate. The existence of a possible seismic gap in this portion of the Cocos plate and a general forecast of a large earthquake having an average recurrence interval of about 35 years had been made in 1981 by McNally. The specific time of the earthquake had not been specified, however. This earthquake was noteworthy because about 400 5-20 story buildings located in Mexico City, about 250 miles from the epicenter, collapsed partially or totally, causing an estimated 5,000-10,000 deaths, numerous injuries, and economic losses of possibly $5-10 billion. The extraordinarily high degree of damage at this large epicentral distance was mainly due to amplification of the long period ground motion by the 50 meter thick, water-saturated ancient lake bed under part of Mexico City (see Figure 3). The lake beds were recognized in 1964 by Zeevaert as having a characteristic site period of about 2 seconds, the natural period of vibration of a typical 20-story building. Past distant earthquakes (e.g., 1957 and 1962 Mexico earthquakes) had also caused damage in Mexico City that was attributed to site amplification. In the 1985 earthquake, six buildings collapsed at the Mexico General Hospital; about 400 doctors, nurses, and patients were trapped in the ruins of the Juarez hospital, just 8 blocks from the Presidential Palace. Government buildings, as a group, sustained considerable damage. Long distance telecommunications with the rest of the world were interrupted for several days after the earthquake due to the destruction of the main microwave transmitter and the lack of a redundant, backup system. Because of prior planning by US and Mexican scientists and engineers, a number of strong motion accelerographs were in place in the epicentral area at the time of the earthquake and recorded ground motions in the order of 0.20g, a low value for a great earthquake. These strong motion data, together with the data acquired in the March 3, 1985 Chile earthquake provided an unprecedented strong-ground motion data sample for subduction zone earthquakes. A building code as strict as any adopted in the United States had been adopted and implemented in Mexico City since 1976. It included a factor for soil conditions.

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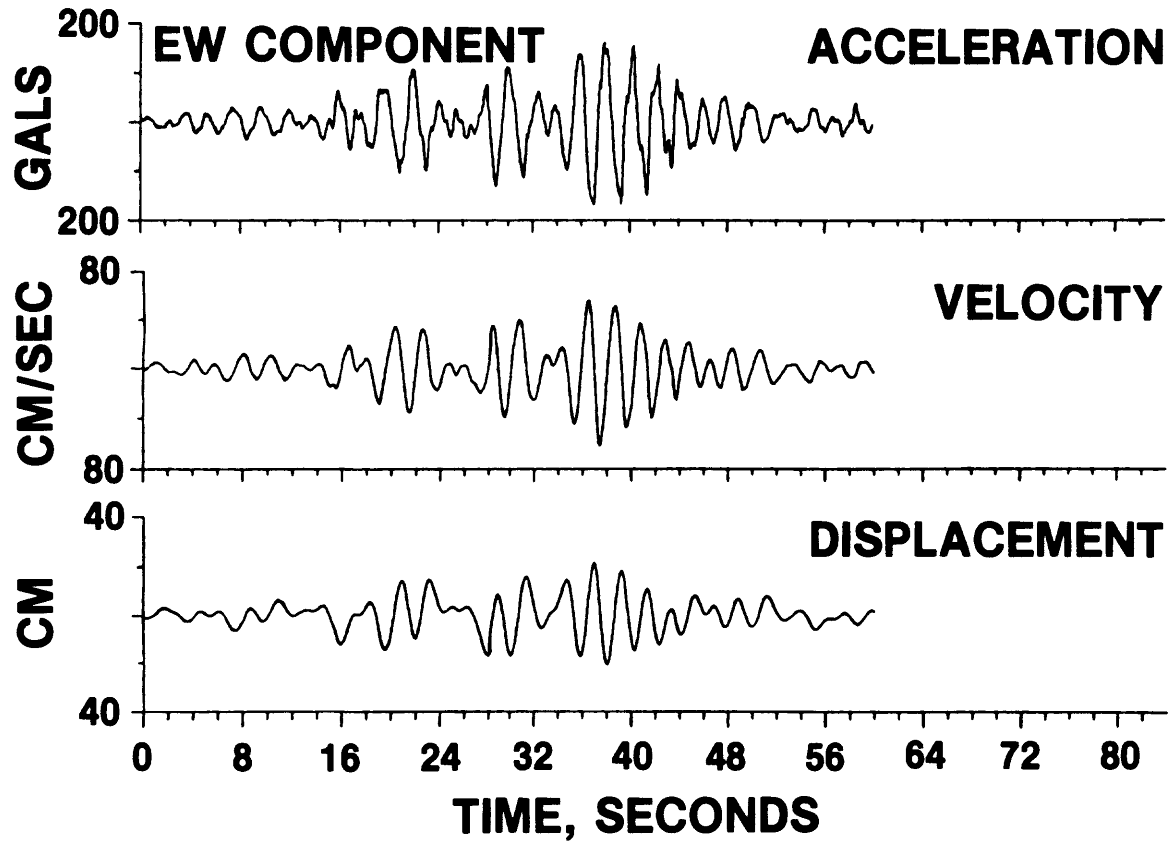

Figure 3.--Accelerogram (top) recorded at a free field location on the surface of the 50-meter thick lake beds forming the foundation in parts of Mexico City. The epicenter of the September 19, 1985 Mexico earthquake was located some 400 km to the west. The strong 2 second period energy in the accelerogram and the velocity (middle) and displacement (bottom) time histories derived from it are a consequence of the filtering effect of the lake beds which amplified the ground motion, (relative to adjacent sites underlain by firmer rock-like materials) about a factor of 5. The coincidence of the dominant period of ground shaking (2 seconds) with the fundamental period of vibration of tall buildings contributed to their collapse. These records were provided by the Universidad Nacional Autonoma de Mexico.

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The 1985 MAMEYES LANDSLIDE DISASTER

A landslide disaster occurred in the Mameyes district near Ponce, Puerto Rico, on Monday, October 7, 1985, during a rainstorm of record intensity for the area.

Because of the relevance of this experience to the subjects of the 1984 and 1985 USGS/FEMA workshops, the basic facts are included in this report.

The disaster was caused by a block slide, the movement of a slab of soil and rock 30 to 50 feet thick by shear displacement along a bedding surface. The rock slab failed in three stages. Stage 1 began between 3:00 and 3:30 a.m. on October 7. Stage 2 followed 15-30 minutes later, and stage 3 occurred about 5 minutes after stage 2. The slide carried most of the Mameyes hillside residential community (population of about 1,500) into the canyon below and covered part of it with debris 40 to 60 feet thick. One hundred nineteen homes were destroyed and about 130 people were killed. The death toll is the largest ever from a single landslide in the United States. Normally 25 to 30 people are killed each year in the United States from landslides.

Several factors combined to cause the disaster in Mameyes:

1) The nature of the soil and underlying bedrock in the area--beds of chalk with clay partings lie approximately parallel to the hillslope, both dipping approximately 20 degrees to the south into an east-flowing canyon.

2) The heavy rainfall prior to the landslide--nearly 20 inches of rain fell between October 5 and October 7. This amount of rain is not unusual--prior hurricanes such as Donna in 1960 and Eloise in 1975 have produced this much rain--but the intensity was exceptional for the area. The rain probably elevated the pore pressures near the base of the slab of soil and rock and provided the principal trigger for the failure.

3) Local manmade conditions--water leaking from broken water mains and seepage from local sewage disposal facilities may have contributed to slope saturation prior to the failure, and combined with the heavy rain, triggered the failure.

At the request of Governor Rafaêl Hernandez Colón, President Reagan declared the area a disaster zone. Emergency response activities were initiated immediately to deal with the disaster.

ASPECTS OF AN EARTHQUAKE VULNERABILITY STUDY

An earthquake vulnerability study of an urban area is a complex task. The essential requirements are : 1) to model the earthquake hazards, 2) to superpose the hazards with a model (inventory) of what is at risk, and 3) to determine the damage and losses that are likely to occur. This report provides information on each topic.

Earthquake-Hazards Model (see papers by Hays, McCann, and Bolt)--The earthquake hazards model requires that the best available gelogic, seismological, and geotechnical data be integrated to define the hazards, either deterministically or probabilistically. The objective is to provide answers to the questions:

1) Where have past earthquakes occurred? Where are they occurring now?
2) Why are they occurring?
3) How often do earthquakes of a certain size (magnitude or epicentral intensity) occur?
4) How bad (severe) have the physical effects (hazards) been in the past? How bad could they be?
5) How widely do the physical effects (hazards) vary spatially and temporally?

Exposure Model (see paper by Molinelli)--The determination of what is at risk from each earthquake hazard is a critically important task. An inventory of structures of various types (e.g., buildings, utility and transportation structures, hydraulic structures, earth structures, and special structures) is needed. An accurate inventory is difficult to obtain and to maintain because of the rapid change in capital improvements as a function of space and time.

Damage and Losses (see papers by Hays, Stratta, and Scholl)--Estimation of damage and losses (economic losses, loss of function, loss of confidence, life loss, injuries) is an essential part of an earthquake vulnerability study. This step provides information that can be used to guide research, mitigation, response, and recovery programs. Damage and losses can be estimated in terms of a wide variety of scenarios such as worst case or the recurrence of a specific past earthquake (e.g., the "1918 Puerto Rico earthquake").

Loss-Reduction (see papers by Nigg, Molinelli, and Stratta)--Once reasonable estimates of the damage and losses have been obtained, loss-reduction measures can be devised to meet specific objectives. These measures include: 1) personal preparedness, 2) education, 3) land-use regulation, and 4) engineering design and building codes, and 5) insurance.

THE EARTHQUAKE THREAT IN PUERTO RICO

Puerto Rico is located in a subduction zone where the North American tectonic plate is sliding under the Caribbean tectonic plate. About 4 million Puerto Ricans live, work, and play in a locale surrounded and underlain by active faults, each capable of producing strong (M = 6, 7, or 8) potentially damaging earthquakes. Current scientific knowledge (personal communication with William McCann) indicates that large earthquakes of magnitude 7.5 are expected to recur, on the average, about once every 80 years. Even a moderate size (M = 6) earthquake along some of the faults could cause significant damage, social disruption, and loss of life and injuries throughout the Island. A magnitude 7.5 earthquake occurred offshore Puerto Rico in 1918 causing losses of approximately $4 million (1918 dollars) and at least 116 deaths (Figure 4). In view of the large increase in population and building wealth since 1918, a recurrence of the 1918 earthquake today is thought to be capable of causing direct losses of about $1+ billion and thousands of deaths and injuries, depending on the time of day and whether or not any buildings collapse. The unique nature of the earthquake threat, including landslides, liquefaction, the potential occurrence of tsunamis, and the potential widespread disruption of life in Puerto Rico call for long-term comprehensive

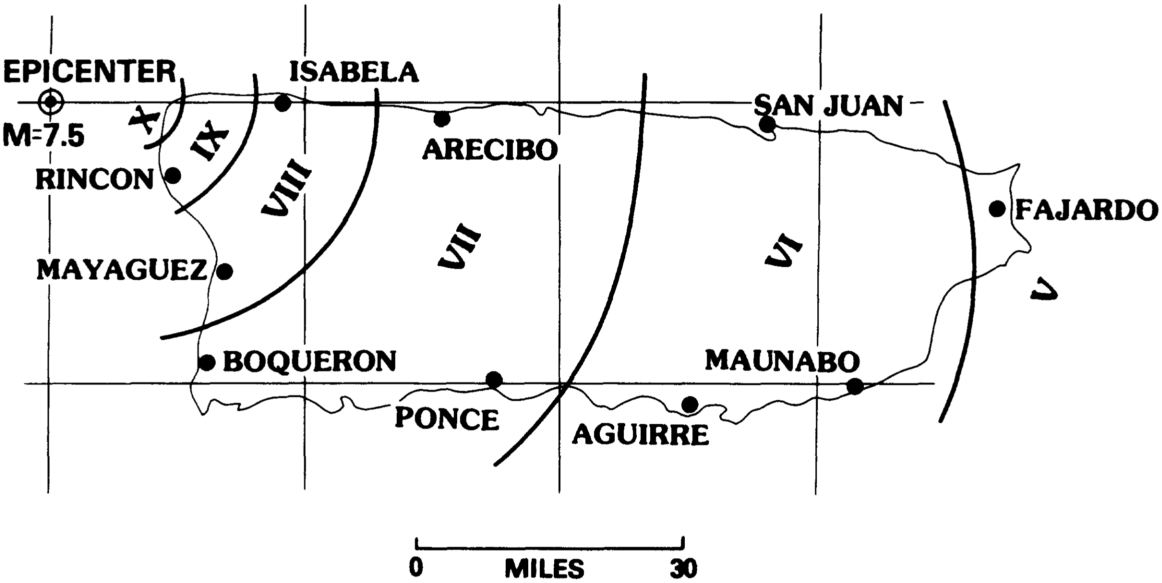

Figure 4.--Isoseismal map of the October 11, 1918, Puerto Rico earthquake. This earthquake affected the entire island and caused $4 million (actual dollars) in losses and at least 116 deaths. It generated a destructive local tsunami. The contours are given in terms of the Rossi-Forel intensity scale. The physical effects for each value of intensity can be estimated from the Modified Mercalli intensity scale (see Appendix B) which is more widely used today than the Rossi-Forel scale. In general, intensities of V - VI affect the contents of a building or facility (e.g., broken china, glassware, etc.), although liquefaction can be triggered if the site geology is favorable. Intensities of VI - VII cause architectural damage (e.g., cracked and fallen plaster, fallen light fixtures and ceilings, overturned water heaters and bookcases, and displaced contents of pantry shelves). An intensity of VIII causes structural damage (e.g., houses shifted on their foundations,ar major cracks to partial collapse in buildings, broken pavements, disrupted utilities, etc.). Intensities of IX - X cause severe structural damage (e.g., total collapse of buildings). Fatalities are largest when buildings collapse. Ground failures (landslides, liquefaction) can occur at intensities ranging from VI - X. Tall buildings may be susceptible to damage from large distant earthquakes if the site geology amplifies the long period ground motion in the range of the natural period of vibration of the building.

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preparedness actions by all levels of government, professionals, volunteer groups, and the private sector.

WORKSHOP PROCEDURES

Following welcoming comments by the Honorable Alejandro Santiago Nieves, Secretary of the Department of Natural Resources, the workshop process began. The overall theme of the workshop was developed in three plenary sessions and two interactive discussion sessions. Three discussion groups were formed after the first and third plenary sessions. The purpose was: 1) to evaluate the progress made since the 1984 workshop, 2) to forge collective goals and action plans, and 3) to devise creative strategies for accelerating progress in critical programs designed to increase the capability of Puerto Rico to reduce potential losses from future earthquakes.

PLENARY SESSIONS

The themes, objectives, and speakers for each plenary session are described below.

Session I: Review of the state-of-the-art in assessing earthquake hazards and mitigating their effects.

Objective: A series of overview-type presentations giving the state-of-the-art for answering the questions: WHERE? WHY? HOW OFTEN? HAT WARE THE PHYSICAL EFFECTS OF GROUND SHAKING, EARTHQUAKE-INDUCED GROUND FAILURE, SURFACE FAULTING, TECTONIC DEFORMATION, AND TSUNAMI WAVE RUNUP? WHAT ARE THE POTENTIAL LOSSES FROM THESE PHYSICAL EFFECTS (HAZARDS)? and WHAT ARE THE OPTIONS FOR MITIGATING THESE LOSSES?

Speakers: Walter Hays
William McCann
Bruce Bolt
Rodolfo Saragoni
James Stratta

Session II: Review of societal and technical lessons learned from recent earthquakes that are applicable for Puerto Rico.

Objective: Presentations describing the societal, scientific, and engineering lessons derived from past world wide earthquakes that are transferrable to Puerto Rico.

Speakers: Joanne Nigg
Roger Scholl

Session III: Current activities in Puerto Rico to reduce potential losses from earthquake hazards.

Objective: Presentations giving the status of important Puerto Rican programs and important results obtained to date.

Speakers: Boris Oxman
Jose Molinelli
Anselmo De Portu
Miquel Santiago

DISCUSSION GROUPS

Three discussion groups were formed to provide the forum for enhanced interaction among the participants. The participants in each group were selected in a way that ensured a good mix of technical and policymaking disciplines. The groups met simultaneously then reported in a plenary session. The moderators of the discussion groups were: 1) Group 1: Walter Hays, Miguel Santiago, and Rafael Jimenez; Group 2: William McCann and Alejandro Soto; Group 3: Paula Gori and Anelsmo DePortu.

Following the first plenary session, the three discussion groups considered the questions:

1. What happened in the 1918 Puerto Rico earthquake (see Figure 4)?

2. If the losses (116 deaths and $4 million) of the 1918 earthquake were scaled to the 1985 population and building wealth in Puerto Rico, would Puerto Ricans find the potential risk acceptable?

3. If the answer to question 2 is "yes," what should Puerto Rico do? What should individuals do?

4. If the answer to question 2 is "no," what should Puerto Rico do. What should individuals do?

These questions prepared the participants for the detailed presentations of Plenary Sessions II and III.

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The three discussion groups met simultaneously again after the third plenary session and addressed the questions:

1. What do we know now about: a) the earthquake and tsunamigenic potential of Puerto Rico, b) the ground-shaking hazards of Puerto Rico, and c) the ground-failure hazards of Puerto Rico?

2. What do we still need to know and what do we need to do in order to accomplish research goals and to foster an implementation process that will reduce potential losses from future earthquakes?

3. What activities should receive the highest priority in the next 3 to 5 years?

The three discussion groups utilized two sets of materials in their deliberations: 1) a questionnaire which called for each research and implementation activity to be ranked on a scale of 1 (lowest) to 5 (highest) and assigned priorities ranging from 1 (highest) to 3 (lowest), and 2) the recommendations made by the participants of the 1984 Puerto Rican workshop on geologic hazards. The questionnaire is repeated for completeness; the 1984 recommendations are contained in Appendix A of this report.

The discussion groups were enriched by the wide variety of backgrounds of the participants. Because some nonscientists and engineers were not familar with the technical terms, a glossary was provided in both English and Spanish (Appendix B) to facilitate communication. The proposed ammendments to the Puerto Rico building code are given in Appendix C. Appendix D gives a list of participants.

LOSS ESTIMATES RELATIVE TO 1918 PUERTO RICO EARTHQUAKE

The participants of the discussion groups concluded that they were not very familiar with the details of the 1918 Puerto Rico earthquake. The concensus was that the 1918 earthquake should be carefully restudied in order to take full advantage of its lessons. Although all of the participants acknowledged

QUESTIONNAIRE I: STATUS OF RESEARCH ON EARTHQUAKE AND TSUNAMIGENIC POTENTIAL IN THE PUERTO RICO REGION

Research topic	Status see definition	Recommended Priority for next 3 to 5 years
A. RESEARCH		
1. Historic seismicity	1 2 3 4 5	1 2 3
2. Current seismicity	1 2 3 4 5	1 2 3
3. Activity of specific faults	1 2 3 4 5	1 2 3
4. Tectonic setting	1 2 3 4 5	1 2 3
5. Seismic gaps	1 2 3 4 5	1 2 3
6. Seismic sources	1 2 3 4 5	1 2 3
7. Earthquake recurrence	1 2 3 4 5	1 2 3
8. Tsunamigenic sources	1 2 3 4 5	1 2 3
B. PRODUCTS		
1. Seismicity maps	1 2 3 4 5	1 2 3
2. Map of seismic source zones	1 2 3 4 5	1 2 3
3. Map of tsunami source zones	1 2 3 4 5	1 2 3
4. Fault activity map	1 2 3 4 5	1 2 3
5. Seismotectonic maps	1 2 3 4 5	1 2 3

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QUESTIONNAIRE II: STATUS OF RESEARCH ON THE GROUND SHAKING HAZARD IN THE PUERTO RICO REGION

Research topic	Status see definition	Recommended Priority for the next 3 years
A. RESEARCH		
1. Seismic source zones	1 2 3 4 5	1 2 3
2. Attenuation laws for acceleration	1 2 3 4 5	1 2 3
3. Attenuation laws for velocity	1 2 3 4 5	1 2 3
4. Attenuation laws for spectral velocity ordinants	1 2 3 4 5	1 2 3
5. Duration	1 2 3 4 5	1 2 3
6. Engineering properties of soil and rock	1 2 3 4 5	1 2 3
7. Local ground response	1 2 3 4 5	1 2 3
B. PRODUCTS		
1. Maps of seismic source zones	1 2 3 4 5	1 2 3
2. Probabilistic maps of ground shaking hazard		
3. Maps of ground shaking hazard for specific scenarios	1 2 3 4 5	1 2 3
4. Maps of seismic risk zones	1 2 3 4 5	1 2 3
5. Engineering properties of surficial deposits	1 2 3 4 5	1 2 3

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QUESTIONNAIRE III: STATUS OF RESEARCH ON THE GROUND-FAILURE HAZARD IN THE PUERTO RICO REGION

Research topic	Status see definition	Recommended Priority for next 3 to 5 years
A. RESEARCH		
1. Liquefaction potential	1 2 3 4 5	1 2 3
2. Landslide susceptibility	1 2 3 4 5	1 2 3
3. Reactivation of old landslides	1 2 3 4 5	1 2 3
4. Characterization of sensitive clay behavior	1 2 3 4 5	1 2 3
5. Characterization of the foundation	1 2 3 4 5	1 2 3
B. PRODUCTS		
1. Regional liquefaction maps	1 2 3 4 5	1 2 3
2. Regional landslide susceptibility maps	1 2 3 4 5	1 2 3
3. Maps of sensitive clay formations	1 2 3 4 5	1 2 3
4. Dams/inundation maps	1 2 3 4 5	1 2 3

the considerable growth of population and building wealth in Puerto Rico since 1918, the ad hoc estimates made in the discussion groups of the potential 1985 losses ranged from $100 million (factor of 25 relative to the 1918 losses) to $2 billion (factor of 500 relative to the 1918 losses). The concensus of the participants was that both estimates of the risk were unacceptable and that a definitive vulnerability study was needed to define as accurately as possible the potential losses and impacts that a recurrence of the 1918 earthquake might cause.

EVALUATION OF PROGRESS SINCE APRIL 1984 (See Appendix A)

The participants of the discussion groups rated the progress since the 1984 Puerto Rico workshop. The reference bench mark was the set of goals that were recommended by the participants of the 1984 workshop. Many of the participants had attended this workshop and, therefore, had taken part in setting the goals. Also some of the participants were in key positions to foster implementation of the loss-reduction goals. Below are the results of a "report card" that the workshop participants spontaneously filed to give their perceptions on the status of recommendations made the previous year, rating the amount of progress on a continum from 0 to 5, with 5 meaning "substantial progress" and 0 meaning "no progress."

Goal I. Status	Mapping of Geologic Hazards
0	a) Probabilistic map of ground shaking.
1.5	b) Mapping landslide susceptible areas.
1	c) Mapping liquefaction susceptible areas.

Goal II. Status	Loss Reduction Measures
0	a) Department of Natural Resources should gather available information to determine cost analysis of hazards in Puerto Rico.
0	b) There should be an exchange of information that is currently available in State agencies.
0	c) Geologic reports are needed for critical facilities.
1	d) Federal Agencies should be aware and concerned about geologic hazards.

Goal III. Status	Information Transfer, Public Awareness, and Community Preparedness
1	a) Develop information banks and campaigns
0	b) Implement evacuation procedures.
0	c) Educational programs:

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1 1) Educate the public
0 2) Educate politicians
2 3) Educate professional engineers

Goal IV. Building Code

Status

0 a) Implement building incorporating new seismic design requirements.

Goal V. Preliminary Vulnerability Study

Status

3 a) Conduct a vulnerability study of the San Juan area.

EVALUATION OF PRIORITIES FOR THE NEXT 3-5 YEARS

The concensus of the participants was that top priority should be given to the following:

1. Hiring a full-time seismologist in Puerto Rico: A seismologist is needed to prepare the research products (e.g., that are needed to foster implementation of loss reduction measures (e.g., maps of seismogenic zones and seismic risk zones, the latter for the building code). The headquarters of the seismologist should probably be in Mayaquez.

2. Deployment of more accelerographs: Arrays of accelerographs to augment the limited number now available are needed in Puerto Rico to acquire strong ground motion data needed to define design levels, seismic wave attenuation laws, and local ground response. These data, lacking now, are needed to construct realistic probabilistic ground shaking hazard maps like those that are now being proposed for the United States (e.g., the 1978 Model Building Code of the Applied Technology Council) and throughout the World (e.g., in Algeria, Hays, 1985). A suggestion was made in the workshop to utilize the concept of a map made for a 50-year exposure time and a 90 percent probability of nonexceedance as the basis for defining the ground shaking hazard throughout the entire Caribbean basin. The goal is to produce a common seismic risk zone map for the building codes of all the countries of the Caribbean Basin.

3. An improved building code. A code such as the one recommended for adoption in the 1984 workshop (see Appendixces A and C) must be adopted in Puerto Rico.

4. The ground failure hazard in Puerto Rico is underrated. The hazard needs to be quantified in a way that can be correleated with the probabilistic ground-shaking hazard maps. The Mameyes disaster could be worsened in an earthquake.

CONCLUSIONS AND RECOMMENDATIONS

At the conclusion of the workshop, the participants reiterated the desire expressed in 1984 for Puerto Ricans to accelerate the process to reduce losses from future earthquakes. The participants were enthusiastic about the preliminary results of the vulnerability study of the San Juan area and recommended that the study be completed for other parts of Puerto Rico as soon as possible. Recommendations other than those produced in the group discussions included:

1. The present building code should be updated to reflect the state-of-the-art in seismic design and hazard mitigation. The proposed building code should be adopted officially as soon as the public hearings are completed.

2) Buildings should be inventoried to rate their potential vulnerability and risk. A program to reduce potential losses to them should be developed as soon as possible.

3) A process should be developed to strengthen existing structures, as needed.

4) An intensive educational program should be developed to make the public aware of the earthquake hazards and ways to mitigate losses.

5) Data, research results, and other relevant information that affects response and recovery should be derived from the 1985 Chile and Mexico earthquakes and transferred to Puerto Rico.

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6) Puerto Rico should start preparing to increase its capability to serve as a "North-South Center" to facilitate the development and transfer of technology in earthquake engineering to other Caribbean Basin countries.

7) The multidisciplinary "working group in earthquake engineering" that was formed during the workshop should work to promote, encourage, and foster the reduction of potential losses from earthquakes in Puerto Rico. Although the working group is an ad hoc entity, it could serve as the forerunner of a future seismic safety organization in Puerto Rico.

USGS and FEMA, together with their partners in Puerto Rico, plan to convene a third workshop on earthquake hazards in May 1986. The workshop will continue the process begun in 1984 and provide another opportunity to advance the vulnerability study reported in 1985.

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FUNDAMENTALS OF GEOLOGY AND REGIONAL GEOLOGY FOR SOLVING EARTHQUAKE ENGINEERING PROBLEMS IN THE PUERTO RICO AREA

by

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EARTHQUAKE HAZARDS

An earthquake is caused by the sudden abrupt release of slowly accumulating strain energy along a fault, a surface or zone of fracturing within the Earth's crust. When a fault breaks or ruptures, seismic waves are propagated in all directions from the source (Figure 1). As the P, S, Love, and Rayleigh waves impinge upon the surface of the earth, they cause the ground to vibrate at frequencies ranging from about 0.1 to 30 Hertz. Buildings are induced to vibrate up and down and side to side as a consequence of the amplitude, spectral composition, and duration of the ground shaking. Damage takes place if the building is not designed and constructed to withstand the dynamic forces accompaning these vibrations. Compressional (P) and shear waves (S) mainly cause high-frequency (greater than 1 Hertz) vibrations which are more efficient than low-frequency waves in causing short buildings to vibrate. Rayleigh and Love waves mainly cause low-frequency (less than 1 Hertz) vibrations which are more efficient than high-frequency waves in causing tall buildings to vibrate.

Earthquake-resistant design requires an evaluation of the primary and secondary phenomena accompanying an earthquake in order to define the forces that a building must resist. These phenomena, called earthquake hazards, are classified as ground shaking, surface fault rupture, earthquake-induced ground failure (landslides, liquefaction, compaction, lurching, and foundation settlement failure), regional tectonic deformation, and (in some coastal areas) tsunamis. Each of these hazards can cause damage to buildings and facilities, economic loss, and loss of life (Figure 2). Fires and floods can also be triggered by these hazards. Aftershocks may last several months to several years, depending on the energy release of the main shock, and can reactivate any or all of these physical phenomena, causing additional damage and loss.

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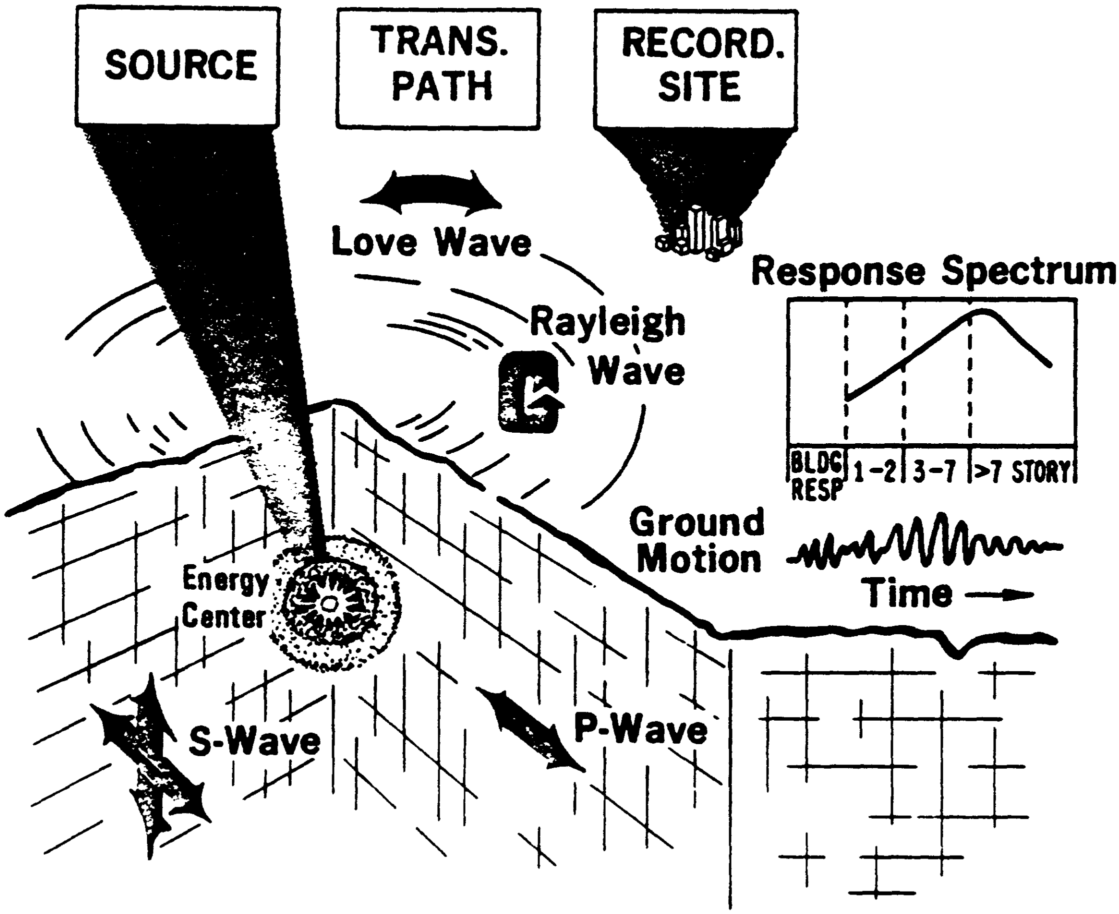

Figure 1.-- Schematic illustration of the directions of vibration caused by body (P and S) and surface (Love and Rayleigh) seismic waves generated during an earthquake. Evaluation of the ground-shaking hazard caused by these waves requires consideration of the physical parameters of the source, transmission path, and the local recording site.

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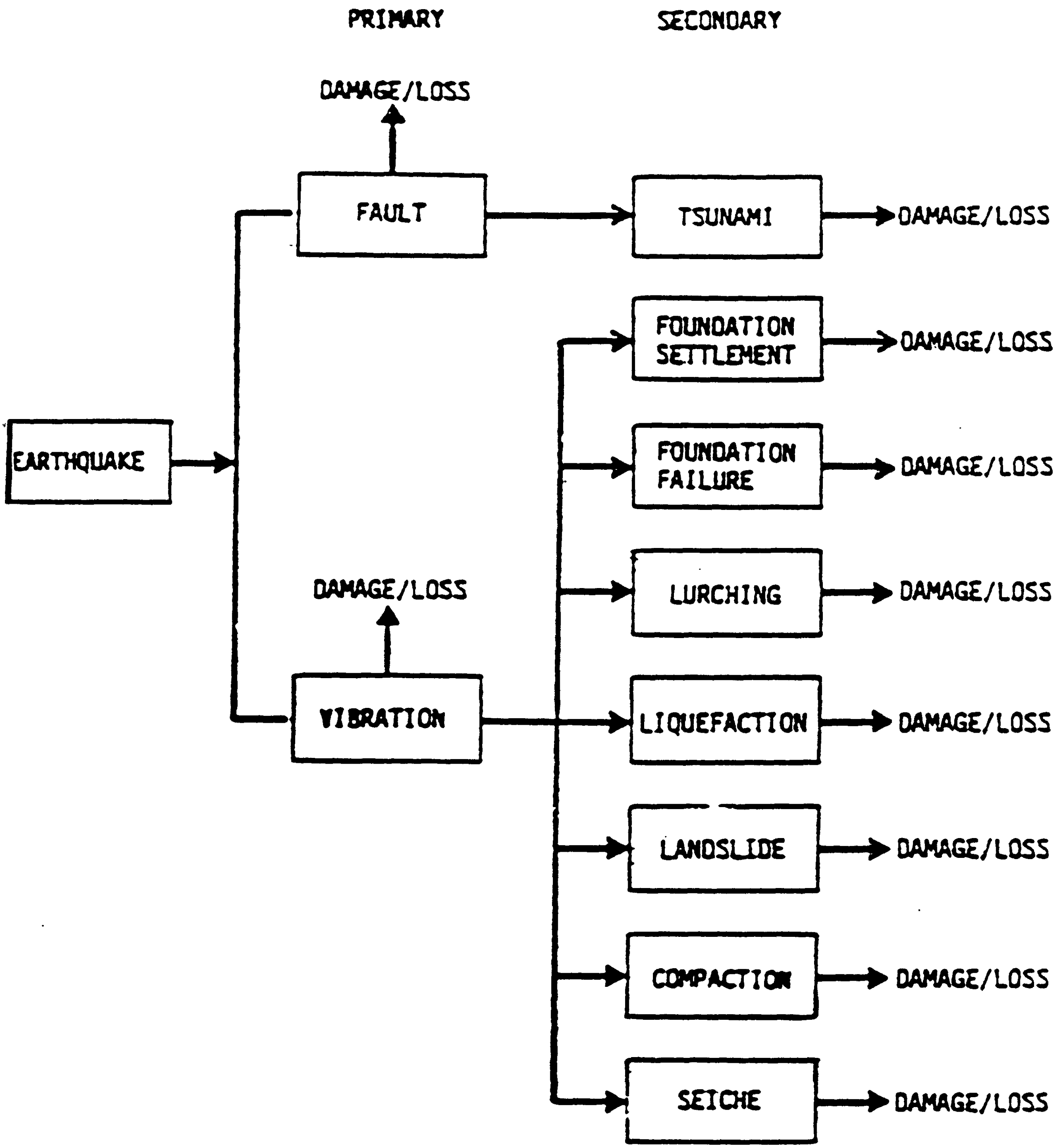

Figure 2.-- Schematic illustration of the primary and secondary hazards caused by an earthquake. Each hazard can lead to damage and loss. The goal of earthquake engineering is to mitigate damage and loss from these hazards through realistic earthquake-resistant design.

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Evaluation of earthquake hazards for earthquake-resistant design is a complex task (Figure 3). A multidisciplinary team of scientists and engineers is required to perform a wide range of technical analyses. These analyses are conducted on three scales: a) global (map scale of about 1:7,500,000 or larger), b) regional (map scale of about 1:250,000 or larger), and c) local (map scale of about 1:250,000 or smaller). Global studies give the "big picture" of the tectonic forces that are at work. Regional studies establish the physical parameters needed to define the earthquake potential of a region. Local studies define the dominant physical parameters that control the site-specific varying characteristics of the hazard. All of the studies seek answers to the following technical questions:

- **WHERE are the earthquakes occurring now? WHERE did they occur in the past?**

- **WHY are they occurring?**

- **HOW OFTEN do earthquakes of a certain size (magnitude) occur?**

- **HOW BIG (severe) have the physical effects been in the past? HOW BIG can they be in the furture?**

- **HOW do the physical effects vary spatially and temporally?**

The answers to these questions are used to define the seismic design parameters (Figure 4). Although these questions appear to be simple, the answers require considerable research and technical judgement.

ROLE OF THE GEOLOGIST

The geologist has an important role in providing information that can be correlated with the amplitude, spectral composition, and duration of the ground shaking, the most important factors that must be incorporated in the earthquake-resistant design of a building or facility. The geologist provides information on all three scales (global, regional, and local) by studying: 1) plate tectonics, 2) faults, 3) paleoseismicity, 4) earthquake potential, 5) seismic

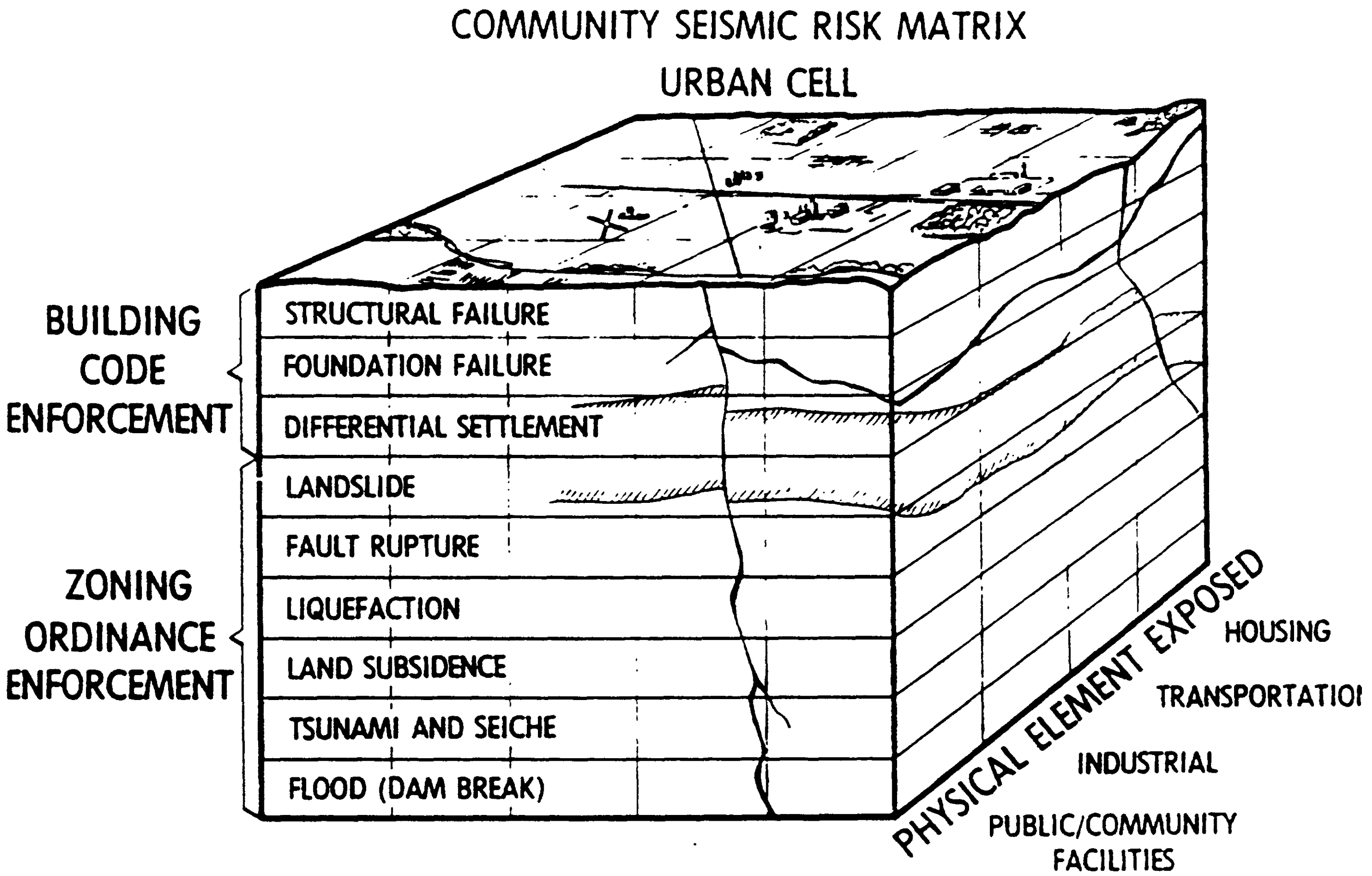

Figure 3.-- Schematic illustration of an urban community having a range of earthquake-reistant design problems. Evaulation of the hazards of ground-shaking, earthquake-induced ground failure, surface faulting, and tectonic deformation is an important part of the process requiring input from the geologist before appropriate earthquake-resistant design parameters can be specified.

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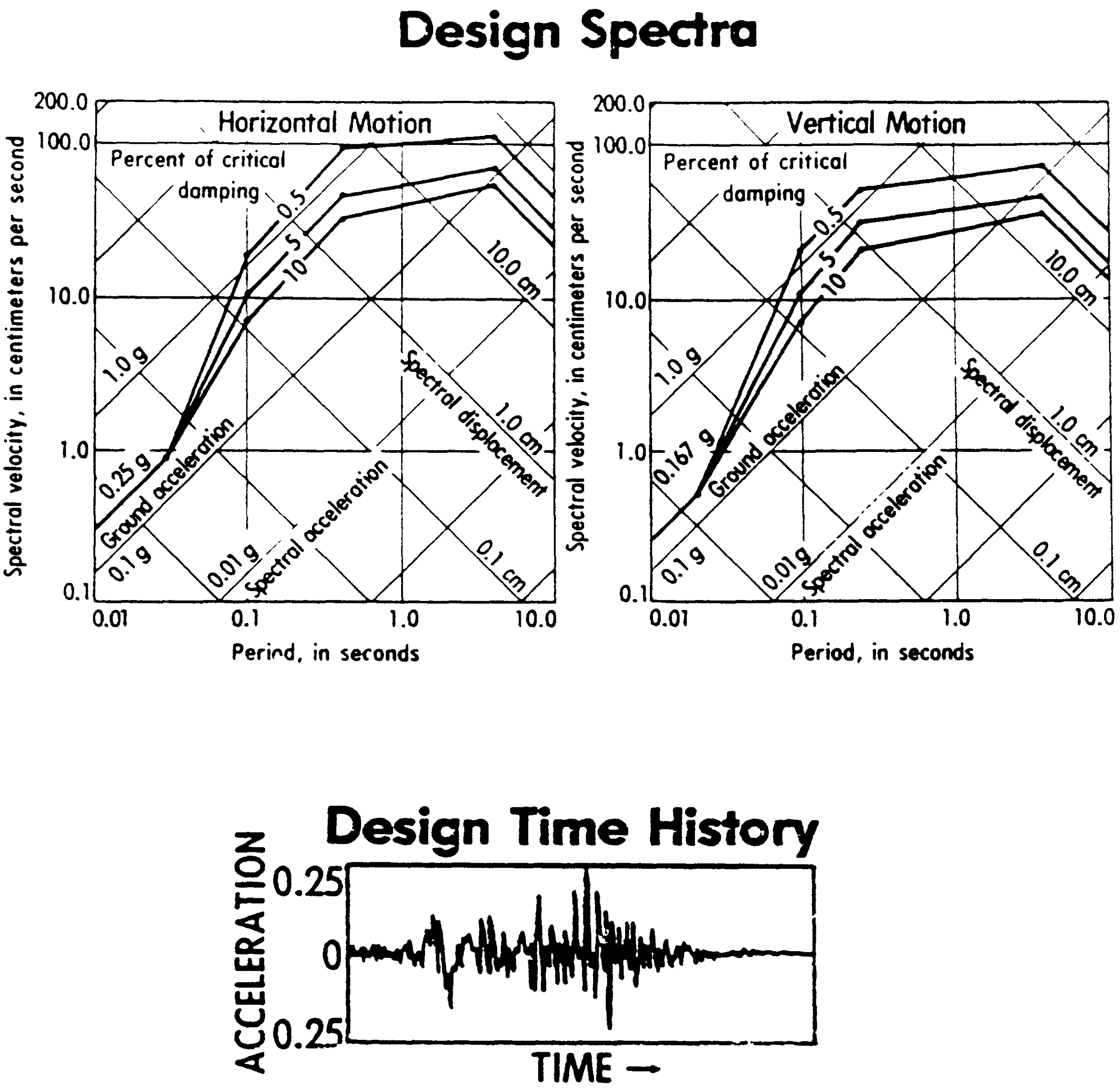

Figure 4.-- Schematic illustration of the design response spectra and time history used in earthquake-resistant design of critically important facilities. In general, the structural engineer requires information about the amplitude, spectral composition, and duration of ground shaking. The geologist provides information that enables reasonable values of these design parameters to be specified.

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source zones, and 6) site-specific characteristics of the soil and rock column underlying the site.

Plate tectonics - Each year, several million earthquakes occur throughout the world. Most of these earthquakes occur along the boundaries of about a dozen 50-to-60-mile-thick rigid plates or segments of the Earth's crust and upper mantle that are moving slowly and continuously over the interior of the Earth (Figure 5). These plates meet in some areas and separate in others, moving with a velocity of relative motion between plates that ranges from less than a fraction of an inch to about 5 inches per year. Although these velocities appear to be slow, they can add up to more than 30 miles in only 1 million years, a short time geologically. As these plates move, strain accumulates. Eventually, faults along or near the plate margins slip abruptly and an earthquake occurs.

Study of faults - The study of faults is critically important in the understanding of where earthquakes are likely to occur, how big they are likely to be, and how often they are likely to take place. The energy released during large earthquakes demands that the fault rupture over a significant fraction of its length. Observational data from historic earthquakes throughout the world indicate that even a moderate earthquake of magnitude 6 requires a fault rupture length of 5-10 km (3-6 miles) and that great earthquakes of magnitude 8 and greater can have a rupture length of as much as 1000 km (600 miles).

The largest known vertical and horizontal fault displacements observed at the ground surface during historic earthquakes are, respectively, 11.5 m (38 feet) during the 1897 Assam earthquake and 9.9 m (33 feet) during the 1957 Mongolia earthquake (Allen, 1984). Geodetic observations suggest that significantly larger displacements have occurred at depth.

Many faults extending to the ground surface have been identified and studied throughout the world by geologists. Studies of faulting have produced the following general rules:

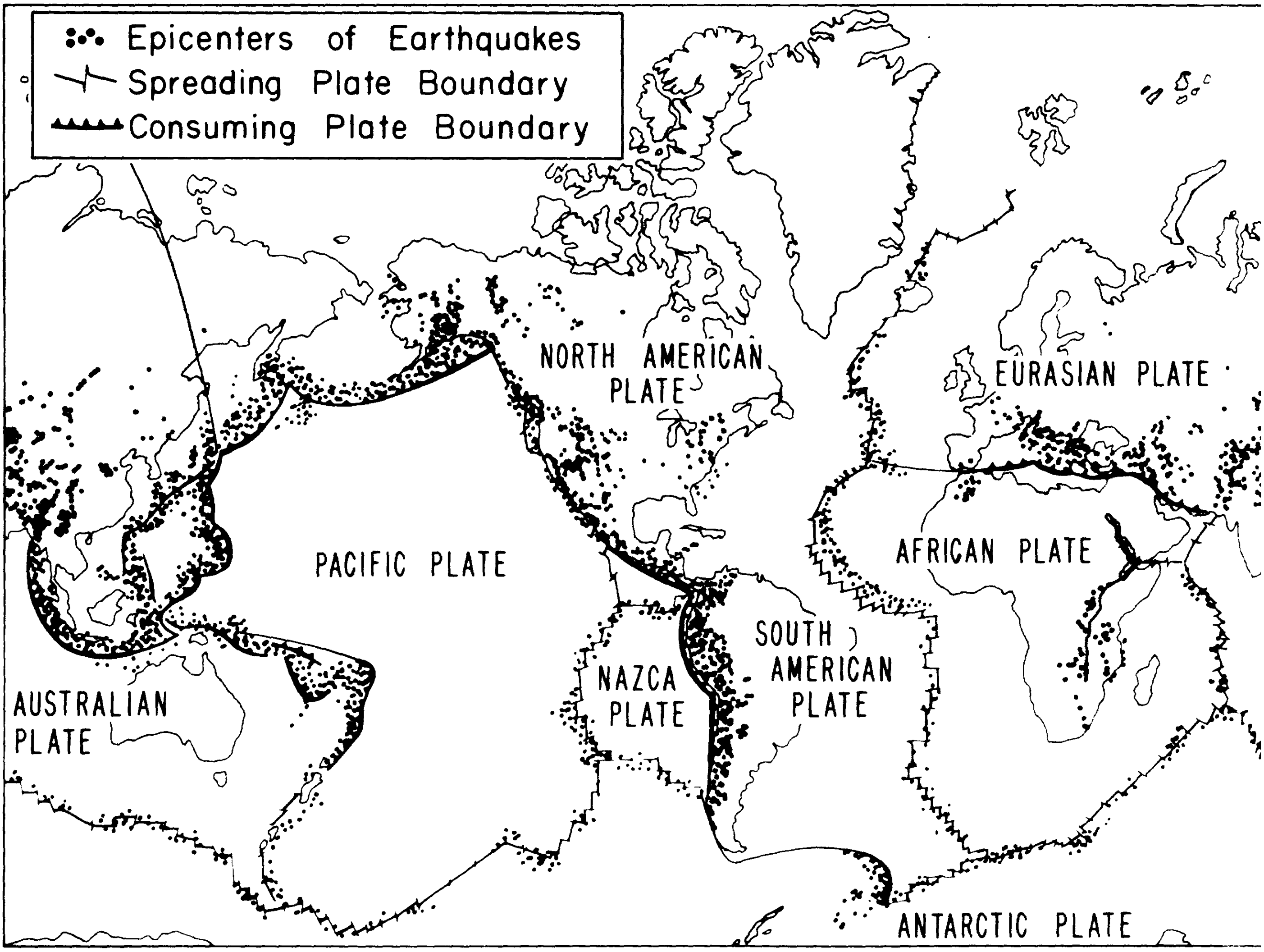

Figure 5.-- Map showing the major tectonic plates of the World. Earthquake activity marks the boundaries of each plate. The double line indicates a zone of spreading from which plates are moving apart. Lines with barbs indicate a zone where one plate is sliding beneath another (subduction). A single line indicates a strike-slip fault along which plates are sliding past one another (compiled and adapted from many sources; much simplified in complex area.)

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- **Almost all large earthquakes have occurred on preexisting faults that have had a previous history of earthquake displacements within the recent geologic past, usually within the past few tens of thousands of years.**

- **Long faults are required to generate large earthquakes.**

- **Long faults grow from the gradual lengthening and coalesing of small faults that rupture in small to medium earthquakes over a period of millions of years. Thus, a long fault such as the San Andreas fault was not born during a single great earthquake in the distant past, but rather is the result of many smaller earthquakes.**

- **If the frequency of movements on a fault during the recent geologic past can be determined, reliable estimates can be made of how likely the fault is to rupture in a future earthquake during a specific time interval.**

Investigations of faults throughout the world have shown that large earthquakes have occurred on strike-slip faults (for example; San Andreas fault) and thrust/or reverse faults (for example; the subduction zone beneath Southern Chile). These two types of faults and the normal fault (for example; Wasatch fault in Utah) are shown schematically in Figure 6. Thrust faults, where one block overrides the other block on a shallowly inclined fault plane, are more difficult to recognize and to evaluate in terms of its activity than strike-slip or normal faults.

A geologist classifies faults as either "active" or "inactive", based on whether they have moved within a specific period of time in the last few tens of thousands of years. Figure 7 illustrates this type of classification. A highly active fault, such as the thrust fault marking the subduction zone in Southern Chile, has the potential for generating a great earthquake, on the average, about once every 100 years; whereas, other faults such as the Oued Fodda fault in Northern Algeria have a longer recurrence interval or repeat time (about once every 450 years) for generating a large earthquake such as the magnitude 7.3 1980 El Asnam earthquake. The activity rate of the fault affects the level of the hazard; to determine it accurately is a major challenge for the geologist.

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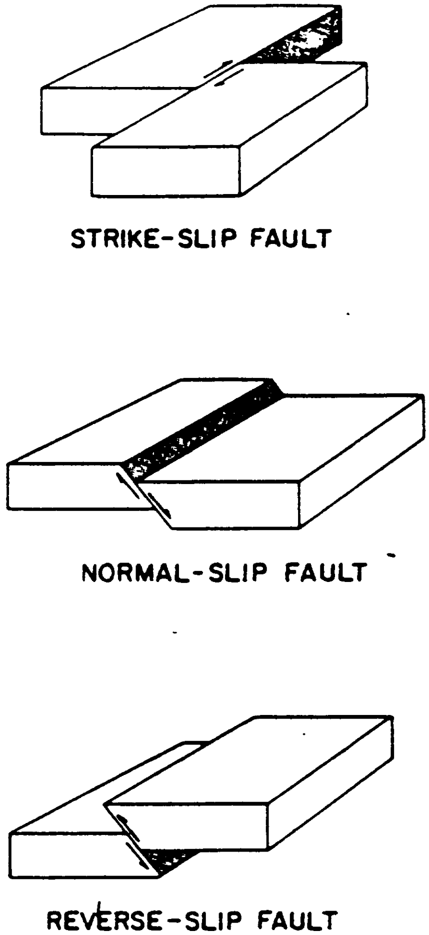

Figure 6.-- Schematic illustration of stike-slip, normal, and reverse faults.

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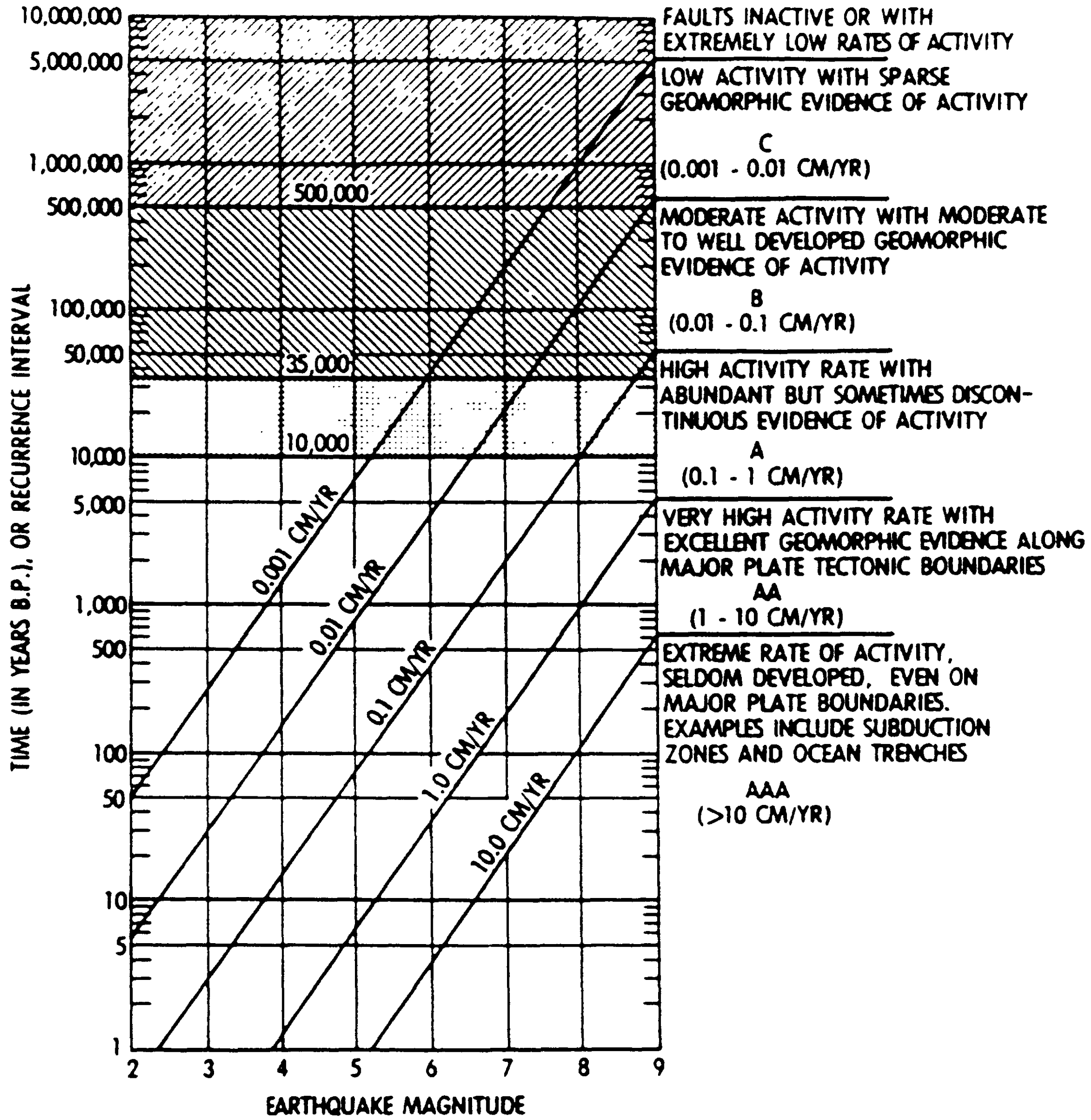

Figure 7.-- Graph showing earthquake magnitude, slip rate, and recurrence interval of active fault zones throughout the world (from Slemmons, 1977)

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In some cases, determination of the activity rate of a fault is very difficult because the fault is not exposed at the surface. An example of this case is the 1886 Charleston, South Carolina earthquake; the causative fault for this earthquake has still not been identified unequivocally (Hays and Gori, 1983). Geophysical investigations (e.g., seismic reflection) are very important in identifying and evaluating the activity of buried faults, both in onshore and offshore areas.

Paleoseismicity - Recently, geologists have developed field techniques to determine the dates of prehistoric earthquakes on a given fault. These techniques involve trenching and age dating, usually with the Carbon-14 method, of buried strata that immediately predate and postdate a historic earthquake. The techniques are called "paleoseismicity." The basic principle of paleoseismicity is:

- **Prehistoric earthquakes cause cummulative surface deformation which manifests itself as stratigraphic and topographic displacements. Hence, a trench having a depth of only 5 m (16 feet) along the San Andreas fault can exibit deformation from prehistoric earthquakes during the past 2000 years.**

The basic assumptions in trenching are:

- **Evidence of significant crustal strain can be isolated at discrete surface locations.**

- **Earthquake-generating fault movements duplicate the near-surface pattern of deformation.**

- **Datable near-surface materials around a fault are preserved for longer periods of time than the recurrence intervals of major fault movements.**

Because several of prehistoric earthquakes are likely to be represented in a single exposure in a trench, the geologic relations can be very complex. Optimal bracketing of the time of the earthquake requires dating of the oldest unbroken post earthquake strata and the youngest deformed pre-earthquake strata.

Useful geologic evidence for paleoseismicity has been developed from stratiographic and geomorphic evidence within active fault zones in the Western United States (Sieh, 1978; Schwartz and Coppersmith, 1984). These relationships provide estimates of the displacements and repeat times of individual paleoseismic events. In the Eastern United States, paleoseismicity studies also are beginning to produce useful results. Late Holocene (10,000 years B.P.) prehistoric earthquakes have been recognized in the New Madrid, Missouri region on the basis of liquefaction associated with two prehistoric earthquakes in the past 2000 years (Russ, 1982). Recently, four large pre-1886 earthquakes in the past 7500 years have been recognized in Hollywood, South Carolina on the basis of liquefaction studies (Obermeier, 1985).

Study of Earthquake Potential - Once tectonic features have been identified, their potential for generating earthquakes is determined. Procedures for assessing the earthquake potential include:

1) Selection of the physical characteristics that enable tectonic features to be differentiated.

2) Comparison with other tectonic features having specified physical charactersitics.

3) Assessment of the probability that a tectonic feature exhibits a particular combination of physical characteristics favorable for generating earthquakes.

Figure 8 shows a matrix that can be used when assessing the earthquake patential of a tectonic feature. All available information should be used to infer the physical characteristics as accurately as pollisble. The following types of questions are asked:

- **Has historical seismicty been associated with the tectonic feature?**

- **Is there evidence of recent crustal strain?**

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PHYSICAL CHARACTERISTICS FOR ASSESSING EARTHQUAKE POTENTIAL

RECENT STRAIN	GEOLOGIC REACTIVATION	LOW STRENGTH OR CHANGE IN STRENGTH	ASSOCIATED SEISMICITY: MODERATE-TO-LARGE EARTHQUAKES				SMALL EARTHQUAKES ONLY				NO SEISMICITY			
		GEOMETRY RELATIVE TO STRESS/ SENSE OF SLIP	FAVORABLE		UNFAVOR-ABLE		FAVORABLE		UNFAVOR-ABLE		FAVORABLE		UNFAVOR-ABLE	
		LOCAL STRESS AMPLIFICATION	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO
YES	YES	YES												
		NO												
	NO	YES												
		NO												
NO	YES	YES												
		NO												
	NO	YES												
		NO												

Figure 8.--Example of matrix containing basic information used to evaluate the earthquake potential of a tectonic feature (from Electric Power Research Institute, 1984).

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- **Is the geometry of the tectonic feature favorable relative to the orientation of the stressfield?**

- **Is there evidence for reactivation of a tectonic feature along preexisting zones of weakness?**

- **Is there evidence that the tectonic feature amplifies the local stress above the abmient level because of structural complexities?**

- **Does the tectonic feature have low crustal strength or exhibit spatial and temporal changes in crustal strength?**

The first two factors, association of the tectonic feature with historical seismicity and evidence for recent crustal strain, are usually the most diagnostic for defining the earthquake potential.

Study of Seismic Source Zones - The geologist and seismologist often work together to define seismic source zones, a region having essentially spatially homogeneous characteristics of earthquake recurrence rates and maximum magnitude. Delineation of source zones requires the integration of seismicity and tectonic framework data. Figure 9 illustrates the types of basic source models: 1) line source, 2) area source, 3) collection of line sources, and 4) a collection of line sources encompassed by an area source. The following general principles can be utilized:

- **A line source model can be used when earthquake locations are constrained along an identified fault or fault zone.**

- **An area source can be used when the seismicity occurs uniformly throughout a region.**

- **A set of line sources can be used to model a large zone of deformation where earthquake rupture has a preferred orientation, but a random occurrence.**

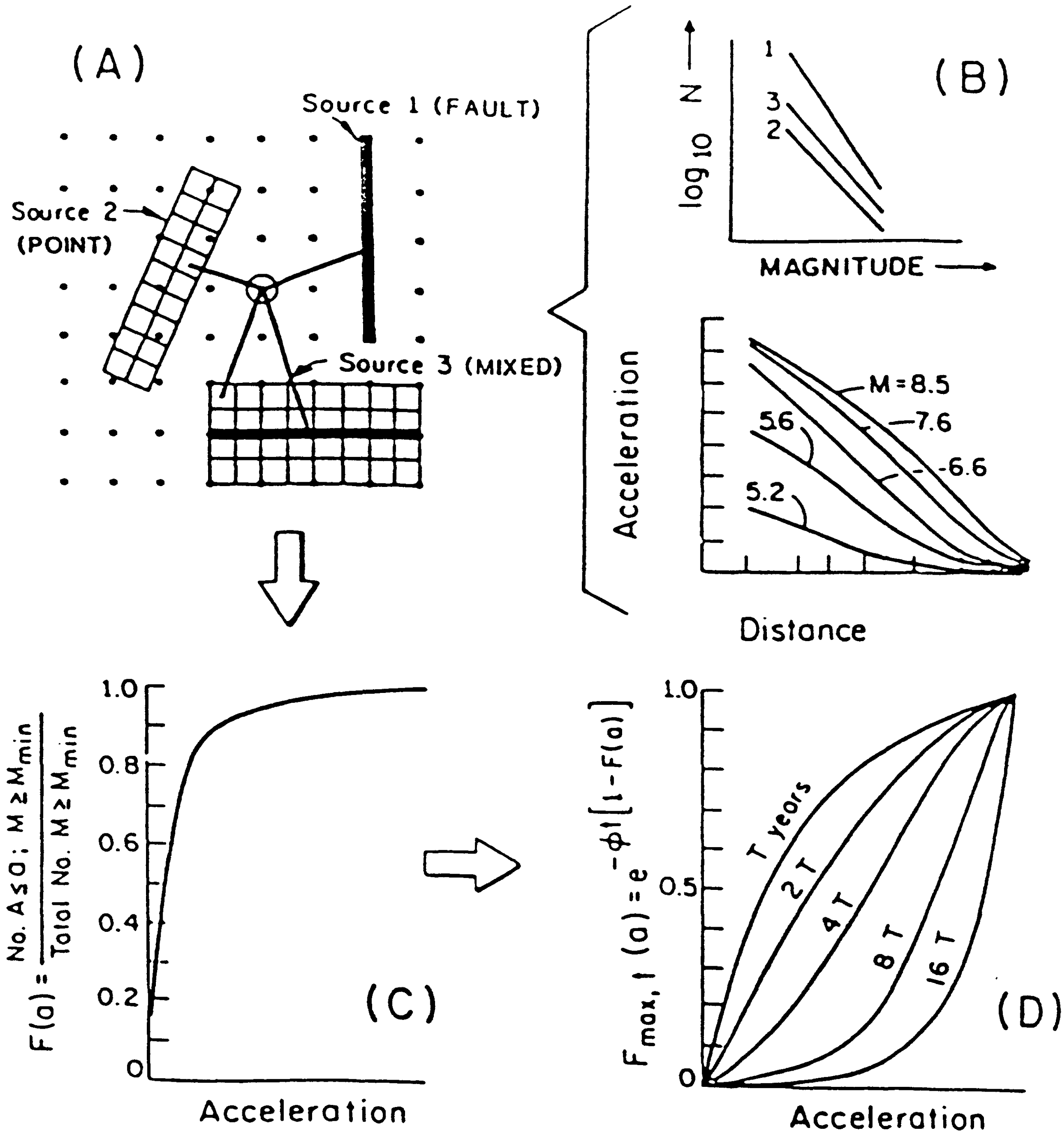

Figure 9.-- Schematic illustration of types of seismic source zones and how they are modeled in a probabalistic analysis.

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- **A collection of line sources encompassed by an area of source can be used when large events are assumed to occur only on identified active faults and smaller events randomly within the region containing them.**

Study of Local Soil and Rock Column - The geologist often works with the geophysicist or geotechnical engineer to define the depth and physical properites of the soil and rock column underlying the construction site (Figure 10). Strong contrasts in the shear-wave velocity between the near-surface soil and underlying rock comprising the upper 30-60 meters (100-200 feet) can cause the ground motion to be increased in a marrow range of frequencies. The peak ampitude, spectral composition, and duration of shaking can all be significantly increased when the velocity contrast is as much as a factor of 2 and the thickness of the soil column is as much as 10-30 m (30-100 feet) (Figure 11). Scientists and engineers are still working to resolve technical issues that center mainly on the question of whether linear ground response occurs at high levels of ground shaking and/or dynamic shear strain (Hays, 1983).

Determination of the physical properties of the near-surface materials is also important in evaluating the potential for liquefaction. Figure 12 gives a flow diagram that can be used to make a preliminary assessment. Additional drilling and geotechnical evaluations are performed if the preliminary assessments indicated such a need.

REGIONAL GEOLOGIC SETTING OF PUERTO RICO

The northern boundary of the Caribbean Sea is comprised of the islands of Cuba, Jamaica, Hispaniola, Puerto Rico, and the Virgin Isalnds. Collectively, these islands consist of volcanic and sedimentary rocks deposited in the last 100 million years and are known as the Greater Antiles. Hispaniola, Puerto Rico, and the Virgin Islands (the eastern Greater Antiles) are the exposed portions of a great linear belt of crustal rocks commonly called the Greater Antilles Ridge. The ridge rises more than 3 miles from the floor of the Caribbean Sea on the south and more than 5 miles above the Puerto Rico trench on the north.

Geologic History - Puerto Rico is very complex geologically. The development of Puerto Rico throughout geologic time can be summarized as follows:

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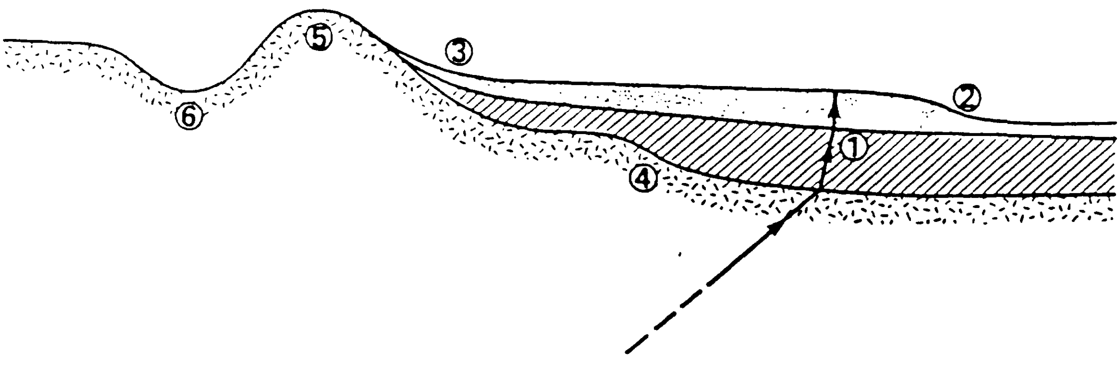

Figure 10.-- Schematic illustration of the effects of the soil and rock column on ground shaking. Each of the six sites will have a different time history and response spectrum because of the varying geometry, thickness, and physical properties of the soil and rock column.

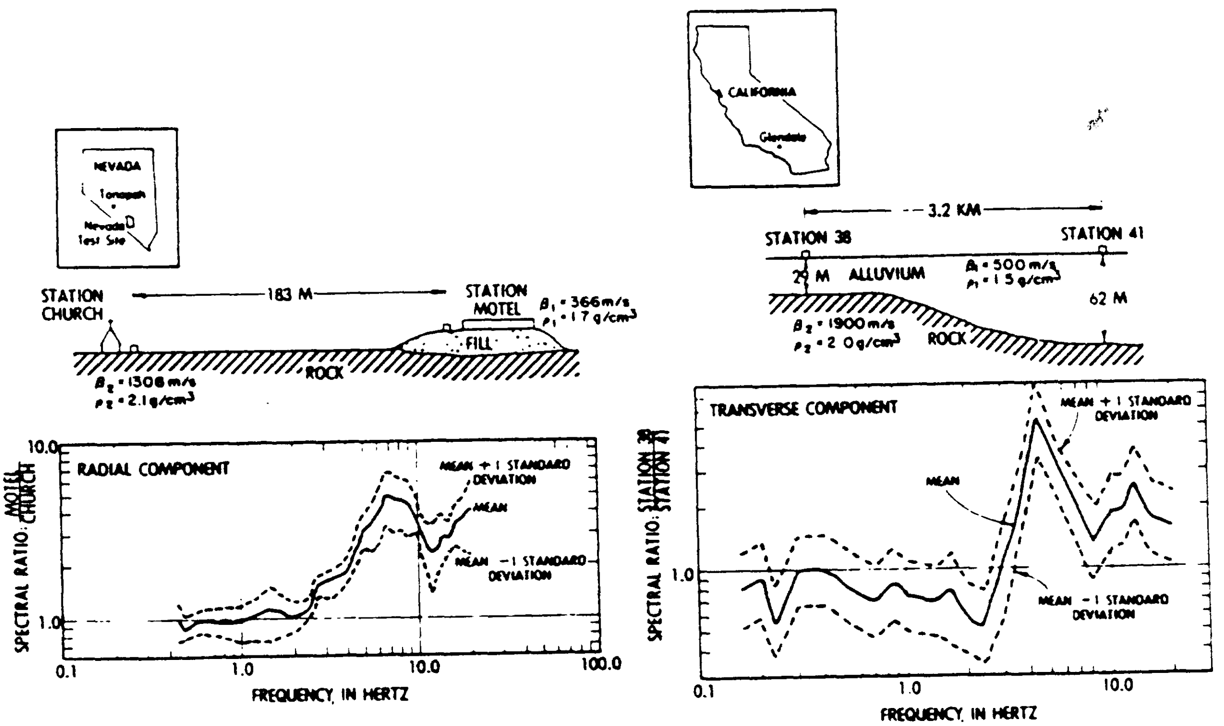

Figure 11.-- Examples of site amplification caused by variations in the near-surface soil and rock column. Variations in the thickness and geometry of the soil and rock and the physical properties (shear wave velocity, density) can cause amplification of ground motion. Amplification can lead to a requirement for larger design ground motion parameters.

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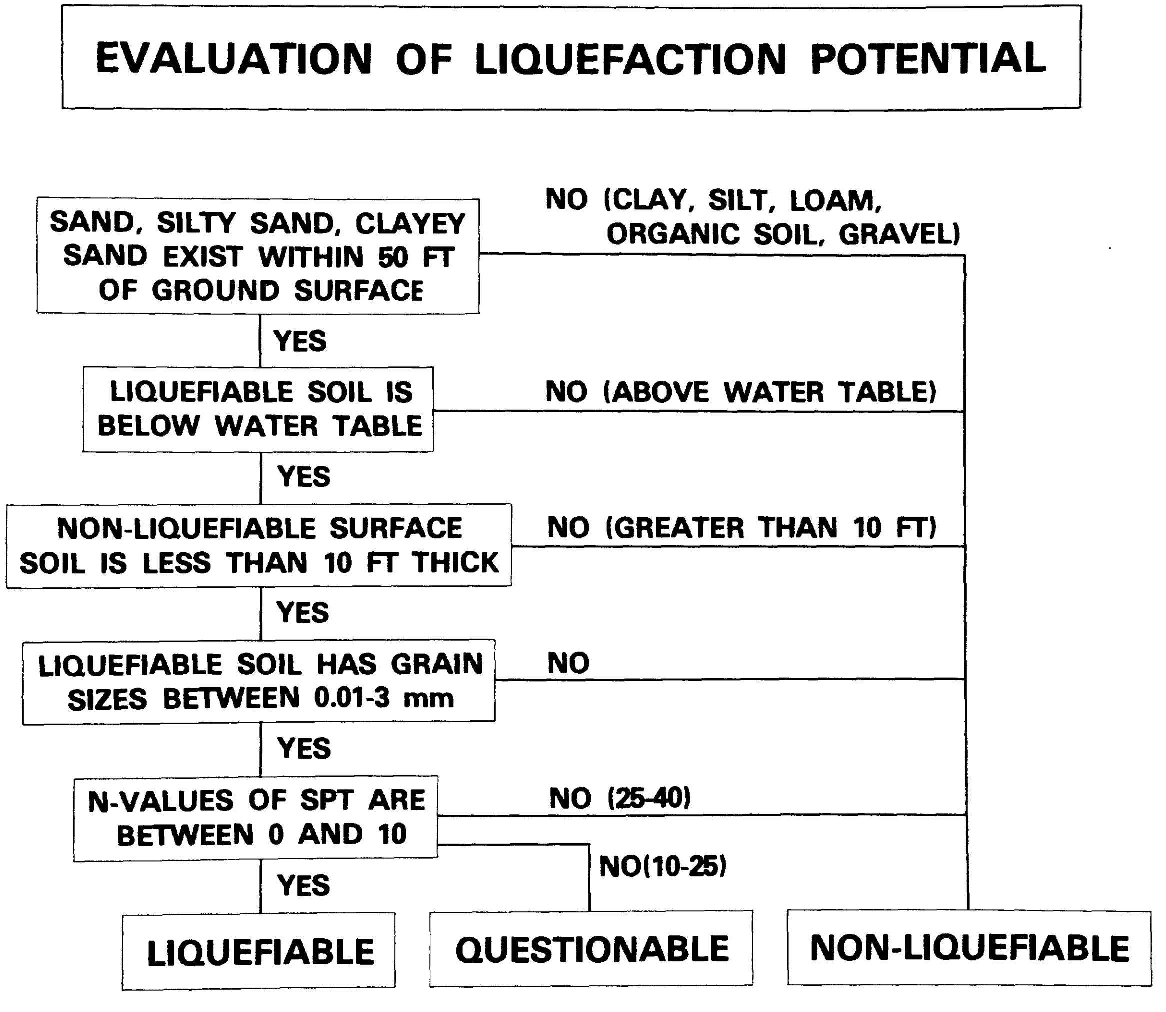

Figure 12.--Flow diagram that can be used when evaluating the potential for liquefaction at a site.

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1) <u>130 million years ago</u> <u>(very early Cretaceous)</u> - the beginning of submarine volcanism with a local build up of volcanic material.

2) <u>120 million years ago</u> <u>(early Cretaceous)</u> - submarine volcanic material continues to accumulate and build up.

3) <u>100 million years ago</u> <u>(late Cretaceous)</u> - the sea floor continues to be built up and submarine surfaces continue to build toward the emergence of a landmass.

4) <u>90 million years ago</u> <u>(early late Cretaceous)</u> - a small volcanic island appears above sea level, the ancestral predecessor of Puerto Rico.

5) <u>70 million years ago</u> <u>(end of late Cretaceous)</u> - extensive volcanism occurs with separate volcanoes acting to form the ancient Puerto Rican landmass.

6) <u>60 million years ago</u> <u>(Paleocene to early Eocene)</u> - period of <u>first</u> major tectonic activity. The volcanic rocks are uplifted and eroded, becoming the source rocks for the Eocene sediments found in Puerto Rico today. The island is uplifted with major northwest trending stike-slip faulting.

7) <u>50 million years ago</u> <u>(middle Eocene)</u> - sedimentary rocks are deposited throughout southern Puerto Rico.

8) <u>40 million years ago</u> <u>(late Eocene)</u> - period of <u>second</u> major tectonic activity. Island was reelevated to alpine heights with recurrence of faulting along preexisting northwest trending faults. The Esneralda fault zone is believed to have developed primarily in this time. This epoch is believed to be the <u>last</u> time that major stike-slip movement occurred throughout the south coastal region. The Juana Diaz formation (conglomerate, shale, limey shale) formed.

9) <u>25 million years ago</u> <u>(late Oligocene to early Miocene)</u> - period of <u>third</u> major tectonic activity. A recurrence of faulting along some

preexisting northwest trending zones, but new faults also formed. The Ponce limestone formation formed.

10) 15 million years ago (late Miocene or younger) - period of fourth major tectonic activity. The island was reelevated with consequent reactivation of some old faults and the development of new ones. Block faulting with displacements up to 200 m (630 feet) occurred. The island began to be under large-scale stresses from the boundary troughs and trenches (Puerto Rico trench, Mona Passage, and Anedaga trough).

11) 1-3 million years ago (Pleistocene-Recent) - the island is relatively stable. However, the island continues to be under stresses with movement to the north, east-west, and south caused by Mona Passage, Puerto Rico trench, and Anedaga trough.

Caribbean Plate - Puerto Rico lies near the northeastern corner of the Caribben plate, a rigid crustal block that is in motion relative to North and South America and the floor of the Atlantic Ocean (Figure 13). The ocean floor to the north and east of Puerto Rico is part of the North American plate and is moving west-southwest relative to the Caribbean plate. On meeting the Caribbean plate, it bends downward, descending into the mantle with a dip of 50-60 degrees, eventually reaching depths as great as 150 km (90 miles) (McCann, 1984). Convergence between the Caribbean and North American plates occur at a rate of at least 37 mm/year (Sykes, et al., 1982). Puerto Rico does not appear to be rigidly attached to the Caribbean plate which appear to be underthrusting western and central Puerto Rico (McCann, 1984).

Puerto Rico, which measures 109 miles east to west and 37 miles north to south, is surrounded by troughs (Figure 14). The Puerto Rico trench is on the north and the Anegada trough is on the east. The Muertos trough is on the south, and Mona Canyon and Mona Passage are on the west. The Puerto Rico trench has a maximum depth of 26,200 feet, the greatest known depth in the Atlantic Ocean. It is also associated with the largest gravity minimum of any oceanic trench in the World. Two major fault zones, the Great Northern Puerto

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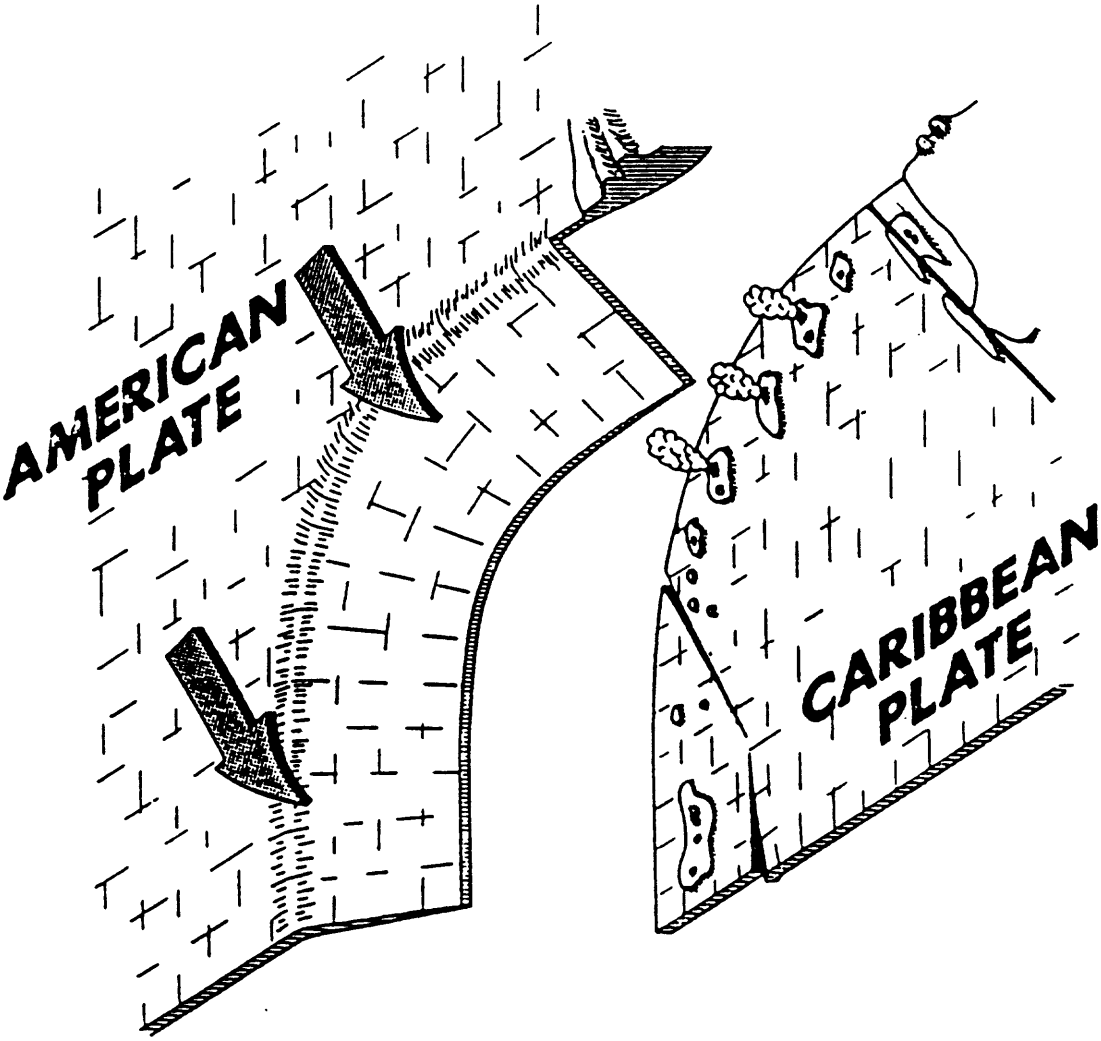

Figure 13.-- Diagram of North American and Caribbean Plates (from McCann, 1984).

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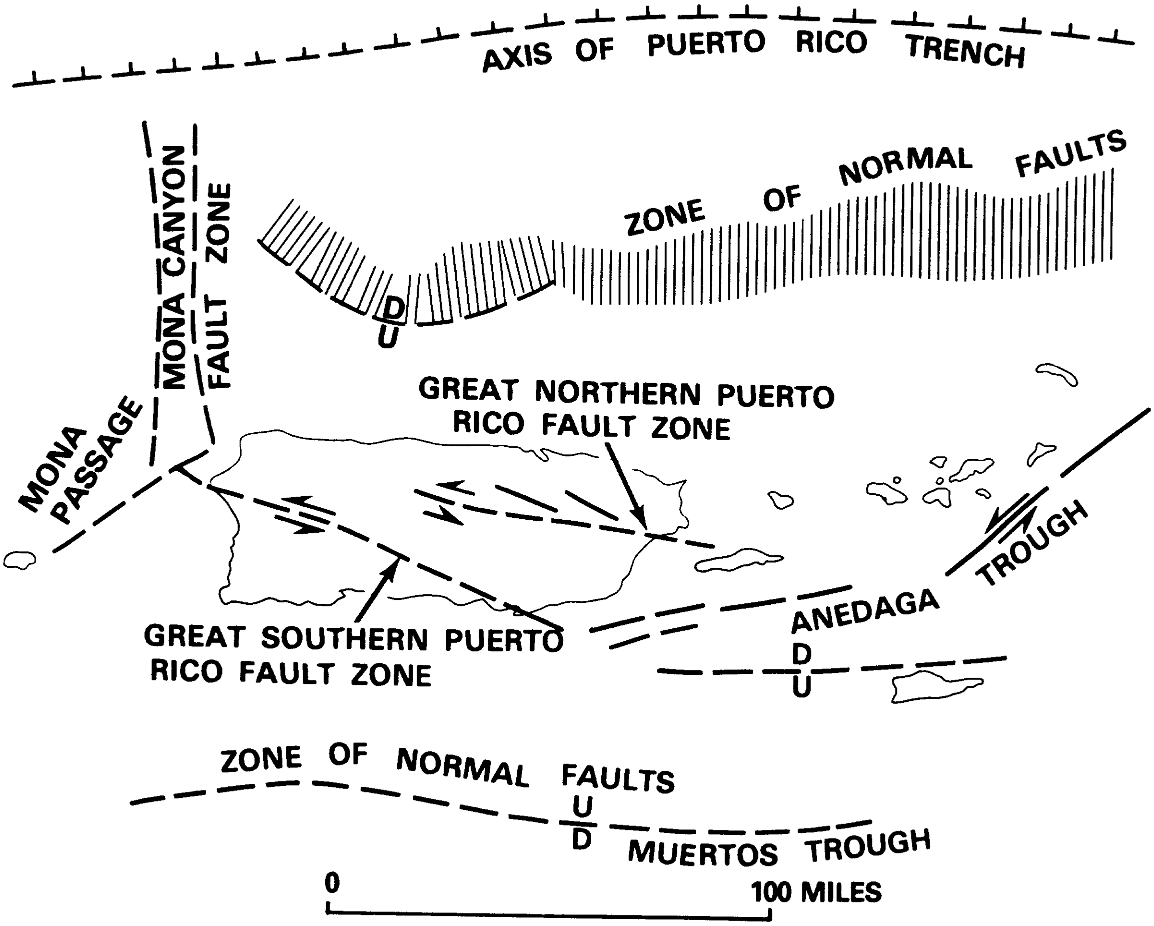

Figure 14.--Diagram showing Puerto Rico and the surrounding troughts and trenches.

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Rico fault zone and the Great Southern Puerto Rico fault zone, divide Puerto Rico into three blocks (Figure 15). These three blocks are called the northeastern, central, and southwestern blocks. The northeastern block is composed mainly of mafic to intermediate composition lava (basalt, andesite), lava breccia, and well stratified volcanistic deposits interbedded with shallow marine shales, sandstones, and limestones ranging in age from middle Oligocene through Miocene. The entire sequence is widely intruded by diorite and quartz diorite. The central block has a similar stratigraphic section as the northeastern block, but differs in that two large granitic batholiths of early Cretaceous to Eocene age are also present. The southwestern block is different from the other two blocks in that a larger percentage of carbonate rocks (mainly reefs) and clastic rocks are present and there are few granitic plutons. Serpentine also outcrops. The age of the carbonate and clastic sequence is middle Oligocene to Miocene.

The Greater Northern Puerto Rico fault zone consists of numerous short, discontinuous normal and transcurrent faults which extend westward from near Punta Lima on the eastern coast of Puerto Rico. It passes through late Cretaceous to early Tertiary volcanic and sedimentary rocks. The last movement is thought to be pre-Mid-Tertiary. The total horizontal offset along the fault zone is more than 60 km (36 miles) in a left-lateral strike-slip sense. The vertical displacement is about 6000 feet along the Damian-Arriba splay. No historic seismicity has been associated with any fault in the fault zone. Little, if any, motion has occurred on this fault in the last 20 million years.

The Greater Southern Puerto Rico fault zone has a total length of about 179 km (112 miles) if the offshore segments are included. It extends from Central Aguirre on the south coast diagonally across the island to the west coast near Punta Higuero. The fault zone is complex, exhibiting left-lateral strike-slip displacement in the western portion of the island and dip-slip displacement near Juana Diaz. No historic seismicity has been associated with any of the faults in the zone. Little, if any, motion has occurred on this fault in the last 20 million years.

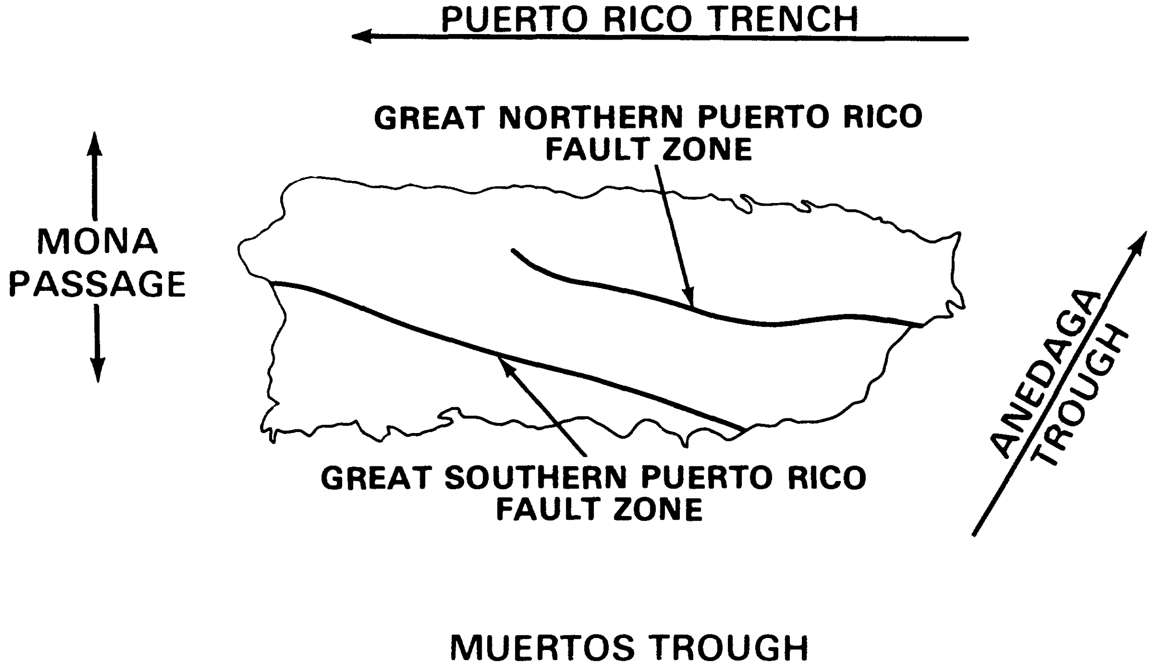

Figure 15.--Diagram showing major faults and tectonic blocks on Puerto Rico.

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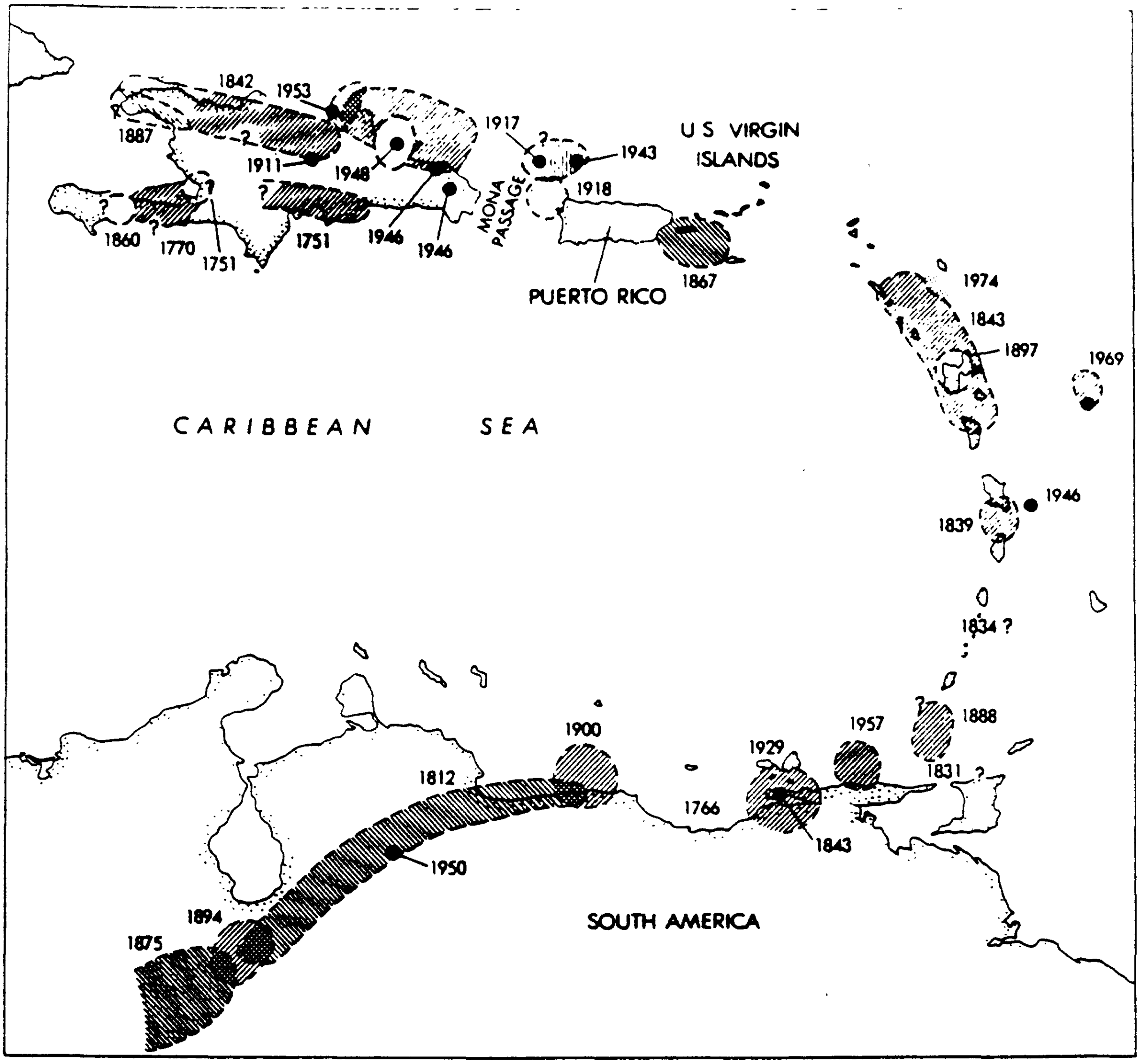

Figure 16.-- Map showing location of major earthquakes in the vicinity of Puerto Rico since 1800.

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HISTORICAL SEISMICITY IN THE PUERTO RICO AREA

Seismicity in the vicinity of Puerto Rico occurs as a consequence of either relative motion between two plates (interplate) or relative motion between blocks within one plate (intraplate).

The historic record of seismicity is more than 400 years long. In the past, major damaging earthquakes have occurred in:

- 1787 (Probably a great earthquake that damaged all of Puerto Rico except the south coast)

- 1867 (Located near the Virgin Islands. It had an epicentral intensity of VIII and also generated a tsunami having wave heights of 3-5 feet in the vicinity of Arroyo.)

- 1918 (A magnitude 7.5 earthquake that also generated a tsunami having 20 foot waves. It was located about 15 km (9 miles) off the northwest coast of Mona Passage.)

- 1943 (The largest earthquake of the 20th century, magnitude 7.75, occurred northwest of Puerto Rico in the Puerto Rico trench.)

Figure 16 shows the location of major earthquakes since 1800.

The regional seismicity falls in seven zones. Each zone is described briefly:

1) Eastern Hispaniola - the most seismically active area within 300 miles of Puerto Rico. The events have a deep focus and large magnitudes (greater than 7.0) and are probably associated with underthrusting of the North American plate beneath the northeastern coast of Hispaniola.

2) Mona Passage - Mona Canyon - seismically active; the locus of several large magnitude earthquakes which are larger and deeper than events further east along the Puerto Rico trench. The damaging magnitude 7.5

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earthquake of October 11, 1918 occurred in Mona Canyon. Mona Canyon is bounded predominantly by normal fault zones.

3) Puerto Rico trench - the location mainly of shallow focus earthquakes. Numerous earthquakes have been recorded in or under the trench. Most events have stike-slip focal mechanisms. Both the magnitude 7.75 event of July 29, 1943 and the magnitude 7.0 event of October 10, 1915 occurred in the Puerto Rico trench.

4) Anedaga trough - the source of a moderate number of earthquakes whose magnitude, depth, and frequency of occurrences increases east of the junction of the Anedaga trough and Puerto Rico trench.

5) Muertos trough - the location of only a few scattered earthquakes of low magnitude. The exact nature of the trough is not known.

6) Zone of intermediate depth seismicity under Puerto Rico - except for a few intermediate depth earthquakes under Puerto Rico, the island is relatively quiescent. Deep events are almost totally lacking. However, a magnitude 7.1 event occurred at a depth of 50 km (31 miles) in 1961.

7) Shallow Puerto Rico crustal seismicity - the source of a few randomly distributed events that are not well correlated with the two major fault zones on the island.

Near-Surface Soil and Rock in Puerto Rico - A wide variety of near-surface materials occur throughout Puerto Rico. They vary in thickness, geometry, and physical properties; therefore, they have the potential at some locations for increasing locally the amplitude, spectral composition, and duration of ground shaking.

The deposits (from youngest to oldest) can be generalized as follows:

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1) alluvial plains deposits (Quaternary) - unconsolidated, waterbearing, sands, silts, clay, and gravel. They range from a few feet to 2,500 feet in thickness.

2) lagoonal and swamp deposits (Quaternary) - unconsolidated clay, fine silt, and organic matter.

3) beach deposits (Quaternary) - unconsolidted sand, gravel, and cobbles derived from the volcanic rocks and shell fragments.

4) Juana Diaz formation (Oligocene) - coarse gravel and pebbles of limestone.

5) sedimentary rocks (Eocene) - interbedded, coarse-to-fine-grained sandstones, siltstones, shales, sandstone breccias, and limestones. These rocks are deeply weathered at some locations.

A typical velocity profile indicates that the S-wave velocity averages about 450 m/sec (1,500 feet/sec) in the upper 15 m (50 feet) and about 1,200 m/sec (4,000 ft/sec) down to a depth of 90 m (300 feet). Such a velocity contrast could lead to amplification of ground motion at some locations.

The intense chemical weathering processes taking place in Puerto Rico transforms the geologic properties of the rock, reducing their shear strength and making them susceptible to mass movements. In general, mass movements are directly related to terrain steepness and rainfall intensity and duration (Molinelli, 1984).

Although records of the 1918 Puerto Rico earthquake make no mention of liquefaction, the evidence suggests that some of the damage may have occurred as a consequence of liquefaction (Soto, 1984).

CONCLUSIONS

Integration of the geologic and seismological data in the Puerto Rico region indicates that the region has the potential to produce moderate to great earthquakes. On the basis of research reported by other investigators (e.g., McCann, 1984), the earthquake potential can be summarized as follows:

- **Large earthquakes (M = 7.5-8.0) may occur with long recurrence times (i.e., thousands of years) in the deeper parts of the trench marking the zone where the North American plate flexes to descend under the Caribbean plate.**

- **Large earthquakes can be expected to occur in the Anegada trough and the Mona Passage-Mona Canyon area.**

- **The broad region encompassing Anegada trough, Muertos trough, and Mona Passage may produce large earthquakes as frequently as the Puerto Rico trench.**

- **Great earthquakes may rupture 200 km (120 miles) long sections of the fault zone south of the Puerto Rico trench about once every 200 years.**

- **Tsunamis are a threat in Puerto Rico.**

- **Landslides may be expected in many locations. Occurrence of liquefaction is also likely.**

- **Surface fault rupture is not considered likely in Puerto Rico.**

The large number of faults off the plate boundary suggests that, on the average, a fault may rupture and produce a major earthquake every few hundred years. Earthquake-resistant design must take this factor into consideration. For example, tall buildings may be potentially vulnerable to low-frequency ground shaking generated by large offshore earthquakes. In general, engineers will be designing for peak ground accelerations in the order of 0.20 g. Design levels will be greater for some important structures and facilities.

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THE EARTHQUAKE HAZARDS OF PUERTO RICO AND THE VIRGIN ISLANDS

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INTRODUCTION

Puerto Rico and the Virgin Islands lie at the eastern edge of the Greater Antilles, a chain of islands composed of volcanic and sedimentary rocks deposited over the last 100 million years (Figure 1); they also lie near the northeastern corner of the Caribbean plate, a rigid block in motion with respect to North and South America, and the floor of the Atlantic Ocean. The ocean floor to the north and east of the islands, which is part of the North American plate, moves WSW with respect to the Caribbean; upon meeting the Caribbean plate it bends downward, descending into the mantle with a dip of 50 to 60 degrees (Figures 2 and 3) eventually reading depths as great as 150 kilometers (Molnar and Sykes, 1969; Schell and Tarr, 178; Frankel et al., 1980; Fischer and McCann, 1984). Convergence between the Caribbean and North American plates occurs at a rate of about 37 mm/year (Sykes et al., 1982).

Seismicity occurring along the margin of the Caribbean plate represents either relative motion between two plates (interplate) or between blocks within one plate (intraplate). Regardless of their origin, strong earthquakes near Puerto Rico and the Virgin Islands pose a hazard to local populations.

The historic record spanning 400 years is clear, strong damaging earthquakes have periodically stricken the islands. The location of their causative faults and the approixmate magnitude of these older shocks is not well determined. The first recorded damaging shock, in the 1520's, reportedly destroyed the home of Ponce de Leon, as well as other structures in western Puerto Rico (Anon, 1972). During succeeding centuries other strong shocks are reported affecting various sectors of the island. The most important shocks

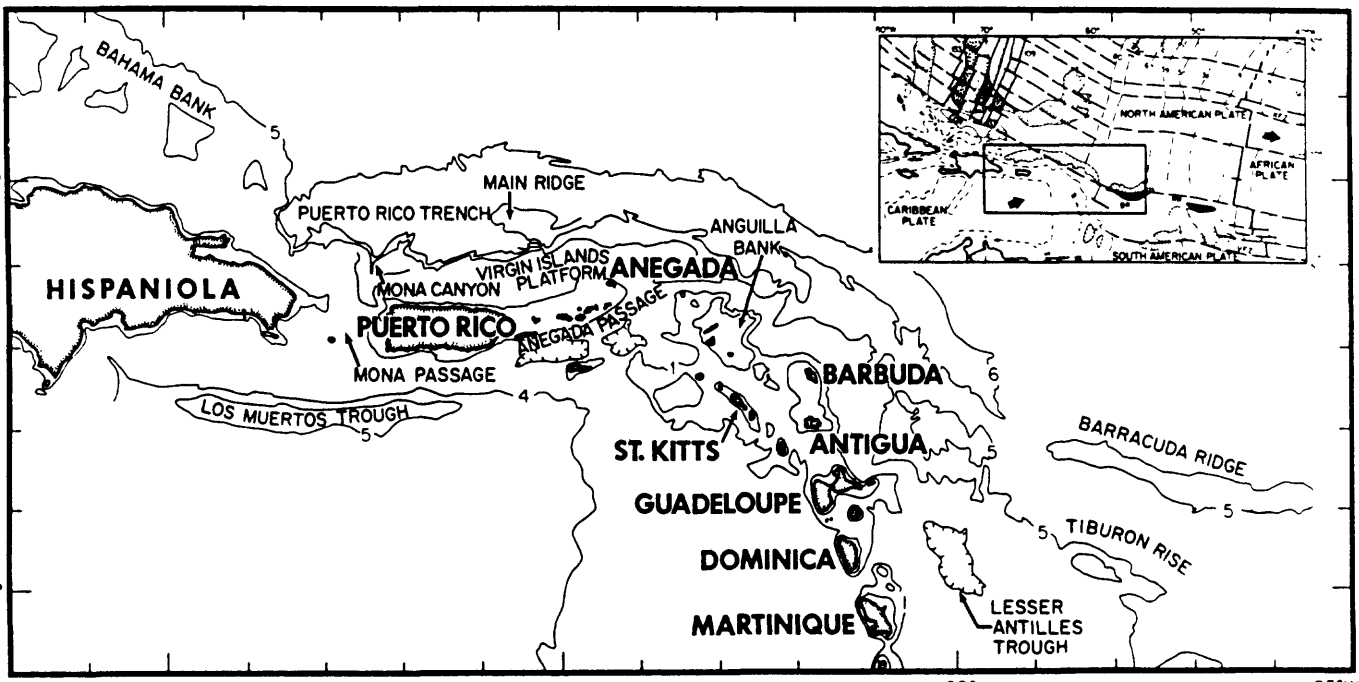

Figure 1. Place names and general bathymetry of northeastern Caribbean. Contours are in kilometers (after Case and Holcombe, 1980). Inset shows tectonic framework fo the eastern Caribbean and Central Atlantic Ocean. Arrows are directions of relative motion of African and Caribbean plates with respect to a fixed North American plate. Double lines represent seafloor spreading. Light dashed lines are magnetic anomalies, numbers are age of anomaly in millions of years. Close stipple pattern is region of Mesozoic anomalies. Heavy dashed lines are fracture zones. Barracuda and Researcher Ridges (BR and RR) are shown in black. Open stipple pattern shows extent of abyssal plains. Northeastern Caribbean is the site of subduction of North Atlantic seafloor. Note the northwesterly trend of fracture zones in the region. Recent motion of the Caribbean plate has carried it over several of these fracture zones. Other labels: VFZ, Vema Fracture zone; KFZ, Kane fracture zone; COR, Caicos Outer Ridge; from McCann and Sykes (1984).

being those of 1787, when destruction occurred everywhere but the south coast of Puerto Rico, and 1867 when a destructive seismic seawave (tsunami) ravaged the coast of southeastern Puerto Rico and various parts of the Virgin Islands (Anon, 1972; Reid and Taber, 1920).

Damage from large shocks in the Dominican Republic to the west, have also affected Puerto Rico. Dominican earthquakes in 1615, 1751, 1776 and 1946 caused considerable damage in the western part of Puerto Rico (Iniguez et al., 1975; Anon, 1972).

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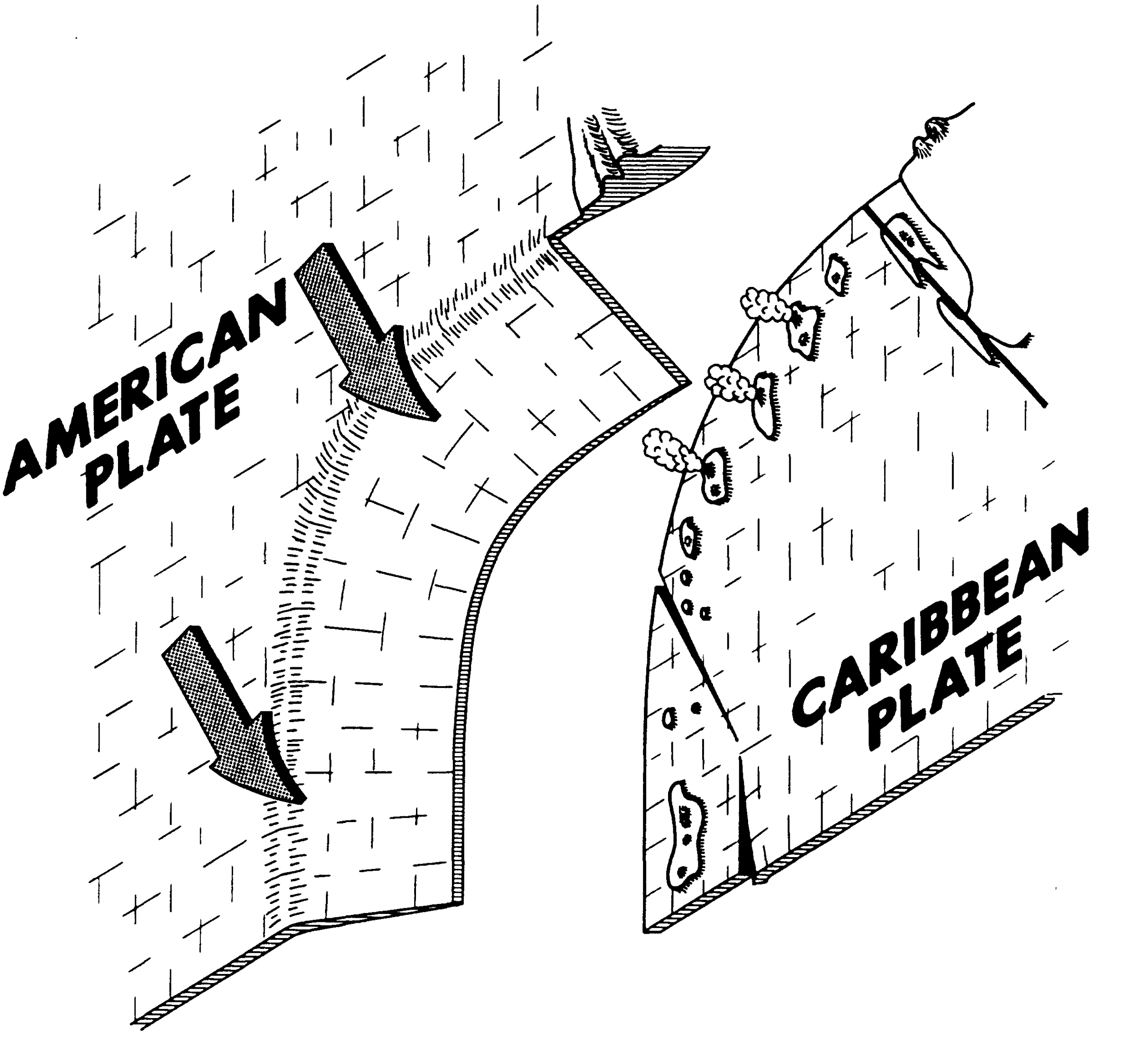

Figure 2. Plate tectonic sketch of eastern Caribbean. North American Plate moves WSW relative to the Caribbean plate. In the view shown here the plates are separated to allow viewing of downgoing section of North American plate. Puerto Rico and the Virgin Islands lie on a block that appears not to be rigidly attached to the Caribbean plate. Caribbean plate underthrusts western and central Puerto Rico; this motion is associated with active faulting south of the Virgin Islands.

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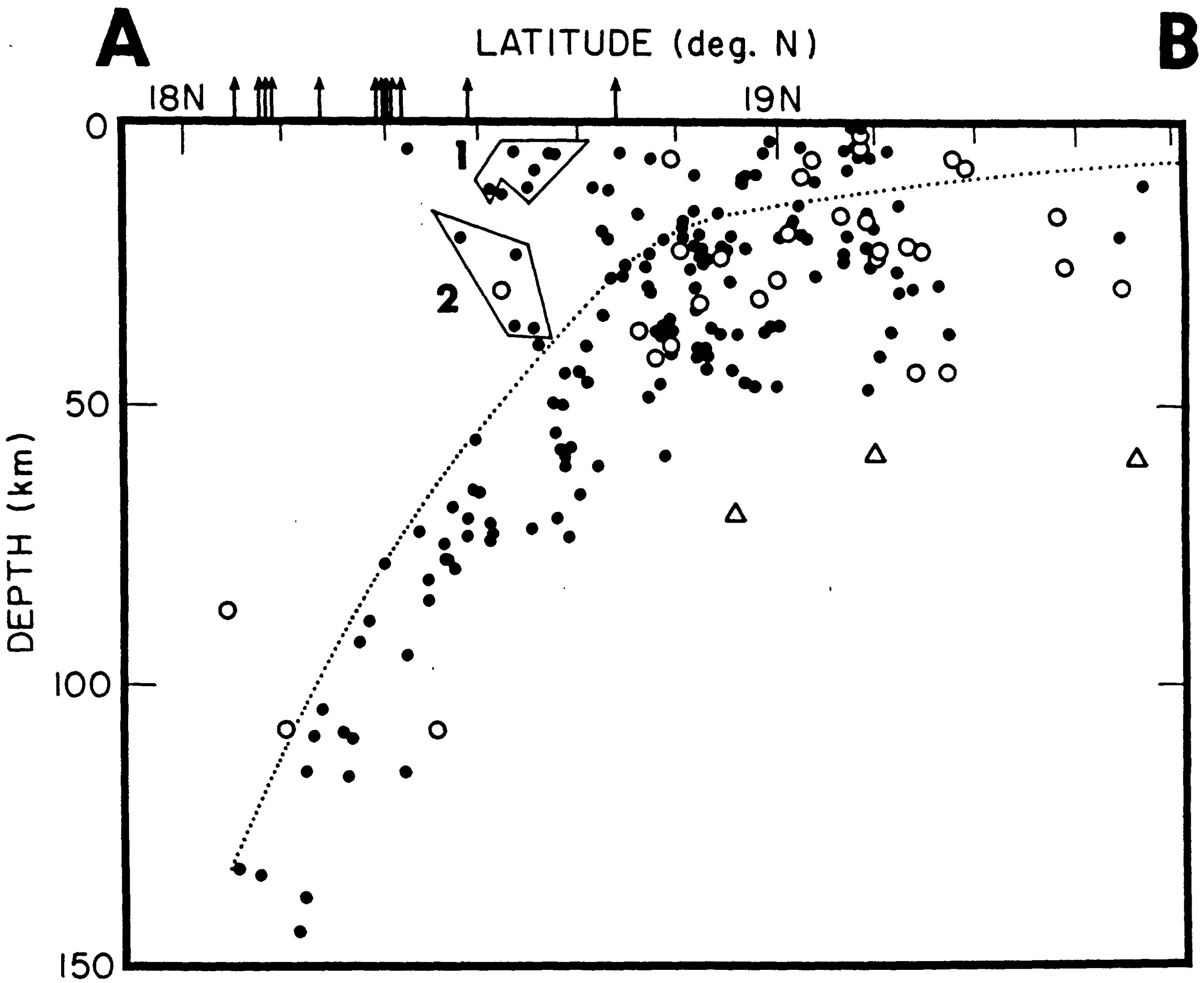

Figure 3. Vertical cross-section of 227 relocated hypocenters, obtained by projecting them onto a vertical plane striking N-S along 64 degrees 40' W in a direction perpendicular to Puerto Rico (only events within 100 km of this line are shown). Open symbols indicate events with residuals $\geq$0.3 sec; solid symbols show events with residuals $\leq$0.3 sec. The two groups of events outlined at the top are long possible intraplate faults. Arrows at top of figure indicate station locations.

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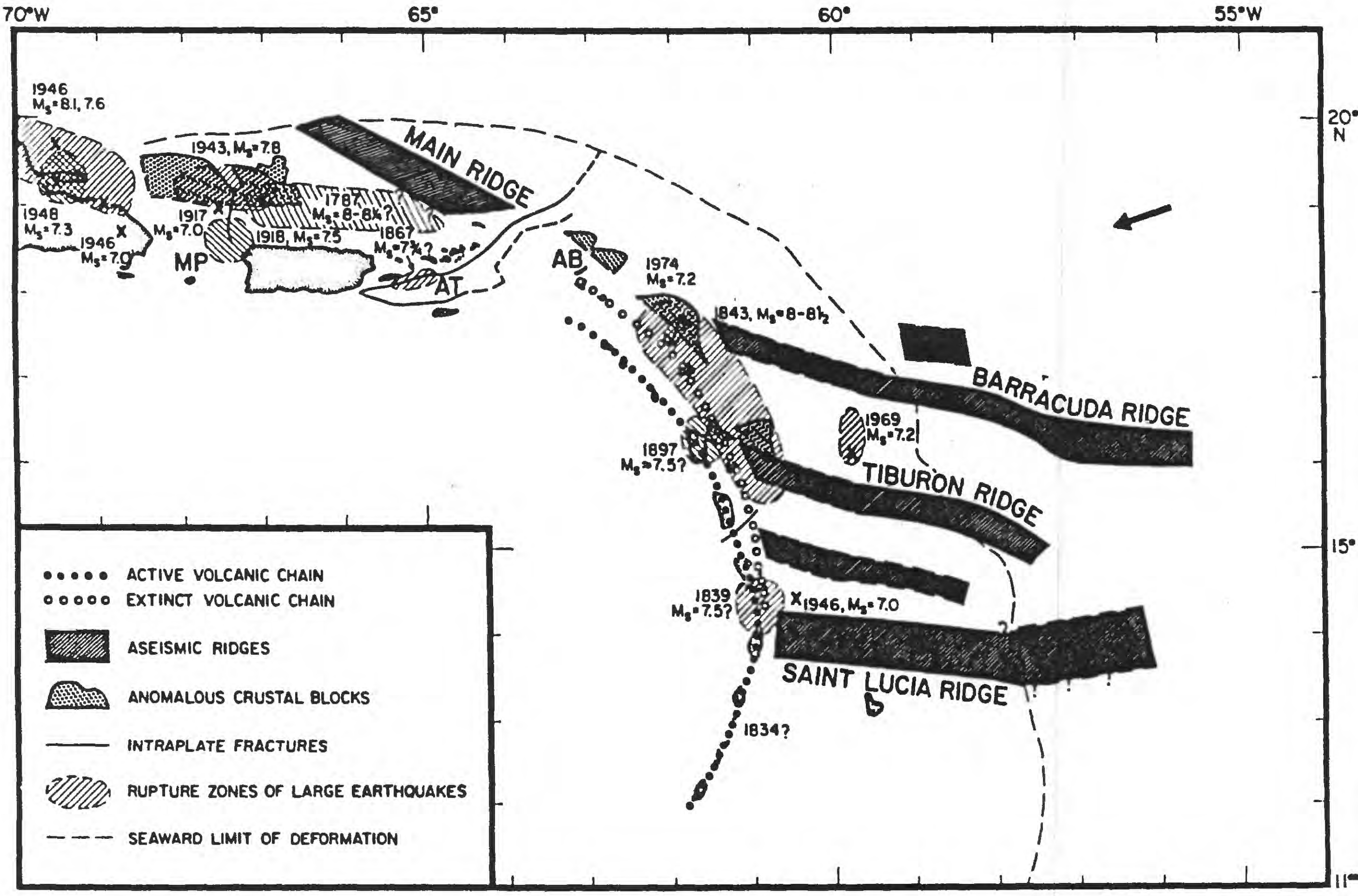

Figure 4. Rupture zones of large earthquakes ($M \geq 7$) in the eastern Caribbean and their relationship to features that bound end of rupture. Several bathymetric highs intersect the plate boundary dividing it into tectonic segments. Rupture during the 1787 event may have been limited by the Main Ridge and the features near Mona Passage (MP). Three anomalously shallow portions of the forearc (stippled areas) may be either exotic blocks accreted to the inner wall of the trench or blocks uplifted by the subduction of aseismic ridges. The large block northwest of Puerto Rico represents a part of the Bahama Bank that has been accreted to the Caribbean plate in the last few million years. AT, Anegada Trough; AB, Anguilla Bank (from McCann and Sykes, 1984).

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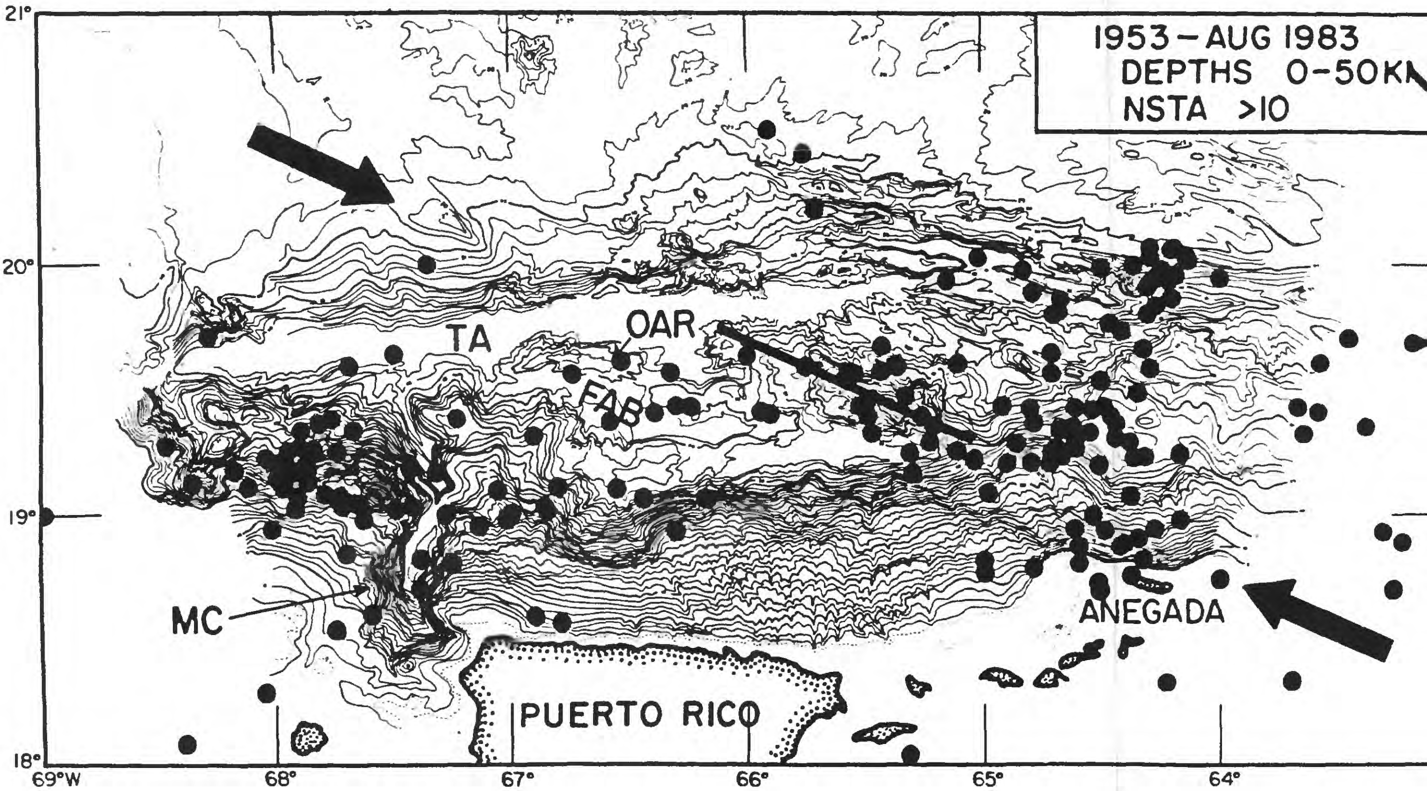

Figure 5. Detailed bathymetry of the Puerto Rico Trench north of Puerto Rico and the Virgin Islands (A. Leonardi, unpublished data). Contours are in hundreds of fathoms (1 fathom = 1.829 meters). Circles are epicenters are moderate-sized shocks from 1953 to 1983 with depths less than 50 kilometers. Only events located using more than 10 statins are shown (nsta >10). Note the clusters of earthquakes near the bathymetric feature northwest of Anegada and near the Mona Canyon (MC). The great earthquake of 1787 probably ruptured a fault segment bounded by these two regions of enhanced seismic activity. Arrows and heavy line lie along strike of Main Ridge. TA is axis of Puerto Rico Trench; OAR is Outer ARc Ridge, a feature composed of sediments deformed by the WSW motion of the North American plate; FAB is a basin of undeformed sediments.

With the advant of instrumental seismic recording (about 1900) information for large earthquakes becomes more complete. The largest shocks of this century (1918, M = 7.5; 1943, M = 7.75) occurred off the northwest coast of Puerto Rico, in the vicinity of the Mona Passage (Figure 4). Instrumental locations of small, more frequent shocks over the last 35 years have allowed a more precise identification of possible causative faults and the distribution of seismicity in general (Sykes and Ewing, 1965; Molnar and Sykes, 1969; Sykes et al., 1982).

Based on the record of historic earthquakes, Kelleher et al. (1973) defined segments of the Caribbean plate boundary most likely to produce large earthquakes in the near future. McCann et al. (1979) and McCann and Sykes (1984) further refined these estimates. They estimate a high seismic potential for a major fault in the Puerto Rico Trench north of Puerto Rico and the Virgin Islands. Recently, work by numerous other authors has helped to define the nature of the main seismic zone extending along Puerto Rico and the Virgin Islands, and to elucidate the relative motion between major tectonic blocks (Minster and Jordan, 1978; Murphy and McCann, 1979; Ascencio, 1980; Frankel, 1982).

This report integrates previous results with new data available from the region south of the islands and presents preliminary estimates of likely earthquake locations and sizes of strong earthquakes.

The conclusion of this report is that, while great earthquakes ($M \geq 7.75$) will occasionally occur in the Puerto Rico Trench 50 to 100 km to the north of the islands, the historic record and regional tectonic framework suggest that major shocks ($M \simeq 7$-7.5) may occur on intraplate faults close to the islands just as frequently. This conclusion, based on a longer historic record than previously available as well as analysis of data from local seismic networks and marine seismic programs, should be taken as a plausible working hypothesis to be refined by further investigations. Clearly more work in several lines of research is needed before definitiave conclusions can be made.

Earthquakes and Structures Offshore

Puerto Rico Trench

The Puerto Rico-Virgin Islands (PRVI) platform is bounded north and south by two deep-sea trenches; to the north the Puerto Rico Trench, to the south the Muertos Trough. The most prominent offshore structure is the west-striking Puerto Rico trench (Figures 1 and 5). Its axis lies at a depth of 8 km about 100 km north of the Puerto Rico-Virgin Islands platform. Here the North American plate moves WSW underneath the sedimentary cover at the northernmost edge of the PRVI platform (Figure 5). The North American plate, as delineated by microearthquakes, dips southerly from the trench, reaching depths of 70 to 150 km beneath the islands (Figures 2 and 3). The shallow-dipping fault zone just to the south of the trench is likely to produce earthquakes with magnitudes as large as 8 to 8.25 (see dotted in Figure 5). In the last 35 years numerous shocks, though moderate in size, occurred in the vicinity of the trench. Most of these shocks are found beneath its south wall; there are two particularly active regions--one where the Mona Canyon meets the trench northwest of Puerto Rico, and the other near where the Main Ridge intersects the easternmost Virgin Islands (Figures 4 and 5).

A broad cluster of seismicity near the Virgin Islands occurs in a triangular region with each side about 100 km long (Figure 5). Seismic activity immediately to the west of this cluster is low. This quiet zone is also similar in structure to classicial subduction zones where rupture during occasional large earthquakes is separated by long periods of seismic quiescence. In contrast, the region typified by high seismic activity of moderate-size shocks lies beneath an anomalous submarine feature on the North American plate, the main ridge. Local network data shows that these earthquakes occur within the PRVI platform, within the downgoing North American plate, as well as the zone of contact between the two plates.

The cluster of activity NW of Puerto Rico lies near a submarine bathymetric high to the west of Mona Canyon. This feature, other submarine highs near it, and the narrow, deep Mona Canyon, are part of a complex tectonic element on the inner wall of the Puerto Rico trench. The geologic history of these features suggest that they are pieces of the Bahama platform carried into the region by the North American plate. Little is known about the details of the distribution of the shocks in this region.

Mona Passage

The regions east, west, and south of Puerto Rico and the Virgin Islands include many complex structures. Some of the structures off the west coast of Puerto Rico are subtle, complex, and difficult to interpret with currently available data. Down-dropped blocks (grabens) striking north or northwesterly are the most prominent features of this region; they extend from the Muertos Trough to the south and from the Puerto Rico trench in the north (Figure 6).

The most prominent of these grabens is the Mona Canyon. A destructive earthquake in 1918 (M = 7.5) probably occurred on one of the faults bounding this canyon (Reid and Taber, 1919). As a destructive seawave accompanied this earthquake, a significant vertical displacement of the seafloor must have occurred and the depth of the shock must have been one of fairly shallow depth, i.e. the upper 40 km. The canyon to the south is a more subtle feature, being less clearly defined bathymetrically than the Mona Canyon. Nonetheless its dimensions approach those of Mona Canyon. Both features should be considered likely sources for strong earthquakes as active faults are observed in seismic reflection records near both features although such shocks may be more frequent and larger near the prominent Mona Canyon.

The grabens do not intersect, but rather terminate against a shallow platform characterized by WNW trending structures. These structures appear to be submarine extensions of the Great Southern Puerto Rico fault zone. This shallow bank is structurally complex, and an estimate of the maximum size earthquake likely to occur there is difficult to determine with existing data.

Muertos Trough

South of Puerto Rico and Saint Croix lies the Muertos Trough. It is probable that, like the Puerto Rico Trench, it accommodates the convergence between two blocks. Along much of this trough the floor of the Caribbean Sea moves underneath the massif of Puerto Rico. So the "rigid" block upon which Puerto Rico and the Virgin Islands lie is at most 300 kilometers wide in the north-south direction and overrides converging seafloor from both north and south. Based on our knowledge of the seismic history, motion along the Muertos Trough appears to be a small fraction of that near the Trench to the north. So Puerto Rico, in fact, is perhaps not an integral part of the Caribbean plate (although nearly so), but is rather a smaller plate or block, separating the larger plates.

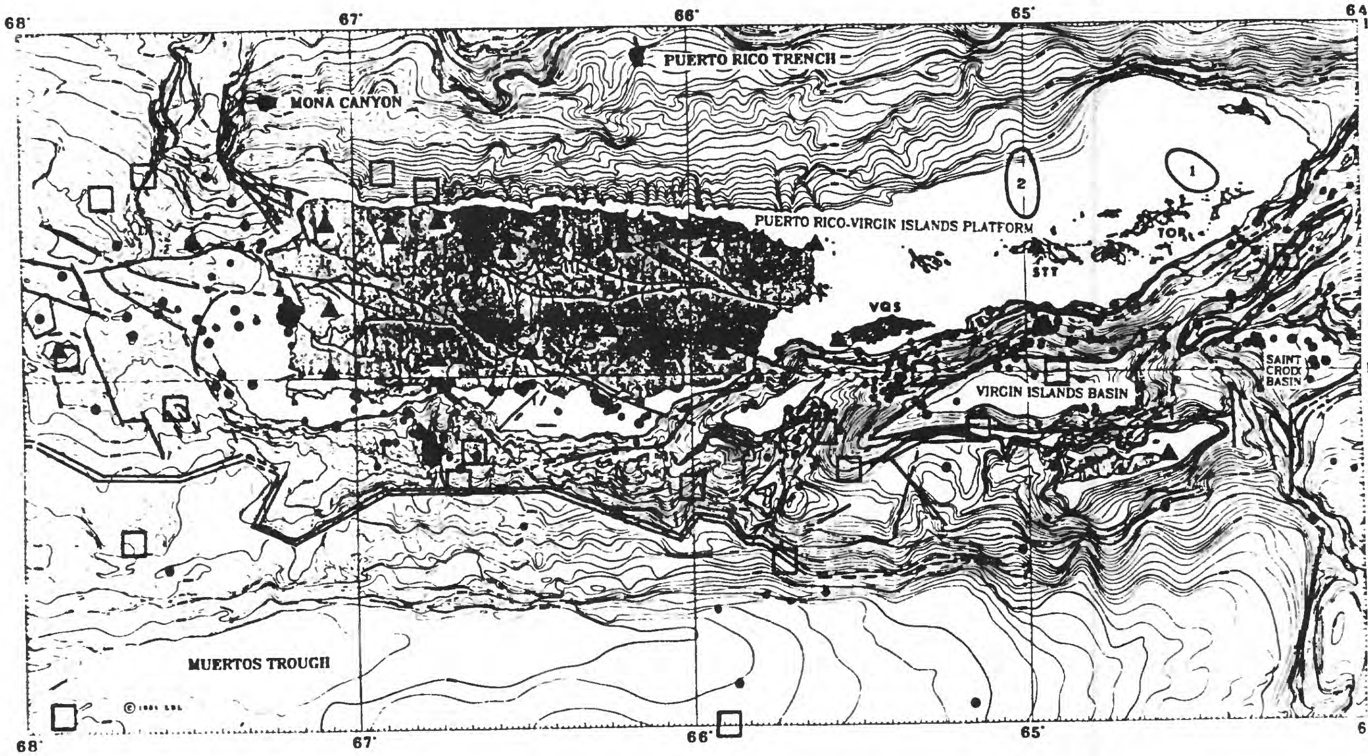

Figure 6. Major, recent tectonic features near Puerto Rico and the Virgin Islands. Contours, showing depth to seafloor in meters, delineate major morphologic features in the offshore region (from Trumbull, 1981). Puerto Rico and the Virgin Islands lie on a long, shallow platform. Saint Croix lies on a narrow bank separated from the PRVI platform by a major basin. The width of the shallow platform off Puerto Rico is highly variable, as is the slope down towards the axis of the Muertos Trough. Closed triangles are stations monitoring microearthquakes (i.e. Puerto Rico Seismic Network). Closed circles are locations of shallow microearthquakes (depth$\leq$ 50 km) south of 18.6 degrees N. east of 66 degrees W locations are from catalog of LDGO network; events occurred during the period 1977-1982; large circles have magnitudes m$\geq$ 2.5, smaller circles represent smaller events; only events reported by 5 or more stations occurring south of the PRVI platform are shown. Events west of 66 degrees W are from catalog of early Puerto Rico network as reported by Dart et al. (1980); only offshore events are shown. Large circles are events with magnitudes m$\geq$ 2. Regions labeled 1 and 2 on PRVI platform are shallow, seismically active faults noted by Fischer and McCann (1984) (see figure 3). Open squares are locations of moderate-sized shocks (M$\geq$ 4) as reported by Sykes and Ewing (1965) and NEIS. Double line south of Puerto Rico is probable southern limit of crystalline rocks of Puerto Rico block. Solid lines are active faults, identified in single-channel seismic reflection records, and their continuation along the strike of obvious morphologic features. Data is from Lamont-Doherty ships VEMA and CONRAD and data reported by Garrison (1972). Beach and Trumbull (1981) and Rodriguez et al. (1977). Single, dashed lines are morphologic features that appear to be fault controlled. Junctures of complex fault systems are found east and west of the Virgin Islands Basin. Northerly striking faults from the Mona Canyon and a smaller graben west of southwestern Puerto Rico are truncated by a WNW trending set of faults.

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Recent sediments on the slope south of Puerto Rico are disturbed by tectonic movements. This slope can be segmented into three regions based on seafloor morphology. In the southwest, the shelf varies in width and the slope is cut by numerous canyons. The central region has a broad shelf, south of which lies an easterly trending ridge-trough pair. The southeast region has a very narrow shelf; it slopes steeply into one of three basins south of the Virgin Islands. This basin is part of a network of complex structures primarily composed of uplifted and down-dropped blocks (horsts and grabens) bounded by short-intersecting fault segments. Of the three morphologic regions south of Puerto Rico, the western two appear to be more coherent blocks bounded by long faults. Therefore, these segments are more likely to generate major ($M \simeq 7.8$) earthquakes, albeit with a long repeat time, as faults segments are probably longer than those to the east. These faults may be nearly horizontal, being associated with motion between Puerto Rico and the seafloor of the Caribbean, or at high angles to the horizontal, representing motion with a part of the Puerto Rico block. In the eastern region earthquakes would probably be smaller in size because any falt breaking during a shock is either short or cut by another fault (Mogi, 1969).

The slope south of Saint Croix is markedly different in character than that south of Puerto Rico. It has a relatively uniform slope from the shallow shelf to the flat floor of the Caribbean Sea. Seismic reflections records of this region suggest a more stable environment than that near Puerto Rico, although high sedimentation rates in this region may mask the effects of slow tectonic movements. This margin can be treated as a coherent, relatively stable block, perhaps attached rigidly to the Caribben seafloor. Hence, it is clear that seafloor morphology, suggestive of active faulting south of Puerto Rico, does not continue along the southern flank of Saint Croix. Instead, active faults appear to pass north of that island into the region near the Virgin Islands Basin, passing to the northeast off the east margin of the PRVI platform, and eventually intersecting the Puerto Rico Trench.

Anegada Passage

Steep scarps characterize the margins of the deep Virgin Islands basin, and microearthquakes are found in association with these features (Figure 6). The large earthquake of 1867 presumably ruptured one of the faults along the northern flank of the basin (Reid and Taber, 1920). Reid and Taber (1920) compared the 1918 earthquake (M = 7.5) near northwestern Puerto Rico with the earthquake of 1867. They said: "The two main shocks had about the same intensity and were felt for about the same distance, namely, 500 or 600 kilometers, and the amounts of energy liberated in the two cases were about the same." Based on their report we assign a magnitude of 7.5 to the 1867 earthquake. The largest clusts of microearthquakes, south of Saint Thomas and Vieques, may lie near the fault which broke during that shock. The relatively simple structure of the Virgin Islands basin, being bounded by long fault segments, is a more likely source of strong shocks ($M \approx 7-8$) than the more complex structures to the west. Complex features separate the Virgin Islands Basin from the smaller Saint Croix Basin. At this complex region northeasterly trending faults extending from the Puerto Rico Trench intersect the westerly trending structures characterizing the series of basins between Saint Croix and the PRVI platform. This complex junction of faults is structurally similar to the region west of the Virgin Islands Basin and therefore is likely to pose a similar earthquake hazard.

The prominent, linear features forming the edges of the ridge-trough structures north of the Saint Croix Basin may pose a hazard similar to the major faults of the Virgin Islands Basin. A large shock in 1785, strongly felt in Tortola and the Northern Lesser Antilles to the east, may have occurred on one of these faults, but the location of this shock is very uncertain (Robson, 1964).

Earthquakes and Faults Onland

The bulk of the rocks comprising Puerto Rico and the Virgin Islands were deposited from 110 to 45 million years ago during a period of sustained convergence between the Caribbean and North American plates. During this time period, and the following 20 million years, two major fault systems, the Great Northern and Southern Puerto Rico fault zones were active, displacing rocks on either side in a left-lateral sense (Briggs, 1968, Seiders et al., 1972). These faults, clearly visible today in the morphology of Puerto Rico, extend into submarine areas to the northwest and southeast of the island, may be

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associated with the formation of the Mona Canyon and Virgin Islands basin, and are the most pominent, inherited zones of weakness in the platform on which Puerto Rico and Virgin Islands lie.

Geologic mapping suggest that little, if any, motion has occurred on these faults in the last 20 million years; none is documented in the last million years. Surprisingly, seismic activity is observed in association with the onland portions of these faults, espeically in Southwest Puerto Rico (Ascencio, 1980). As offshore expressions of these faults appear to be active, some of the onland faults may also be active. The apparent lack of recent faulting observed on land may result from high erosion rates coupled with low rates of slip of the faults. More mapping is needed to carify the relationship between onshore and offshore faults and to identify recent faulting onland if it exists. Nevertheless, most of the recent deformation associated with plate movement appears to occur in the offshore regions. As noted before, deformed sediments and displaced blocks of seafloor are found off all portions of the Puerto Rico-Virgin Islands platform.

Expected Long-term Seismic Activity

The observations presented above provide a tectonic framework in which to estimate the likely sources of strong earthquakes. The conclusions that follow should not be taken as definitive, but they do suggest a high level of hazard for the region; more research is needed to further define the hazard. The spatial distribution of recent seismic activity is remarkably similar for events in the magnitude range 2.0 to 4.0 recorded in the last 10 years and magnitudes 4.0 to 6.0 recorded in the last 30 years. Events during the first half of the century also show a similar pattern, but their locations are less precise (Sykes et al., 1982). Seismic activity is high along limited segments of the Puerto Rico Trench. These active segments are separated by zones of relatively little seismic activity. The relatively long period of time over which this consistent distribution of seismicity is observed (up to 80 years) and the ability to correlate the level of seismic activity with features on the inner wall of the trench strongly sugggests that the distribution of seismicity is not random, but rather is associated with long-term tectonic processes occurring near the plate boundary.

The Mona Canyon region and the Main Ridge are anomalous features that appear to concentrate stress along the major thrust faults in the Puerto Rico Trench. They are presntly seismically active and, because they are stress concentrators, are likely to be sites of large earthquakes ($M \geq 7$) more often than the large, seismically quiet region that separates them. This quiet region is probably the only region near the PRVI Platform capable of producing a great earthquake with a magnitude greater than 8.0. In the eastern, western, and southern regions off the PRVI Platform, some seismic activity correlates with known or suspected submarine faults. Seafloor morphology varies in these regions and therefore the margin can be subdivided into regions based on an apparent density of faulting. Figure 7 is a recent estimate of the long-term seismicity activity for the northeastern Caribbean. Neither figures 7 or 8 should be considered predictions of earthquakes. Figure 7 estimates the likely long-term character of seismicity activity indicating the likely maximum size of an earthquake in a regin, given the tectonic framework provided above.

The main seismic zone in the Puerto Rico Trench is characterized by variations in the expected frequency of moderate and large earthquakes. Those portions of the PRVI Platform interacting with the Main Ridge to the east of Puerto Rico, as well as the feature at the western end of the Puerto Rico trench may be expected to experience relatively short repeat times for moderate and large shocks. The intervening segment of smooth seafloor may tend to be relatively quiescent for shocks of similar magnitudes. This zone of little seismicity, as well as the adjacent active areas is likely to experience great earthquakes with rupture zones about 200 km (?) long and magnitudes about 8 to 8.25 perhaps every 200 years. An example of such an earthquakes is that of 1787. The estimated rupture lengths and magnitudes are probably maximum values, the repeat time is a minimum value. Maximum event size is likely to be limited by the distances between the seismically active areas on the main fault zone ($\sim$ 200 km).

The Mona Canyon west of Puerto Rico as well as the coherent blocks south of west and central Puerto Rico may generate shocks as large as 7.5 to 8.0. A graben southeast of Mona Island and the region south of eastern Puerto Rico and northeast of Saint Croix may generate shocks of magnitude 7.0 to 7.5. The

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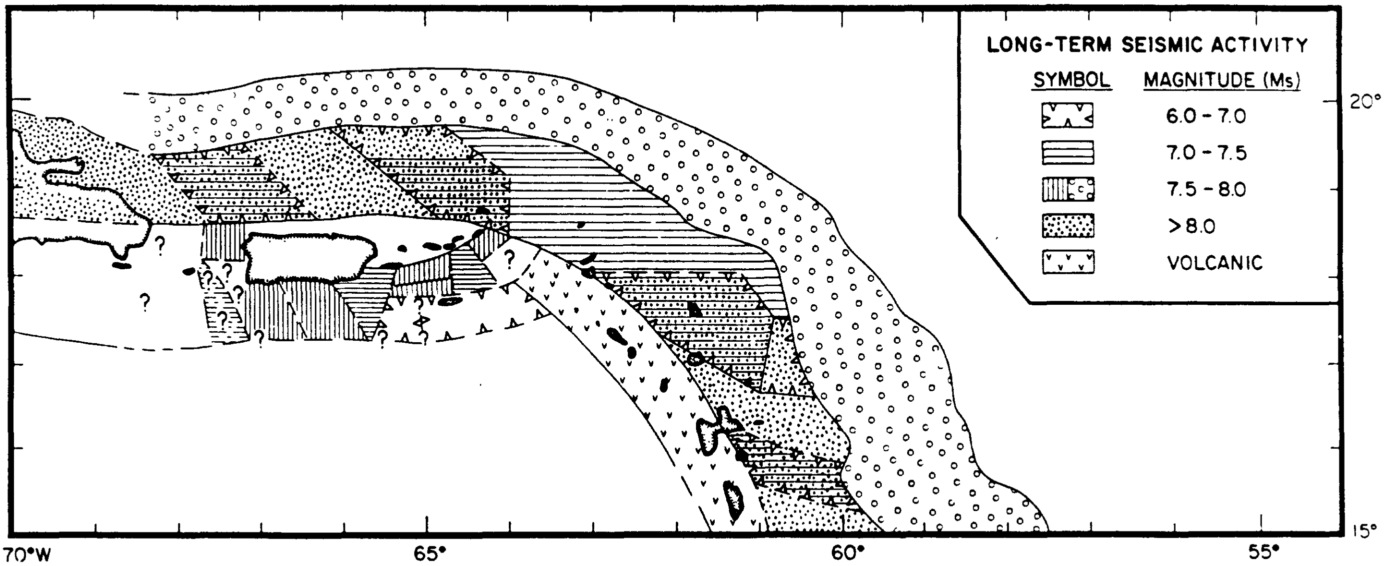

Figure 7. Estimate of long-term seismic activity of shallow focus along the Caribbean - North American plate boundary. Moderate-sized events ($M \simeq 6-7$) are expected to be more frequent along those portions of the seismic zone where bathymetric highs have entered the trench. Large shocks ($M \simeq 7.5-8.0$) may occur occasionally, but with long repeat times (i.e. thousands of years) in the deeper parts of the trench as the North American plate flexes to descend beneath the Caribbean plate. Large shocks can be expected to occur infrequently along the Anegada Passage; events with similar sizes may occur in the region of the Mona Canyon off NW Puerto Rico. Major blocks with some, as of yet poorly defined, seismic potential also exist along the southern flank of Puerto Rico. In total, the region including the Anegada Passage, Muertos-Trough and Mona Passage, but excluding the Puerto Rico Trench, may produce large shocks as frequently as the Puerto Rico Trench. Great shocks ($M \geq 7.75$) may rupture large sections of the fault zone south of the Puerto Rico Trench. The extent of rupture in great events would probably be limited by tectonic barriers such as those that may have delimited rupture during the large shock in 1787. Great shocks may not occur along the plate boundary in the transition region from normal underthrusting to oblique slip, where the Anegada trough intersects the subduction zone. Areas of seismic potential for great shocks appear to exist along the northern Lesser Antilles and to the north of Puerto Rico (from McCann and Sykes, 1984).

relatively large, steep walled Virgin Islands Basin and the linear structures leading to the Puerto Rico Trench from this basin may generate magnitude 7.5 to 8.0 earthquakes. Any given fault segment not on the main plate boundary near the Puerto Rico Trench may produce strong earthquakes every few thousand years rather than hundreds of years. The prominent Mona Canyon and Virgin Islands Basin, having produced shocks in historic times, may be more active than other, more subdued features. The larger number of off-plate boundary faults in this region suggests that, on average one fault may break every few hundred years.

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Estimates of Seismic Potential

Estimates of the likelihood that a major fault will experience a large earthquake (seismic potential) can be made by use of the historic record and inferences of the likely sites of future shocks based on regional tectonics. McCann et al. (1979) estimates seismic potential based on the time elapsed since the last large earthquake. Regions of greatest seismic potential are those with the greatest elapsed time since the last large shock. McCann and Sykes (1984) revised those estimates (Figure 8). Better knowledge of the current tectonic deformation will further refine these results. Although more precise determinations of seismic potential can be made in regions with numerous historic or prehistoric events, the general lack of historic detail for this region prohibits the use of such techniques.

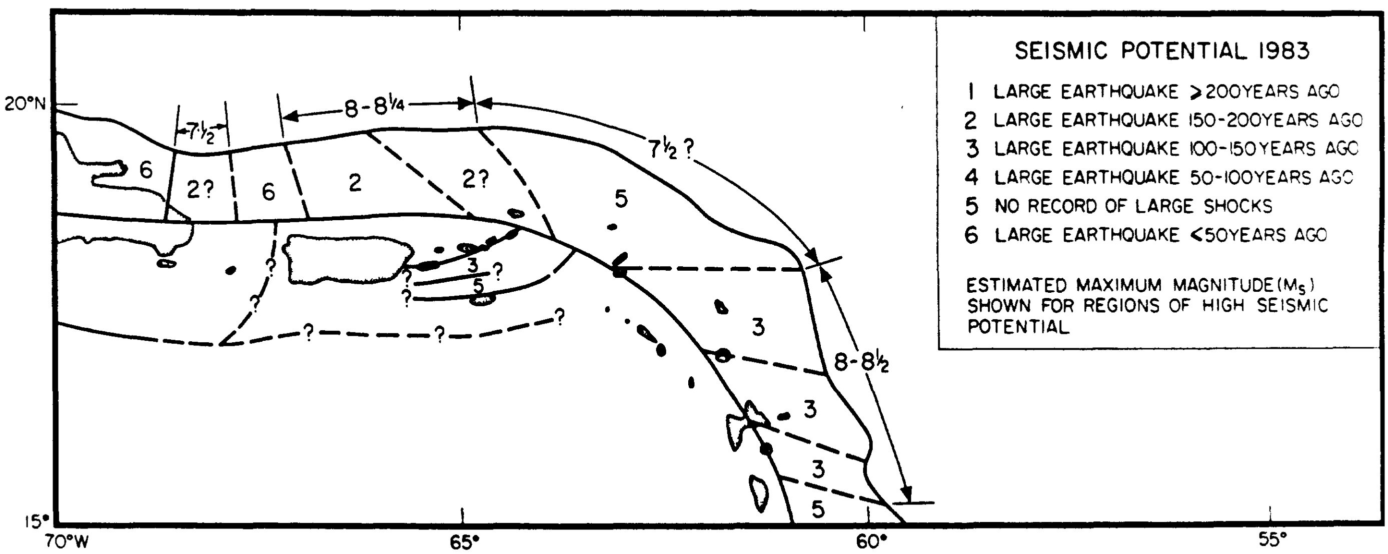

Figure 8. Estimate of seismic potential for the northeastern Caribbean. Potential for large or great shock to occur is estimated by the time elapsed since the last large earthquake. This method assumes repeat times throughout the region are about the same. Magnitudes of future shocks are estimated for those regions of high potential. Question markes (?) denote uncertainty in boundaries of seismic zone or level of seismic potential.

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We implicitly assume that the repeat times for shocks of the same size are approximately the same. Whereas this may be true for regions where smooth seafloor abuts the Puerto Rico Trench, those regions interacting with features such as the Main Ridge and the features near the Mona Canyon are likely to have shorter repeat times for significant shocks ($6 < M < 7.5$). Most of the regions off the main plate boundary (i.e. Puerto Rico Trench) appear not to have experienced a large shock in historic times. The two that have, the Mona Canyon and the Virgin Islands Basin are the largest, most prominent features. Hence, because of a lack of historic information, it is probably too early to extend the seismic potential analysis, intended for more simple structures, into all of this region.

McCann et al. (1979) placed the Puerto Rico-Virgin Islands region in a neutral category for seismic potential. At that time it was not clear that this region was capable of producing large interplate shocks. Now with better understanding to the tectonic structure of the region, and with a more complete historic record, it is clear that this region does have the potential to produce strong and great earthquakes.

CONCLUSION

The earthquake of 1787 appears to have originated in the Puerto Rico Trench, 50 to 100 kilometers to the north of the islands. While the probable magnitude of this event ($M = 8 - 8.25$) makes this shock the largest in the historic record, more damaging quakes of somewhat smaller magnitude ($M = 7 - 8$) occurred much closer to land (10-50 km). A major shock on one of the many faults nearer to the islands may, on average, occur just as frequently as the great earthquakes in the Puerto Rico Trench. The main earthquake hazard in this region, therefore, may come not from great earthquakes to the north, but rather from major ones occurring closer to land.

The information collected in the last decade has clarified our understanding of the nature of the seismic zone near Puerto Rico and the Virgin Islands. Numerous active faults are located in the offshore region; some may extend onshore. The framework developed here represents a plausible working hypothesis for the evaluation of the earthquake hazard of the region. More research is needed to validate this hypothesis. Identification and detailed mapping of active faults, focal mechanisms and more precise locations of small earthquakes, more detailed investigations of the historic record and collection of geodetic data are a few of the areas of research deserving expanded effort.

ACKNOWLEDGEMENTS

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REVIEW OF RECENT ADVANCES IN SEISMOLOGICAL ASPECTS OF EARTHQUAKE HAZARDS

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INTRODUCTION

Growth of Observational Material on Seismic Shaking

Reduction of earthquake hazards depends in the most fundamental way on our understanding of earthquakes. We must understand the causal tectonic forces and deformations, the seismic fault source, the rupture mechanics, the effect of rock properties on the waves, and the modifications to shaking produced by surficial soils and topography.

All these matters have been significantly clarified in recent years[1] although, of course, many problems remain to be solved. What is the scaling of ground motions from moderate (magnitude 6 to 7) earthquakes to great (magnitude 7.5 to 8.5) earthquakes? What is the best way to parameterize ground motions for risk mapping and engineering design? The basic clarification has come because many more records of strong ground motions are now available - both in the free field and in structures. In the March 3, Chile earthquake (magnitude 7.8) over 20 clear accelerograms were recorded throughout the area of damage, compared with only one recording in the 1971 Chile earthquake.

Of importance are the recent ground motion recordings from specially designed arrays of accelerometers, particularly in California and in Taiwan (SMART 1). Such arrays measure the shaking variability in time and space. The data

[1]Note. The present theoretical position is treated in the revised text, K.E. Bullen and B. A. Bolt, "Introduction to the Theory of Seismology," Cambridge University Press, 1985.

set is weak, however, on measurements of foundation-structure and sub-surface (down-hole) motions.

Interpretation of Strong Ground Motion

Nowadays, an experienced seismologist can go a long way in interpreting the wave pattern observed on a seismogram of a distant earthquake. This is because the assumptions of linear elasticity hold and ray theory can be used. The source can usually be approximated by a point or small sphere and at large distances the wavefronts are effectively planar, so that motions can be separated into longitudinal and transverse components.

There are, however, complications which are common. When an elastic wave encounters a boundary which separates rock of different elastic properties, it will, like sound and electromagnetic waves, undergo reflections, refraction, and diffraction. Within an homogeneous, isotropic, elastic medium, there are two body waves which propagate. The fastest is the dilatational wave, called the P or primary wave, and the slower is the shear wave, called the S or secondary wave. When such body waves encounter a boundary, a conversion between these types occurs, with either an incident P or S wave yielding a reflected P and S wave as well as a refracted P and S wave. In addition, the effects of rapid variations in the rock structure can be often observed in the form of scattering of the waves, producing seismic energy in regions which, on simple ray theory, there should be a shadow.

The free surface of the Earth permits the existence of additional seismic waves of surface wave type. Rayleigh waves have particle motions near the surface of the ground that are elliptical in a vertical plane. In addition, when layers are present near the surface or there is a gradient in elastic properties, horizontally polarized surface waves, called Love waves, also exist. At considerable distances from the source, the P, S, Rayleigh, and Love waves can be seen on seismograms clearly separated, according the their respective velocities and, as well, there are often waves such as PP, SS, and so on which correspond to reflections of these waves at internal boundaries.

In addition to usual phases mentioned above on conventional seismograms of distant or small earthquakes, there are certain types of seismic waves (often pulselike) that are observed specially in the near field of the seismic source. These include "stopping phases" which are due to the intermittent stopping of the dislocation front and the final (sudden) cessation of the rupture. A special form of this stopping phase is called a "breakout" phase, which arises from the generation of a pulse when the rupture reaches the free surface of the Earth.

A full elementary (non-mathematical) treatment of the physics and types of seismic waves and their relation to faulting can be found in B. A. Bolt, "Earthquakes - A Primer," W. H. Freeman, New York, 1978, or B. A. Bolt, "Terremotos," Reverte, Barcelona (in Spanish), 1981. Helpful background material on earthquake hazards and ground shaking related to Puerto Rico is given in the papers by W. McCann and W. Hays in "A Workshop on Geologic Hazards in Puerto Rico," Open File Report 84-761, U.S.G.S., 1984.

Present Earthquake Source Models

In 1964 and 1966, N. Haskell developed a model "in which the fault displacement is represented by a coherent wave only over segments of the fault and the radiations from adjacent sections are assumed to be statistically independent or incoherent." The physical situation in this model is that the rupture begins suddenly and then spreads with periods of acceleration and retardation along the weakly welded fault zone. In this model, the idea of statistical randomness of fault slip or "chattering" in irregular steps along the fault plane is introduced.

More recently, Das and Aki (1977a, b) have considered a fault plane having various barriers distributed over it. They conceive that rupture would start near one of the barriers and then propagate over the fault plane until it is brought to rest or slowed at the next barrier. Sometimes the barriers are broken by the dislocation; sometimes the barriers remain unbroken but the dislocation reinitiates on the far side and continues; sometimes the barrier is not broken initially but, due to local repartitioning of the stresses and possibly nonlinear effects, it eventually breaks, perhaps with the occurrence of aftershocks.

The elastic rebound model involving a moving dislocation along a fault plane over which roughnesses of various types are distributed stochastically is thus the starting point for the interpretation of near-field records. Based on this model, there have been recently quite a number of attempts to compute synthetic seismograms from points near to the source and comparisons have been made with observations.

From geological evidence, there are, of course, different kinds of fault ruptures. Some involve purely horizontal slip (strike-slip); some involve vertical slip (dip-slip). It must be expected that the wave patterns generated by fault mechanisms of different kinds will be different to a larger or lesser extent, due to the different radiation patterns produced.

The theory must also incorporate effects of the moving source. These Doppler-like consequences will depend on the speed of fault rupture and the direction of the faulting (Boore and Joyner, 1978). The physical problem is analogous (but more difficult) to the problem of sound emission from moving sources. The problem can be approached both kinematically and dynamically. The acoustic problem shows that in the far field the pressure is the same as when the source is at rest. However, in the near field, the time dependence of both frequency and wave amplitude is a function of the azimuth of the site relative to the moving source.

We now summarize the main lines of approach to modeling mathematically the earthquake source. The first model is the kinematic approach in which the time history of the slip on the generating fault is known a priori. Several defining parameters may be specified, such as the shape, duration, and amplitude of the source (or source time function and slip), the velocity of the slip over the fault surface, and the final area of the region over which the slip occurred. Theoretically, a Green's function representation is usually used to calculate the resulting displacements of the medium. Green's functions for the various classifications of faulting have been constructed, and numerous theoretical papers using this approach have been published. The process is a kind of complicated curve fitting whereby the parameters of the source are varied in order to estimate by inspection the closeness of fit with distant radiated seismic waves. Once the seismic source is defined by this process, using distant recordings, then the near-field parameters can be used to calculate the ground motions near to the source for engineering purposes.

A second approach is to use the differential equations involving the forces which produce the rupture. This dynamic procedure has received considerable emphasis lately. The basic model is a shear crack which is initiated in the pre-existing stress field and which causes stress concentrations around the tip of the crack. These concentrations, in turn, cause the crack to grow. Many of the articles on this subject have been built on the work of Kostrov (1966). For example, Burridge and Willis (1969) obtained analytic expressions for particle accelerations in given directions from a uniformly growing elliptical crack, although they did not include the effect of crack stoppage. (This unrealistic boundary condition is included in most work of this kind.) The key to the crack problem seems to be in modeling the physical processes of the typical crack where there is interaction between the rate of crack growth, the criterion of fracture, and the stress accumulation. Most of these studies on dynamic shear cracks are concerned primarily with the actual rupture process, and so the crack is assumed to be imbedded in an infinite homogeneous medium. Studies more concerned with the seismic waves that are recorded in the field need a numerical approach, such as finite elements or finite differences, to handle realistic structural conditions.

The studies mentioned under kinematic and dynamic models are built around the elastic rebound theory of slip on a fault. There are, however, more general studies that take a less specific view of the earthquake source. Recent work by Backus (1977a, b), for example, has taken up the important idea of the uniqueness of the various source descriptions; the representation of an arbitrary source of seismic waves is given in terms of moment tensors. Any seismic source can, in principle, be expanded in terms of spatial moments, that of the long wave lengths compared to the fault dimensions; only the low degree terms of the expansion need to be included. Thus, for small earthquakes or far-field problems, it is sufficient to represent a seismic source in terms of a single first-degree moment of the equivalent force, which is a symmetric second-rank tensor. Then, the waves calculated can be interpreted in terms of any specific model. It turns out, however, that in practical attempts to represent thy near field in this way, higher terms give very complicated tensor components and analytic evaluation may not be worthwhile. It should be mentioned here that the scalar seismic moment (direction of force couples along the fault ignored) is given by

$$M_o = \mu AD \tag{1}$$

where A is the slipped area.

Let us now summarize the physical model for the earthquake source now generally accepted (see Figure 1). The source extends over a fault plane in the Earth which is ruptured by a series of dislocations which initiate at some point (the focus) and spread out with various rupture velocities. The dislocation front changes speed as it passes through patches of roughness (barriers on the fault). At the dislocation itself, there is a finite time for a slip to take place and the form of the slip is an elastic rebound of each side of the fault leading to a decrease of overall strain. The slip can have vertical components, as well as horizontal components, and can vary along the fault itself. The waves are produced near the dislocation front due to the release of the strain energy in the slippage.

This model resembles in many ways radio waves being radiated from a finite antenna. In the far field, the theory of radio propagation gives complete solutions for the reception of radio signals through stratified media. However, when the receiver is very near to the extended antenna, the signal becomes jumbled due to the finiteness of the source and interference through end effects.

The main parameters in the model are:

Rupture length	L
Rupture width	W
Fault slippage (offset)	D
Rupture velocity	V
Rise time	T
Roughness (barrier) distribution density	$\phi(x)$

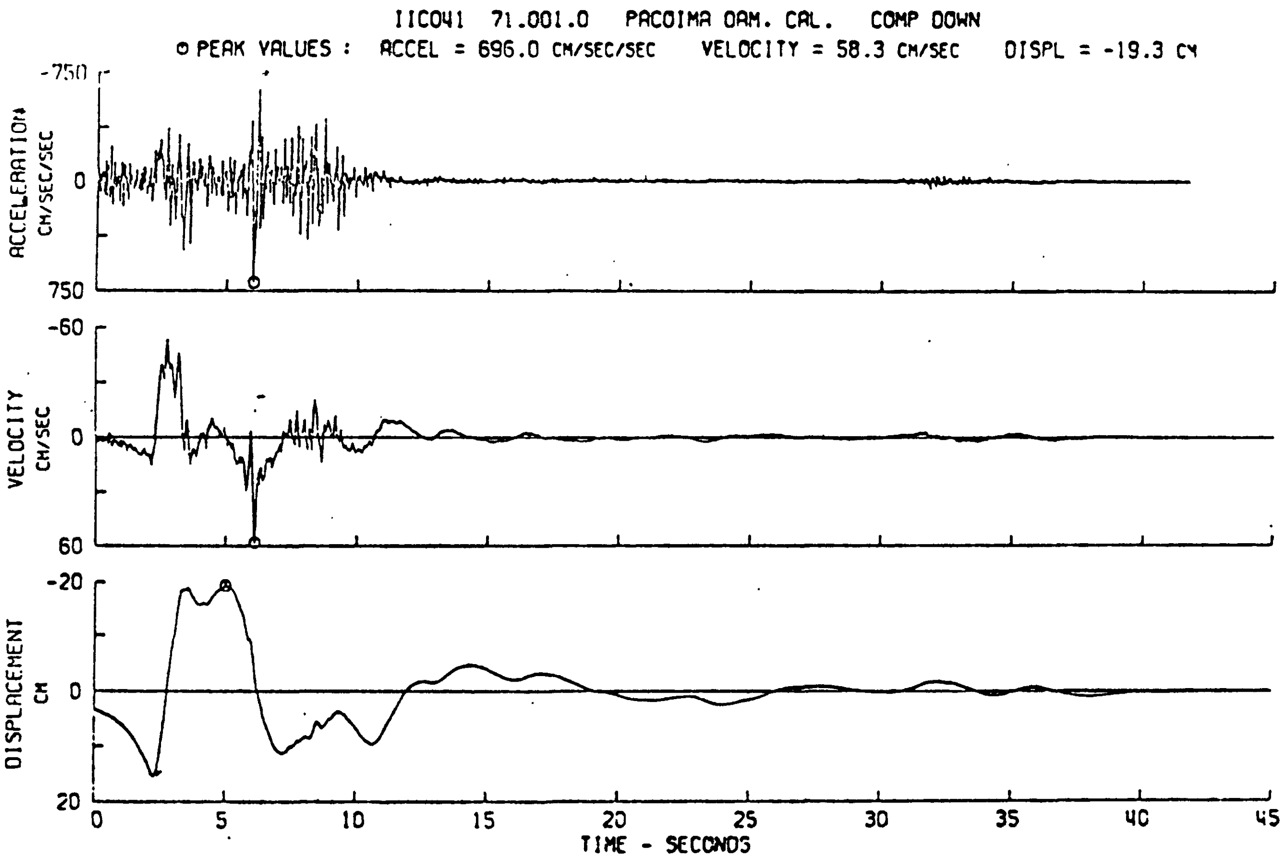

Figure 1.--San Fernando Earthquake of February 9, 1971.

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The main work in theoretical seismology on source properties today is to determine which of these parameters are essential, whether the set is an optimal one, and how best to estimate each parameter from both field observations and analysis of the seismograms made in both the near and the far field.

A number of papers have now been published that demonstrate that, in certain important cases, synthetic seismograms for seismic waves near their source can nnow be computed rather realistically. The synthetic motions can be compared with the three observed orthogonal components of either acceleration, velocity, or displacement at a site (see Figure 1). There remains difficulties, however, in modeling certain observed complexities and there is a lack of uniqueness in the physical formulations which lead to acceptable fits with observations.

MAJOR NEAR-FIELD PROBLEMS IN ENGINEERING SEISMOLOGY

Maximum (Peak) Amplitudes

For some time, a key scaling parameter in the specification of ground motion for engineering purposes has been the maximum (peak) acceleration. These peak values are used to scale not only the seismograms (time histories), but also to anchor the high-frequency end of ground response spectral curves. The methodology was evolved in the 1960's when there were few strong-motion records for large to moderate earthquakes available and the maximum amplitudes seen on accelerograms were about 0.3g to 0.5g.

The situation has not changed for several reasons. First, many instrumental measurements have now been obtained of peak accelerations greater than 0.5g. Indeed, in the Imperial Valley, California, earthquake of October 15, 1979, a high-frequency peak accleration of about 1.7g was observed in the vertical direction and on the Pacoima record (see Figure 1) a peak horizontal high-frequency amplitude of 1.2g was measured. At the same time, it is observed that these high-acceleration values often are represented on the record by only one or two spike-like features. In other words, they are not representative estimates of the accelerations which were being experienced through the strongest ground shaking. Indeed, in some cases they could be characterized as abnormal samples of a more typical frequency distribution of peak amplutides.

A second observational property has also recently come to light. Near to the source of quite small earthquakes, strong-motion instruments often record high accelerations. Well-known examples of this are the Bear Valley, California, earthquake of September 4, 1972 (M_L = 4.7, peak horizontal acceleration = 0.69g), and the Ancona earthquake of June 21, 1972, in Italy (M_L = 4.5, peak horizontal acceleration = 0.61g).

These observations of high peak accelerations at high frequencies from small-magnitude earthquakes show that raw peak acceleration taken alone can be a deceptive parameter so far as scaling ground motions for engineering purposes. Another aspect of the problem is that in synthesizing ground motions for engineering design it has been common practice to emphasize the peak acceleration parameter. For example, this procedure has been followed by the Nuclear Regulatory Agency in terms of the safe shutdown earthquake for a particular site. The procedure, of course, breaks down when it is accepted that a given peak acceleration (0.5g, say) could apply to strong ground motions of vastly different overall seismic energies and spectra.

Another difficulty with the emphasis on peak accelerations stems from the high-frequency nature of the observed peaks in almost all cases (see Figure 1). It is now realized that an engineering response spectrum can be drawn which would be anchored at the peak acceleration specified for the predicted earthquake at the site, while the spectral amplitudes at longer periods, say beyond one second, could be quite deficient for the predicted type of earthquake. For this reason, demand is growing for not just a peak acceleration as the dominant scale parameter, but also suitable scaling parameters for maximum velocity and even maximum displacement. An illustration comes from the recent Applied Technology Council's (ATC) risk maps for the United States (Donovan, Bolt, and Whitman, 1976) where the free-field ground-motion response spectra were scaled at short periods to an effective acceleration parameter and at longer periods to an effective velocity parameter. Partly for this reason, in the analyses that follow, discussion will be given not only on acceleration records, but also to their first and second integrals (i.e., velocity and displacement).

Because of the central role that has been played by peak accelerations in estimating strong ground motions, they have been correlated against a number of parameters. One of the most important is the correlation of near-source acceleration with local magnitude. Figure 2 illustrates the marked differences in estimates due to different assumptions. A few observed values from actual strong-motion records are added to Figure 2 to indicate some of the scatter of data that went into these extrapolations. The dashed line comes from a study by Page et al. (1972) in which they give peak accelerations against magnitude for very near-source distances. The curve is fixed by extrapolations back from a few earthquakes of moderate size (generally up to about magnitude 7) at distances greater than 5 km from the ruptured fault. The assumptions used lead to a curve which rises rather steeply above magnitude 6 to about 1.2g for the largest earthquakes. The second curve on Figure 2 is based on attenuation curves (by Schnabel and Seed, 1973) for peak acceleration as a function of magnitude. The different assumptions used lead to an extrapolation with almost no increase in the horizontal peak acceleration at near-source distances for magnitude above 6.5. It should be mentioned that the general physical properties of the source model (discussed above) would seem to favor the second hypothesis over the first. This is because the amount of seismic energy produced in any frequency band along the rupturing fault would be a function of the elastic properties of the rocks near the dislocation at any time, rather than the summation at a given time of energies over the whole fault plane. In the former case, the emitted wave energy is limited above a threshold while in the latter it would be greater for larger magnitude earthquakes than for smaller magnitude earthquakes. This central problem of scaling from low-magnitude to high-magnitude earthquakes remains unresolved.

Duration

The concept of the duration of strong motion at a site is a crucial one in terms of understanding the dimension of the source and also in estimating the overall energy which should be incorporated in the input ground motions for any structure.

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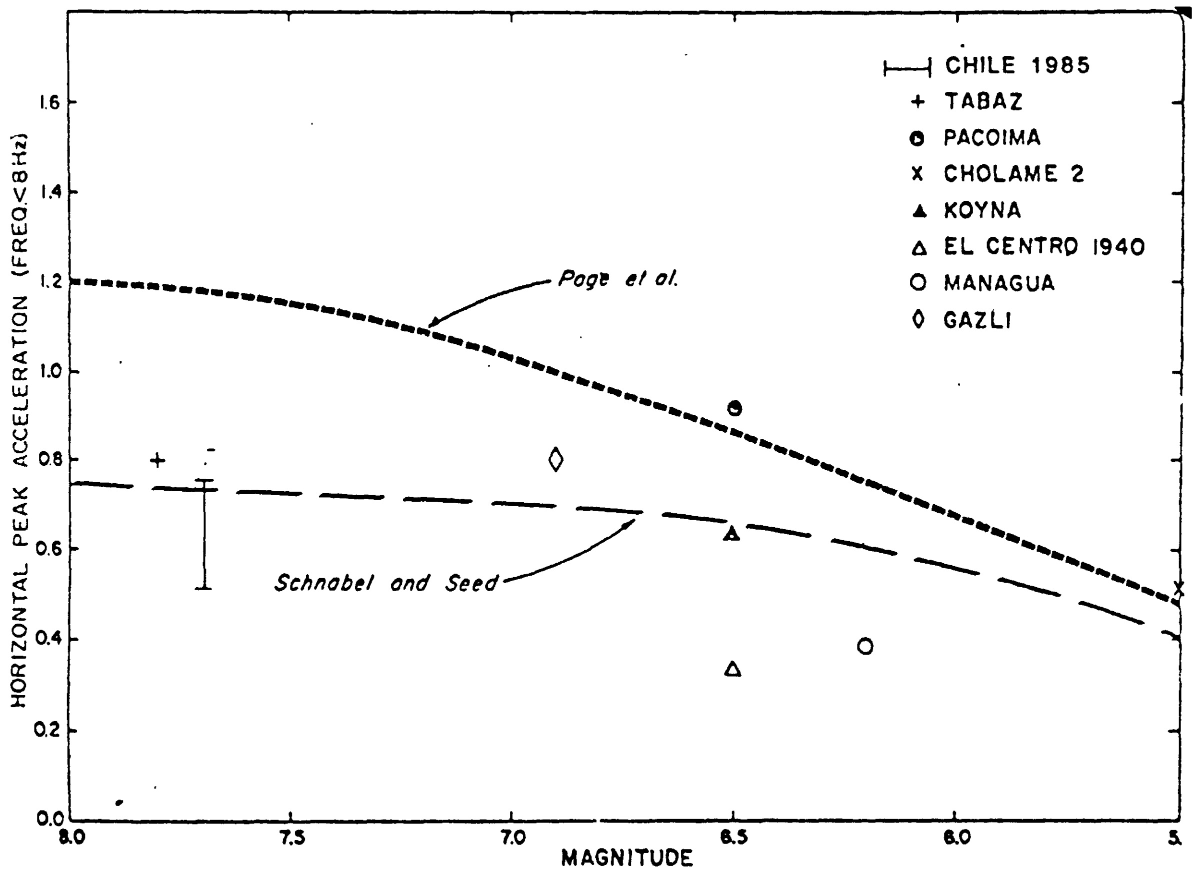

Figure 2.--Correlation between magnitude (M_L) and maximum acceleration (freq. < 8 Hz) within 10 miles of the causative fault. The curves are after Page et al. (1972) and Schnabel and Seed (1973).

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The physical model outlined above predicts that the duration, if defined in a quantitative way, will be significantly dependent upon the dimensions of the faulted surface. The waves are radiated from the moving dislocation across the full dimensions of the plane; both magnitude and seismic moment reflect this dimension. (The magnitude estimate used for great earthquakes is the surface wave magnitude which is estimated from waves of period of 20 sec or even larger, which correspond to wavelengths of 50 km or greater. These wavelengths effectively sample the whole source dimension. The seismic moment defined by equation 1 is proportional to the dimensions of the faulted area.) Considerable weight can thus be given to the duration parameter in both the interpretation of strong-motion records and in the synthesis of time histories for a particular site.

Estimates for the bracketed duration (amplitudes greater than 0.05g) can be obtained from Figure 3. Three instrumental measurements (Bolt, 1973) are shown; a recent point is from the Tabas earthquake and is particularly important since it represents a measurement of duration of strong ground accelerations in the near field for a very large earthquake. (The end of the curve in Figure 3 for the large magnitudes was originally only weakly based on felt reports from large earthquakes.)

Deviations from the mean duration curve usually arise because of multiplicity of the earthquake source and also the special side effects of layering and soil conditions.

Patterns of Arrivals - Deterministic and Stochastic

After the appropriate duration of a strong-motion record is estimated, based on the seismic moment M_0 or magnitude M_L of the earthquake, it remains to analyze the detailed patterns of groups of waves on the strong-motion record. An observatory seismologist becomes efficient at recognizing patterns of arrival of P and S body waves and surface waves when working with seismograms from distant earthquakes or small local ones. The question is to what extent can similar sets of stable patterns be recognized for strong ground motions in the near field. Let us consider four aspects of the problem.

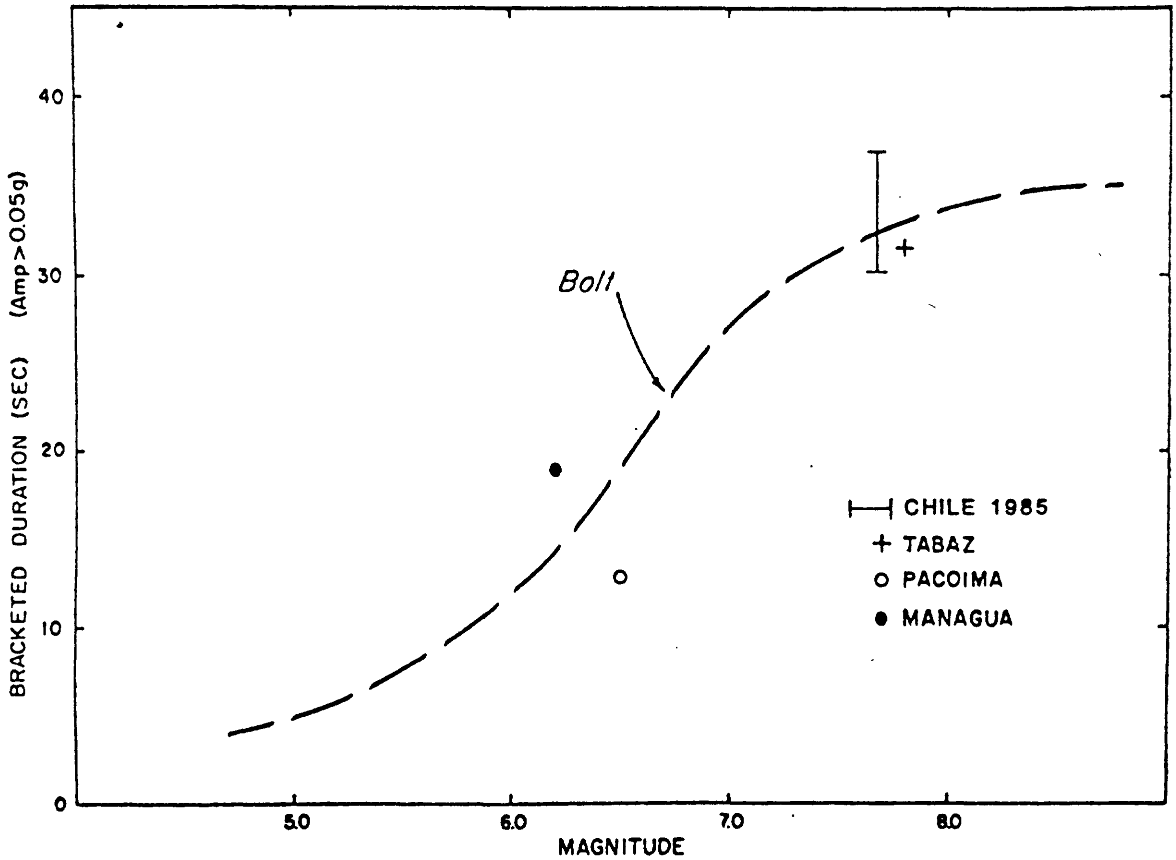

Figure 3.--Correlation between magnitude (M_L) and bracketed duration (seconds) for amplitudes >0.05g. The curve is after Bolt (1973).

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1. We have available three types of records. The primary seismogram in most cases is the accelerogram since such instruments are designed to record ground accelerations in the frequency range normally of interest to engineers. We also have the complementary records of wave velocity and displacement (see Figure 1). There thus could be three different dominant patterns, one for each of the three variables - acceleration, velocity, and displacement. In fact, the availability of these three time functions is of great assistance in the interpretation of strong-motion records. Accelerograms appear more structured, with many high-frequency pulses and considerable variability in amplitudes. The first integration to wave velocity considerably smooths these records and emphasizes frequencies in the middle range of interest. A third integration produces usually quite smooth displacement-grams with fewer fluctuations and a simpler pattern of dominant waves, usually with periods beyond one second. Sometimes, however, because of problems with baseline corrections and instrumental drift, the integrations produce large long-period bays and variations in the displacement records which may or may not be physically related to the seismic waves themselves. This type of long-period noise makes interpretation almost impossible.

2. It has been known for some time that the general shape of strong-motion records can be simplified into three parts. The first is an increase in amplitude which is the envelope of the (largely) P-wave motion rising from zero up to the longer amplitudes. A middle section follows where the amplitude fluctuation remains more-or-less the same and which can be bounded by lines parallel to the base line. The final part of the pattern is a descending taper which encompasses the coda of the record and whose slope may be small. These attempts at simplification of the pattern certainly work for certain records, but are not very satisfactory in characterizing other important strong-motion records (see, e.g., Figure 1). Nevertheless, this tripartite division is a useful one. Deviations are not likely to seriously affect the overall spectrum of the time history for engineering design purposes.

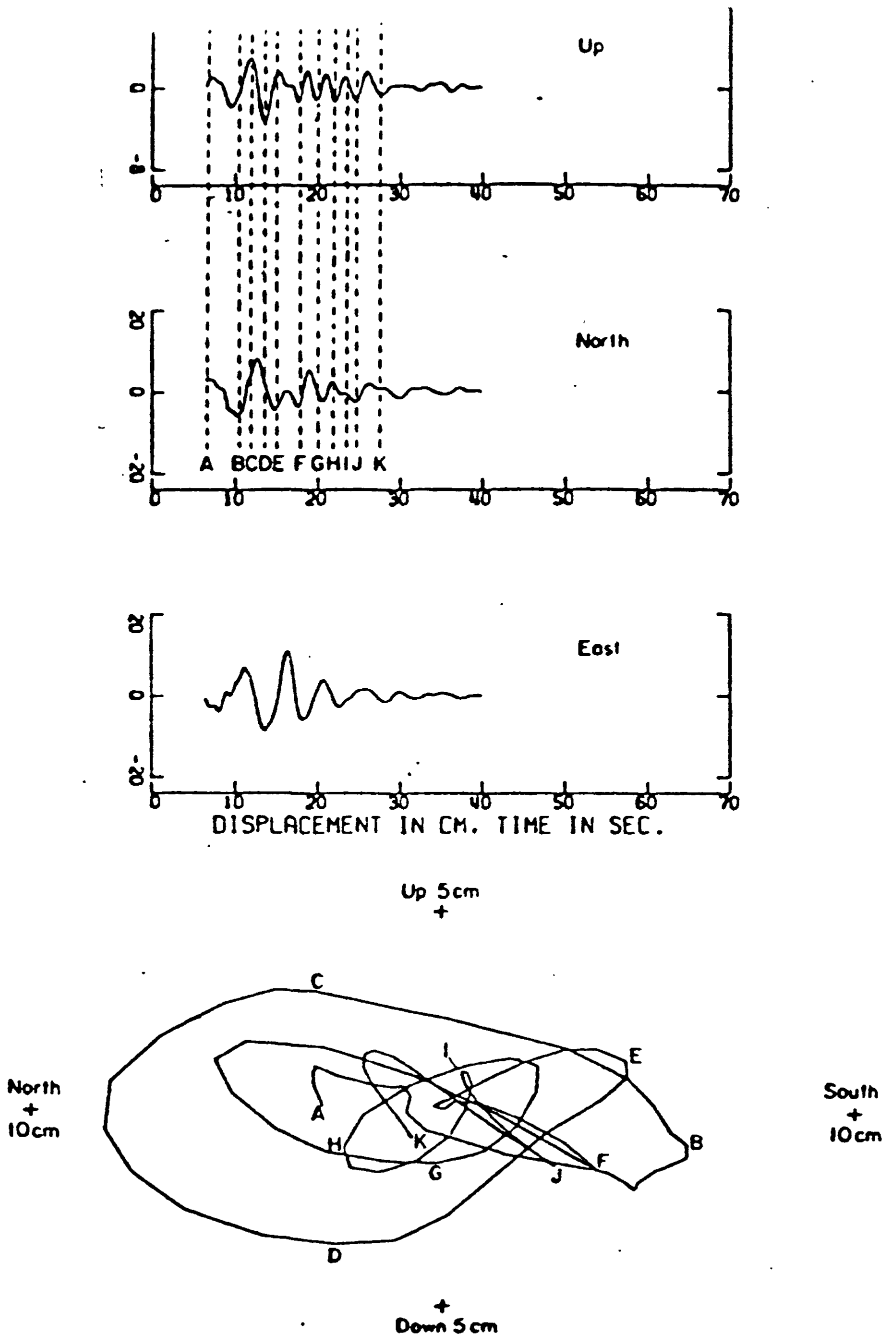

Figure 4.--Three components of ground displacement obtained by summing several displacement records at strong-motion instrument sites about 20 km from the San Fernando fault rupture, 1971. The UP and NORTH components correlate well and give the particle motion plotted at the bottom. The motion is retrograde elliptical - typical of Rayleigh waves. The EAST (transverse) component does not correlate and may be interpreted as a Love wave train (after Hanks, 1975).

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3. By analogy with regular seismograms of smaller ground motions we would expect there to be a wave pattern which follows the following properties. There should be an initial portion of ground motion made up mainly of the longitudinal P waves. Depending on the distance between the site and the source, there will then be an onset of S waves which will be superimposed on P waves still arriving from other parts of the moving dislocation. Greatly enhanced shaking will continue, consisting of an unknown mixture of S and P waves, but with the S motions becoming richer as the duration increases.

Later in the horizontal component records there will be surface waves of both Rayleigh and Love type, in general mixed with S body waves (see Figure 4). Again, depending on the distance of the site from the causative fault and also on the structure of the intervening rocks and soils, the surface waves will be dispersed into trains with certain frequency characteristics as a function of time (Hanks, 1975). This record coda is likely to be significantly affected by the focal depth of the faulted surface; the greater the depth, the less likely that a significant train of surface waves will be contained in the strong motions.

As we will see later other portions of the record will contain pulses which can be explained in terms of special properties of the finite but extended source of the motions. If the dimensions and dynamic properties of the source were known, then the appropriate patterns could be built up in the time history for such wave pulses as the "break-out phase" and "stopping phase" (see Introduction). Since this is not likely to be the case a priori, these details are often not included.

4. One pattern should be an ingredient of any realistic strong ground motion near to the causative fault. As mentioned in earlier, there is seismological evidence that near to a ruptured fault a pulse of approximately one-second duration propagates outwards and affects structures on the surface. This pulse, however, may not have the largest accelerations on the record, although it may be associated

with the greatest kinetic energy. It has been pointed out from studies of the damaged Olive View Hospital in California in the 1971 San Fernando earthquake (Bertero et al., 1978) that failures in that structure apparently occurred during the long-duration pulse that can be seen in the Pacoima velocity record (see Figure 1) about 3 seconds after the instrument triggered. The hospital structure was forced out of its elastic range of response by this motion, with significant damage to the supporting columns of the lower floors. The subsequent strong ground motion of higher frequency (peak acceleration greater than 1.0g) then shook the damaged buildings without further significant inelastic displacements.

It must be regarded as good practice, therefore, to include at an appropriate portion of a near-source record (see below) a longer period pulse which corresponds to the elastic rebound or "fling" along the fault as the dislocation passes by the site. The effects of this in engineering terms are important since the presence of this fling ensures that the longer periods parts of the response spectrum are realistically energetic.

The above expectations have been based largely on the theoretical model. Such deterministic explanations of the observed wave patterns will normally be found to leave a residual portion of the record unexplained. These unexplained residuals are found particularly in studies with synthetic displacement records for wave frequencies above 1 Hz and for acceleration records. The unexplained portion must be dealt with stochastically, as suggested, for example, by Haskell (1964). An example of the problem is discussed in Bolt (1981). From a theoretical point of view, this random component of strong ground motion can be thought of as arising primarily from the unknown distribution of roughness along the fault and, consequently, the unknown roughness distribution density $\phi(x)$. If this could be specified, then the stochastic problem would become a deterministic one. This stochastic component of strong ground motion has been one reason why one approach to modeling artificial time histories has been by random number generators (Penzien, 1970).

As yet, no roughness distribution densities have been proposed for different classes of earthquakes. In their recent barrier model for the earthquake source, Aki et al., (1977) propose three ways to estimate the interval between the significant barriers along an extended fault of the

Spectral Content

In this report, emphasis is given to the time histories of the strong ground motions. By contrast, in engineering practice at the present time, design requirements usually demand the provision of response spectra representing the ground motion at the site or its effect on a harmonic oscillation. Time histories, however, are also used, particularly for mechanical engineering tests and special analysis of critical structures. In the mathematical sense, the treatment of ground motion in either the time domain or the frequency domain is a matter of convenience and in certain interpretation problems it is essential to compare the representations in both domains. While in this work no general comparison is given between spectra of strong ground motions, there are two points about the spectral content of strong ground motions that are important in interpreting strong-motion records.

First, the spectrum from any artificial strong-motion record should not contain either gaps at certain frequencies or should not be deficient in energy at the longer period end of the spectrum. Of course, comparison of actual Fourier amplitude spectra from strong ground-motion records indicates significant fluctuations in the amplitude peaks of the spectra. For some time "average" design ground motions, however, have been used to avoid this problem.

There is also a measurement deficiency with many widely-used analog strong-motion accelerometers. Statistical analysis of the strong-motion records from the 1966 Parkfield earthquake and the 1952 Taft earthquake (Shoja-Taheri, 1977) indicates that the useful limit of long periods of velocity and displacement calculated by integration of analog accelerogram records is restricted by human reading and by baseline correction errors. The long-period limits due to the combined errors vary between 7 and 14 sec. Beyond these limits, components of displacement spectra from the present analog accelerograms are not a reliable measure of ground motion. It has also been found that the usable long-period limit with the standard analog paper records, T_c, varies (increases) with record length L. For L equal to 40, 50, and 60 sec, T_c is estimated to be about 10, 12, and 14 sec, respectively. At a period of about 16 sec, the combined errors for the majority of cases of strong-motion records studied exceed 25 percent of the accumulative displacement amplitude spectrum. Digital records, however, from the newly available digital strong-motion accelerometers should allow the above limit to be significantly extended.

Secondly, the spectrum of strong ground motion is in two parts. The first is the amplitude spectrum, which is normally all that is considered in strong-motion seismology and earthquake engineering. The second part, however, is the phase spectrum, and this phase defines the pattern of seismic waves, which is the subject of these interpretation studies. This property has not been as widely used in the construction of artificial strong ground motions as it deserves. For example, an amplitude spectrum from a magnitude 7.5 earthquake with adequate maximum amplitudes can be combined with the phase spectrum from another earthquake (with smaller amplitudes, say, than required) but with a phase spectrum appropriate to the wave pattern for very near-fault motions.

A computed illustration is given in Figure 5.

Directivity and Focussing

A major practical question in strong-motion interpretation and construction of artificial time histories is "to what extent is the time history at a particular site dependent upon the location of the rupture on a given fault?" It is known both theoretically and observationally that each seismic wave type has a directivity function which depends on the azimuth relative to the center of the earthquake source.

Consider the seismic sources in the form of superimposed force couples (or a seismic moment tensor). This representation entails that each type of seismic wave has its own radiation pattern. Thus, for example, a vertical strike-slip fault can be represented by a double couple with center at the focus; the

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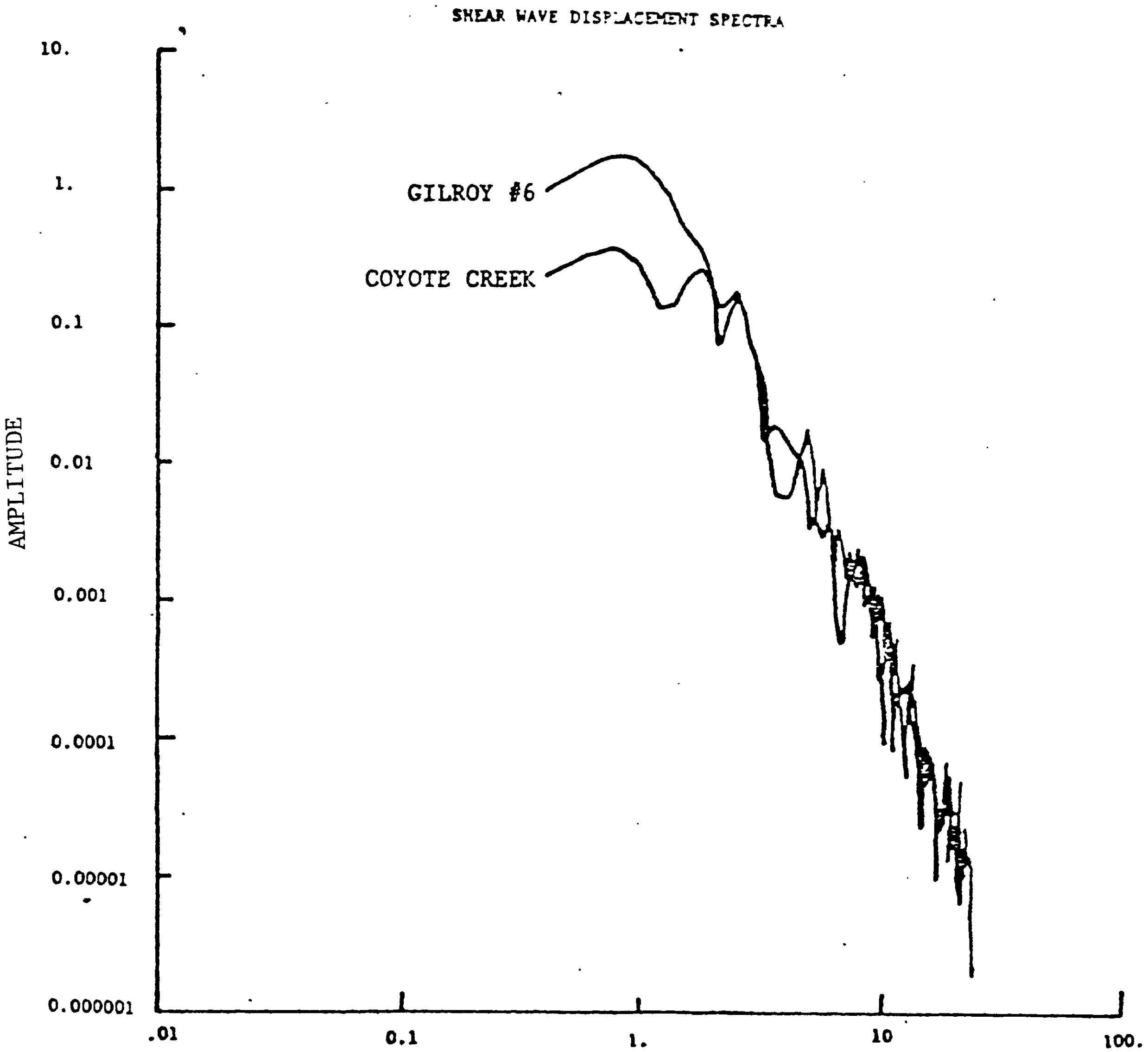
SHEAR WAVE DISPLACEMENT SPECTRA



FREQUENCY - CYCLE/SEC

Figure 5.--Wave displacement Fourier spectra calculated from two strong-motion records of the August 6, 1979, Coyote Lake, California earthquake (after Singh, 1981).

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radiation pattern for SH waves will consist of a four-lobed pattern with maximum amplitudes at directions normal and along the faulting. Similarly, P waves and Rayleigh waves will have appropriate radiation patterns (Bullen and Bolt, 1985). Because the intensity of ground shaking is the effect of all the waves that arrive at a point, these radiation patterns are not always obvious by looking at isoseismals based on the assessed intensity. Nevertheless, in the interpretation of accelerograms and numerical modeling of synthetic strong-motion records, seismic radiation patterns are essential ingredients.

In the case of large earthquakes where the rupture length L is significant compared with the wave lengths considered, the radiation pattern becomes more complicated. Rather than the usual symmetric pattern typical of a stationary point source, the radiation pattern lobes for the various seismic waves become retracted or extended, depending on the direction of rupture along the fault. There are now published a number of reasonably representative radiation patterns for moving earthquake sources which are helpful in the interpretation of strong-motion records (see, e.g., Ben-Menahem and Singh, 1972). This effect of rupture velocity is called dynamical directivity, and it is an important matter to detect this directivity on strong-motion near-field records. Because of various complications, this has not yet been clearly accomplished in the near field, although these patterns are widely verified in the far field.

Another aspect of the moving seismic source is the occurrence of a Doppler-like effect analogous to sound radiation on an acoustic point source that moves in a medium at rest. If, when stationary, the source has a symmetric radiation pattern, its radiation would be expected to be focussed in the direction of motion when it is moving with a finite velocity. The amount of focussing, in general, will be different for the case when the source velocity V is subsonic ($V < \alpha$ for P waves) or supersonic ($V > \alpha$ for P waves). A purely geometrical argument (Morse and Ingard, 1968) gives rise to the well-known focussing factor.

$$F = (1 - M\cos\theta)^{-1} \qquad (2)$$

where is the angle subtended by the direction of the wave from the source and rupture direction and M is the Mach number V/α. The result is that, depending on the angle , there is a Doppler shift in both the wave amplitude and frequency.

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Various examples have been cited by seismologists (e.g., Benioff, 1955) that strong-motion data obtained from stations along the direction of the ruptured fault evidence the focussing of earthquake motions. Reasonable values for the parameters in the above formula indicate that the focussing effect might (for a perfectly elastic non-attenuating medium) change the wave amplitude by a factor of up to 10, with an increase in front of the rupture.

Effect of Complex Propagation Paths in Ground Structure

In earlier sections the effect of horizontal layering in the crustal rocks on seismic waves has been outlines. In many situations, however, particularly in fault zones, the variations in soil and rock structure are not restricted to plain parallel horizontal layering. Particularly in sedimentary basins there will be significant lateral variations and often irregular shaped and sloping rock structures. The behavior of elastic waves encountering such physical obstacles is mathematically complicated and only a few special cases have been treated theoretically. Indeed, mode conversions, scattering, diffraction, and resonance make even numerical estimates for standard procedures extremely difficult. In these circumstances, the elementary ray approximation may perhaps be misleading, so that the simple approaches must be used with caution wherever there is evidence of marked inhonogeneities in structure.

The problem is that normally the presence or absence of seismic structural anomalies is unknown. Even with deep borehole data and geophysical profiling, large-scale anomalous bodies of arbitrary shape along active fault zones may not be well defined. Nevertheless, it has been common for seismologists and engineers to call on this hypothesis to explain rapid variations in intensity in areas of heavy shaking. It is one way to explain, for example, the reason why a pocket of high intensity is seen at one place and yet no damage occurred to similar structures in another part of the area about the same distance from the earthquake source. The explanation, therefore, is ususally open to question even though such structurally anomalous bodies would certainly focus seismic waves by refraction in the same way that light waves are focussed by a lens. This mechanism of seismic focussing is, of course, quite different from that described above in the previous section.

EXAMPLE ANALYSIS

Two examples are given below to illustrate the concepts of this paper.

Coyote Lake, California, Earthquake - August 6, 1979

References: Porcella *et al* (1979); Uhrhammer (1980)

Location:	Earthquake:	37°6.12'N, 121°30.2'W
	Accelerometer:	1. Coyote Creek C217/SMA-IT
		2. Gilroy Array #6 1413/SMA-1
	Foundation:	Both instruments on conglomeratic sediments
Size:	Magnitude:	$M_L = 5.9$
	Moment:	$M_o = 6 \times 10^{24}$ dyne cm
	Mean Stress Drop:	$\Delta p = 18 \pm 5.6$ bars

Fault Source Characteristics: (See Figures 5 and 6)

Faulting:	Minor breaks and cracks along about 8-10 km of the Calaveras Fault, which strikes N30W. Right-lateral displacements up to 5 mm were observed.
Focal Mechanism:	Strike $N(27 \pm 7)^oW$ Dip $(90 \pm 15)^o$
Parameters:	$L = 23.1$ km $W = 5$ km $D = 0.21 \pm 0.066$ m (from M_o) $= 0.005$ m (observed at surface)
Mean Rupture Velocity:	2.2 km/sec

Peak Wave Amplitude Values

1. Coyote Creek	UP	S70W	S20E
Acceleration (cm/sec^2):	98	225	157
Velocity (cm/sec):	-	-	-
Displacement (cm):	-	-	-

2. Gilroy Array No. 6	UP	N40W	S50W
Acceleration (cm/sec^2):	167	333	412
Velocity (cm/sec):	-	-	-
Displacement (cm):	-	-	-

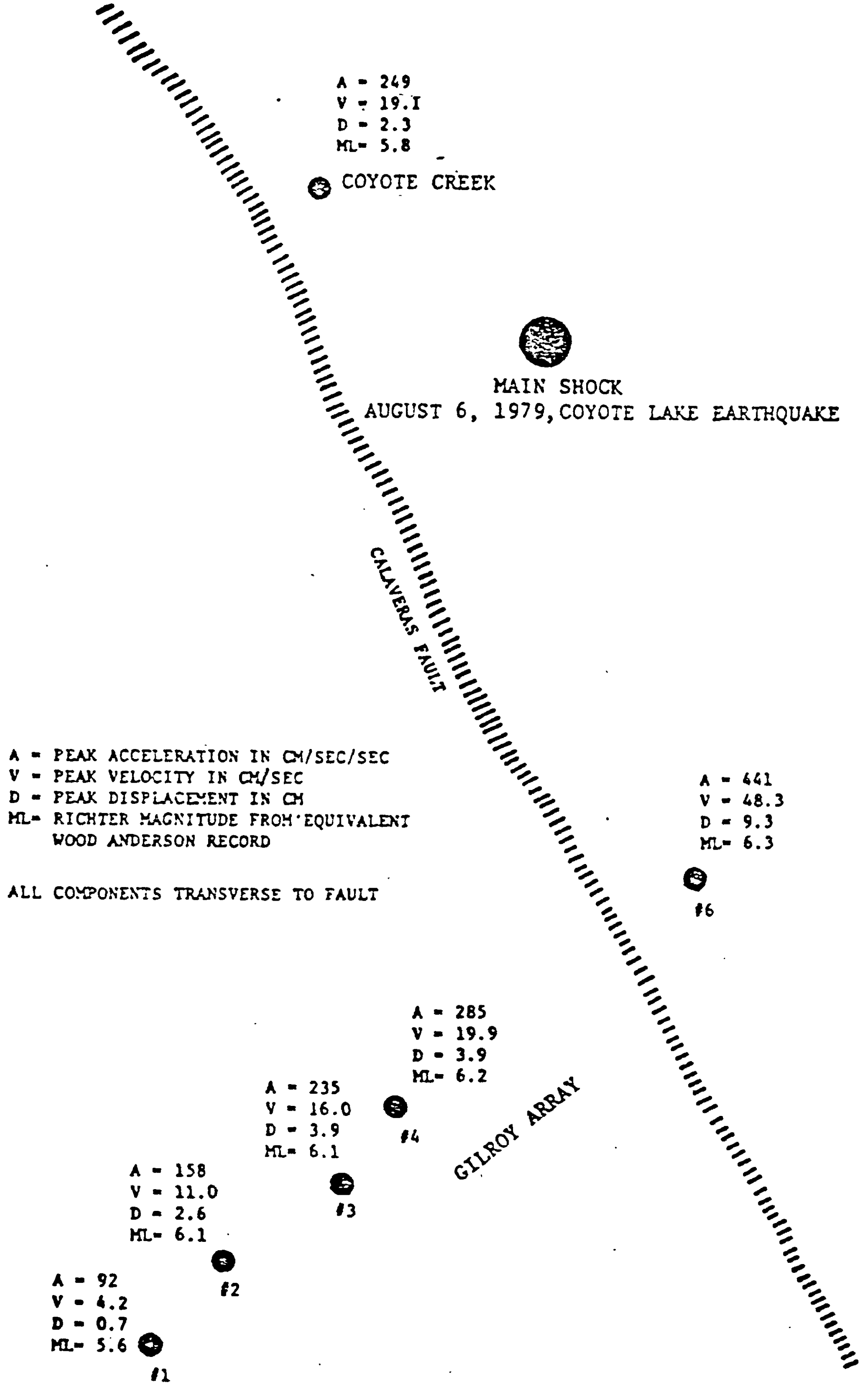

Figure 6.--Map of Calaveras fault and accelerometer stations for the Coyote Lake earthquake. Peak accelerations, velocities, and displacements are given for component transverse to the fault (from Singh, 1981).

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Central Chile - March 3, 1985

The principal shock occurred at 7:47 p.m. local time on Sunday, March 3. The preliminary epicenter was off the coast about 15 km northwest of Algarrobo at 33.25S, 77.75W with a focal depth of 6 km (Figure 7). The surface wave magnitude is M_s = 7.8 (NEIS), compared with M_s = 7.9 (NEIS) for the July 9, 1971 earthquake located approximately 100 km north of the 1985 epicenter. Widespread but localized damage occurred to various types of structures in central Chile. The great majority of modern reinforced concrete structures were not damaged.

There was a foreshock sequence before this earthquake that began about one month prior to the mainshock. The foreshock sequence consisted of a swarm of about 300 earthquakes with magnitudes up to about 4.5. The swarm was centered just east of the mainshock epicenter. The swarm began to decrease about ten days before the mainshock and the last event in the swarm occurred 200 hours before the mainshock. No prediction of the mainshock was made based on the occurrence of the foreshocks.

The principals shock was followed by an extensive aftershock sequence. Even after 24 hours, about 22 earthquakes per hour with magnitude about 3.5 could be distinguished on the seismograms. The aftershock region is shown in Figure 7. The area of the aftershocks is about 140 km long (north-south) by 70 wide (east-west) with focal depths ranging from 5 to 40 km.

In Figure 7 are shown locations of the strong-motion accelerometers operated by University of Chile groups. The measured peak horizontal acceleration at Milipilla is about 0.55 g and the duration of strong shaking is about 40 seconds. Of particular interest in the strong-motion recordings is the high amplitude, high frequency content on the vertical component during the S wave arrival.

Upon inspection of the Milipilla record, it is clear that this earthquake was a double event. The intial P wave arrival has small amplitude and is followed byu a small S wave. Later in the record, the amplitude of the S waves

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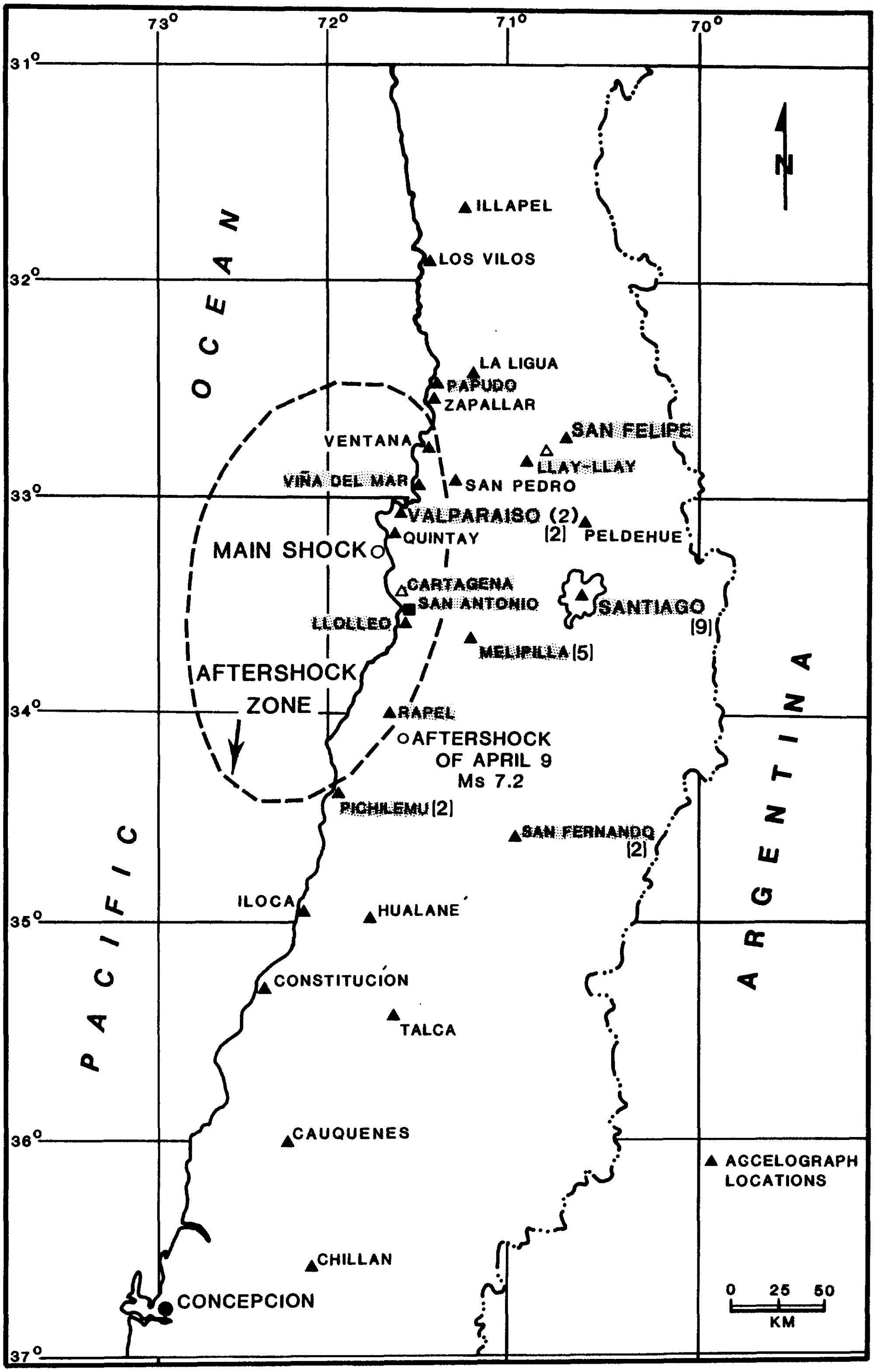

Figure 7.--Location map of central Chile showing the epicenter of the March 3, 1985 main shock and the extent of the aftershock zone. The locations of accelerographs that recorded the main shock and many aftershocks are shown by solid triangles. Numbers in parentheses indicate locales where more than one accelerograph is installed. Shaded place names and open triangles identify sites of USGS digital seismographs. Numbers in brackets indicate that more than one digital seismograph was installed in the same vicinity.

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abruptly increases by an order of magnitude indicating that the mainshock had been preceded by about 10 seconds by a smaller foreshock. The hypocenter given above is possibly for the smaller magnitude foreshock. The location of the second and main source of seismic energy is important in order to interpret the distribution of the damage.

The largest accelerations were recorded on the coast at Llolleo just south of San Antonio. The peak vertical and horizontal accelerations are about 0.80 g and 0.75 g, respectively. There is a dramatic change in the amplitude and frequency content in the records obtained north of Llolleo at the coastal cities of Valparaiaso and Vina Del Mar. These records show peak accelerations of about 0.25-0.3 g. The peak accelerations are at periods of 1 Hz compared with the high frequency (>4 Hz) on the Llolleo and Millepilla recordso.

Also of interest is the variation of the strong ground motion from the coast toward the Andes. The record obtained at Llay-llay in the center of the valley east of Valpariaso has a peak accleration of about 0.48 g. Further east, the record from San Felipe has a high frequency content with a peak horizontal acceleration of about 0.39 g. This station is almost 100 km from the edge of the aftershock zone (Figure 7). Detailed studies are required to understand how these high-frequency waves are propagating over such large distances.

The ground motions in this Chile earthquake were more energetic than from previous recordings of California earthquakes. The Chilean strong motions are recorded at sites with different soil and rock foundations and will allow correlations with the amount of damage to buildings and other structures.

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IMPORTANT CONSIDERATIONS FOR ASSESSING THE POTENTIAL FOR LOSSES FROM EARTHQUAKE HAZARDS IN PUERTO RICO

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INTRODUCTION

The assessment of the potential losses from earthquakes in an urban area is a complex task requiring:

1. An earthquake hazards model.
2. An exposure model (inventory).
3. A vulnerability model.

A schematic illustration of the total range of considerations is shown in Figure 1. Each model is discussed below.

EARTHQUAKE HAZARDS MODEL

Loss estimation is closely related to the capability to model the earthquake hazards of ground shaking, surface fault rupture, earthquake-induced ground failure, tectonic deformation, and in some cases, tsunamis (Figure 2). Most of the spectular damage and losses in an earthquake are caused by partial or total collapse of buildings as a consequence of horizontal ground shaking (Figure 3). However, ground failures triggered by an earthquake can also cause substantial damage and losses. For example, during the 1964 Prince William Sound, Alaska, earthquake, ground failures accounted for about 60% of the estimated $500 million total loss with landslides, lateral spread failures, flow failures, and liquefaction causing damage to highways, railway grades, bridges, docks, ports, warehouses, and single family dwellings (Figure 4). Surface faulting, which generally affects a long narrow area, has

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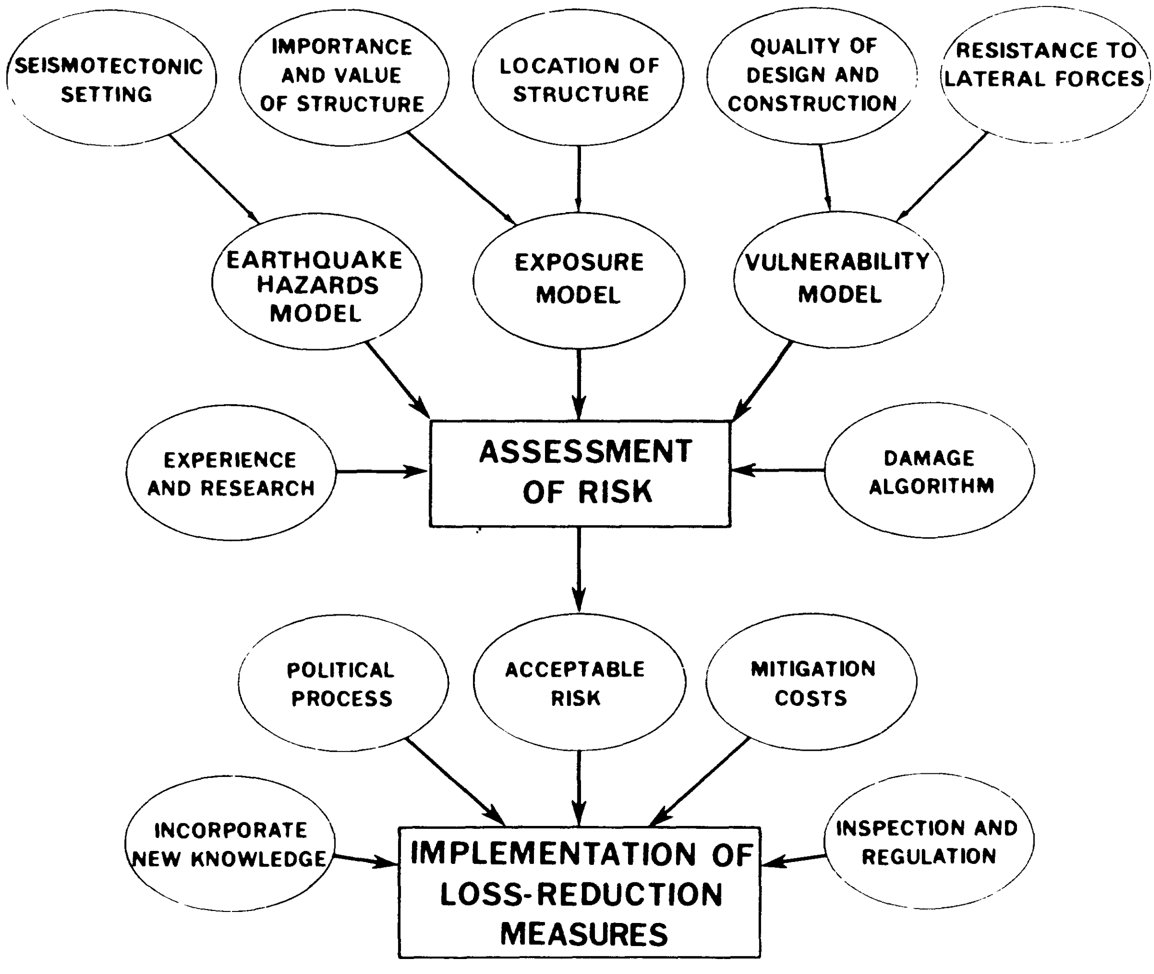

Figure 1.--Schematic illustration of the overall process involved in the assessment of earthquake hazards and risk in a geographic area. The earthquake hazards, exposure, and vulnerability models are critically important parts of the total process.

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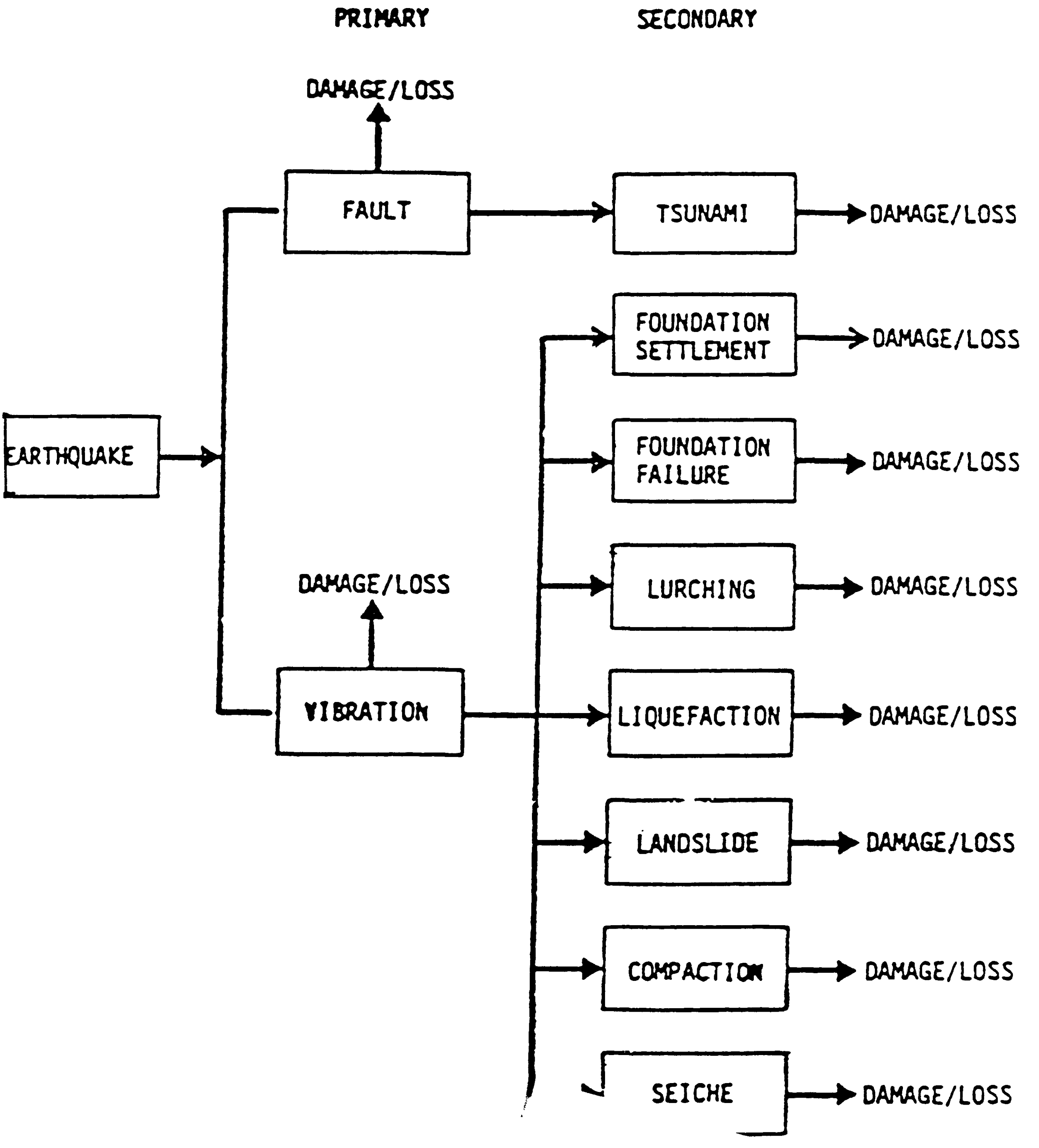

Figure 2.--Schematic illustration of the primary and secondary hazards caused by an earthquake. Each hazard can lead to damage and loss, depending on the inventory of structures exposed to the hazards and their vulnerability to the hazard.

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Figure 3.--Olive View Hospital. Note stair tower at left which fell away from main building. Roof in right foreground collapsed on parked ambulances. Eight hundred occupants were successfully evacuated from the main building. One person was killed due to a partial building collapse. (Los Angeles City Fire Department photo).

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Figure 4.--Damage from landslides triggered by ground shaking in the 1964 Prince William Sound, Alaska, earthquake. Ground failures can occur in ground shaking ranging from Modified Mercalli intensity VI to XII.

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damaged lifeline systems and single family dwellings in the Western United States, but has not directly caused deaths and injuries (Figure 5). Tsunamis, long period water waves caused by the sudden vertical movement of a large area of the seafloor during an earthquake, occur fairly frequently in Hawaii and have produced great destruction and loss of life. Although occurring much less frequently, destructive tsunamis have also affected Puerto Rico, the Virgin Islands, Alaska, and the west coast of the United States.

The earthquake hazards model must answer the following questions:

1. Where have past earthquakes occurred? Where are they occurring now?

2. Why are they occurring?

3. How often do earthquakes of a certain size (magnitude) occur?

4. How bad (severe) have the physical effects (hazards) been in the past? How bad can they be in the future?

5. How do the physical effects (hazards) vary spatially and temporally?

The answers to these questions are used to define the amplitude, frequency, composition, and duration of horizontal ground shaking--the three parameters that correlate best with damage.

EXPOSURE MODEL

The spatial distribution of things and people exposed to the earthquake hazards is called inventory. The inventory is one of the most difficult models to characterize. For loss prediction, the term structure is used to refer to any object of value that can be damaged by the earthquake hazards of ground shaking, surface faulting, ground failure, tectonic deformation, and tsunami run up. The various categories of structures include:

1. Buildings (residential, agricultural, commercial, institutional, industrial, and special use).

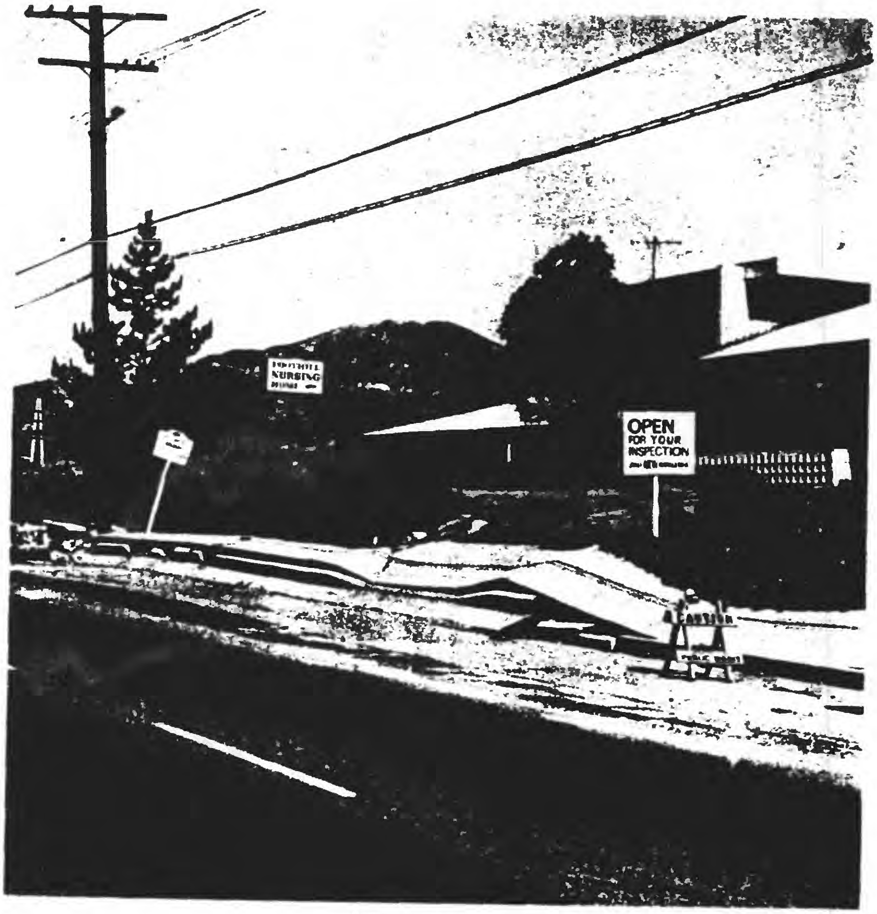

Figure 5.--Surface Fault Break. Area beyond curb was raised about 3 feet relative to the street. Unoccupied nursing home was damaged but did not collapse. (Los Angeles City Department of Building and Safety photo)

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2. Utility and transporation structures (electrical power structures, communications, roads, railroads, bridges, tunnels, air navigational facilities, airfields, and waterfront structures).

3. Hydraulic structures (earth, rock, or concrete dams, reservoirs, lakes, ponds, surge tanks, elevated and surface storage tanks, distribution systems, and petroleum systems).

4. Earth structures (earth and rock slopes, major existing landslides, snow, ice, or avalanche areas, subsidence areas, and natural or altered sites having scientific, historical, or cultural significance).

5. Special structures (conveyor systems, sky lifts, venelation systems, stacks, mobile equipment, tower, poles, signs, frames, antennas, tailing piles, gravel plants, agricultural equipment, and furnishings, appendages, and shelf items in the home).

A structure consists of many elements. To predict losses, the contributions of each individual element to the total response of the structure from the dynamic forces induced by ground motion (or any of the other hazards) must be modeled (Figure 6).

Vulnerability Model

Vulnerability is a term describing the susceptibility of a structure or a class of structures to damage. The prediction of the actual damage that a structure will experience when subjected to a particular hazard (such as ground shaking) is very difficult as a consequence of:

1. Irregularities in the quality of the design and construction.

2. Variability in material properties.

3. Uncertainty in the level of ground shaking induced in the structure as a function of magnitude and epicentral distance.

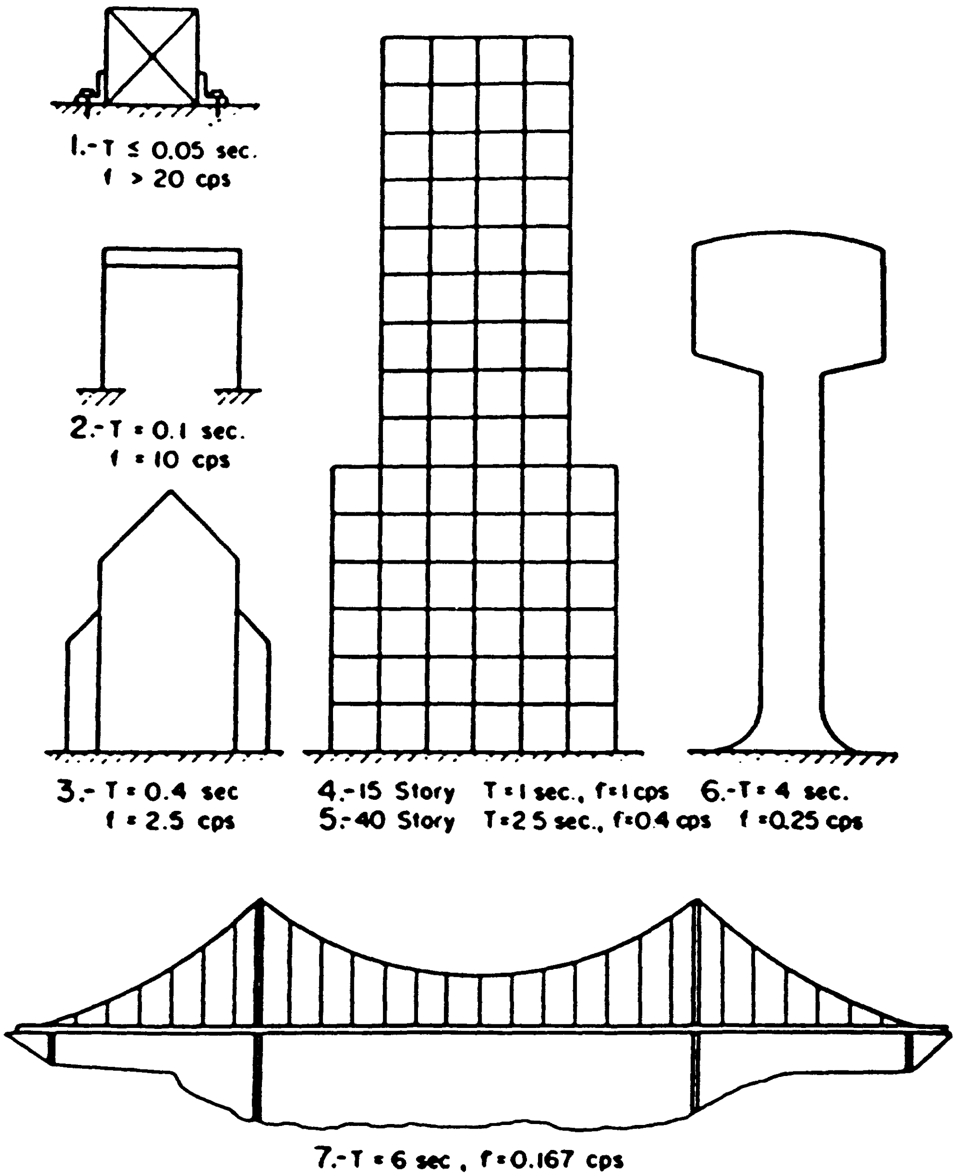

Figure 6.--Schematic illustration of various kinds of structures ranging from low-rise to high-rise buildings, water tanks, and long bridges. Each type of structure has a different fundamantal period of vibration which causes it to respond differently to ground motion. The assessment of potential vulnerability considers the characteristics of each structure.

4. Uncertainty in structural response to earthquake ground shaking, especially in the range where failure occurs.

A fragility curve (Figure 7) can be used to represent failure of a specific type of structure (or a structural system) when it is exposed to the dynamic forces induced by ground shaking. For most structures, damage occurs as a function of the amplitude, frequency composition, and duration of ground shaking and manifests itself in varyious states ranging from "no damage" to "collapse." Specification of the damage states of a structure is very difficult because each State is a function of the lateral-force-resisting system of the structure and the severity of the hazard.

The capability to model the potential vulnerability of a structure is improving. Many efforts have been made throughout the World to assess damage and losses following a major historical earthquake. However, quantitative procedures for assessing vulnerability and predicting losses have only been developed since the 1970's. Some of the investigators in the United States who have contributed to the current state-of-knowledge include: Blume (1970); Algermissen, et al., (1972, 1973); Scholl (1974); Hopper et al., (1975, 1983); Rogers, et al., (1976); Scholl and Kustu (1981); Davis, et al., (1982); Kircher and McCann (1983); and Steinbrugge, et al., (1984).

ASSESSMENT OF POTENTIAL LOSSES

On the basis of the current knowledge of earthquake hazards (McCann, 1984) in the Puerto Rico area, the earthquake potential can be summarized as follows:

- Large earthquakes (M = 7.5-8.25) have occurred in the past and are expected to occur again.

- The regional seismicity falls in seven zones: 1) Eastern Hispaniola, 2) Mona Passage-Mona Canyon, 3) Puerto Rico trench, 4) Anedaga trough, 5) Muertos trough, 6) intermediate depth zone under Puerto Rico, and 7) shallow zone on the island.

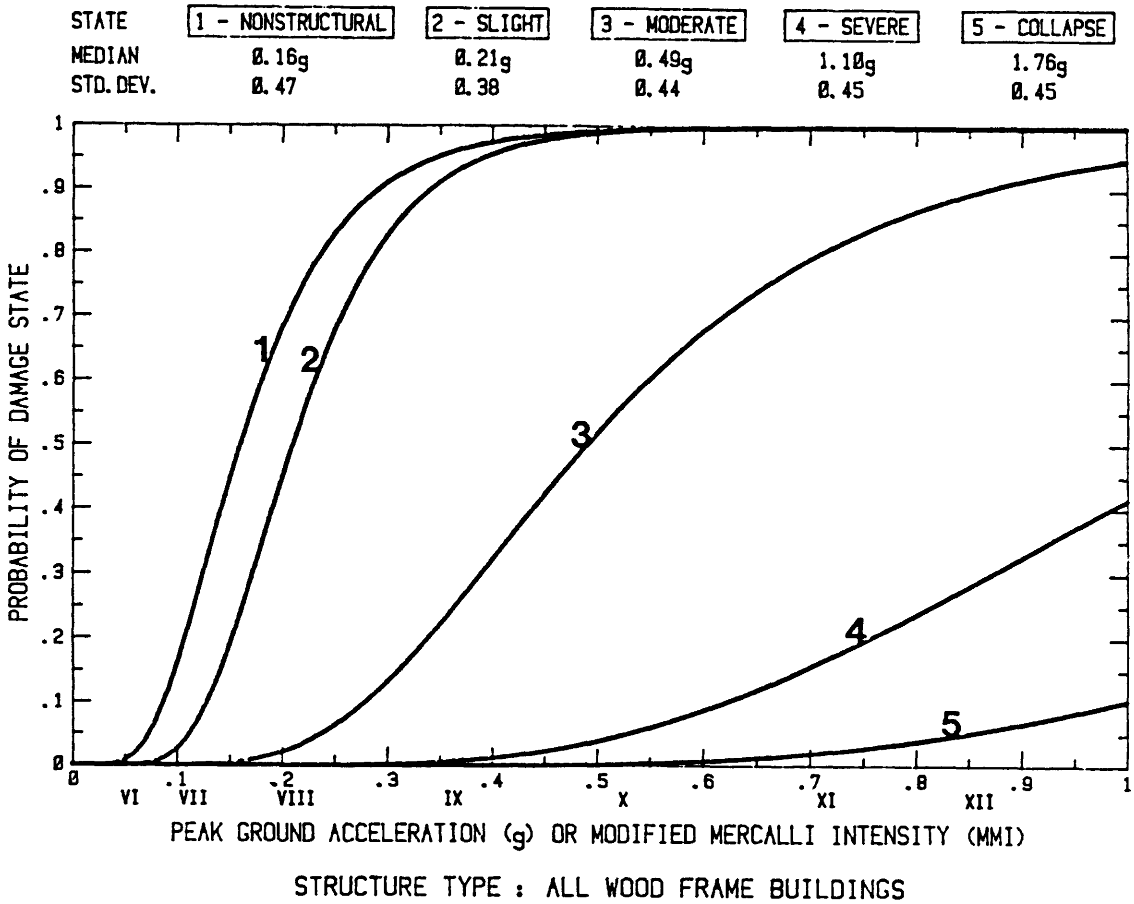

Figure 7.--Example of a fragility curve proposed by Kircher and McCann (1973) for wood and frame buildings in the Central United States. Five damage states ranging from "no damage" to "collapse" are generally considered.

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- The largest earthquakes are expected in the Puerto Rico trench, Mona Passage-Mona Canyon, Anedaga trough, and Muertos trough. Great earthquakes may rupture 200 km (120 miles) long segments of the Puerto Rico trench about once every 200 years. (McCann, 1984).

- Tsunamis have occurred in the past and the potential for recurrence is high.

- The potential for surface fault rupture on the island is probably low.

- The potential for landslides and liquefaction on the island is high.

- Peak ground accelerations in the order of 0.20 g are expected in an exposure time of 50 years. The level of ground shaking will vary as a function of the magnitude, epicentral distance, and the local geology.

Estimating the potential losses from ground shaking in Puerto Rico requires the following steps, illustrated schematically in Figure 8:

1) Divide the island into zones and determine the inventory in each zone.

2) Predict the ground motion for each zone, using either deterministic (Hays, 1980) or probabilistic (Algermissen et al., 1982) methodologies and the concept of a scenario earthquake.

3) Predict the losses for all structures and zones, using fragility curves such as in Figure 7 or Figure 9.

4) Sum the losses for all zones to estimated loss the total loss.

This process will produce estimates of the potential losses. The estimates will vary as a function of the earthquake selected for the planning scenario. For example, one scenario might consider the magnitude 7.5 earthquake of October 11, 1918, which caused considerable damage and generated a tsunami having 20 foot waves. In the recurrence of such an earthquake, a

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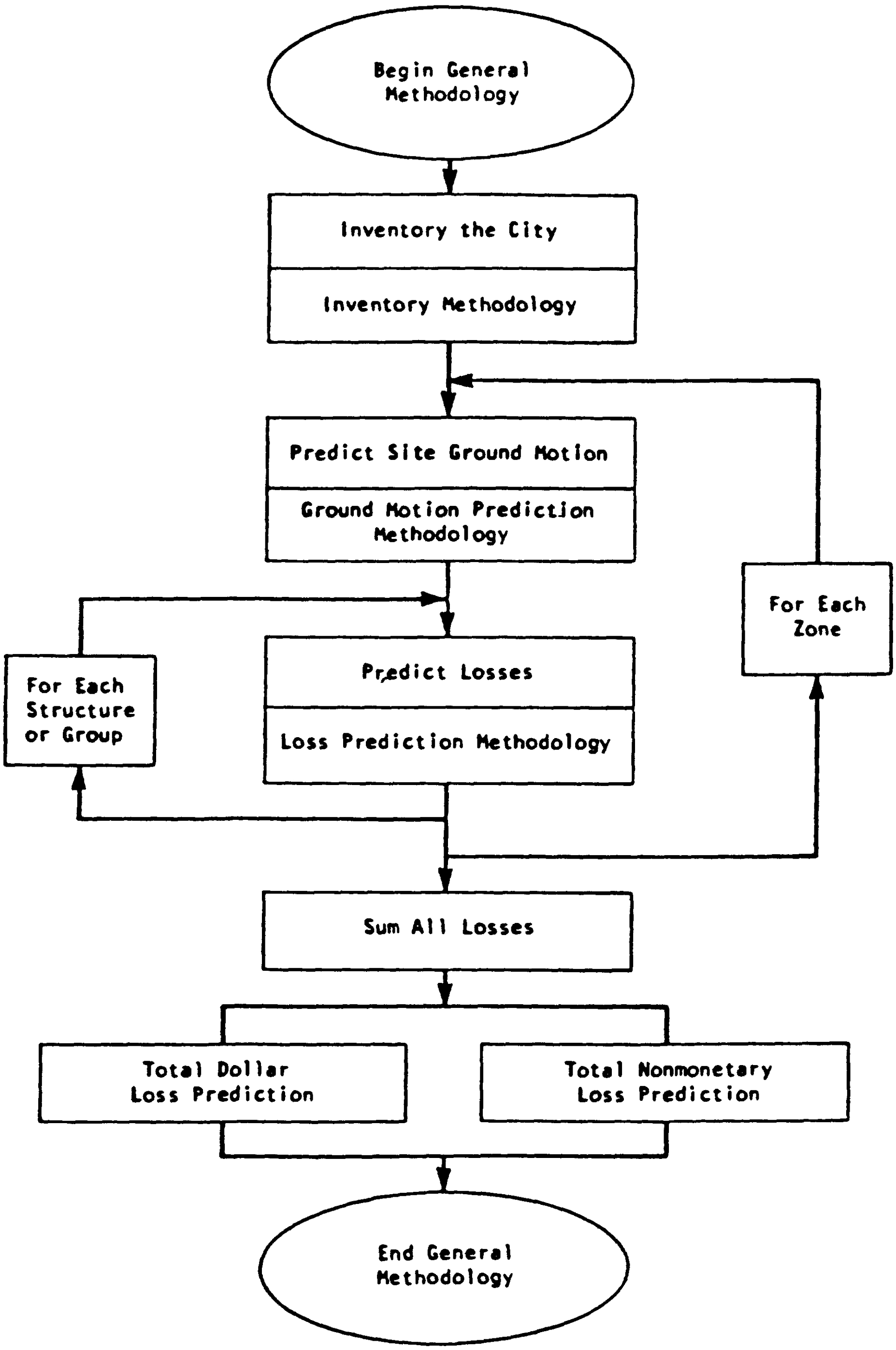

Figure 8.--Schematic illustration of a procedure for estimating potential losses from earthquake ground shaking (from Scholl and Kustu, 1981).

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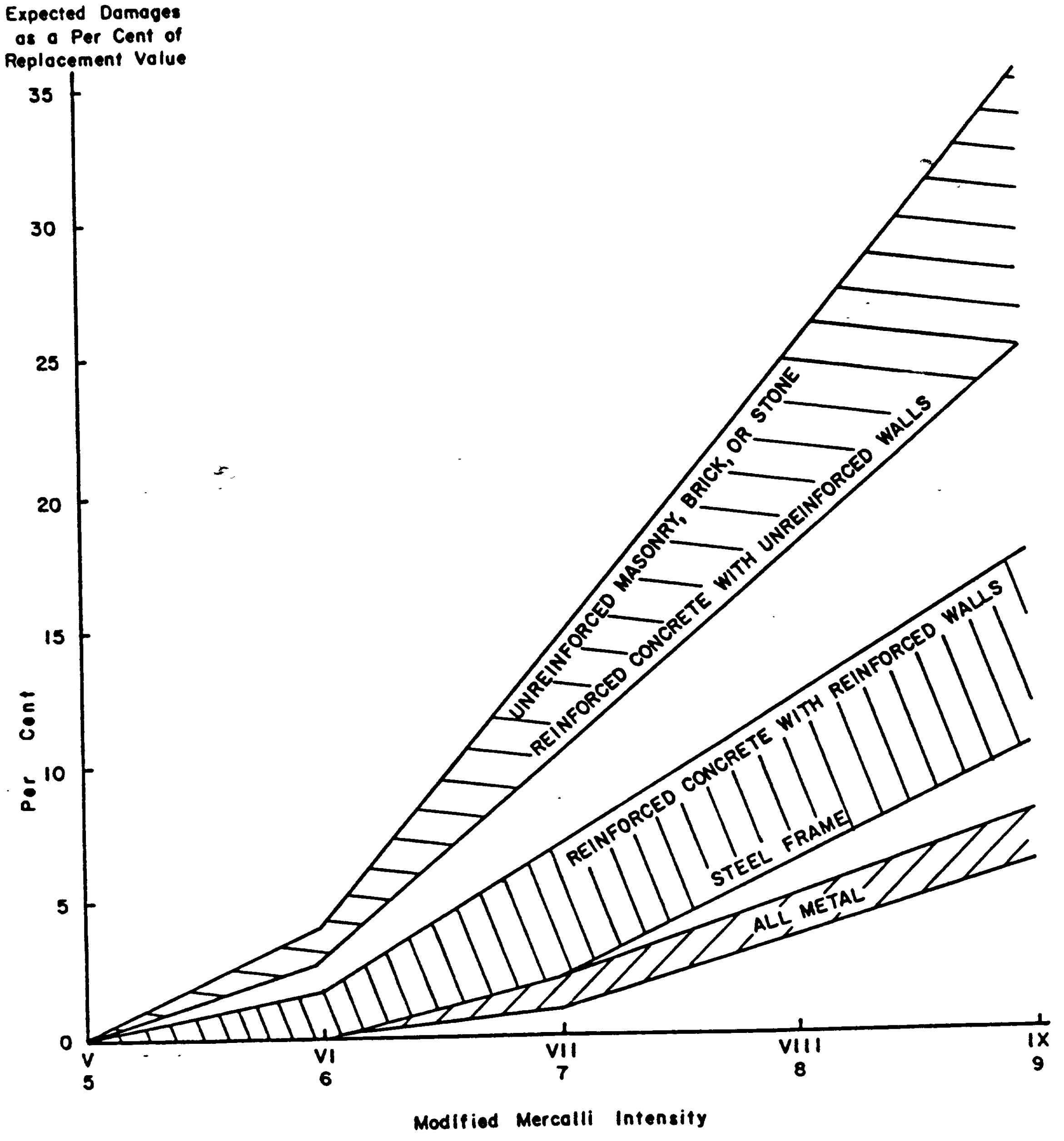

Figure 9.--Example of curves for estimating losses to various types of structures using Modified Mercalli intensity as a parameter.

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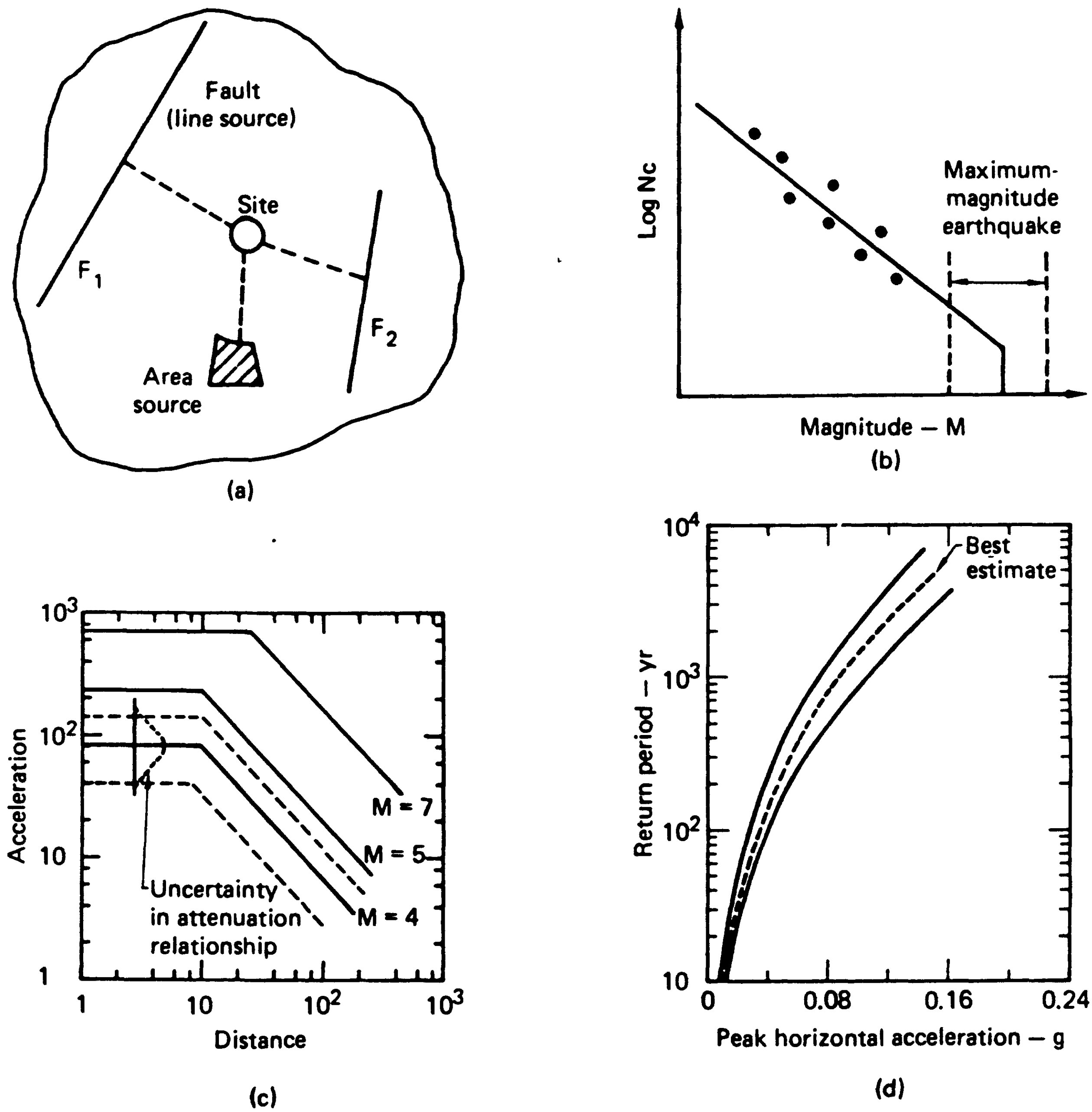

Figure 10.--For a probabilistic hazard assessment, an analyst generally: (a) specifies the geometry of important seismic regions; (b) characterizes the relative frequency of earthquakes of various sizes; developes an earthquake recurrence model (usually a Poisson distribution in time, not shown); and (c) selects a transfer function that transforms information about the earthquake at the eipcenter into information at the site, such as ground acceleration, that a structure engineer can use. The result of such an assessment is a plot of return period vs peak horizontal ground acceleration (d).

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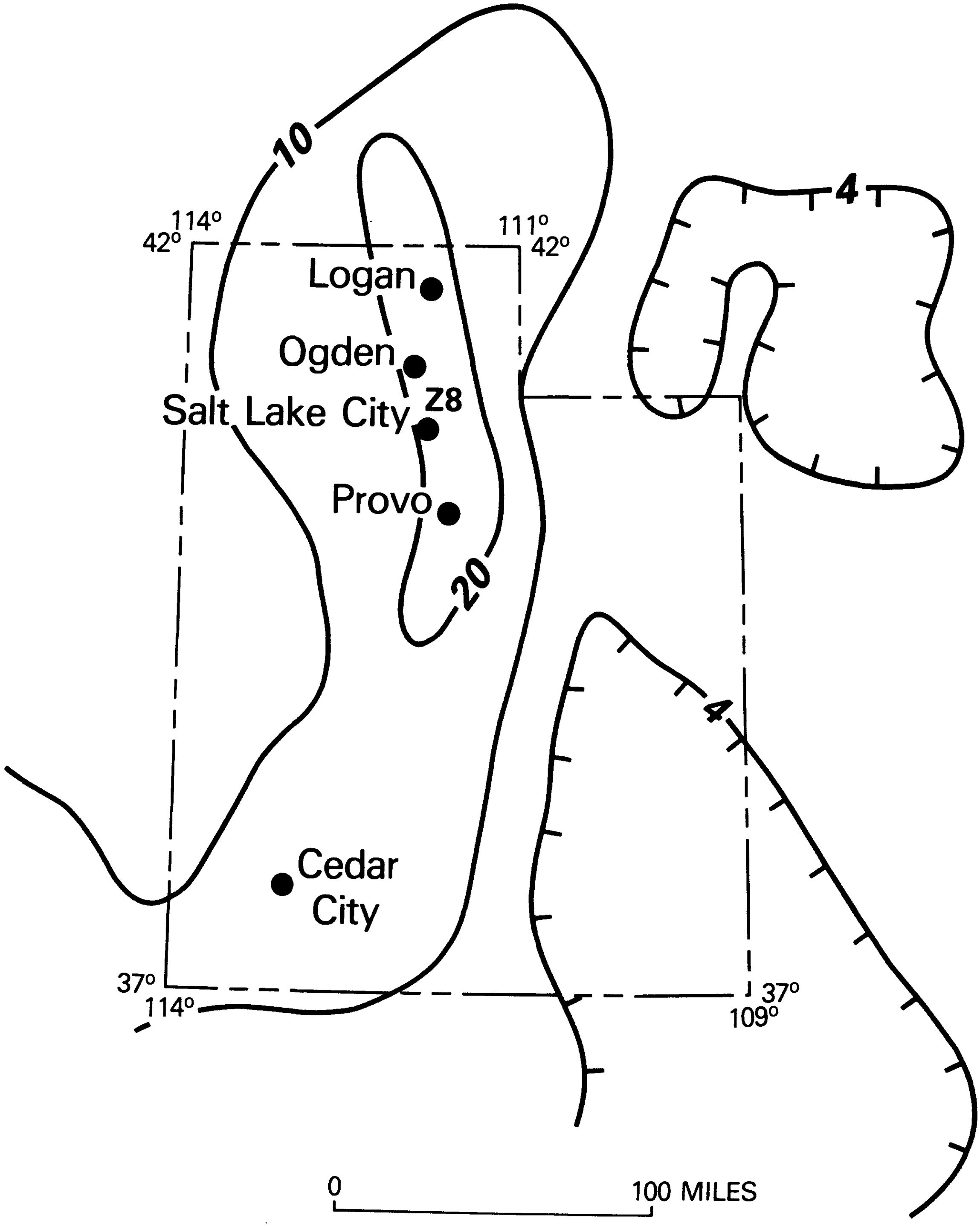

Figure 11.--Example of map showing maximum level of peak horizontal ground acceleration expected in Utah in an exposure time of 10 years at sites underlain by bedrock (Algermissen et al, 1982). The corresponding return period is approximately 500 years (actually 95 years). The values of peak bedrock acceleration have a 90 percent, probability that they will not be exceeded in a 10-year period. Soil effects must be considered separately.

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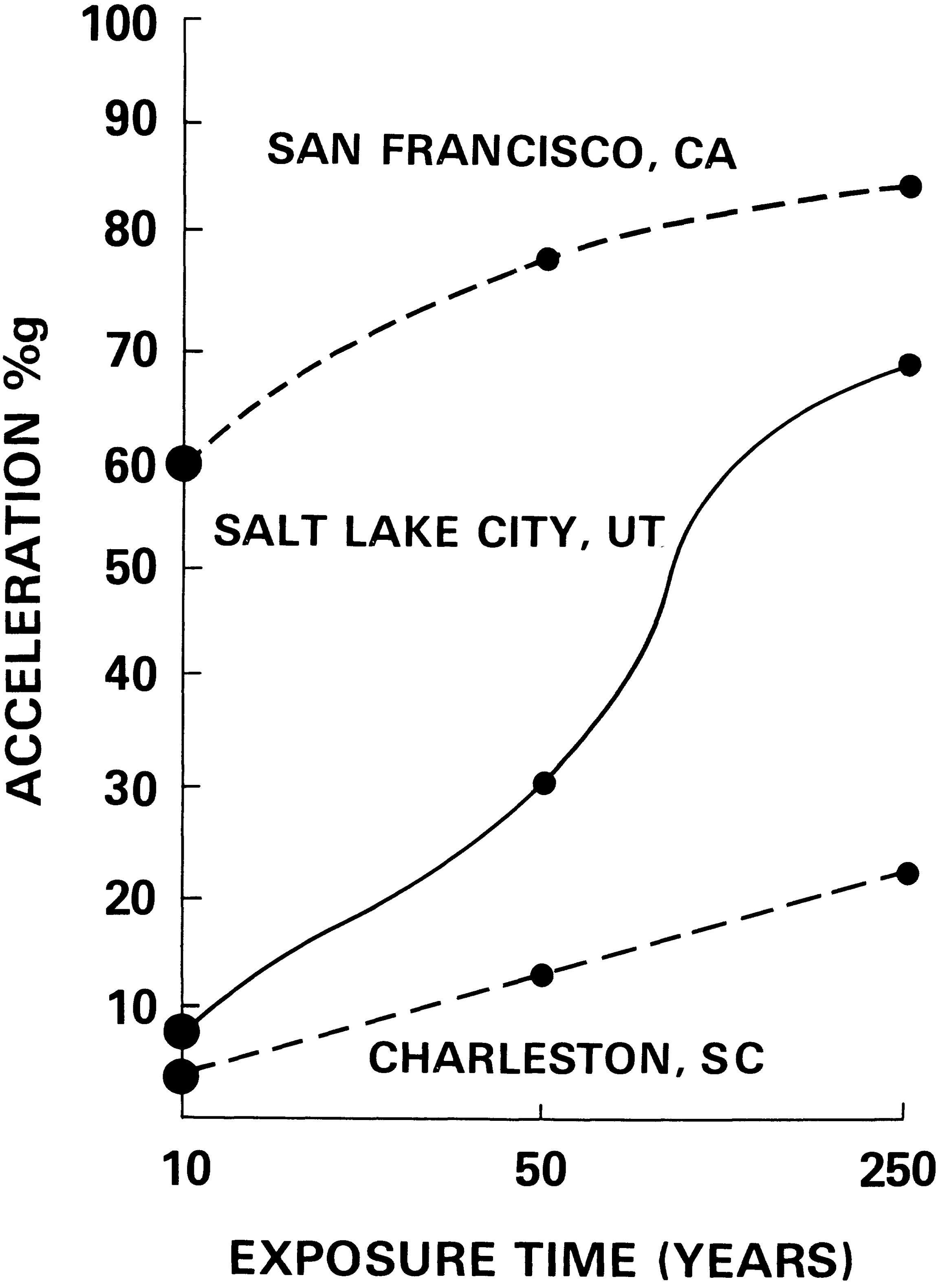

Figure 12.--Examples of a way to depict the ground-shaking hazard at a site. These curves show peak bedrock acceleration as a function of exposure time and a 90 percent probability of nonexceedance. Although some controversy exists about the absolute values of peak acceleration, the relative values between two locations are stable. These curves permit a choice by the acceptable level of risk.

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wide range of ground shaking intensities would be expected. Intensities of IV to VI would affect the contents of structures, intensities of VI to VII would cause architectural damage, and intensities of VIII or greater would cause minor to major structure damage. The potential losses, deaths, and injuries would be substantially greater now because of the greater building wealth exposed to earthquake hazards. When adequate data are available, coupling ground-shaking hazard maps produced with the probabilistic methodology (Figures 10-12) for a given exposure time (for example, 50 years) with the inventory is the most effective way to develop estimates of losses.

The procedures for assessing losses from, the ground failure, surface faulting, and tsunami hazards are similar, but are not as well developed as the procedures for ground shaking.

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RETROFIT OF EXISTING BUILDINGS

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INTRODUCTION

Many of our present day structures may be considered to be "Potentially Hazardous Buildings." This term may include structures either not conforming to codes or those types of structures which have been known to be susceptible to seismic activity. The types of buildings which can be so considered are discussed below:

> Before beginning the list, it must be emphatically pointed out that not all structures in the separate categories can be considered potentially hazardous. Some of the structures within some categories could well be considered to be well constructed buildings capable of resisting seismic forces as evaluated in the Introduction.

Some of the potentially hazardous buildings located in the United States can be categorized as follows:

1. Unreinforced masonry structures.
2. Buildings constructed using concrete beams and columns but having weak infill walls.
3. Non-ductile concrete frame buildings.
4. Concrete tilt-up buildings constructed prior to the 1973 code requirements.

5. Some current concrete tilt-up buildings which are really precast concrete structures with large window openings whose frame does not comply with either ductile concrete frame requirements or shear wall requirements.

6. Buildings having a "Soft Story" within the structure.

This is usually the ground floor level.

7. Large open structures with high walls such as theaters, auditoriums, churches, and gymnasiums which were constructed prior to 1933.

8. Buildings with exterior heavy cladding of granite, marble, precast concrete, or masonry panels not properly attached to the structure to allow for motion during seismic activity.

9. Structures having high unreinforced and/or unbraced parapet walls.

10. New construction which has been built not in conformance with good seismic design or construction practice.

A short commentary on the retrofitting of each category follows:

1. Unreinforced masonry structures are discussed in some detail later in this paper.

2. Buildings with infill walls must be braced with rigid elements such as shear walls or braced frames in order to prevent motion which would damage the structure. Removing and replacing the infill with other material would help, however, it is highly unlikely that the concrete frame would be capable of resisting seismic loading conditions.

3. The non-ductile frame designed and constructed prior to ductility requirements can best be braced by introducing braced frames or shear walls.

4. Tilt-up buildings constructed prior to 1973 generally are lacking in diaphragm strength and connections to the walls resisting lateral forces. The diaphragm in general may be upgraded to resist the increased load code force requirements of about forty percent (40%), and both in-plane and out-of-plane connections at the walls must be strengthened. All of this work can be accomplished at a time when the structure is to be reroofed. With all of the roofing material taken off, the upgrading of the nailing of the diaphragm especially at the edges can easily be accomplished. Additional bolts for forces parallel to the walls can be added and connections for the out-of-plane forces installed. Chord forces should be checked and angles similar to those used for unreinforced masonry as detailed later in this paper can be added if necessary.

5. These structures must be looked at on an "as is" basis. Interior shear walls or braced frames may be used to upgrade these buildings.

6. "Soft Story" structures are usually found in the mid-rise to high-rise structures, however, it may be possible that three story buildings could be constructed with a soft first story. The term "soft story" is merely a designation for a flexible moment resisting type of bracing system. Its drift will be greater at the soft story than at other stories. The Olive View Hospital and the El Centro County Services Building are two structures which fit this category and have been damaged by earthquakes. Their upgrading would be easiest through the addition of shear walls or braced frames.

7. Large open structures have had many collapses in past earthquakes. Churches in Italy in 1976 and in 1980, in Peru in 1970, in the Philippines in 1976, in Chile in 1985 collapsed or were severely damaged. A large auditorium collapsed in the Philippine earthquake of 1976. A gymnasium was damaged in the Anchorage quake of 1964. A theater in Coalinga in 1983 suffered heavy damage, while in Peru in 1970 a theater collapsed killing over 400 persons. These structures all have in common several problem areas:

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a. Their high unbraced walls may not be adequate to resist out-of-plane forces

b. The connection of these high walls to the roof structure to withstand out-of-plane forces will in all probability be insufficient.

c. The roof bracing system which is probably a horizontal steel truss will also not be able to carry and distribute seismic forces

d. If the roof has a wood diaphragm, in all probability it too will not be able to resist the lateral forces adequately.

To correct some of these problems, roof diaphragms or horizontal trusses may be utilized as required. Chords must be developed and connections properly added. Shotcrete (Gunite) may be applied to exterior walls where in-plane or out-of-plane forces cannot be handled by the existing walls.

8. Exterior cladding not properly attached is a difficult issue to resolve. Upgrading of connections is practically impossible. Removal and replacement of cladding using new seismic connections which will allow for some motion during seismic activity may possibly be the only viable solution

9. High unstable or unreinforced parapet walls are dangerous and in almost all earthquakes collapse of these walls is noted. The solutions are relatively simple. They may either be braced back to the structure, or be removed or at least lowered in height to a safe dimension.

10. New structures which do not conform to code or to good seismic construction due to configuration, design assumptions, final design, or construction must also be handled on an "as is" basis. Differing alleged deficiencies must be corrected as required. Our present litigious society makes it imperative that prope design and construction be implemented to the best capabilities of all involved. It is unfortunate that these structures exists, but they do. The solution to this category is to attempt to avoid it by continuing education programs for Architects, Engineers, Building Officials, and Contractors.

In all of the above areas where retrofitting may be required, a strong effort should be dedicated toward the sensitivity for the motion of the various portions of the building with some emphasis on the relative motion of differing resisting elements.

The installation of concrete shear walls always creates problems due to the added weight. This weight contributes to higher seismic forces and increased soil pressures under footings. The introduction of collectors to pick up the shear forces to be delivered to the shear wall also creates additional construction problems

Shear walls and braced frames also introduce overturning effects which must be analyzed along with increased soil pressures.

The all important issue to address in retrofitting is a life safety issue. What can be done to prevent collapse of the structure and prevent injury or death to occupants? Some retrofit requirements address that new issue only, acknowledging that structural damage will occur but hopefully the life safety issue has been resolved.

Unreinforced Masonry Structures

The retrofitting of existing structures to conform to seismic requirements has lately drawn much attention, especially since the City of Los Angeles introduced an ordinance requiring certain unreinforced brick masonry buildings to be retrofitted to conform to new seismic code requirements. Other cities in California have also considered implementing some similar requirements. A few words relating to the proper retrofitting concepts are therefore certainly in order. First of all, most of the buildings to be retrofitted are buildings which are constructed of unreinforced brick masonry bearing walls with wooden floor and wooden roof structures, such as those which collapsed in earthquakes in San Fernando, Santa Barbara, Bakersfield, Coalinga, Long Beach and others.

The Coalinga earthquake demonstrated that extreme damage can and will occur to a variety of structures if they are not earthquake resistant.

However, as previously discussed, the question of deformation, and how to account for it, which is of extreme importance, is even more so in this type of construction since these unreinforced masonry buildings are very brittle structures which can withstand only very minor deformations before failure occurs. Therefore, proper logical concepts which take into account this limitation of motion must be utilized. For example, taking a one story building, rectangular shape, with a flat roof it would be possible to retrofit the roof, so that diaphragm action can be introduced thereby tying the building together. This can be accomplished by removing the roofing, placing a layer of plywood, to act as a diaphragm, on the existing structure properly nailed to take lateral loads, and then reroofing. The deflection of the new diaphragm should be carefully considered in light of brittle unreinforced walls. Anchoring the walls to the roof structure may be accomplished with bolts "A" or "possible anchors" as shown in Figure 1. Anchors bolts "A" must be embedded sufficiently to prevent pullout under design loads. It could be necessary to provide through bolts with a plate washer on the exterior face of the wall (shown with dotted lines in Figure 1). Testing for the capacity or allowable load of the anchors in the wall must be performed prior to design in order to establish reliable values for these bolts.

It is also necessary to introduce chords at the sides of the diaphragm to provide a complete design. This new diaphragm now transfers the loads to the four walls. These walls, of unreinforced masonry, may not be sufficiently strong to take the lateral forces, and therefore, the introduction of some new bracing element may be necessary depending upon the strength and dimensions of these walls. These new elements, however, should not be very flexible moment resisting frames designed to take the lateral forces, since if too flexible the frames would tend to deflect much more than the masonry wall could distort without cracking. If so, the masonry wall would still tend to take the load and, in fact, would fail prior to the frame being able to take the load. The frames that have been introduced could perhaps prevent the structure from collapsing because they would take the load after the brick has failed, but the proper design and retrofit for a building of this type would be to introduce a structural system which would be equally rigid or more rigid than the masonry wall. This could be either a poured-in-place concrete (or shotcrete) wall on the inside of the masonry wall, a very stiff moment resisting frame, or a braced frame.

An example of a possible solution to provide for this retrofit is shown in Figure 1.

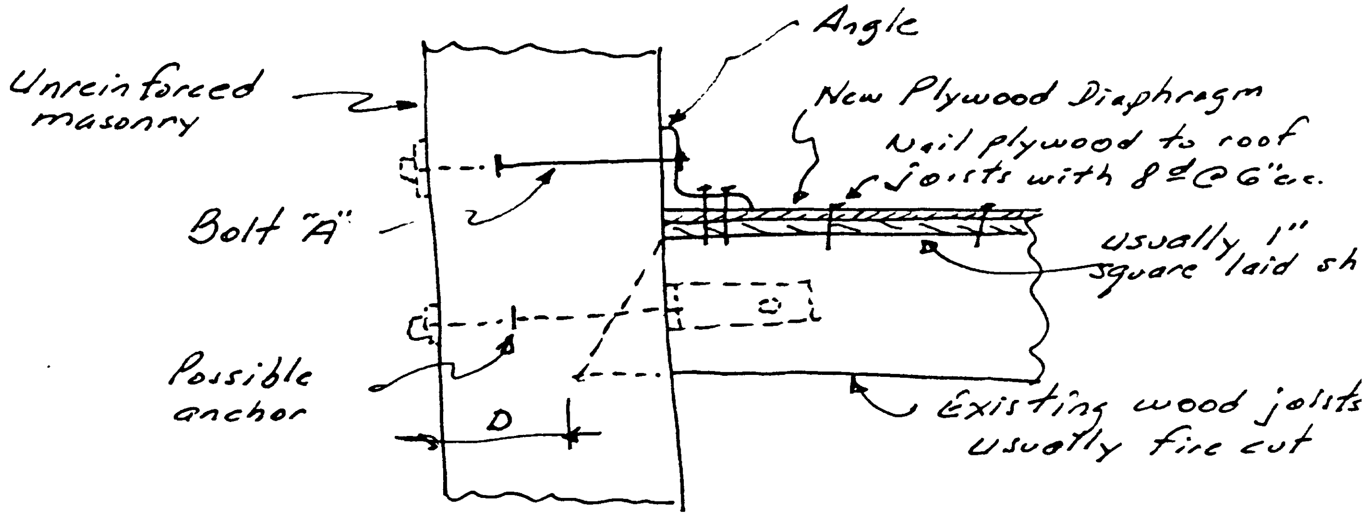

Figure 1.

The retrofit work may be done from either above or below the floor or roof in question. Figure 1 shows how it can be done from above.

Prior to commencement of any work, testing must be performed, in random locations, to determine allowable in-plane values for unit shear, shear modulus, and tensile strength values of the wall and also for allowable shear and pullout values of bolts. Determination of out-of-plane stresses must also be checked and verified by testing. The in-plane shear should be checked within the area shown in Sect. X-X in Figure 1. For example, if joists are at 16" centers and a brick wall 13" thick is being used, then the shear area is 13" X 16" less a 2" X 2 1/2" hole (approximately) or a net area of 203 sq. in. and if a 7 p.s.i. stress on the net area of the wall has been deemed to be acceptable, then the wall can carry a shear of 1421 lbs./16" or 1068 p.l.f. (In-plane requirements are those parallel to the wall and out-of-plane requirements are those perpendicular to the wall).

The walls should also be carefully checked for cracks which could have developed due to differential settlement, temperature stresses, prior earthquakes, excessive loading, etc. These cracks could effect the ability of the existing structure to resist new loads as determined by using a new diaphragm.

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The procedure for the construction sequence to retrofit the simple rectangular one story unreinforced brick masonry building would be:

1. The existing roofing membrane is removed.

2. A new 1/2" plywood diaphragm is installed making sure that at edges of plywood, the shear can be properly transferred to the adjacent piece, being careful of possible joints in existing sheathing matching joints in plywood where shear transfer could be interrupted.

3. Drill and install anchors and bolts into the unreinforced masonry wall. Spacing should be determined by both in plane and out of plane requirements. Special inspection and testing of drilled anchors should be made to ensure good reliable installation. Prior testing had determined allowable value for properly installed bolts.

4. Install the chord members. An angle, as shown, could be used as a chord which can be nailed to the plywood to develop the proper shear transfer. This shear transfer could be accomplished by one or two rows of nails as required by the calculated shear. See Figure 1. The chord must have a splice capable of transferring the chord stress as needed. As long a piece as possible could be used at the center of the chord thereby reducing the force to be transferred at the splice

5. After completion of this work around all four walls, the building can be reroofed and while expenditure has not been great, much has been done to improve the seismic resistance of this building.

After completing the design for the diaphragm, it becomes necessary to check the resistivity of the four walls. This requires the checking of in plane as well as out of plane stresses. First we will deal with in plane forces.

In general, the front wall of a building, especially in a business district, is mostly open. Therefore, the existing masonry wall would, in all probability, be unable to resist the seismic forces. A new bracing element, therefore, would be necessary to prevent in-plane failure of the wall. There are several possible solutions:

Prior to discussing the possible retrofit solutions, a brief discussion of motion is necessary.

As was previously mentioned, the allowable drift limitation from floor to floor is given by the code as .005 times the story height. This translation of .005H will, in all probability, be increased considerably due to seismic events which can easily develop force levels of several times the design coefficient.

This translation is reflected in the structure in two significant ways:

1. This is the out-of-plane distortion. This out-of-plane distortion is due to three factors

    a. The distortion due to the horizontal diaphragm;
    b. The distortion of the vertical elements resisting the diaphragm forces; and
    c. The bending of the wall itself as it spans between horizontal elements.

    Items A and B make up the drift while Item C contributes only to the bending stresses in the wall itself.

2. The second is the in-plane distortion. This in-plane distortion is due to two factors:

    a. The flexural distortion of resisting elements; and
    b. The shearing distortion of the same resisting elements.

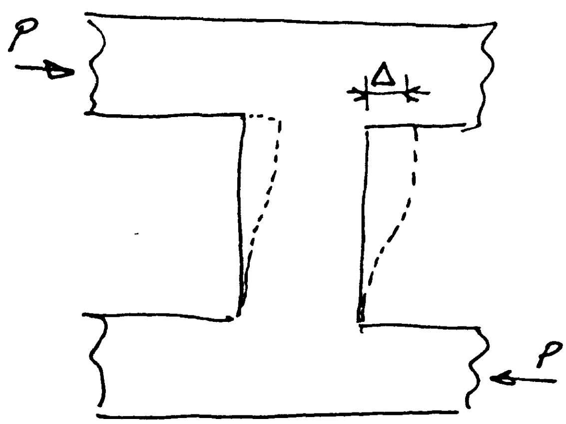
The very important issues to keep in mind are these distortions. The out-of-plane distortion creates primarily bending stresses while the in-plane distortion stresses create primarily shear stress (and sometimes bending stresses).

Since unreinforced masonry buildings are essentially constructed with few openings, they will act primarily as shear wall structures, and as such, the shearing distortions may well constitute the majority of the lateral motion. In a structure such as the one shown in Figure 2, the flexural distortion will exceed the shearing distortion:

The total distortion of piers is given as:

$$\Delta = \frac{Ph^3}{12E_mI} + \frac{1.2\,Ph}{AE_v}$$

Flexural Shearing Distortions



Where:

Δ	=	Total distortion
P	=	Load at top of pier
h	=	Height of pier
A	=	Area of pier
I	=	Moment of inertia of pier

E_m = Modulus of elasticity of masonry

E_v = Shear modulus of masonry

A good reference to use would be the "Masonry Handbook" which explains the calculations very well.

It is easy to predict that moment resisting frames will have a difficult time resisting the load without deflecting beyond the capability of the masonry wall to deflect without damage. In this instance a maximum deformation in the magnitude of about 0.1 inches is about all the movement the wall can take without damage. However, if the brick can be supported by the moment resisting frame to keep it from collapsing, then the only problem after an earthquake would be a "cosmetic" reconstruction rather than a structural reconstruction. The methods of possible retrofit are described as follows: 1) Moment Resisting Steel Frame, 2) Moment Resisting Concrete Frame, 3) Braced Frame (Steel), and 4) Concrete Shear Wall.

1. Moment Resisting Steel Frame

Assume the existing front of the structure to be as shown in Figure 2. Note the path of the lateral forces in Figure 3. They pass from the new plywood diaphragm through the nails to the steel angle chord, then from the angle through the bolts to the masonry wall, down the wall through the restricted area at "D" (Section A), then through the bolts into the steel channel, through the weld from the channel to the frame, then down the columns into the ground. Quite a path. Yet, each link is absolutely required. Any inadequacies and the entire design could prove useless. As mentioned before, a flexible moment resisting frame may not suffice. The important criteria in this scheme will be the lateral deflection of the frame when loaded with force F. If the deflection exceeds the distortion that can be accommodated by the brick piers, the piers will fail. Therefore, a limiting deflection should be established. This could now mean a redesign of the frame on a basis of deflection rather than stress. The allowable distortion of unreinforced brick is a difficult parameter to assess, quite likely it will vary substantially from structure to structure depending on quality of workmanship and materials used at time of construction. Testing of walls prior to design will give some indication of usable data.

2. Moment Resisting Concrete Frame:

The most expedient way to construct this frame would be to follow the same dimensions as the brick walls. This will result in details shown in Figure 4.

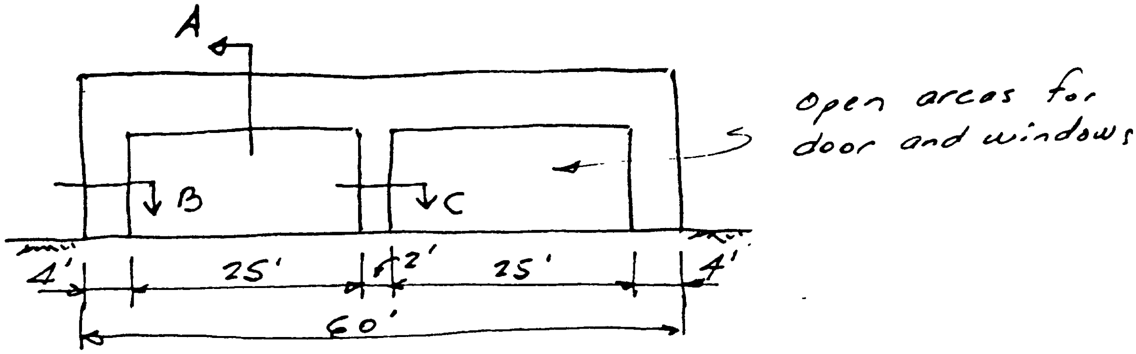
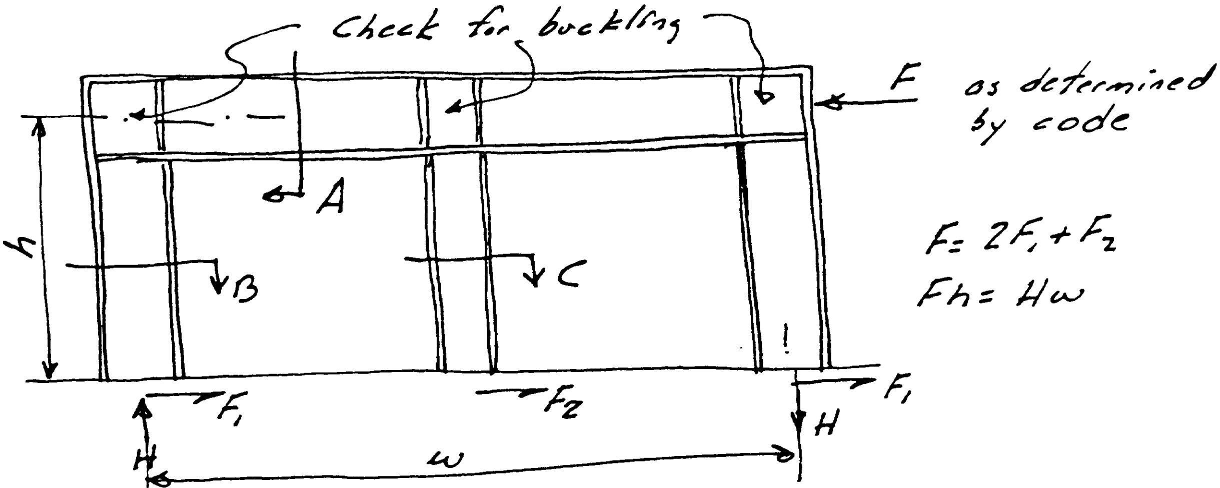


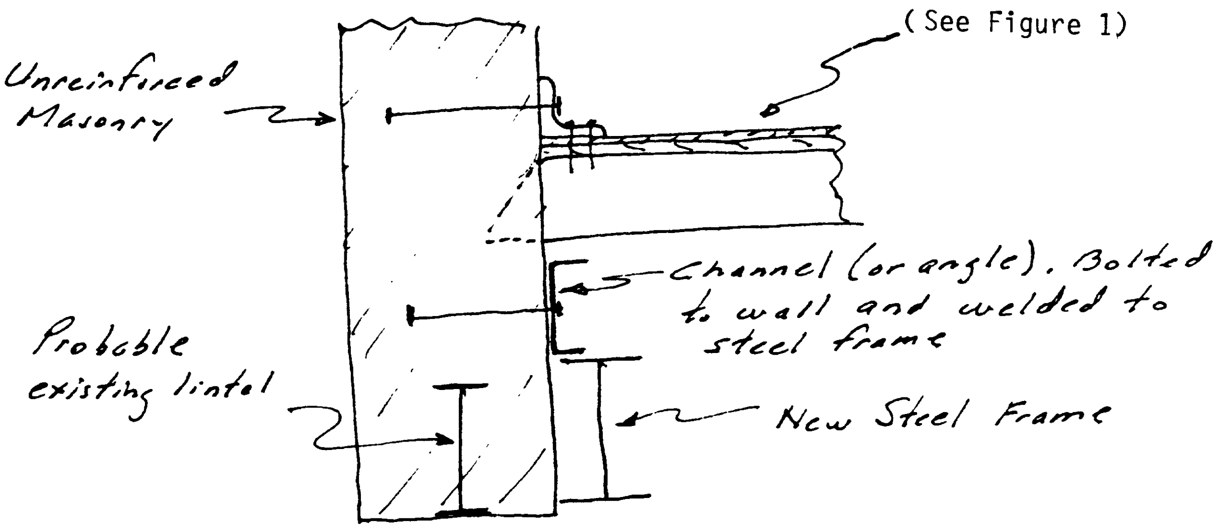
Figure 2

1. The moment resisting steel frame:



Steel frame with Wide Flange or built-up sections to resist design loads and allowable distortion.

ELEVATION OF FRAME:



SECT A

Figure 3

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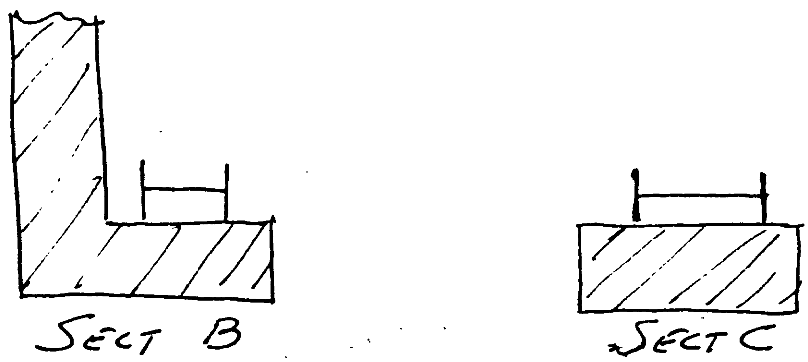

(Figure 3 continued)

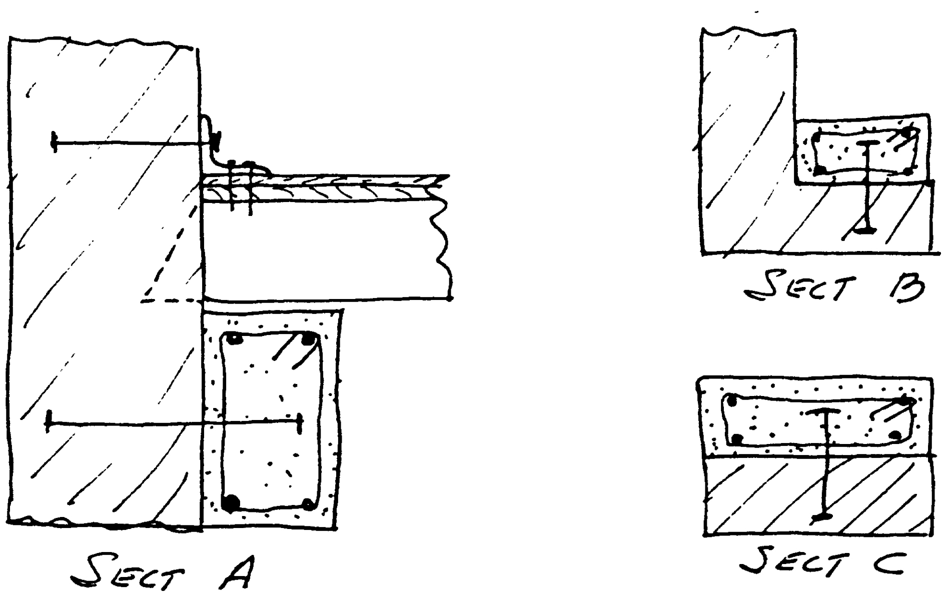

Figure 4

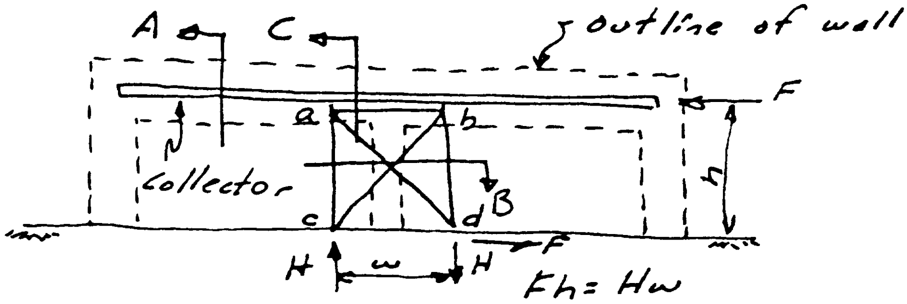

Figure 5.

The procedure here follows exactly the same pattern as for the steel frame. See also Chapter Six for information regarding Ductile Concrete Moment Resisting Frames.

3. Steel Braced Frame:

The braced frame will provide a stiffer bracing element because members are in tension and/or compression an not in bending. The configuration of the braced frame could be as suggested in Figures 5 and 6.

Observe that in using the braced frame, a new element of design is introduced; the overturning problem of the frame itself. Forces H in this case can be considerably greater than the forces of the moment resisting frames. These larger forces create a situation where adequate foundations may have to be introduced to resist this overturning. In addition, since the frame is rather light there will be insufficient weight to adequately "hold down" the frame. Dead weight in the form of heavy foundations, to which the frame is anchored, must be introduced. In Figure 6 member bd could be bolted to the exterior wall to utilize some dead weight of the existing structure to overcome overturning forces.

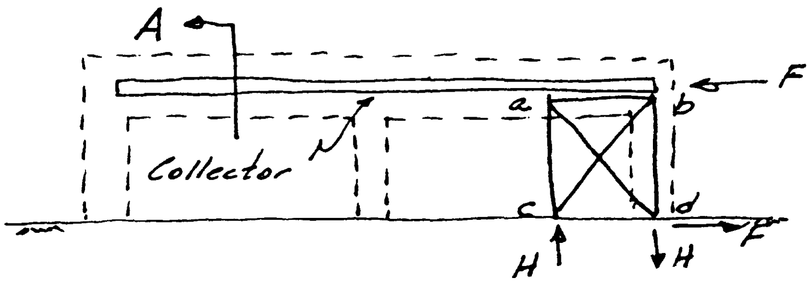

Figure 6.

Again the same basic transfer of loads must beaccommodated, as shown in Figure 7. Observe that collector in Figure 6 will have to carry about twice the load of the collectors in Figure 5. However, the load in the braces are identical.

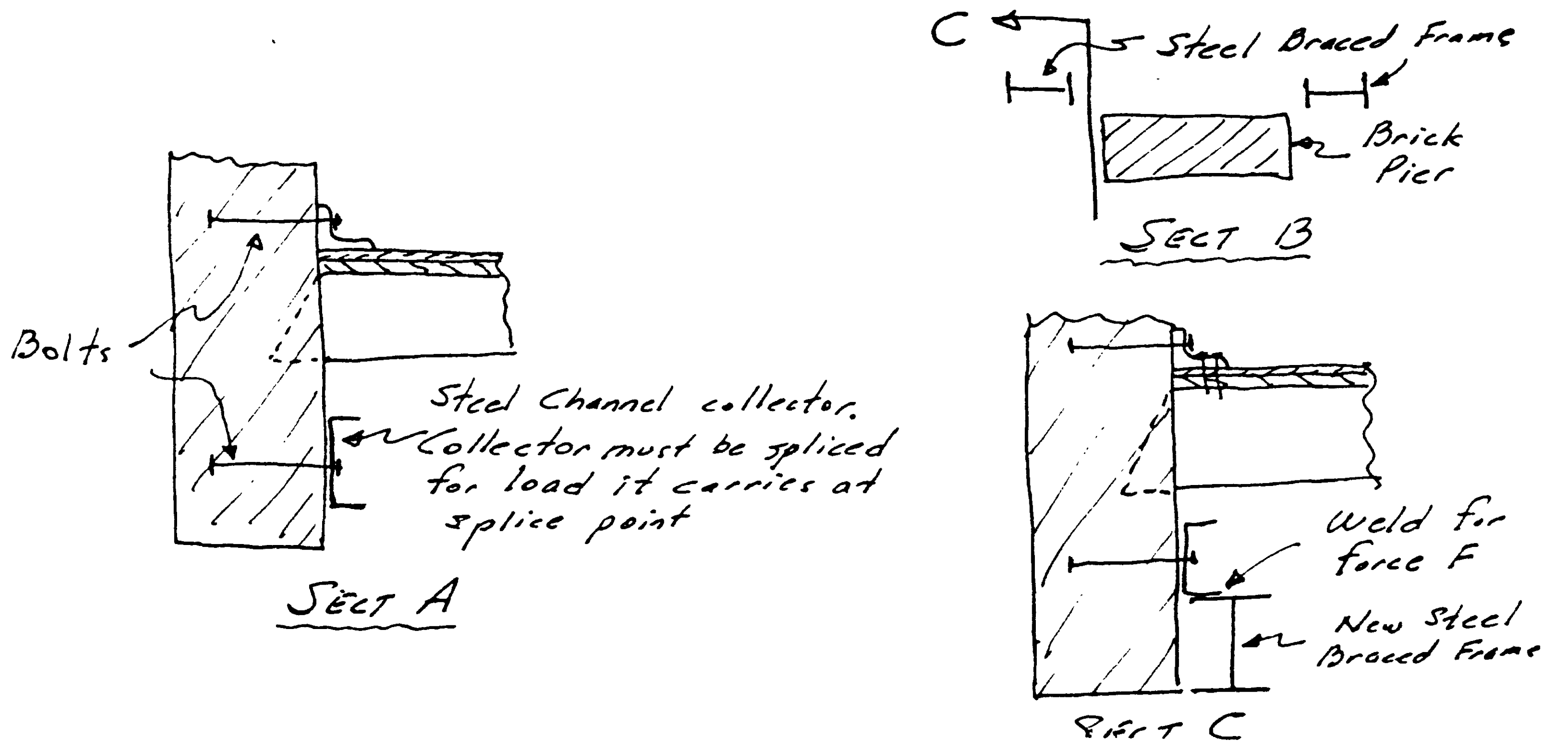

Figure 7.

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Overturning introduces another item to be investigated in the design. In addition to the requirements to prevent overturning, consideration of its affect on the structure should be investigated. If, for example, the soil settlement due to the increased soil pressure is one-eighth of an inch then in the case above where the height is about one and a half times the width, the motion at the top along the collector would be approximately three-sixteenths of an inch. This results in a horizontal motion due to rotation. Other elements of motion must be added to this to arrive at the final distortion.

4. Concrete Shear Wall:

The possible locations for the shear walls will be the same as for the braced frame. See Figure 8.

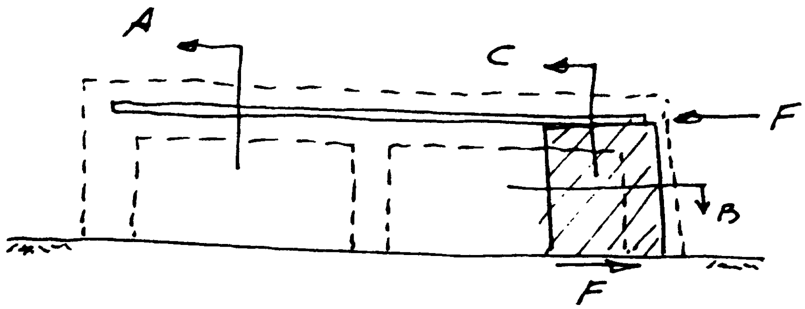

Figure 8.

The same overturning considerations must be included. In this case, the weight of the concrete wall will aid slightly to help overcome the need for all of the dead weight that was required for the steel braced frame. Section A of Figure 8 will be similar to that used for the steel braced frame. Sectins B and C are shown in Figure 9.

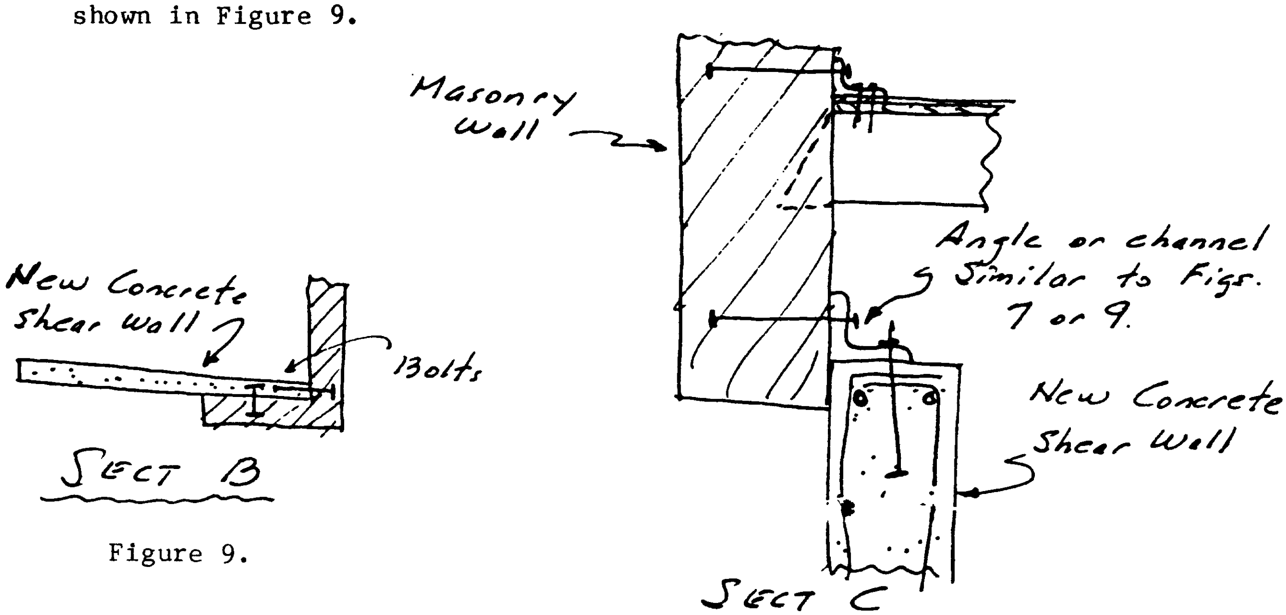

Figure 9.

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In this case, replacing the weld from the collectors to the braced frames, we have the connection of the angle collector to the top of the concrete shear wall with anchor bolts.

A good exercise fpr the reader would be to design each of the five configurations, shown above, detailing all connections for each different scheme and writing a brief description of the path that the lateral forces are taking to get to the ground.

Advantages and Disadvantages

An additional exercise would be to add to each scheme the anticipated cost of construction with the sequence of work and time required to complete the work. All design work must always be adequate for the forces involved and the construction must be economical and constructable within a time frame agreeable to the tenant.

In view of the above, it will be well to go back to the above five configurations and point out advantages and disadvantages of each scheme. Cost factors will not be included here since they will vary from area to area, and in fact may even vary within an area due to the locale of the structure to be retrofitted. Therefore, the reader should keep in mind that the cost factor, which may in fact govern, is not taken into account in this discussion. Items, however, leading to the differences in cost will be investigated.

1. Moment Resisting Steel Frames:

   Advantages: The principal advantage is that this scheme will probably fit behind the existing walls thereby eliminating the necessity to do any work related to the front of the building. It is relatively light, so that additional foundations to carry its weight will be easily handled. The transfer of lateral forces at the ground level may be relatively easy in this case.

   Disadvantages: The sections may be heavy and require a crane for installation. Details at beam-column areas will require expensive fabrication. Field welding or high strength bolts will be required to develop moment connections. The frame may be relatively flexible and may require large sections to reduce its flexibility.

2. Moment Resisting Concrete Frame:

Advantages: The configuration of the frame MAY fit behind the existing walls. All materials, forms reinforcing, bolts, concrete can be easily delivered and put in place without use of hoisting equipment. Only a pump for the pumping of concrete will be necessary and that piece of equipment can remain outside of the structure.

Disadvantages: The dead weight will be greater than the steel frame and will probably require foundations in addition to making provisions for the taking out of lateral forces at the ground level. There may not be enough headroom over the openings to develop the depth necessary for a concrete beam, this would necessitate an impingement of the window area. Sizes of columns may also impinge upon the window area.

3. Steel Braced Frame:

Advantages: Steel members will be much smaller in size than the frame. All connections are simple connections requiring only bolting. Handling is easy.

Disadvantages: Taking care of the overturning moment and the resulting large uplift forces will present the greatest difficulty in completing this scheme. The foundation requirement and dead weight problem may be expensive. The space requirements may be such that the configuration will require a redesign of the front window areas. The scheme to be selected will depend upon architectural requirements.

4. Concrete Shear Wall:

Advantages: The shear wall is the least sophisticated construction method. It will require only simple forming, reinforcing, and pouring of concrete.

The collector can easily be anchored to the wall, and the weight of the wall will help with overturning requirements.

Disadvantages: The weight of the wall will require foundations. Also, overturning will have to be considered. The bolting to the existing masonry walls will help in one direction, but provisions will still have to be made for overturning in the other direction.

One aspect to note is that all schemes are practically alike in the transfer of the seismic load to the new resisting elements.

Odd shaped structures; such as an L shaped building, can be handled by using extended chord members to act as "collectors". See Figure 10.

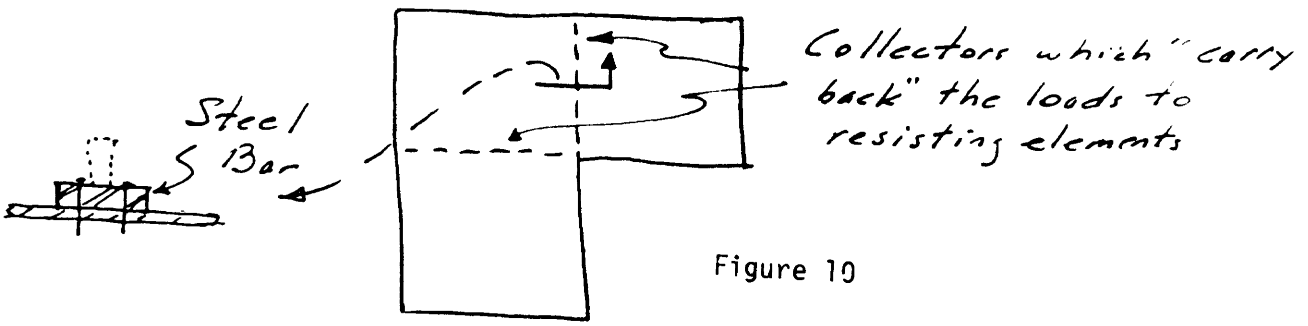

Figure 10

These collectors can be flat bars nailed to the diaphragm and welded to angle members at the walls. Care should be taken to consider compression forces in these members and possible buckling problems. A tee could serve well as a member, however, roof drainage problems must also be considered

In order to reduce costs of retrofitting and provide someseismic resistance, the City of Los Angeles has reduced the lateral load requirements for these buildings in their code. A suggestion would be, however, that when a scheme has been selected as the bracing configuration, that as much load as possible be assigned to that scheme where it can still function without a major change. Sometimes it is possible to increase the carrying capability of a system by the addition of a few more bolts or slightly heavier plates. In the overall project, these small additional details might impact the costs in the one to five percent range but the capability of the system could be appreciably increased. Good engineering judgment will serve the owners well in these situations.

Narrow vertical resisting elements with long steel collectors should be carefully analyzed. Overturning could be one problem to overcome, but more seriously could be the elongation of the collector. Assume a collector is one hundred feet long and carries back to the bracing element a force of one hundred thousand pounds. If anchors drilled into the unreinforced masonry are good for two thousand pounds each, then fifty such connectors could be drilled at two foot centers. The problem lies in the fact that the bolts nearest the brace element will take considerably more load than the bolt farthest away. This is due to the elongation of the collector at the most highly stressed area. This means that motion will occur which cannot totally be accommodated by the first bolt. Therefore, it is possible a "zipper effect" could take place which would result in a failure of the connections performing as anticipated. The designer should investigate the potential problem.

In solid walls, in plane stresses should be checked to assure the capability of the wall to take these stresses. If weak or fractured walls are found, the possibility of the use of epoxies should be investigated to check the likelihood that the wall could be strengthened sufficiently to take the anticipate loads. Sometimes it may be necessary to inject epoxies into walls in order to activate all units, since at times hollow spaces may be found in these walls which would tend to reduce the capability of the in plane forces to be resisted by bot outside masonry units forming the two faces of the wall.

If through testing studies it has been found that the wall does not have sufficient strength, then some scheme must be devised to strengthen these walls.

Testing procedures for the determination of allowable stresses in existing unreinforced brick walls are usually made by coring holes or by removal of certain bricks and replacing them with jacks to determine mortar strength. However, after the Italian earthquake of 1980 had devastated much of the housing in the Campania-Basilicata area, a special testing program for determining in plane strength of masonry walls was developed by Italian engineers. Figures 11 and 12 show some examples of damage.

This on site test will result in the development of allowable stresses for a type of construction used over one hundred years ago. It also served to bolster the

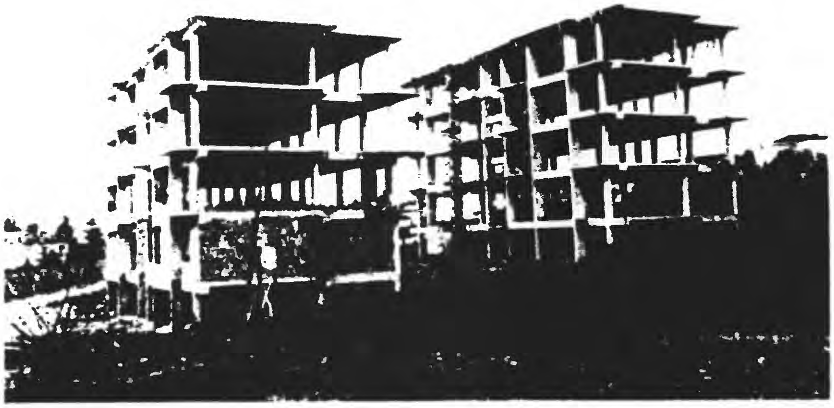

Figure 11. Damage to one of twin residential units under construction. Note that infill walls have caused damage, but the "pure" frame had no damage.

Figure 12. Relative new 10 story apartment building in Naples that failed in earthquake.

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confidence of the populace for the reconstruction of damaged houses, plus the construction of new structures. The above test shown in Figures 11 and 12 were witnessed by a group of American Engineers and Architects sent to Italy by the National Science Foundation in order to exchange information on the retrofitting of unreinforced masonry construction.

Configuration Difficulties

The retrofitting of structures other than unreinforced masonry may sometimes be deemed to be necessary.

The following is an actual case of a retrofit: Figure 13 shows a plan view of the structure, and Figure 14 shows a cross-section.

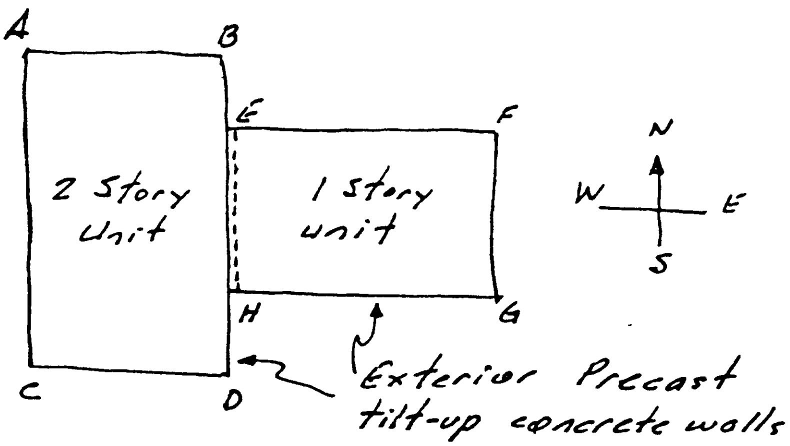
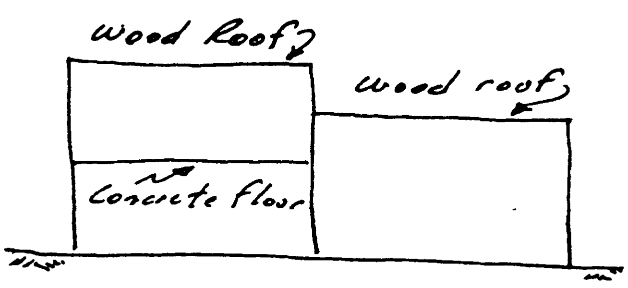


Figure 13

Figure 14



The structure shown comprises a two story concrete tilt-up unit with a concrete diaphragm at the second floor and a wood roof diaphragm. The one story unit is a concrete tilt-up building with a wood roof diaphragm.

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From a configuration point of view, there are no particular problems in the north-south direction. However, in the east-west direction there were several:

1. When the motion of the earthquake would force the two story units to sway toward the west, this two story portion would tend to pull away from the one story unit at points E and H, because of the deflection of the diaphragms and because of the rigid walls EF and HG in that direction. In addition, the two story unit would impose additional load to the one story unit by pulling on its roof diaphragm connection between the second floor and roof.

2. When the motion of the earthquake would force the two story unit to sway toward the east, this two story unit would now push against walls EF and HG, forcing them to absorb practically all of the seismic load. In addition, the deflection of the roof diaphragm toward the east would impose a loading condition onto the roof diaphragm of the one story structure.

3. There were no connections at points E and F to transfer these loads. (It is questionable whether or not such a connection could be developed in the tilt-up panels.)

4. The deflection of the diaphragm of the one story unit would tend to push against wall BD of the two story unit or pull away from wall BD depending upon the direction of motion.

5. Since the two units have an obvious difference in the period of vibration, the two units would at times be pounding together and at other times trying to pull apart.

The possible consequence of the above conditions could have led to the following possible failure modes:

1. With the two story unit tending to more toward the west and the one story unit toward the east, three possible modes could develop:

a. The one story unit roof would pull away from wall BD and having no support would collapse to the ground in a failure similar to ones noted in the San Fernando, California quake of 1970.

b. If the connection at wall BD were strong enough (doubtful) then a failure of wall BD between the second floor and roof could occur due to the bending at that location. See Figure 15.

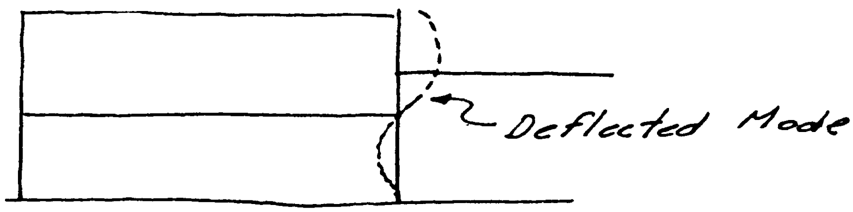

Figure 15.

c. Failures along the walls at points E and H could be anticipated due to the movements.

2. With the two story unit tending to move toward the east and the one story unit toward the west, two possible modes of failures could occur:

a. Wall BD could fail due to distortions of diaphragms, as shown in Figure 16.

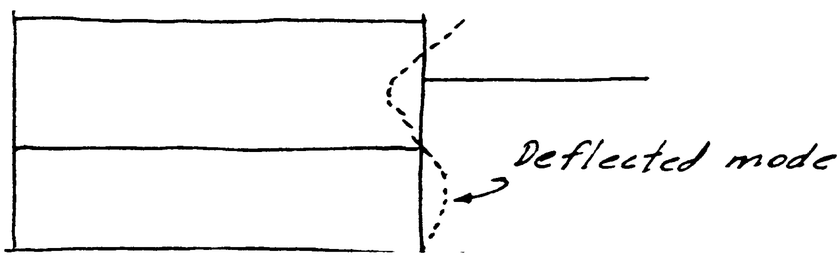

Figure 16.

b. The concrete second floor diaphragm would impose almost its full load onto walls EF and HG at points E and H. Localized failure at these points could occur. Also walls EF and HG could fail due to this load.

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c. An additional source of problem could be the difference in distortions of diaphragms of the two story unit. Since the wood roof diaphragm is more flexible than the concrete diaphragm at the second floor, stresses in the exterior walls could be introduced by motion as shown in Figure 17.

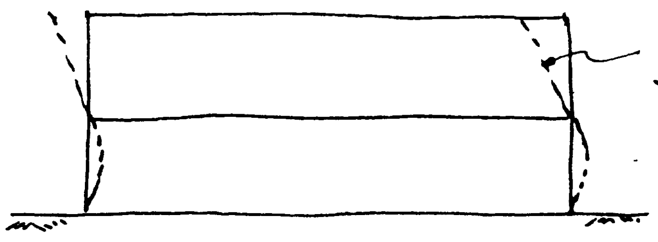

Figure 17.

The retrofitting of this structure to avoid the above problems was a relatively simple one. The structure was cut at line EH, thereby separating the two story unit from the one story unit. The separation of four inches was easily made and an "X" bracing system was introduced for the one story unit near line EH to take the load of the one story unit in the north-south direction. In addition, the vertical load of the roof at that point was also carried by the new frame.

Had this structure been constructed using poured in place concrete walls and roofs, it would have been possible to brace the structure with the same configuration. The reasons being that:

1. The motion of the diaphragms would be much smaller, and

2. That transfer of loads at points E and H could have been accomplished with embedment of reinforcing steel to transfer the computed loads.

Additional improvement to the poured in place structure to prevent damage would be to place the roof diaphragm of the one story unit at the same elevation as the second floor diaphragm of the two story unit. With a poured in place structure, reinforcing steel could have connected the two units to provide a good seismic resistant design. The situation described above would be nonexistant.

We can conclude, therefore, that in addition to the configuration of buildings, the type of materials used and their interconnection is vital.

REMODELLING OF EXISTING STRUCTURES

The addition to or the remodeling of structures may at times create problems with the structural integrity of the original structure. Adding to a building will not necessarily make it stronger. An example where adding to a structure could cause a collapse during an earthquake is shown in Figure 18.

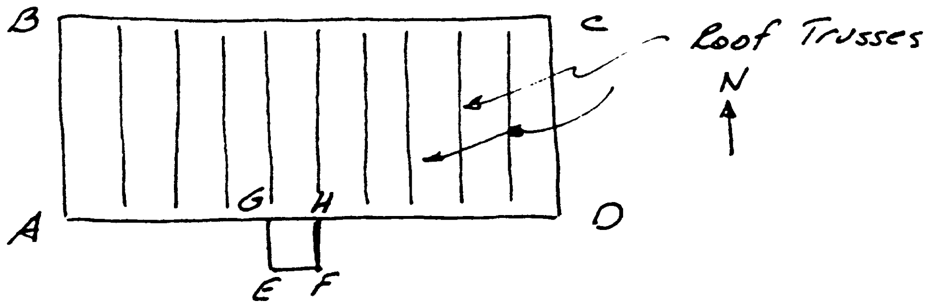

Figure 18.

Assume we have an existing simple one story building with a plywood roof diaphragm noted ABCD. It is desired to add a small flammable storage area about twenty feet by twenty feet EFGH. Since it is a flammable storage area masonry walls must be used. In order to do a good job the walls GE and HF will be anchored to the building columns at G and H. As was shown in an earlier chapter, during an earthquake the plywood diaphragm ABCD will deflect, but now walls GE and HF will tend to prevent this motion, but there are no collectors at lines G and H nor adequate anchorage of the roof structural elements at columns G and H to deliver any sizable load. During an earthquake, therefore, when the building deflects in a northerly direction, columns G and H cannot deflect because they are restrained by walls GE and HF and the roof structural elements could simply fall off of their supports at these points. If the motion were to be in a southerly direction, the connection of the roof structural members would fail and in addition walls GE and HF would probably be cracked by the deflection of diaphragm ABCD.

Another situation noted occurred in a reinforced concrete basement with a concrete waffle slab overhead. The columns wer on a 24' module, as shown in Figure 19.

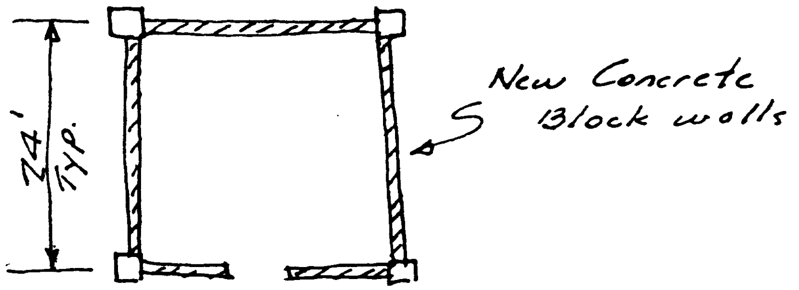

Figure 19.

New concrete block walls were tightly placed between the concrete columns with only one door into the area. These new masonry walls now created a situation of infill walls, such that during an earthquake these walls will not allow the columns to distort when the overhead diaphragm tries to deflect. The res@lt will be that the masonry walls will fail, and in so doing will in all probability cause a failure of the concrete columns as well.

Again the important point that is brought out here is that the relative deflections of the structural components must be considered for proper seismic design.

Unfortunately, many similar situations to the above occur all too frequently. The results will be noted following destructive earthquakes. The main cause of these types of problems are the lack of the owners seeking engineering advice on these apparently simple problems in remodeling. It is strongly recommended that owners seek professional advice whenever they consider doing any remodeling or additions to their structures regardless of the nature. It is not always evident what the implications may be of any proposed revisions.

In the remodeling of a existing structure, drawings of the structure should be carefully reviewed to determine the intended method of seismic bracing. A check of stresses can be quickly made with a few calculations. Lacking drawings, calculations should be performed to determine the probable means of seismic resistance that was used for the original construction. In both cases these calculations are necessary to prevent the removal of or cutting through walls which could have been or were used as shear walls.

The removal of steel "X" bracing systems to provide for easy access most certainly cannot be condoned without extensive research and additional structural bracing and probably collectors to make up for the elimination of part of the original design concept.

Cutting holes through existing concrete shear walls is one of the very difficult issues of remodeling. Great care must be taken. Problems of relative stiffnesses will immediately be created and furthermore provisions for bending moments in the walls which require reinforcing at opening edges create difficult conditions to resolve.

Another item to be kept in mind is the gradual upgrading of codes which might require some bracing elements that were not needed in the original design.

SOCIETAL RESPONSES TO EARTHQUAKE THREAT: APPLICABILITY TO PUERTO RICO

By

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INTRODUCTION

Although damaging earthquakes in most countries are rare events, substantial areas of the world are exposed to high levels of earthquake threat. By looking at those portions of the world that are threatened and by reviewing the research conducted following some major destructive quakes, there are some conclusions we can draw about how societies handle earthquake threat. This paper outlines some of these research findings (or "lessons") and investigates how they may be applicable for Puerto Rico.[1]

PUBLIC PERCEPTION OF EARTHQUAKE THREAT AND ACTION

There are three major findings which address the relationship between one's perception of the threat and how one responds to it.

1. People who live in seismically active areas are likely to believe that a future damaging earthquake could strike the community in which they live, but they may be no better prepared for that quake than people in less seismically active areas. That is, their definitions of acceptable risk are likely to be similar but for different reasons.

2. There are three factors that can raise public awareness: personal experience in an earthquake; recurring small earthquakes; and predictions that heighten public awareness of and preparedness for an

[1] The "lessons" presented in this paper were summarized from an article by D. D. Mileti and J. M. Nigg, "Earthquakes and Human Behavior" in Earthquake Spectra 1 (1984): 89-106.

earthquake. This heightened awareness, however, usually lasts for only a short period of time.

3. Public information and education programs about earthquake hazards and preparedness may raise people's awareness and concern levels, but their effect on long term preparedness has not been established.

Applicability for Puerto Rico.

Puerto Rico can capitalize on its "recent" (1918) experience of a damaging earthquake to remind people of the hazard and what problems occur when households and communities are unprepared. This is an especially useful technique when a comparison can be made between the characteristics of the effected area (in terms of population size and density, building stock, industry) then and now. Since local "dramatic events" (predictions or recurring earthquakes) can not be relied on to heighten the awareness of residents, damaging earthquake events in other Caribbean, Central American, or South American countries could be used as examples of problems caused by earthquakes. From research conducted in California,[2] it was clear that the 1976 Guatemalan earthquake had a dramatic impact on the Hispanic community in Los Angeles where extensive, long term coverage by the Spanish language media raised their levels of concern substantially.

From research recently conducted in the Central United States (along the New Madrid Fault zone)[3] where people are less likely to see themselves at risk from an earthquake (as in Puerto Rico), it appears that public education and information programs can be successful in three ways:

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2 For more information, see Ralph H. Turner, Joanne M. Nigg, Denise H. Paz, and Barbara S. Young, Community Response to Earthquake Threat in Southern California, Los Angeles: Institute for Social Science Research, UCLA, 1979 (especially Parts 2 and 6).

3 An NSF-sponsored research project is currently being conducted by Alvin H. Mushkatel and Joanne M. Nigg on "The Development of Earthquake Awareness and Seismic Policy: A Regional Approach in the Central States."

1. Raising the general level of knowledge about the existence of the threat.

2. Instructing people about what to do during an earthquake.

3. Instructing people about the things they can do to reduce potential injuries to their families.

PUBLIC DISASTER RESPONSE

There are four major findings concerning the general public's response to a disaster.

1. Communication systems, if still functional, rapidly become overloaded for two reasons. First, people try to contact relatives and friends to reassure them of their safety or to determine whether assistance is needed. Second, in emergencies, people require information to determine what to do and will often use the telephone to gather this information.

2. People generally act in rational and pro-social ways following a damaging earthquake.[4] Neither panic flight nor looting have been substantiated in the United States following an earthquake. People in the disaster areas frequently engage in altruistic acts and helping behavior. People who have jobs that are directly related to emergency work usually stay at work (especially if they know their families are safe). Services and goods are frequently donated to victims by those in nearby areas.

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4 This finding has been well established in the United States following natural disasters. However, whether it reflects the experiences of other countries is an empirical question. Given the relationship between the Commonwealth of Puerto Rico and the United States, it is highly likely that these "lessons" would be relevant for Puerto Rico.

3. Despite this pro-social behavior, major problems may still occur (e.g., spontaneous evacuations, over-crowding or under-utilization of medical facilities, uncontrolled convergence of people and goods).

4. Housing needs increase as time goes on, often resulting in the development of conflicts within the community and between the community and other levels of government.

Applicability for Puerto Rico

It is assumed that the same types of pro-social, rational, and information-seeking behaviors would be exhibited following a damaging earthquake in Puerto Rico as has been observed elsewhere, where neighbors try to help each other in the immediate post-impact period to restore normalcy to their lives.

One problem that could quickly develop following a destructive earthquake is temporary shelter. Large, public use buildings with cooking facilities (such as schools or auditoriums) are often unsafe following a destructive earthquake (e.g., in Mackay, Idaho) and cannot be used to house or feed the homeless. Even when people are told that their homes (especially multi-storied or brick units) are structurally sound, they may be unwillingly to reside indoors for several days (e.g., in Coalinga, California), necessitating providing them with camping tents or trailers for short periods of time. Does the Commonwealth have sufficient supplies and delivery systems to distribute them in a short time span?

GOVERNMENTAL PLANNING AND EMERGENCY RESPONSE

Five major findings can be summarized concerning the ways in which governments respond to earthquake disasters.

1. The larger the earthquake disaster, the more quickly the local emergency response units are overwhelmed.

2. To be most effective then, emergency response plans must be integrated across all levels of government with respect to communication, authority, and coordination of activities.

3. Damage assessment and reconnaissance teams are often duplicative in effort and may actually impede disaster recovery and confuse residents (as in Coalinga).

4. The ability to rapidly mobilize search and rescue teams and equipment (including medical personnel) is essential in reducing fatalities following earthquakes.

5. Governmental ability to successfully respond to major earthquake events is frequently impeded by incompatible or nonfunctional communication systems.

Applicability to Puerto Rico

Since many of the agencies that will have to respond to emergencies do not have a presence in every local community (e.g., the Department of Transportation and Public Works, the Department of Natural Resources, and the Red Cross), have they developed integrated plans to work with community governments? Will emergency service units be able to communicate with each other (across agencies and among the different levels of government) following a large magnitude earthquake that incapacitates the phone system? Because many local hospitals do not have emergency rooms or triage capacity and because paramedic units are scarce in parts of the Commonwealth, planning must be directed toward the provision of emergency medical care and toward incorporating medical resources into the coordinated response plan. For search and rescue operations of collapsed buildings to be successful, the availability of large machinary is a must. The cataloging of this equipment (whether publicly or privately owned) is essential to saving those who become trapped. Since Puerto Rico has not yet adopted building codes that require seismic design elements, this is an extremely important aspect of disaster response planning.

HAZARD MITIGATION

The issue of "acceptable risk" is especially pertinent when hazard mitigation measures are considered. In the States, the adoption of mitigation measures has often led to conflict between certain segments of the public (who are willing, seemingly, to accept higher levels of earthquake risk) and government entities. Therefore, it is necessary for government agencies that are responsible for public safety and welfare to know and present clearly what the losses are likely to be from a damaging earthquake (including the loss of jobs and production of goods). The problems faced in Puerto Rico with respect to major mitigation measures (improving building codes for new structures, improving and monitoring construction practices, and regulating land use) are similar to those in the Central United States.

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LESSONS LEARNED FROM RECENT EARTHQUAKES

by
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INTRODUCTION

The destructiveness of earthquakes has been studied for hundreds of years and records of those studies are available, dating at least as far back as the Lisbon, Portugal, earthquake of 1755. From that time until the early 1900s, there were few reports of damage from other earthquakes, but in the twentieth century, reports on earthquake damage have been abundant.

In 1971, the San Fernando, California, earthquake struck the seismic heart of the United States. An important lesson was learned from that event: that there simply were not enough earthquake specialists available to record all the valuable information that was laid bare. The need for a more organized effort was recognized. Subsequently, the Earthquake Engineer Research Institute (EERI) established what is now commonly referred to as the Learning from Earthquakes project to ensure that investigative responses to future earthquakes would be more comprehensive.

Table 1 lists the various disciplines involved in the study of earthquakes and indicates the current scope of a typical post-earthquake damage investigation. These investigations clearly cover a great deal of territory, but earthquakes can and do have an extensive and very detrimental effect on man and manmande works. In the past three decades, EERI has looked into literally hundreds of earthquakes and has reported on more than 100 in its newsletter and in special reconnaissance reports. Post-earthquake reconnaissance reports and other EERI documents consulted in preparing this paper are the first 22 in the list of references.

A report on the post-earthquake damage investigation typically presents only a sketchy description of the type and extent of the damage caused by the earthquake. For this reason, the lessons to be learned directly from these investigations are limited. More is learned from the in-depth studies

Table 1 Disciplines Involved in the Earthquake Problem

Discipline	Function/Purpose
GEOSCIENCES Geology Seismology Soil Mechanics	• Predict ground motion • Evaluate site soil stability • Evaluate soil structure interaction
ENGINEERING Civil Structural Mechanical Electrical	• Design structures to resist earthquakes and predict structural performance for specified ground motion levels
ARCHITECTURE	• Design architectural aspects of structures to resist earthquakes and predict the performance of architectural components for specified ground motion levels
URBAN PLANNING	• Provide urban development (or redevelopment) guidelines for minimizing earthquake hazards
SOCIAL SCIENCES	• Provide sociological guidelines for minimizing the impact of earthquake hazards on man

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conducted by dedicated researchers during the many years following an earthquake. Of course, the need for the more definitive studies is sparked by observations made during post-earthquake investigations.

Several recent EERI damage investigation reports conclude with statements like, "No new lessons were learned except . . . ," indicating that we have now seen virtually all types of earthquake effects. This does not imply that we should discontinue conducting post-earthquake investigations, however. We are still learning, as will be discussed, and there are other reasons for continuing the investigations as well. For example, people who are visiting an earthquake-damage-stricken area for the first time are left with a lifelong respect for the horror of earthquakes, which will help to make them more concerned earthquake specialists. Another reason for continuing to send investigative teams is that seriously damaging earthquakes occur infrequently in the United States. Without continued reminders, we may forget that the need to improve our earthquake-resistive designs still exists, as will be discussed later.

This paper is not intended to be a definitive evaluation of recent lessons learned. Instead, I have chosen to focus primarily on the engineering aspects of the earthquake problem. More detailed evaluations of the lessons learned and the lessons that need to be learned are planned as part of the EERI Learning from Earthquakes project in the coming years.

ON FAULT MOVEMENT CAPABILITY

On January 25, 1980 at 11:00 a.m., a magnitude 5.5 earthquake occurred in the vicinity of Livermore, California. The first question for the geologist was: Which fault broke? Geologic and seismic evidence indicated that it was the Greenville fault. Prior to this earthquake, the Greenville fault (Fig. 1) has been classified as "inactive" (more than 2 million years old). Detailed field investigations conducted during the first week after the earthquake revealed Holocene (the most recent geologic epoch spanning the last 10,000 years) activity indicating that the Greenville fault should be classified as "active."

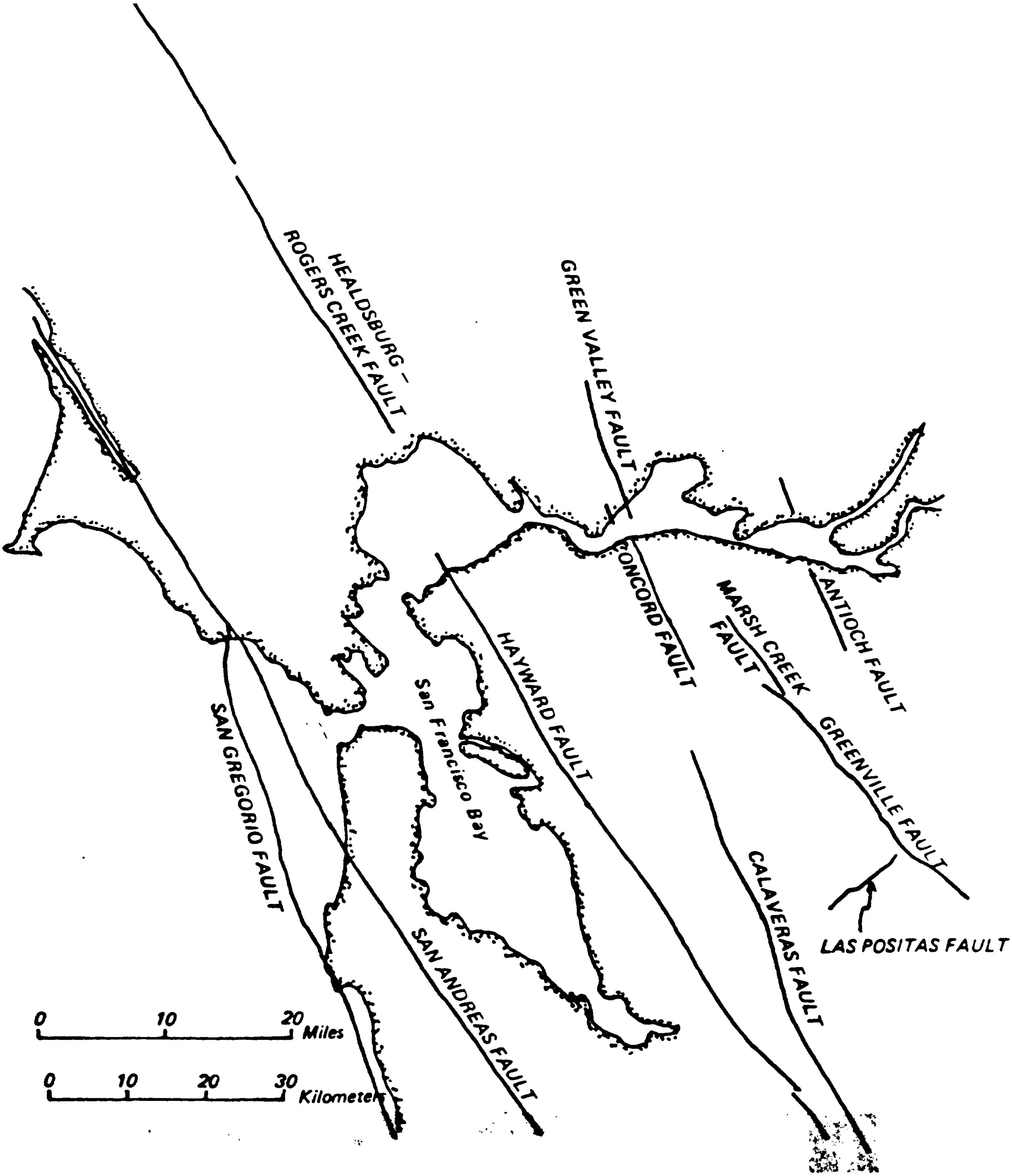

Figure 1.--General map of major faults of the central San Andreas fault system. Prior to the February 22-24, 1980 Greenville earthquake, the obscure Greenville fault had been classified as "inactive" (more than 2 million years old). Detailed field investigations conducted a week after the earthquake revealed Holocene (most recent geologic age spanning the last 10,000 years) activity, which indicated that the Greenville fault should be classified as "active."

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This experience illustrates the tremendous advances that have occurred in recent years in accurately predicting fault movement capability. Many new techniques for evaluating historical fault movement (e.g., paleomagnetics, carbon 14 dating) have recently been proven viable and ground motion prediction is rapidly improving in reliability. With accurate, reliable ground motion predictions, it will be possible to accurately predict structural response and performance.

ON THE PERFORMANCE OF PIPELINES

The failure of buried pipelines has been reported for every major earthquake. But the 1976 Tangshan, China, earthquake exposed a large, urbanized, and geographically diverse area to damaging ground motion, and thus provided a unique opportunity to establish statistical data on pipeline failures.

Table 2 shows pipeline failure rates for various earthquake intensities and ground conditions. Note that pipes in firm ground perform much better than those in soft ground. Table 3 reveals that pipe damage varies inversely with pipe diameter. This indicates that as the pipe's diameter increases, its stiffness also increases and the pipe is increasingly able to resist the surrounding soil deformation, thus resulting in less pipe deformation and a decreased damage rate. The Tangshan earthquake also revealed the large-diameter (500 to 600 mm) prestressed concrete pipes perform better than pipes of either steel or cast iron.

ON MOBILE HOMES

Mobile homes have undoubtedly been damaged in past earthquakes, by they have not been considered a significant problem until recently. Clearly, this is primarily due to the increased use of mobile homes as permanent residences that has occurred over the past 15 years in the United States.

The 1971 San Fernando, California, earthquake produced the first record of significant damage to mobile homes. Subsequently, mobile home damage was reported for the 1978 Santa Barbara, California, earthquake, the 1979 Imperial

Table 2 Relationship of Cast-Iron Pipe Damage to Earthquake Intensity and Ground Condition

Locality	Intensity*	Ground Condition†	Damage Rate (number/km)‡	Remarks
Tianjin	VII-VIII	Class 3	0.18	
Tanggu	VIII	Class 3	4.18	The geological conditions are worse than at Tianjin
Hangu	IX	Class 3	10.00	The geological conditions are worse than at Tanggu
Tangshan	X-XI	Class 2	4.00	

*Chinese intensity, but similar to modified Mercalli intensity.

†Class 1: rock; Class 2: firm, stable; Class 3: soft, miscellaneous.

‡Number of breaks per kilometer.

Table 3 Effect of Pipeline Diameter on Damage

Locality	Pipe Diameter (mm)	Damage Rate (number/km)*
Yinkou	100	1.8
	150	0.88
	300	0.13
Tianjin Urban Area	50	1.13
	75 - 600	0.20
	600	0.04
Tangshan	150	5.23
	300	4.63
	600	1.89

*Number of breaks per kilometer.

County, California, earthquake, and for the January 1980 Greenville, California, earthquake.

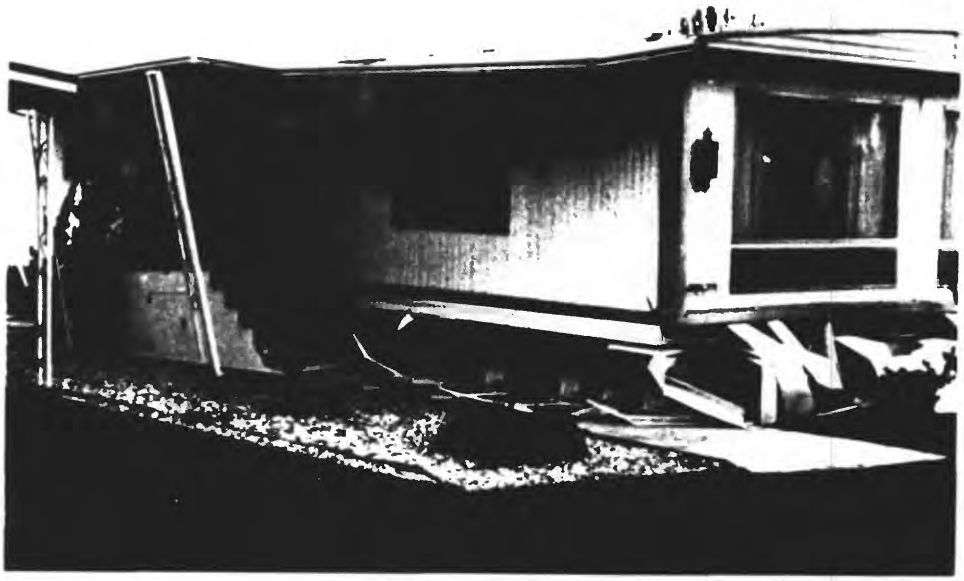
Examples of mobile home damage are shown in Fig. 2. Although relatively minor structural damage is sustained by the mobile homes themselves, in most cases considerable damage to utilities, porches, awnings, stairways, skirts, and other attachments is common. Because gas-line ruptures commonly result from the failures of mobile home foundations during earthquakes, this situation represents a dangerous threat to life and property for a segment of society that is often particularly ill-prepared to cope with such problems, notably senior citizens.

Earthquakes impose high lateral forces on the tops of the small footprint concrete or metal piers commonly used to support mobile homes. Various failure details are shown in Fig. 3 (e.g., bending of the bases of metal piers, buckling of the angle struts of metal piers, and chipping of the bases of concrete piers) but the basic cause of failure is simply overturning of the piers.

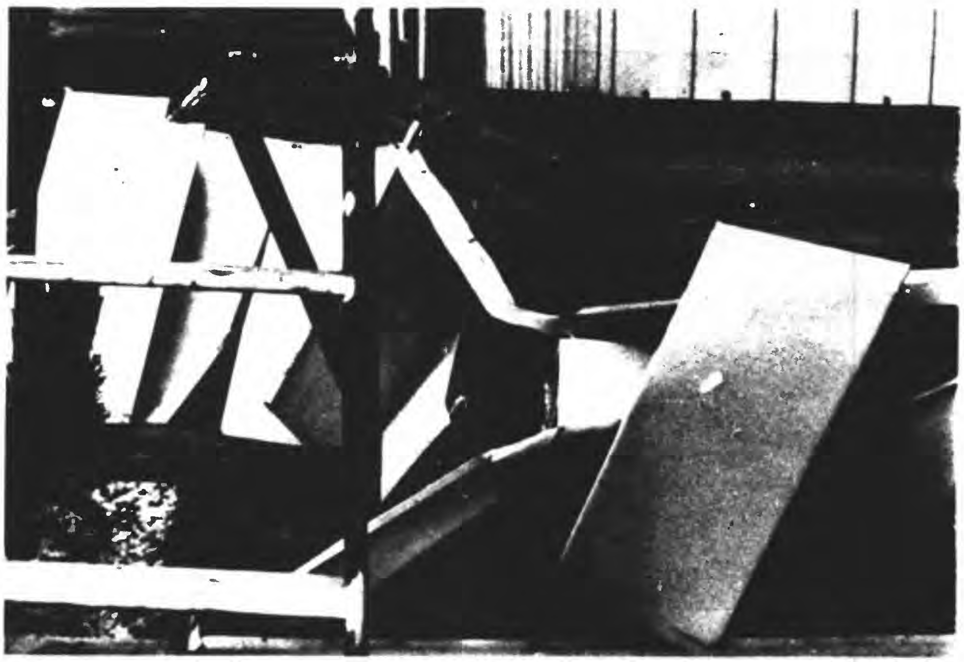
Current state law does not allow mobile homes, which are considered vehicles for tax purposes, to be permanently attached to their foundations. The effect of this requirement on earthquake losses is vividly depicted in Fig. 4--mobile homes suffer substantially greater damage during earthquakes than do permanent residences. Clearly, a change in the law regarding mobile home foundations is in order if we are to seriously pursue earthquake hazards reduction. Fortunately, such changes are currently being considered.

ON THE PERFORMANCE OF TANKS

The seismic vulnerability of surface-mounted tanks has been as well recognized fact since the 1964 Alaska earthquake. Recent observations of tank failures have been made in connection with the 1978 Miyagi-ken-Oki, Japan, earthquake, the 1979 Imperial County, California, earthquake, and the 1980 Greenville, California, earthquake.

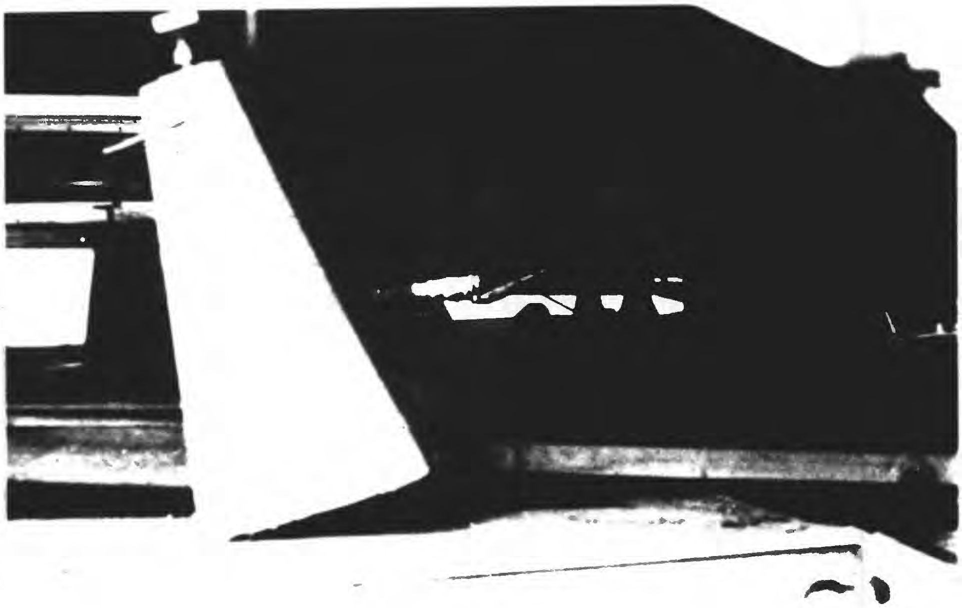


a. Mobile home at Santa Barbara West Mobile Home Park on Winchester Canyon Road was shaken off its foundation.

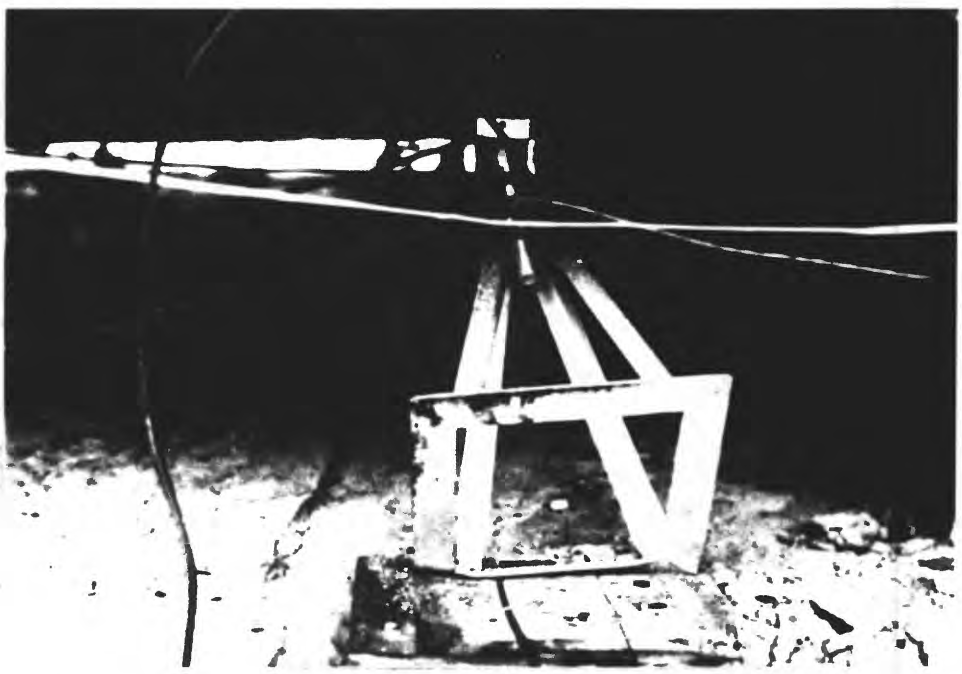


b. Close-up of fallen mobile home showing total horizontal shift of approximately 1 to 2 ft.

Figure 2.--Typical mobile home damage resulting from the August 13, 1978 Santa Barbara earthquake.



a. View of overturned concrete piers supporting a mobile home that fell off its foundation.



b. Typical metal pier upon which other mobile homes are mounted.

Figure 3.--Details of mobile home foundation failures resulting from the August 13, 1978 Santa Barbara earthquake.

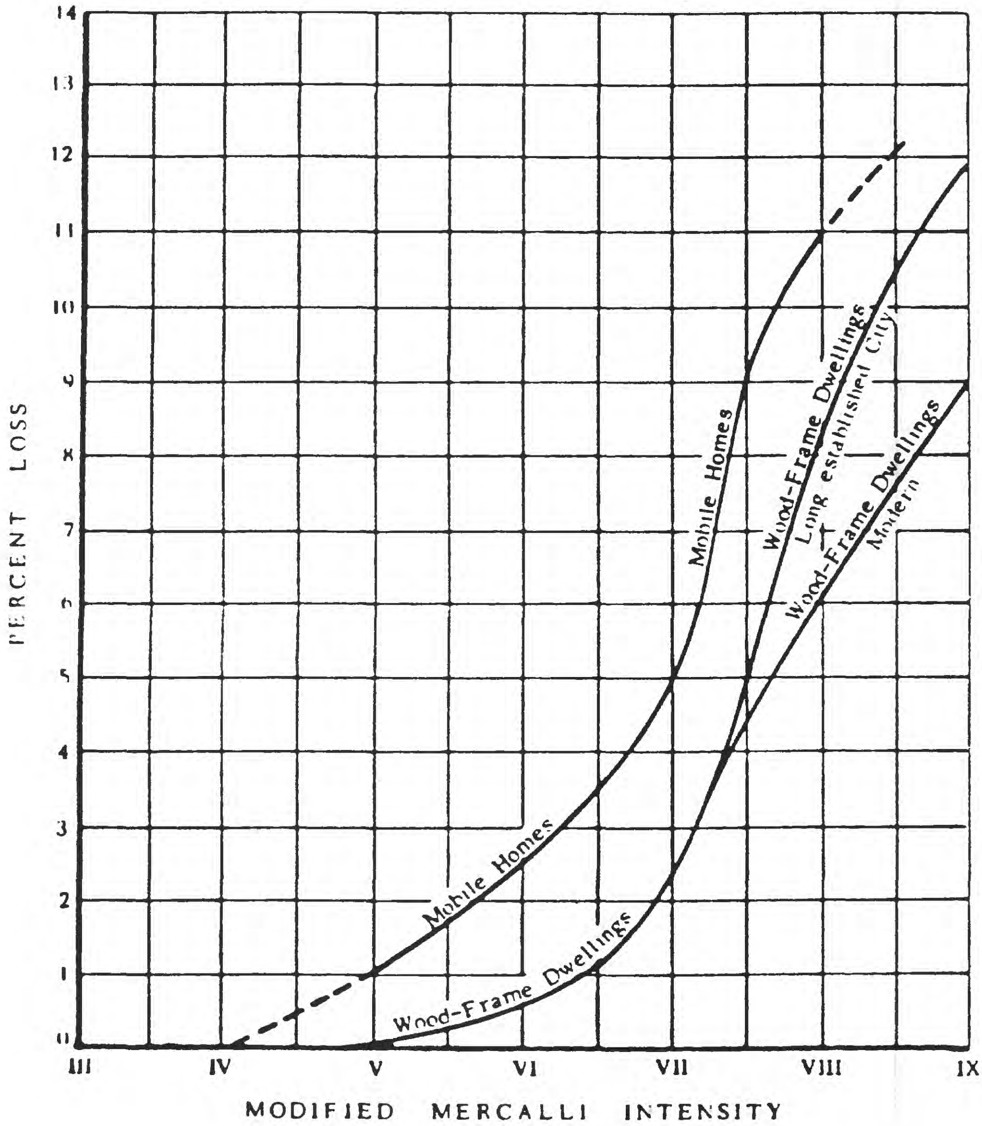


Figure 4.--Monetary loss (in percent of value) as a function of modified Mercalli intensity for mobile homes and wood-frame dwellings (from Reference 24).

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The Greenville earthquake provided a unique opportunity to acquire statistical information on damage to tanks. The Wente Brothers Winery is located in the area that experienced the strongest ground motion during the earthquake. There were other tanks at the winery, but the 177 stainless steel tanks sustained the most extensive damage. A general view of tank damage at the winery is shown in Fig. 5a. These tanks are vertical cylinders made of 12- to 14-gage stainless steel sheets. The diameters vary from 6 to 22 ft. with a height-to-diameter ratio (H/D) ranging from 0.8 to 3.0. The tanks are positioned on elevated concrete pads 2 to 4 ft above the ground. The tops of the pads slope slightly to facilitate drainage. Most tanks are anchored to the concrete pads at two points at the high side of the pads, although a few are anchored at six to eight points. Some of these stainless steel tanks have cooling jackets consisting of a second layer of steel wrapped around the tank to allow space for the coolant to circulate.

During the earthquake, 47 stainless steel tanks were empty or partially full. Forty of these were undamaged or sustained only minor damage. Of the 130 tanks that were completely full at the time of the first shock, 10 suffered no damage and 24 sustained minor damage consisting of minor concrete spalling, anchorage welds failing, or minor local buckling. Seventy tanks suffered a medium level of damage consisting of concrete spalling at the pads, anchorage welds and bolts failing, and some shell buckling with peak-topeak buckle amplitudes of less than 2 in. Twenty-six of the tanks sustained damage considered to be severe. Most of the anchors for these tanks failed, and the shells buckled extensively exhibiting peak-to-peak buckle amplitudes of more than 2 in. Most of the severely damaged tanks sustained permanent overall deformations, such as up to 3 in. of uplift at the base and visible tilting from the vertical. Only one tank was reported to have ruptured at the base.

A cursory study of the damage data indicates that the pattern of damage or failure was a function of the following factors:

- <u>Amount of liquid in the tanks</u>: Empty tanks suffered little or no damage.

a. General view of 177 stainless steel tanks at the Wente Brothers Winery after the earthquake.

b. Typical elephant-foot buckling pattern.

Figure 5.--Damage to tanks caused by January 24, 1980 Greenville, California, earthquake.

c. Typical diamond-shaped buckling pattern.

Figure 5.--Continued

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- Height-to-diameter ratio (H/D): The tanks with low H/D values ($H/D < 1.5$) sustained predominantly large amplitude, elephant-foot buckles all around (Fig. 5b). The tanks with intermediate H/D values ($1.5 < H/D < 2.0$) exhibited varying patterns and combinations of diamond-shaped buckles (Fig. 5c) and elephant-foot buckles. Tanks with high H/D values ($H/D > 2.0$) suffered minor or no damage to the shell, but exhibited some anchorage weld or bolt failures.

- Location of the cooling jackets: Where the extra sheet of steel was close to the bottom of the tank, there was no damage to the shell. Where the jacket was located 3 to 4 ft above the base, the major buckling occurred between the cooling jacket and the base.

This statistical data on the performance of surface-mounted tanks offers needed additional insight into the factors to be considered in their seismic design.

ON CHARACTERIZATIONS OF EARTHQUAKE SHAKING

For decades, earthquake engineering professionals have used earthquake magnitude as a measure of the expected performance of structures during earthquakes. This approach is exemplified by the seismic design philosophy presented in the Recommended Lateral Force Requirements and Commentary of the Structural Engineers Association of California (SEAOC). "Structures designed in conformance with the provisions and principles set forth herein should, in general, be able to:

1. Resist minor earthquakes without damage;

2. Resist moderate earthquakes without structural damage, but with some nonstructural damage;

3. Resist major earthquakes, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as non-structural damage."

In addition, for an even longer period of time, at least since the 1906 San Francisco earthquake, earthquake engineers have used peak ground acceleration as an indicator of an earthquake's destructiveness. Instrumental data, including both ground motion and structural response, taken from several recent earthquakes indicate that both earthquake magnitude and peak ground acceleration may be inadequate as measures of the destructiveness of earthquakes. Specifically, damage observations from several recent earthquakes have led investigators to conclude that neither of these measures correlate consistently with damage. A good case in point is the 1979 Imperial County, California earthquake. This moderate earthquake was marked by a broad spectrum of peak ground accelerations and caused the near collapse of a modern high-rise building, although it left many older unreinforced masonry buildings virtually unscathed.

SEAOC is currently revising its stated philosophy, and the phrases "minor earthquake, moderate earthquake, and major earthquake," are being replaced with "minor ground motion, moderate ground motion, and major ground motion." For the inexperienced engineer who is producing a seismic design, this represents an important step forward. If he reads the SEAOC Recommended Lateral Force Requirements and Commentary (Ref. 25), he will immdiately recognize that it is the strength of shaking he is designing his structure to resist--not the earthquake's size.

What then are measures of minor, moderate, and major ground motion? Peak ground acceleration, as stated above, has long been used as an indicator of shaking strength, but in recent earthquakes, instruments have recorded high levels of acceleration without correspondingly significant damage, leading investigators to question the reliability of peak ground acceleration as a damage indicator.

The earthquake ground motion characteristics that are important for structural design are:

- o Amplitude
- o Frequency content
- o Periodicity
- o Duration

The importance of amplitude is obvious. Both frequency content and periodicity affect dynamic amplification. Duration is important only for nonlinear (ductile) response when low-cycle fatigue is a factor.

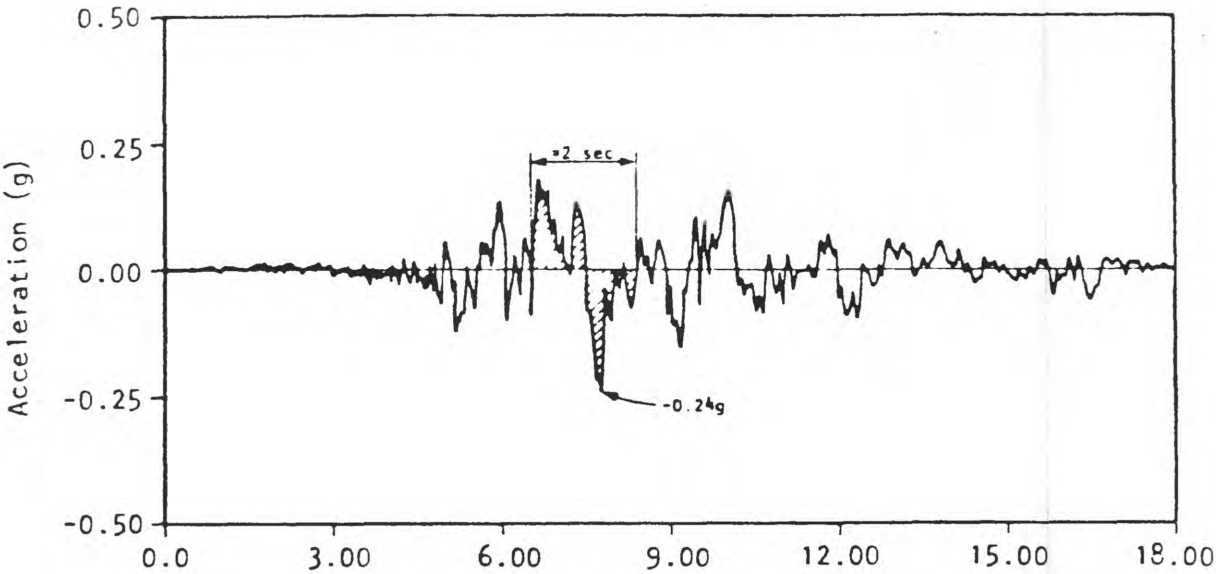
Earthquake ground motion can also be appropriately identified in terms of:

- o Acceleration
- o Velocity
- o Displacement

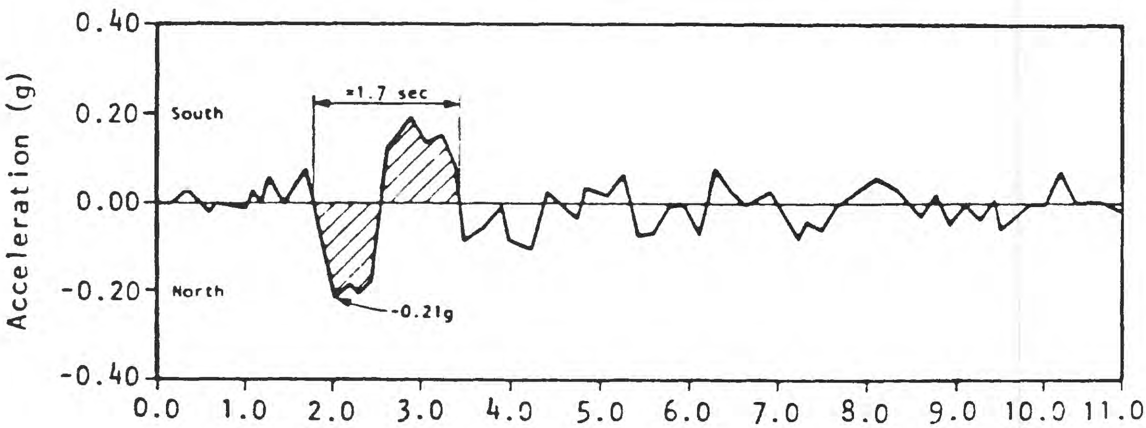
The importance of these latter three characteristics to the field of structural design has not been rigorously established to date. But several recent post-earthquake damage observations, and the accompanying instrumental data, have shed some important light on this matter.

Detailed analysis of the response behavior of Olive View Hospital led investigators to conclude that high-amplitude, long-duration acceleration pulses (individual acceleration pulses of long duration) were probably an important factor in the building's failure. No ground motion was recorded at the hospital site, however, and this conclusion was based largely on observations of the characteristics of the Pacoima Dam record. Similar observations have been made for both the 1977 Romanian earthquake, and the 1979 Imperial County, California, earthquake. The ground motion acceleration records from these two earthquakes reveal relatively high-amplitude, long-duration acceleration pulses (Fig. 6).

For the analysis of structural response and for design purposes, it is important to realize that these high-amplitude, long-duration acceleration



a. Imperial County, California, earthquake of October 15, 1979: Ground motion record from Imperial County Services Building free-field site (Trace 1, 92°).



b. Romanian earthquake of March 4, 1977: Ground motion record at Romanian Building Research Institute.

Figure 6.--Examples of strong ground motion records from recent earthquakes revealing long-period, high-amplitude acceleration.

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pulses result in a large-amplitude ground motion displacement demand being placed on structures. A convenient way to view the effect of these pulses on structures is to calculate a response spectrum and plot the curve on 4-way log paper. An important feature of a response spectrum is that the values are asymptotic to peak ground acceleration at short periods and asymptotic to peak ground displacement at long periods.

The 5%-damped response spectra for three different ground motion events are shown in Fig. 7. The two El Centro curves (Curves 2 and 3) reveal that the maximum displacement demand of the 1979 event was greater than that of the 1940 event. Comparing the underground nuclear explosion spectrum with the earthquake curves reveals that the spectral acceleration demand for the explosion was greater than that for the earthquakes, but that the spectral displacement demand of the explosion was two orders of magnitude less than that of the 1949 El Centro, California, earthquake. Damage from the underground nuclear explosions was insignificant when compared with that resulting from these two earthquakes (Ref. 26).

Many observations can be made with implications for design and for estimating the actual performance of structures subjected to these various spectra. Consider, for example, the case of a one-story, reinforced-concrete-frame structure with an initial elastic fundamental period of 0.1 sec. Obviously, the spectral acceleration demand of the explosion is greater and would govern for a force design. Assume, however, that the structure is designed according to current philosophy for an elastic force less than the actual demand spectral acceleration, and further assume that the structure is exposed to the actual demand spectral acceleration and a mechanism failure occurs. If the mechanism failure were to cause the fundamental period to shift substantially, say to 1.0 sec., the following scenario would apply. For the underground explosion, the maximum displacement demand would be 0.8 cm and a 0.8-cm interstory drift would most likely result in unnoticeable damage. For the 1979 Imperial Valley record, the displacement demand is about 10 cm and significant damage would most likely be revealed. The entire process is not quite as simple as this illustration implies, but these trends prevail. In the extreme case, if the structure were to lose all its stiffness ($K \simeq 0$), the mass would remain stationary while the ground below moved equal to the peak

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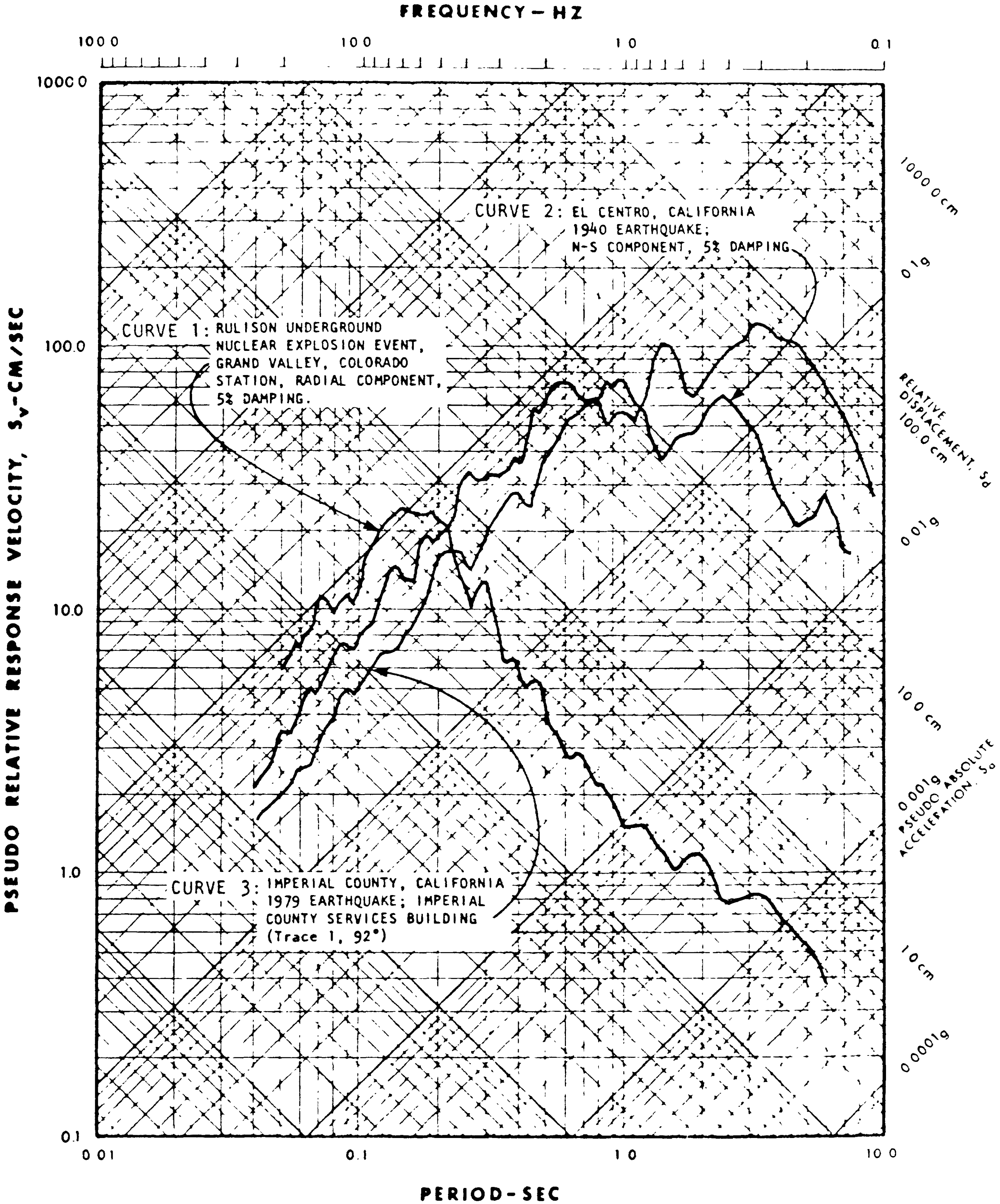

Figure 7.--Example response spectra: A comparison of various spectra revealing vast differences in the accelerations and displacements of the ground motion records.

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ground displacement. It follows that a highly appealing candidate as a general measure or characterization of ground motion is the response spectrum plotted on a 4-way log graph. All important ground motion parameters, except ground motion duration as it affects low-cycle fatigue, are included in this characterization.

ON THE PERFORMANCE OF HIGHWAY STRUCTURES

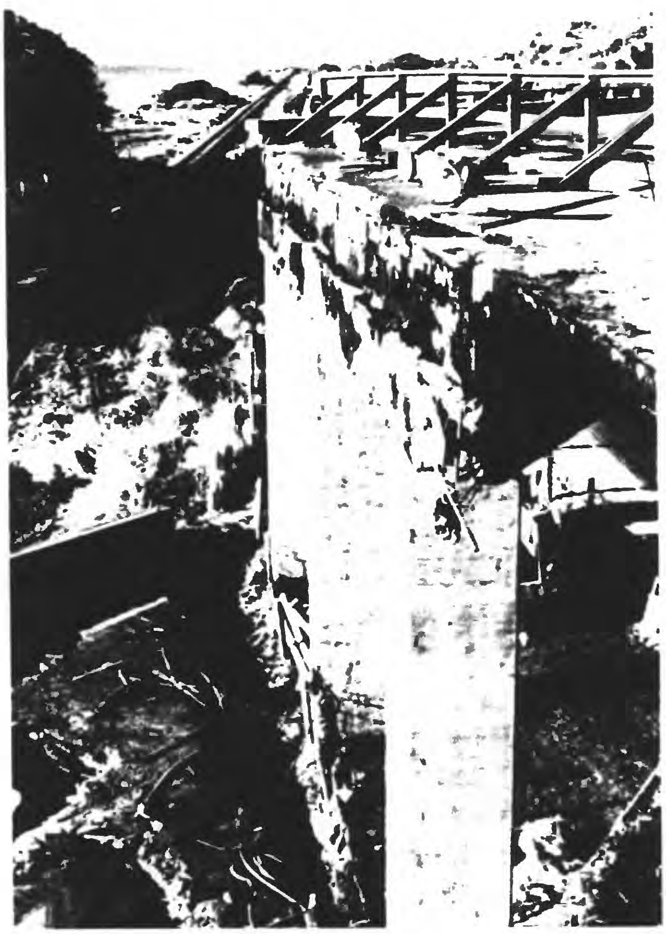
The seismic vulnerability of simple span bridges has long been acknowledged. Since the 1971 San Fernando, California, earthquake, the vulnerability of these structures has been recognized as a serious earthquake problem here in the United States. The 1979 Imperial County, California, and the 1980 Trinidad Offshore, California, earthquakes provide a clear illustration of the importance of tying bridge components together.

The collapse of two spans of one of the two Fields Landing bridges located near Fortuna, California is clearly illustrated in Fig. 8a. The collapse resulted from inadequate beam-bearing support and a failure to tie the decks of simply supported spans at both a pier and an abutment (Fig. 8b). Both conditions were aggravated by the significant bridge skew of 56°. This collapse is consistent with previous earthquake-induced bridge failures, which further underscores the need to strengthen older bridges in seismically active regions. There were no ground motion recording instruments at the bridge site, but based on the records available in the general area, the peak ground acceleration at the site was estimated to range from 0.1g to 0.15g.

Several highway bridges were subjected to strong ground motion during the 1979 Imperial County, California, earthquake. There are a large number of bridges in the earthquake-affected area, but only three will be discussed here: the New River Bridges at Brawley and the Meloland Road overpass.

The New River Bridges (Fig. 9a) were constructed in 1953. They are 197 ft-8in. in length and have eight spans made of continuous reinforced-concrete slabs. During the 25-year life of the bridges, the embankments have been moving toward the channel (both bridges, both sides) and had covered about one-half the distance recorded after the Imperial County earthquake of October

a. Aerial view showing two collapsed spans.



b. Close-up of the third bent revealing the almost complete absence of superstructure component interconnection.

Figure 8.--The collapse of the Field Landing Bridge caused by the November 8, 1980 Offshore Trinidad, California, earthqauke.

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a. The initial shock (M = 6.7) caused only slight damage, but the bridge was closed to traffic following the third aftershock (M = 5 to 6).

b. Ground failure (precipitated by the movement of the embankments toward the river) was the primary cause of damage.

Figure 9.--Damage to New River Bridge caused by the October 15, 1979 Imperial County, California, earthquake.

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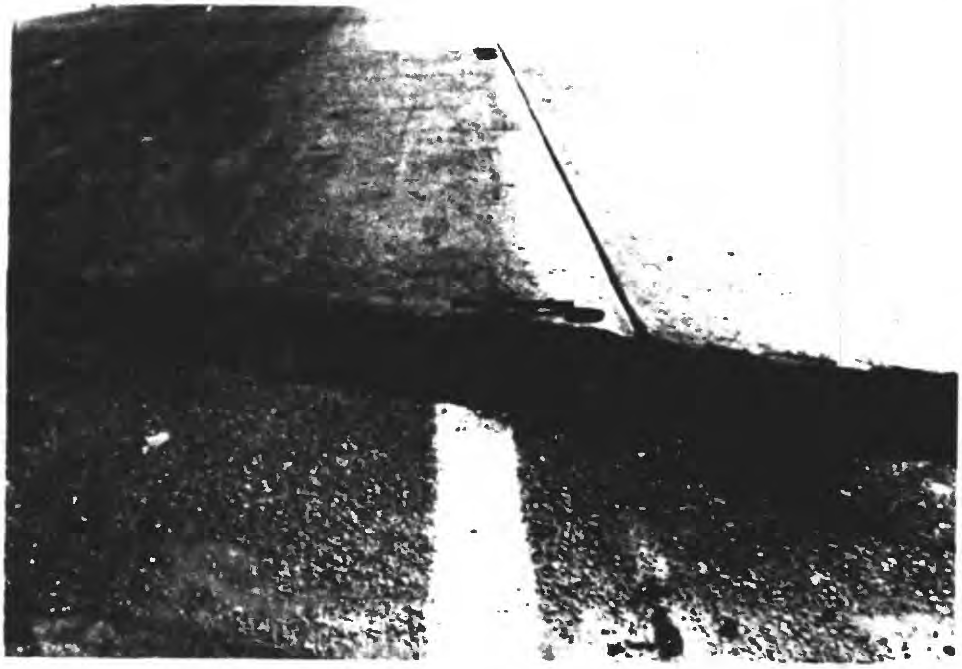
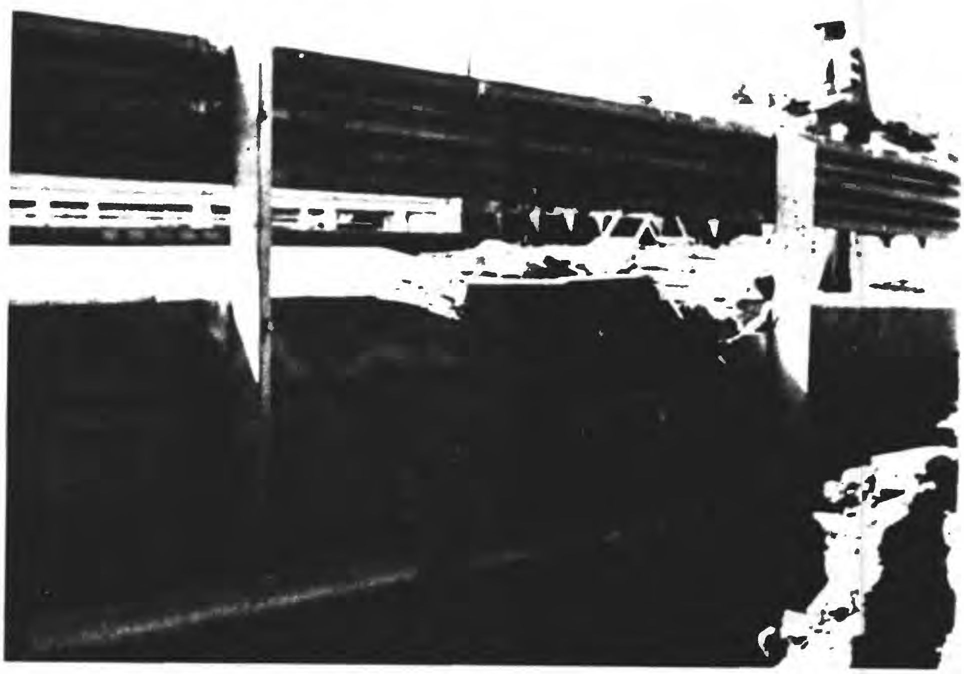


c. Ground failure caused the relative vertical and horizontal movement between the approach and the bridge. The horizontal offset was probably caused by the 16^{o} skew of the bridge.



d. Heavy reinforcement between the abutment and the deck kept the bridge together despite the strong motion (0.2g) and the significant ground failure.

Figure 9.--Continued

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1979. Some of this movement was attributed to small earthquakes during that period. Immediately after the initial M 6.6 shock, local Caltrans personnel surveyed the damage and embankment movements at New River and judged them to be slight enough to permit traffic to continue to use the bridges. But the bridges and the embankment continued to move with each aftershock, and the three aftershocks ranging from M 5 to M 6 that occurred between 6-1/2 and 7-1/2 hours after the initial shock caused structural damage and embankment settlement severe enough to require closing both bridges to traffic. Due to the skew of approximately 20°, the bridge superstructure had rotated clockwise relative to the abutments by as much as 4 in. at one abutment. Another abutment had moved about 6 in. toward the river (Fig. 9d). In spite of the significant ground failure at the bridge site, and the estimated 0.2g acceleration at the site, the bridges did not collapse. They were subsequently repaired and are now in service.

The Meloland Road bridge (Fig. 10) is constructed with continuous reinforced-concrete 3-cell box girders on open-end diaphragm abutments and a reinforced concrete column bent, all on reinforced concrete piles. The two spans are each about 104 ft in length and there is no skew. The bridge was built in 1971 as a one-piece structure without joints or sliding details. Because of its structural characteristics, size, and location in a highly active seismic area, the bridge was selected for instrumentation under the California Strong-Motion Instrumentation Program. In November 1978, it was fitted with two 13-channel, kinemetric CRA-1, remote-accelerometer central-recording accelerograph systems. The bridge was not damaged in spite of the fact that the maximum horizontal acceleration recorded on the ground adjacent to the bridge was 0.33g. Several other bridges with continuous tied decks in the area of strong ground motion also sustained no significant damage.

These observations from the 1979 Imperial County, California, earthquake clearly indicate the superior performance of bridges that are tied together.

ON STRENGTHENING

Based on the recognition that old, seismically weak structures will be in use for many decades to come, several communities in the United States are

a. Overall view of grade-separated, continuous-deck bridge located approximately 0.5 km from the Imperial fault.

b. View of the underside showing complete absence of damage and one of the 26 seismometers installed at the site. The peak horizontal ground motion component recorded at the free-field site was 0.33g.

Figure 10.--Meloland Bridge after the October 5, 1979 Imperial County, California, earthquake.

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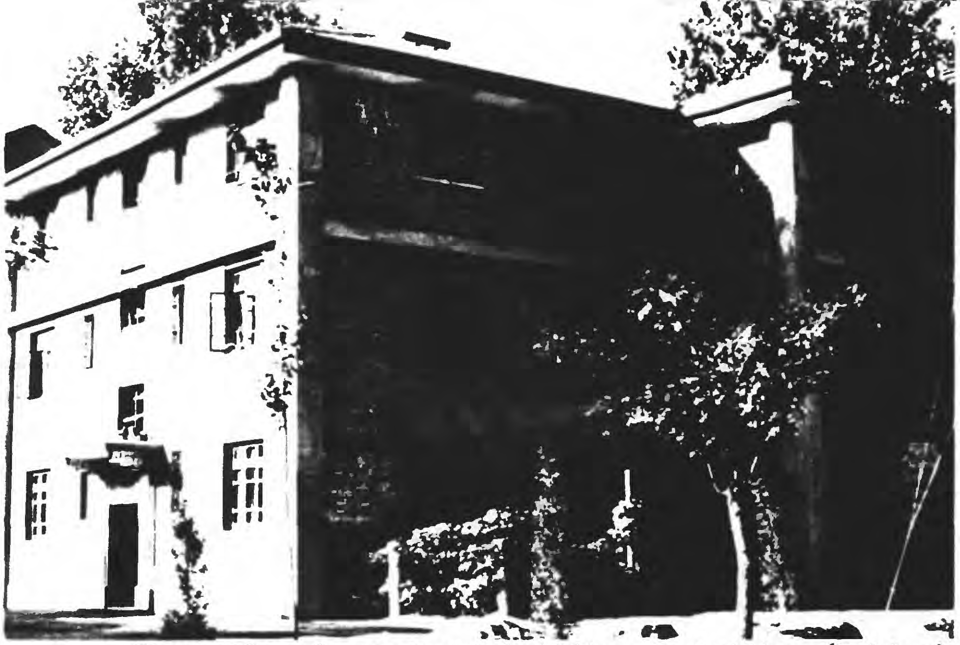
actively implementing seismic strengthening recommendations. Buildings repaired and strengthened after previous earthquakes, or merely strengthened prior to an earthquake, have performed well in three recent earthquakes: 1974 Lima, Peru, 1976 Tangshan, China, and 1980 El Asnam, Algeria.

Figure 11 illustrates techniques used in China to strengthen unreinforced masonry buildings. This procedures reportedly reduced damage in Tianjin during the 1976 Tangshan earthquake. Figure 12 shows a building in El Asnam, Algeria, that was repaired and strengthened after the 1954 earthquake. The building performed well during the 1980 earthquake when many other buildings sustained substantial damage or even collapsed. Both of these buildings were simply tied together, and the principal goal was to keep the building standing as a unit during future earthquakes. Generally, this strengthening procedure involves first constructing pilasters and bond beams on the exterior of the building, and then running tie rods through the building at floor and wall lines to tie the existing walls of the structure and the new concrete frames together. On the basis of the reports cited, this strengthening procedure--caging the buildings--appears to be effective.

Figure 13 illustrates another type of strengthening used in Managua. In the Teresiano School, a three-story classroom building, the ground-story columns were damaged in the 1968 Managua earthquake. Reinforced-concrete piers were added along the columns and extended up to the level of the second-story windowsills. Above that level, the columns were unchanged. In the 1972 earthquake, the second and third stories were damaged above the level where the piers ended (Fig. 13). The alterations that strengthened the lower part of the building virtually guaranteed that the unaltered part of the building would suffer greater distress during the next earthquake. Repairing or strengthening a part of a building will alter the dynamic behavior of the entire structure. This must be borne in mind by those involved in earthquake repair of strengthening.

ON NONSTRUCTURAL ITEMS

Nonstructural items have only been seriously included in the scope of post-earthquake investigations since about the time of the 1964 Alaska



a. Strengthened apartment building in Beijing (Peking).

b. Details of strengthening method used in China.

Figure 11.--Building strengthening procedure used extensively in China and consisting of a cage constructed around the building to tie it together. This procedure reportedly reduced damage in Tianjin during the 1976 Tangshan earthquake.

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Figure 12.--Building strengthened after the 1954 El Asnam, Algeria, earthquake using the tied-cage procedure. This structure experienced only minor damage from the October 10, 1980 El Asnam earthquake.

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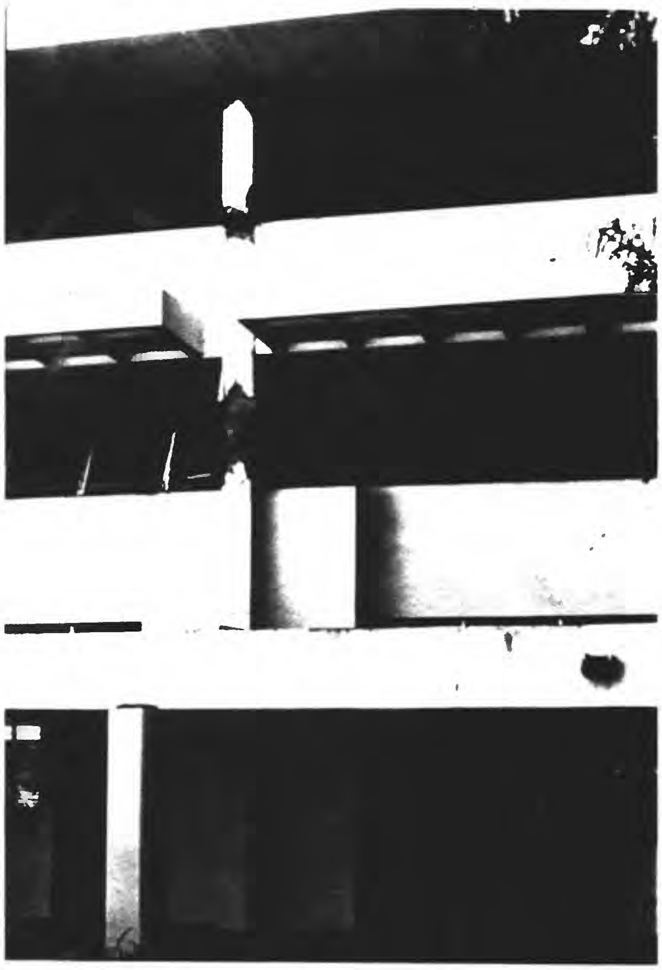

Figure 13.--Building strengthened after the 1968 Managua, Nicaragua, earthquake by adding reinforced-concrete piers along the columns up to the level of the second-story windowsills. Note the damage above that level caused by the 1972 Managua earthquake.

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earthquake. Prior to that time structural performance occupied the concern of the available earthquake specialists and, in addition, the importance of nonstructural items was only vaguely understood and appreciated.

Detailed investigation of the 1971 San Fernando, California, earthquake, has made it clear that modern buildings contain many nonstructural elements and assemblies that often suffer severe damage in earthquakes and may, in turn, damage the main structure and present a real hazard to life (e.g., stairs and brittle nonstructural walls).

Nonstructural items can be conveniently divided into three categories:

- Architectural Components: glass, partitions, cladding, ceilings, stairs

- Mechanical/Electrical Components: heating, ventilation, water, gas, lighting, fire prevention systems, elevators

- Contents: furniture, shelving, filing cabinets, other goods

The economic importance of nonstructural items is revealed by their initial cost. In a typical commercial or institutional building, the value of nonstructural items will average somewhere between 3 and 10 times that of the structure, so that economic loss from structures subjected to minor and moderate ground motion can be significant. In addition, business interruption caused by nonstructural damage can result in significant economic loss. Recent earthquake damage investigation reports reveal frequent nonstructural damage: elevators that jammed, bookshelves that overturned, suspended ceilings and light fixtures that have fallen, and supports for spring-mounted HVAC units that have failed.

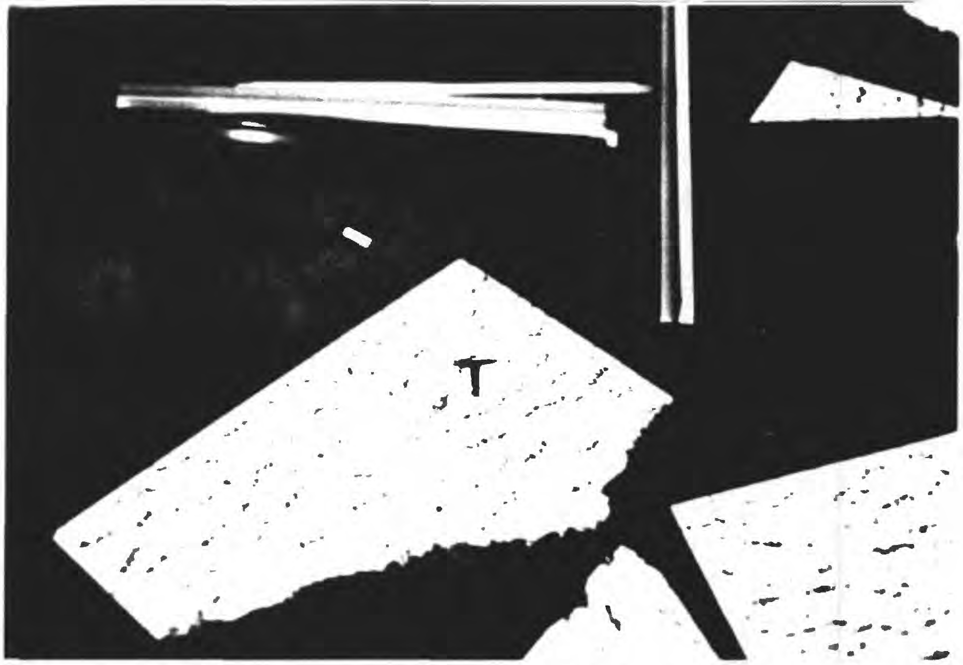
The life-endangering hazard that nonstructural items pose has not been as widely acknowledged as the danger of a structural collapse has been. Accordingly, we see failures of suspended ceilings in even moderate earthquakes such as the 1980 Offshore Trinidad, California, the 1980 Greenville, California, and the 1980 Northern Kentucky earthquakes

(Fig. 14). The failure of a suspended ceiling always poses a threat to the life of the building's occupants.

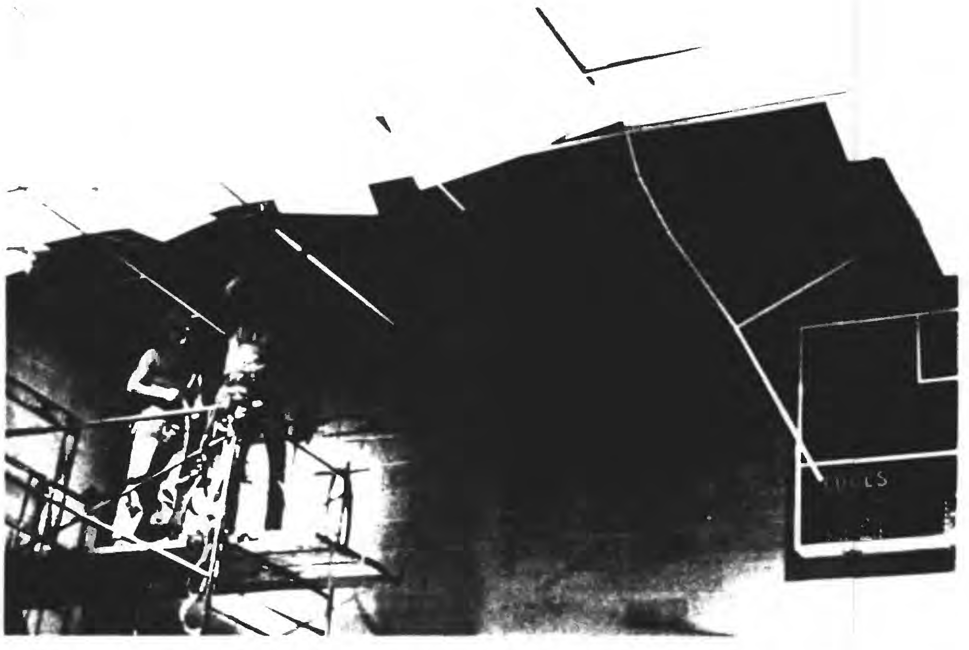
Based on observations of suspended ceiling failures in the 1964 Alaska earthquake and in the 1971 San Fernando, California, earthquake in particular, the California Office of the State Architect (OSA) changed the design requirements for suspended ceilings. In the 1978 Santa Barbara, California, earhtquake, no failures of suspended ceilings in newly constructed schools were reported, demonstrating that the improved design requirements were instrumental in reducing this hazard.

Another example of a serious, life-threatening hazard was revealed in the observation of the performance of the Banco Central building during the 1972 Managua, Nicaragua, earthquake. The Banco Central, a 15-story, reinforced-concrete-frame building was a relatively flexible structure, and as a result, substantial partition damage occurred, causing significant debris to be deposited in the stairwell, as shown in Fig. 15c. Egress is seriously impeded by such debris, and the damaged handrail shown in this figure indicates that the stairwell was not a safe place to be during the earthquake. Examples of other common types nonstructural damage to architectural components, mechanical/electrical components, and building contents are shown in Figs. 15, 16, and 17, respectively. Nonstructural damage continues to occur, although the earthquake specialists who conduct post-earthquake damage investigations feel that such damage can and should be reduced.

Awareness and action are the lessons to be learned from nonstructural items. In general, it is neither difficult not costly to design nonstructural items to be earthquake resistant. Accordingly, if the appropriate design professionals are aware of the problems with seismic performance, remedial action to reduce earthquake hazards from nonstructural items can be readily undertaken. Where the awareness has been followed by appropriate action (e.g., improved design requirements for suspended ceilings issued by California OSA), nonstructural component hazards have been reduced.



a. Fortuna High School after the November 8, 1980 Offshore Trinidad, California, earthquake. Note that a T-bar penetrated the tile.



b. Northern Kentucky earthquake of July 27, 1980: Workmen replacing the T-bars to repair the suspended ceiling at the gymnasium of St. Patrick's Scholl in Maysville, Kentucky.

Figure 14.--Nonstructural damage to architectural components: Suspended ceiling failures caused by very moderate shaking.

a. Window at the Imperial County Services Building damaged by the October 15, 1979 Imperial County earthquake.

b. Damage to stiff and brittle nonstructural exterior walls of moment-resisting-frame building (La Protectora Insurance Building) caused by the 1972 Managua, Nicaragua, earthquake.

Figure 15.--Nonstructural damage to architectural components: Windows and walls damaged by drift greater than these brittle elements could withstand.

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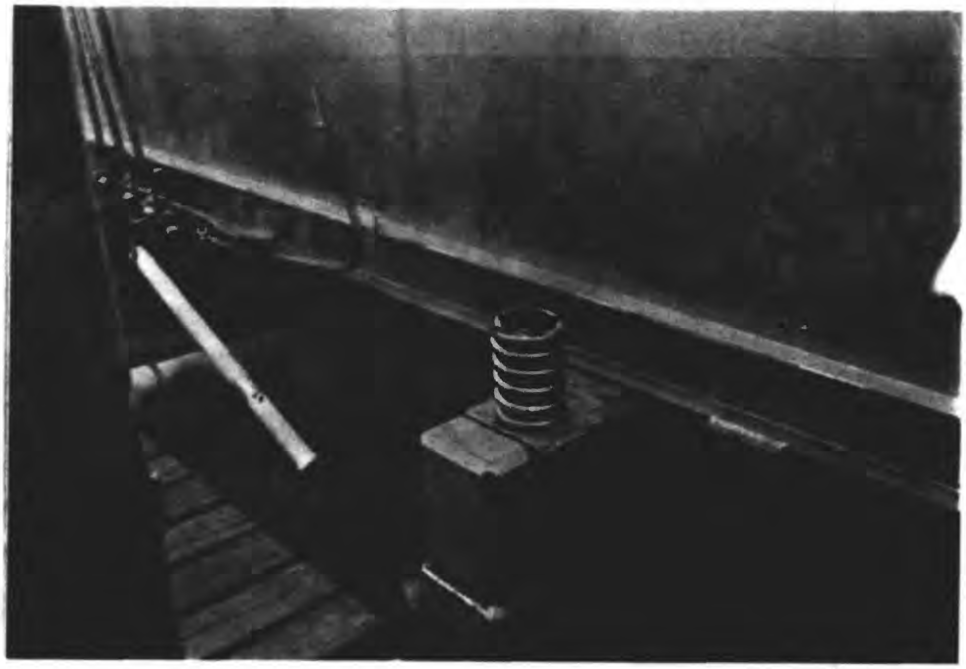
c. Stairs at the Banco Central building littered with debris after the 1972 Managua, Nicaragua, earthquake.

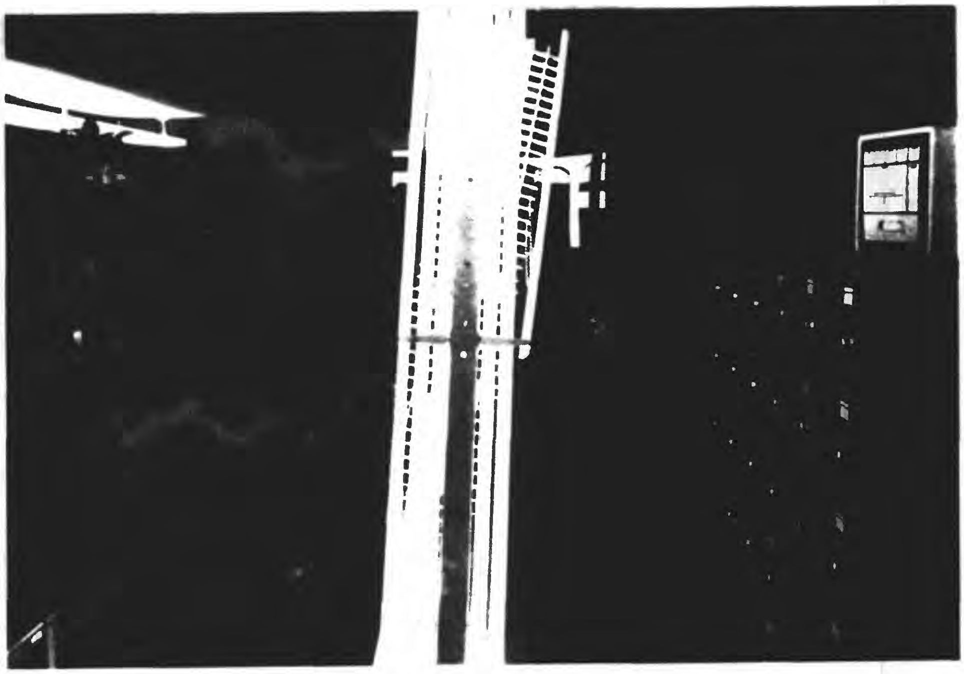
Figure 15.--Continued



a. Base support failure of air-handling unit on the roof of the Imperial County Services Building caused by the October 15, 1979 Imperial County, California, earthquake.

b. Broken mounting base of hot-water pump on top of Library III Building at the University of California at Santa Barbara caused by the August 13, 1978 Santa Barbara, California, earthqauke.

Figure 16.--Nonstructural damage to mechancical/electrical components.



c. Fluorescent light fixture in a Main Street office building in El Centro fell yet remained lit during the October 15, 1979 Imperial County, California, earthquake.

Figure 16.--Continued

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a. Filing cabinet at the Imperial County Services Building (ICSB) opened and tipped over.

b. Failed light-gage shelving rack at the ICSB.

Figure 17.--Damage to building contents caused by the October 15, 1979 Imperial County, California, earthquake.

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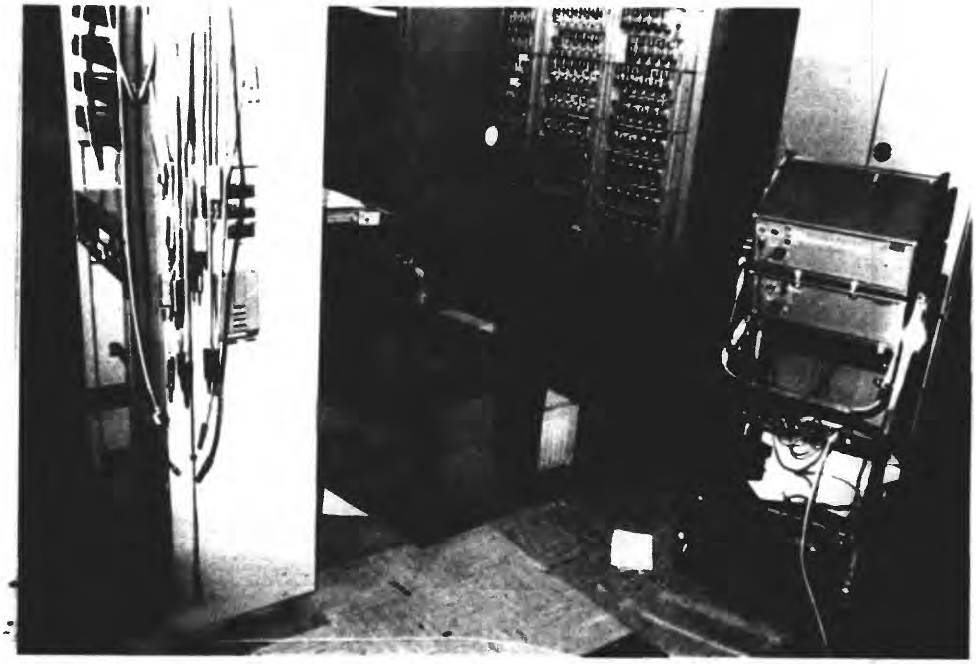


c. Toppeled storage cabinet at the office of the telephone company in El Centro.

Figure 17.--Continued

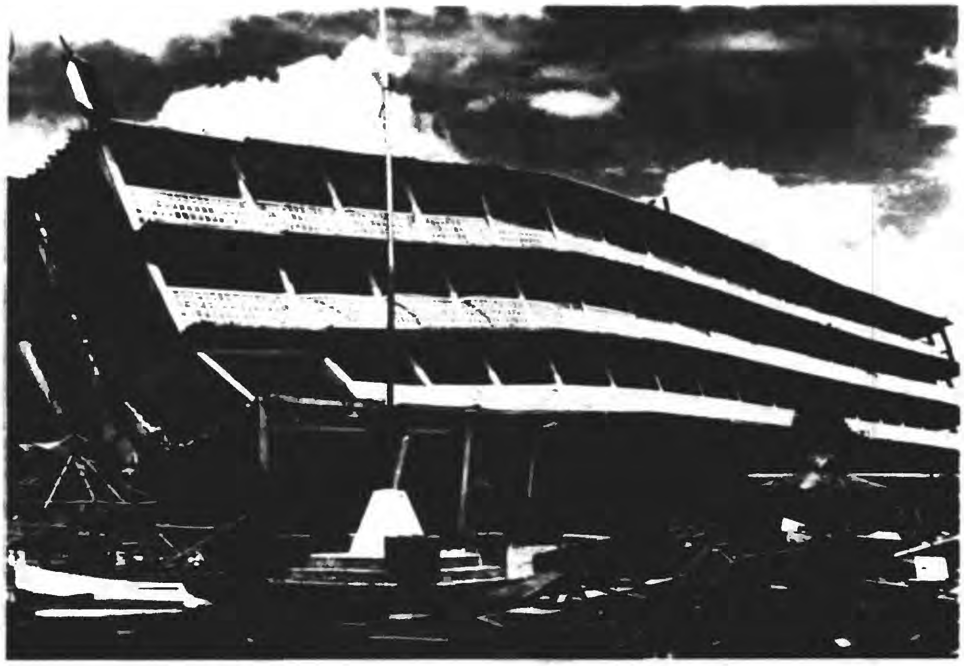
ON THE PERFORMANCE OF ENGINEERED BUILDINGS

The poor performance of engineered buildings, mostly reinforced-concrete-frame buildings, has long been a popular subject of post-earthquake damage investigation reports. A recent example of a reinforced-concrete structure that collapsed in the 1976 Mindanao, Philippines, earthquake, is shown in Fig. 18a. By contrast, a reinforced-concrete structure that survived the Mindanao earthquake essentially unscathed is the 4-story Tison building shown in Fig. 18b. Assuming that the two buildings experienced similar ground motion (ground motion was not recorded), then it is clear that reinforced-concrete structures can be designed to withstand earthquake forces. Reportedly, the Tison building was the only structure in the earthquake affected area whose design included seismic considerations.

Another recent failures of a reinforced-concrete, moment-resisting frame structure is the Ain Nasser Market (Fig. 19), which collapsed during the October 19, 1980 El Asnam, Algeria, earthquake. The collapse of this huge shopping mall and apartment complex initially trapped some 3,000 persons, and the final tally included several hundred deaths. The structural design included heavy waffle floor slabs supported on slender columns that were not designed to have sufficient moment capacity to resist the ground motion, as the photos clearly show.

The 1972 Managua, Nicaragua, earthquake provided many opportunities for learning, especially in connection with the response of engineered buildings. In particular, two high-rise buildings strongly affected by the earthquake afforded the opportunity of comparing the performance of stiff and flexible construction. Both are relatively modern buildings and were built in the mid-1960s (see Fig. 20).

The Banco de America was a 17-story shear-wall building and was relatively stiff. Reinforced-concrete shear walls formed the four angle-shaped enclosures for the stairwells and elevator shafts. These enclosures were connected with pairs of girders to form a large central shear core. The girders had central openings to allow the passage of air conditioning ducts. After the earthquake, the girders in the east-west direction were badly



a. Harvardian College collapsed as did most other engineered buildings in Mindanao that were not designed with seismic considerations in mind.

b. The Tison Building, reportedly the only building in Mindanao designed in accordace with seismic considerations, was essentially undamaged.

Figure 18.--Performance of engineered buildings during the August 17, 1976 Mindanao, Philippines, earthquake.

a. View showing near collapse of a portion of Ain Nasser Market.

b. View showing complete collapse of the upper two floors of a portion of Ain Nasser Market.

Figure 19.--Damage to Ain Nasser Market (shopping mall and apartment complex covering one entire block) caused by the October 1980 El Asnam, Algeria, earthquake.

c. Tie-beam damage at Banco de America. Note exposed reinforcing steel below

Figure 20.--Continued

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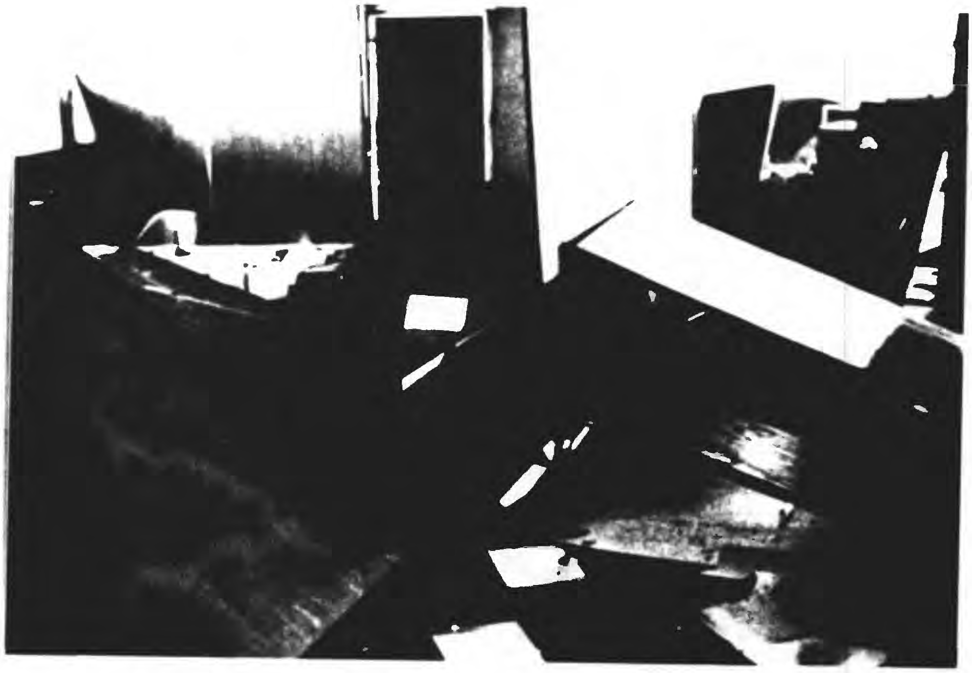

d. The President's office of the Banco Central showing significant nonstructural damage.

e. An office at the Banco de America showing negligible nonstructural damage and dislocation of office equipment.

Figure 20.--Continued

sheared at the central air duct opening, all the way up the building (Fig. 20c). Although the connecting girder failed, the shear walls performed well and the building and its contents were little affected by the shaking. Floor distress appeared in some locations over the sheared girders. There was minor cracking in the shear walls. The elevators were not working because the counterweights had been displaced, but the stairs remained clear and could have been used to evacuate the building without difficulty if it had been occupied at that hour. The offices remained in good condition (Fig. 20e), and although the building required structural repair, it could have been reoccupied in short order and remained in use while structural repairs were being made.

The Banco Central was a relatively flexible, 15-story, moment-resisting reinforced-concrete-frame building with a few shear walls around the elevator enclosure at the west end. Apart from the collapse of the auditorium wing roof, the structural performance of the building was good. The frame members were not seriously damaged, although beam and column cracks were evinced and a few columns were spalled. The floor diaphragm cracks at the stair landings, were the worst structural distress in the main building. In addition, however, moderate structural distress was evinced in the form of elevator core spalling, spalled bridge beams between elevator cores, and a permanent offset of 2 to 3 in. to the east of the upper floors of the building with respect to the lower floors. This displacement occurred gradually between the fifth and ninth floors. The building sustained serious nonstructural damage: brittle clay tile infill walls shattered, lay-in ceilings fell, poorly anchored interior partitions toppled, and fixtures were strewn about (see Figs. 15c and 20d). Although the building remained structurally sound, the nonstructural components were a shambles and rehabilitation would have been very slow and expensive. The combination of a flexible structure and brittle nonstructural components was disastrous.

Repairs on the Banco de America building were made using the epoxy injection technique. Engineers proposed repairing the Banco Central building, but the appearance of extensive damage caused the owners to decide to remove the upper 12 stories of the building. The two-level basement containing the national vault was not damaged by the earthquake, so it was deemed desirable to preserve the vault and an aboveground structure sufficient to service it.

In recent United States earthquakes, two modern high-rise buildings in California have failed. Figure 21 shows the first-story column failure at the Olive View Hospital, caused by the 1971 San Fernando earthquake. Figure 22 shows the failed first-story columns of the Imperial County Services Building (ICSB) caused by tyhe 1979 Imperial County earthquake. In both cases, the failures are primarily attributable to the substantial difference in stiffness above and below the tops of the failed columns. At the Olive View Hospital, a classic example of a flexible first-story building, a significant change in stiffness occurred in the longitudinal direction with some interior shear walls at the first story in the transverse direction. However, post-earthquake evaluations indicated that a significant change in the configuration of structural components above and below the seond floor at least contributed to the failure. Subsequent detailed analyses of both these buildings have revealed that large deformations (large interstory drifts) occurred in the lower, flexible portions of these buildings imposing ductility demands in excess of the ductility capacities of these components.

The question of whether or not to demolish the Olive View hospital was never posed. Structural damage was so severe that demolition was the only viable option. By contrast, serious consideration was given to repairing the ICSB. It was estimated that the damaged building could be repaired for about three-quarters of the cost of replacement. But social, rather than cost, considerations were the deciding factor, and the building was demolished and will be replaced with a two-story structure. The county employees who would spend 8 hours a day working in the building were polled and expressed considerable reluctance to spend any more time in the building.

Important lessons to be learned from the effects of earthquakes upon the two Managua buildings and the two California structures discussed above are that both structurual damage and extensive nonstructural damage are directly related to the lateral deformation of buildings during earthquakes and that deformational compatibility of structural and nonstructural elements is essential if earthquake damage is to be minimized. Importantly, damage can be

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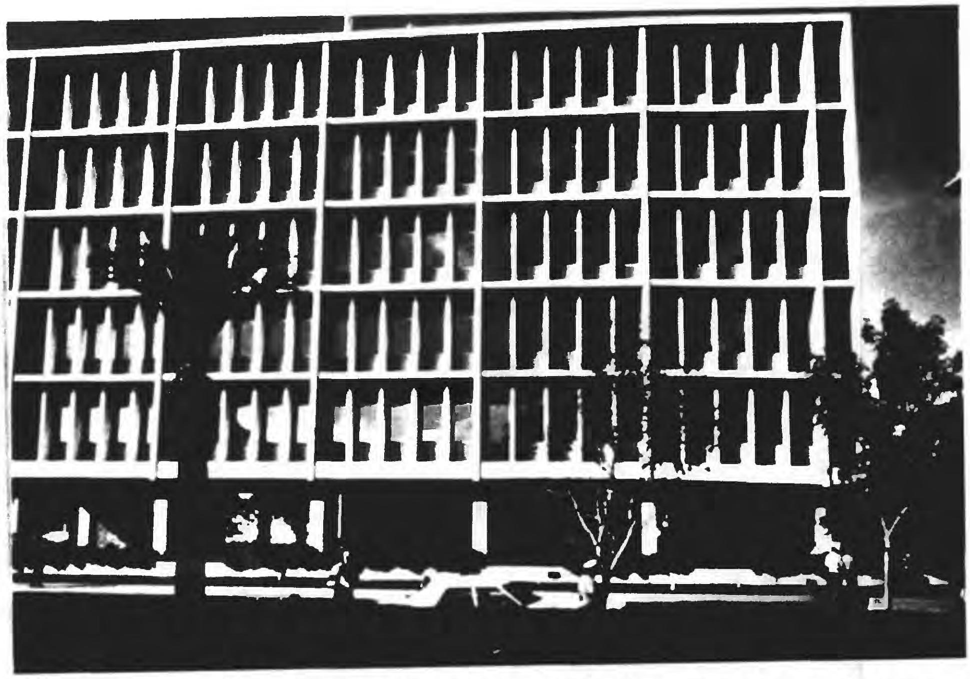
a. Overall view showing overturning at end stairwells, but generally good condition of upper portions of building.

b. Details of first-story column failure. Note obvious inadequacy of reinforcing steel tie bars.

Figure 21.--Olive View Hospital failure caused by the February 1971 San Fernando, California, earthquake.



a. View of Imperial County Services Building showing slight drop at the east end (right) resulting from crushing failure of the columns at that end of the building.

b. Crushing failure of the columns at the east end of the building.

Figure 22.--Damage to Imperial County Services Building caused by the October 15, 1979 Imperial County, California, earthquake.

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identified in terms of interstory drift. Interstory drift predictions can be used to estimate damage during the early stages of design. The recent inclusion of drift calculation requirements in the SEAOC Recommended Lateral Force Requirements and Commentary represents a positive step toward reducing future earthquake damage. In addition, the Banco Central building and the ICSB experience have shown that acceptable damage is not much damage at all.

Uniform building configuration has been regarded as important to good earthquake performance since at least the 1920s when torsional eccentricity about a vertical axis was recognized as perilous. Recent experiences with the poor performance of the Ain Nasser complex, the ICSB, and the Olive View Hospital have underscored the importance of building configuration uniformity. Our present Uniform Building Code (UBC) includes various admonitions against abrupt changes in stuffness, discontinuous shear walls, irregular shapes, and other such architectural configurations and aspects of design.

Irregular building configurations do not necessarily increase demand (i.e., load) on the overall structure, but they commonly impose an increased demand on some elements of the structure. To ensure that an irregular structure will perform well during strong ground motion, the capacity (i.e., strength) of those highly stressed elements must be increased. Just how the necessary increase in the capacity of specific members and joints should be affected in seismic design is currently widely debated. The necessary amount of increased capacity is also under debate.

Currently the code specifies that the distribution of forces in the structure shall be based on dynamic analysis for irregular structures. Thus, the overall seismic design lateral force is not increased but the demand on certain members is greater and the code design capacity of those members would be correspondingly increased. Earthquake specialists who investigated the ICSB failure are of the opinion that the overall seismic design force should be increased for such irregular buildings. Another expert has suggested that the code should only be applied to regular buildings.

The problem of the seismic design of irregular buildings can and will only be resolved when more information becomes available from future earthquake damage investigations. In addition, a clearer and more elaborate explanation of ductility in design codes would be beneficial. The SEAOC Recommended Lateral Force Requirements and Commentary only implicitly includes ductility: design force requirements are some percentage of the actual expected earthquake demand. The Applied Technology Council's Recommendations [ATC-3(06)] explicitly include consideration of demand ductility in the R factors for various types of structural systems. Neither set of recommendations discusses the procedure for quantitatively evaluating member or joint ductility demands or capacities. If member and joint ductility demands can be analytically predicted, and if member and joint ductility capacities are known, irregular structures can be designed with confidence. Substantial experimental and analytical research is needed to achieve this goal.

CONCLUDING REMARKS

This paper offers a synopsis of the engineering aspects of earthquake damage observed in recent years. Earthquake damage is still an important problem as evidenced by the failure of a relatively modern engineered building in the 1979 Imperial County, California, earthquake; the extensive damage and building failure revealed in the 1980 Algerian earthquake; and the large amount of potentially life-threatening nonstructural damage resulting from low to moderate levels of ground motion.

Perhaps the most important lesson from recent earthquakes concerns acceptable damage. The engineering community was surprised and disappointed by the failure of the Imperial County Services Building. Despite the fact that engineers proposed that the structure be repaired, the local community, particularly the people employed by Imperial County who would be spending eight hours a day in the building, deemed that proposal unacceptable and the building was demolished. A similar scenario is presented by the 15-story Banco Central building, which was damaged by the 1972 Managua, Nicaragua,a earthquake. Repair was proposed, but it was decided to remove the upper 12 stories. Accordingly, it appears that very little damage is acceptable, particularly to the public. Our seismic design codes classify damage as slight, moderate, severe, and so forth. The ability to quantify anticipated damage, to make even a rough estimate, as part of the design process would be very helpful in reducing future earthquake damage. If the designer is aware of the amount of damage that may result from a particular design, he is more likely to be motivated to modify the design to reduce the potential damage. Although damage estimates are not an established and accepted part of our current technology, I feel that it is important to focus on accurate damage estimation as a goal to work towards.

Finally, it is only appropriate to comment on nondamage. Nondamage, as indicated in the preceding sections, has been routinely reported, but has not received as much attention as damage. Without instrumental ground motion recordings, a critical investigation of structures suffering little or no damage is not very revealing. Substantial instrumental ground motion data is available for several recent earthquakes, in particular the 1979 Imperial County, California, earthquake. Detailed earthquake response studies of non-damaged buildings, such as the Imperial County Courthouse, would be very informative.

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Note: References 1 through 22 are published by the Earthquake Engineering Ressearch Institute, 2620 Telegraph Ave., Berkeley, California.

EARTHQUAKE VULNERABILITY STUDY FOR THE METROPOLITAN AREA OF SAN JUAN, PUERTO RICO

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Introduction

Among natural hazards earthquakes are one of the most devastating catastrophic events. When an earthquake occurs near a populated area, widespread destruction of life and property takes place. The island of Puerto Rico is situated in a tectonically active zone and has experienced the effects of large earthquakes in the past. The 1918 and 1867 earthquakes had an estimated magnitude of 7.5 and were accompanied by destructive tsunamis. These events caused hundreds of deaths and millions of dollars in losses. In 1787 an earthquake with an estimated magnitude of 8-8.25 severely shocked the northern coast of Puerto Rico. Similar events are likely to occur in the future.

Fortunately, a large earthquake has not affected the island in the past 62 years. During this period the population has tripled and urban areas have expanded proportionally. Presently, a significant portion of the residential, commercial, industrial, and transportation infrastructure is located on geologic materials that are vulnerable to earthquake induced geologic hazards. Thus, the potential damage created by future earthquake events in greater today than ever before.

This study examines the seismic vulnerability of the San Juan metropolitan area by mapping the spatial distribution of geologic hazards and estimating the likely damage in these zones. Three important geologic hazards are considered: ground shaking, liquefaction, and landsliding. Evaluation of the tsunami hazard is beyond the scope of this study.

Each geologic hazard is mapped according to three levels of susceptibility determined by the geologic hydrologic, and geomorphic characteristics of each zone. Damage is estimated by adapting the procedures recommended by the Rice Center (1983) for the application of earthquake risk analysis techniques to land use planning. The tasks of the earthquake vulnerability analysis are to

1) define tectonic setting and regional seismicity
2) identify sources of seismicity
3) define regional attenuation
4) select an earthquake hazard level for the analysis
5) define the geology of the study area
6) define and map ground shaking hazard
7) define and map liquefaction hazard
8) define and map landslide hazard
9) estimate damage ratio for each of the hazard zones

Identification of risk situations is necessary for local disaster preparedness, land use planning, estimation of economic losses, identification of measures for reducing expected economic loss, and for the selection and implementation of mitigation strategies.

Tectonic Setting and Regional Seismicity

The present tectonic regime of the Caribbean region differs markedly from that of the past. Malfait and Dinkelman (1972) proposed that the Caribbean and East Pacific Plates formed a single unit that separated during the Eocene (fig.1). Most of the northern boundary of the northeastern Caribbean Plate changed from a convergent to a trascurrent type of boundary (fig.2). Recent work by Sykes et al., (1982) shows the opposite; the Plate's margin is convergent, suggesting that only the angle of subduction changed as the Plate evolved. The present seismicity results from the North American Plate moving 3.7 cm./year WSW with respect to the Caribbean Plate. (Sykes et al., 1982).

Seismic activity in the Caribbean Region extends northward from South America through the Atlantic side of the Lesser Antilles and Puerto Rico, then streaks westward through Hispaniola, the Cayman Trough, and Middle America. This belt of high seismicity corresponds to the boundary of the Caribbean Plate, which is nearly aseismic below the Caribbean Sea.

Earthquakes epicenters along the Caribbean Plate margin coincide with convergent and transcurrent plate boundaries. The Cayman Trough is characterized by relatively narrow belts of seismicity caused by left lateral strike-slip motion along steeply dipping fault planes. Right lateral strike-slip motion characterizes the southern boundary of the Caribbean Plate north of Venezuela. Wider belts of seismicity are present in zones where convergent processes are ocurring. Plate convergence in presently active from Hispaniola to Trinidad at the north and east portions of the Plate, and on its western boundary along Central America.

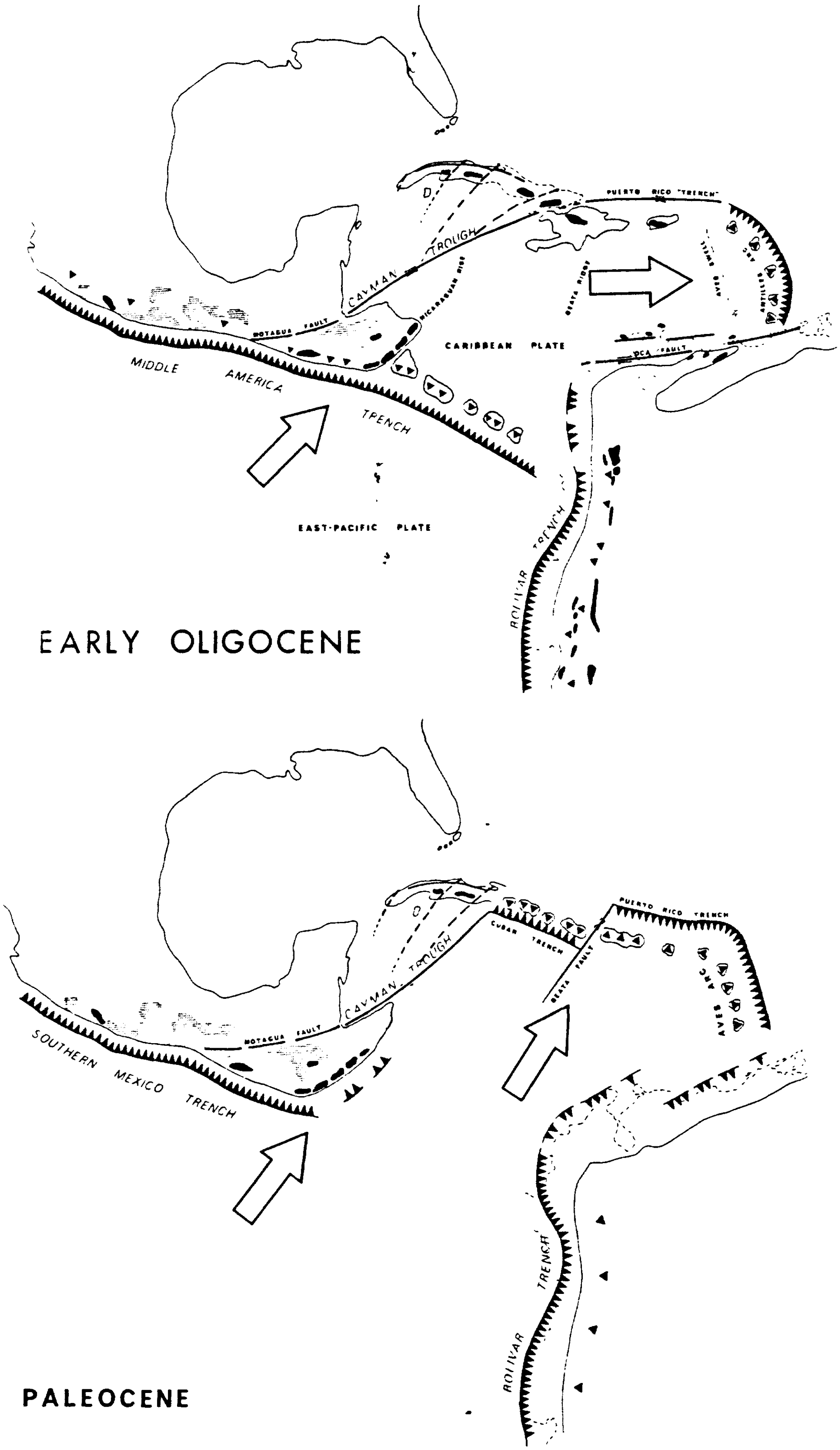

Figure 1. Distribution of plate boundaries and movement during the Paleocene and Early Oligocene (From Malfeit and Dinkelman, 1972).

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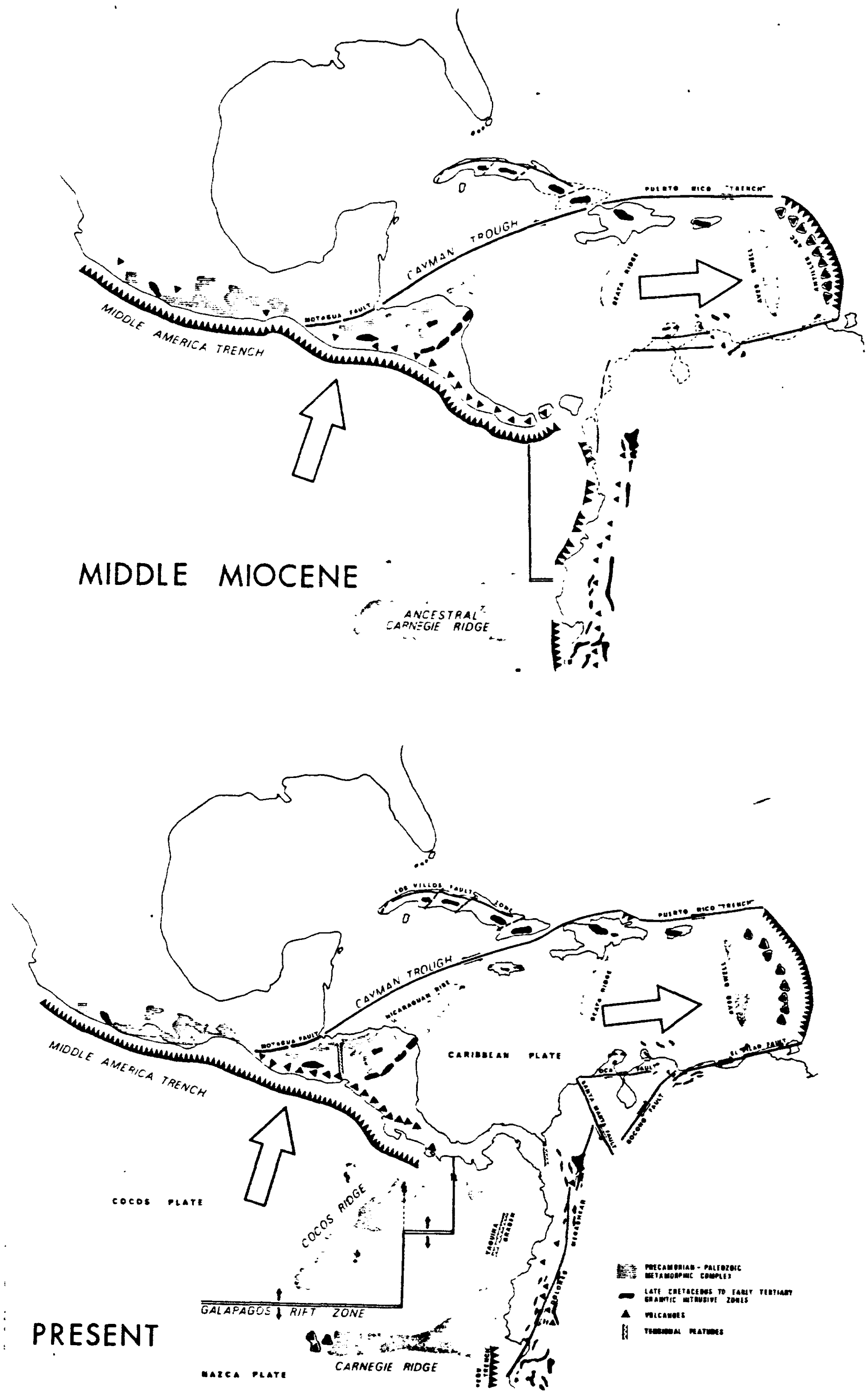

Figure 2.--Distribution of plate boundaries and movement during the Middle Miocene and Holocene (From Malfait and Dinkelman, 1972).

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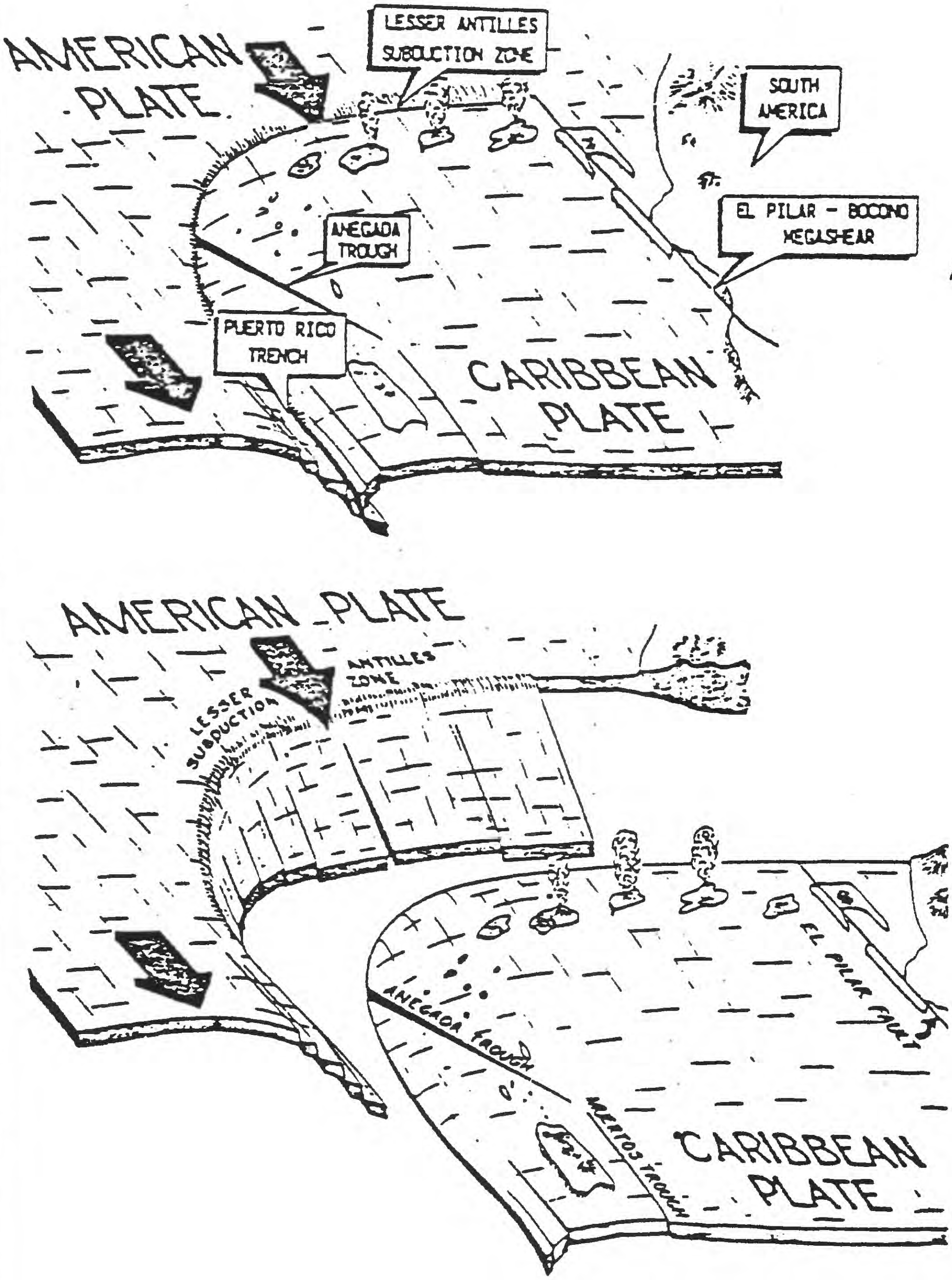
Hypocentral distribution of seismicity in the Caribbean indicates a dip of seismic activity from the Atlantic Plate margin toward the Caribbean Sea. The foci of these earthquakes are distributed on well-defined planes that dip into the mantle. Dipping planes of seismicity define the position of the North American Plate which is plunging into the Earth's mantle beneath the Caribbean Plate (fig.3). The results from data collected by the Puerto Rico Seismic Network firmly establish the existence and configuration of the North American lithospheric plate below the Puerto Rico-Island block. Intermediate-depth earthquakes located by the Puerto Rico Seismic Network form a prominently inclined seismic zone dipping about 45-60 degrees from the Puerto Rico Trench to a depth of about 150 km.under the island (fig. 4).



A. View is toward the southeast with American and Caribbean plates intact.

B. Plates pulled apart to allow visualization of subsurface configuration.

Boundary configuration of the North-American and Caribbean plates

Fig. 3 From Schell and Tarr 1978

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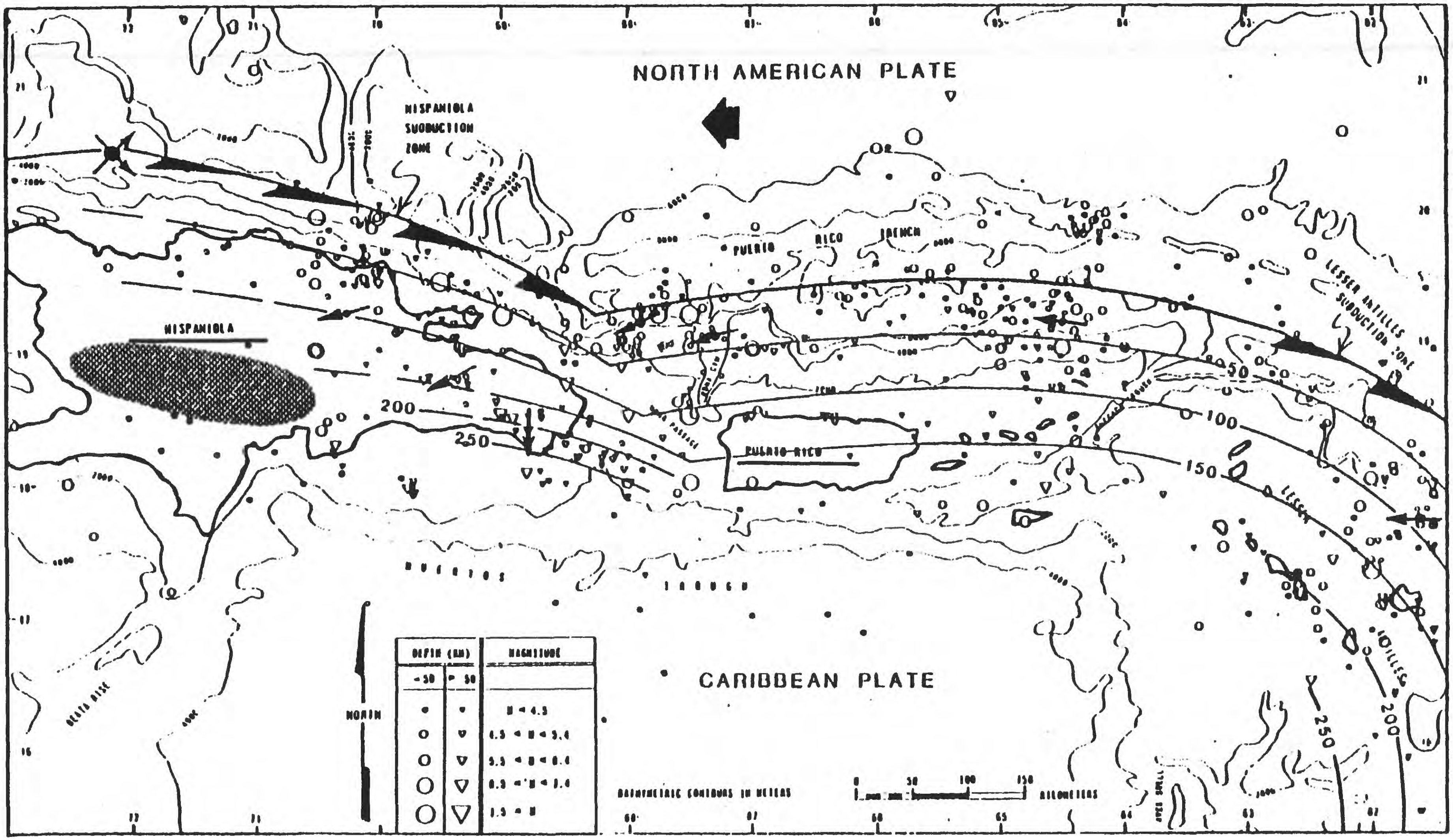

Figure 4. Major plate-tectonic features and seismicity in the northeastern Caribbean sea region (From Schell and Tarr).

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The Sources of Seismicity

The on-site seismicity of Puerto Rico is characterized by the general absence of large and shallow events on the island itself. Small magnitude events of generally less than magnitude 3 typify its seismicity (Dart et. al. 1980). The largest shallow earthquakes on the island were located west of Guajataca in the northwest and near La Parguera in the southwest (NORCO-NP-1-ER, 1972).

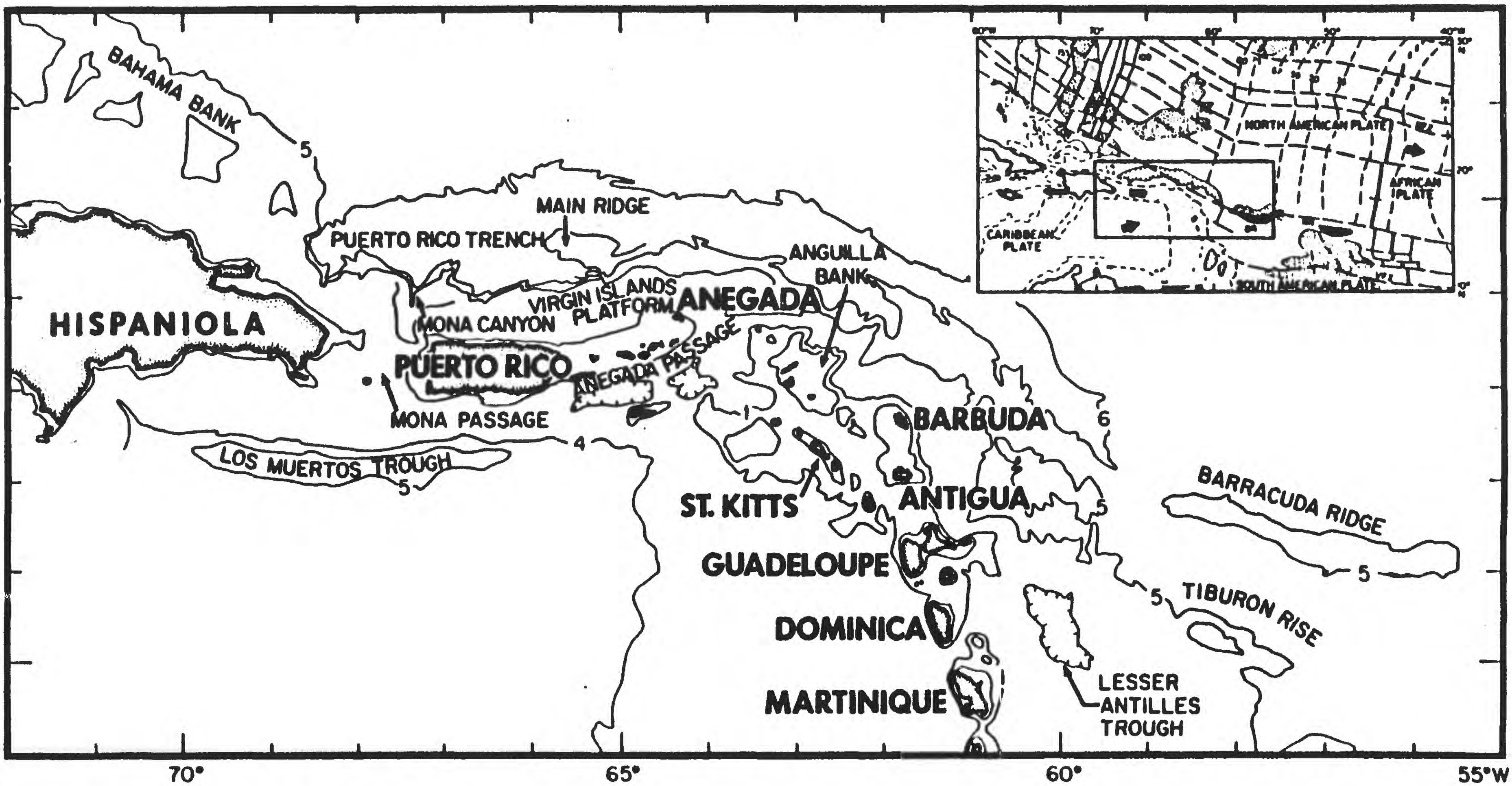
Seismic events with epicenters in Puerto Rico are not likely to cause significant damage. The essentially undeformed nature of Middle Terciary limestones and the absence of evidence of faulting indicate a long period of tectonic stability with respect to surface faulting. Thus, the probabilities of ground rupture due to faulting in San Juan are very low.

The off-site seismicity is the product of seismically active off-shore zones where large magnitude events have occurred in the past. The most significant, seismically active, tectonic features capable of generating large earthquakes are the Puerto Rico Trench, the Mona Canyon-Mona Passage area, the Anegada Passage, and the northern portion of the Muertos Trough along the southern slope of the Puerto Rico insular shelf (fig.5).

The Puerto Rico Trench forms an arc that extends about 100 km north of the eastern cape of Hispaniola to approximately 200 km east of Barbuda. It parallels the north-eastern Caribbean arc system. The Trench axis lies at a depth of 8 km north of the Puerto Rico-Virgin Islands platform. The Puerto Rico Trench is bounded by high angle faults with a structural configuration suggestive of a downdropped block. Most seismic events are of shallow focus and occur in clusters where the Mona Canyon meets the Puerto Rico Trench northwest of Puerto Rico and in the area immediately northwest of Anegada. Fault zones just south of the Trench are likely to produce earthquakes with magnitudes as large as 8 to 8.25 (McCann, 1984). Puerto Rico is aproximately 60 km from the southern wall of the Trench. The closest fault zone south of the Trench that extends to the sea floor is about 35 km north (NORCO-NP-1-ER pag 9.c-15) of the north central coast.

The Mona Canyon-Mona Passage area is located between Puerto Rico and the Dominican Republic. Seismic activity is largely concentrated on the western side of the Mona Passage. The most prominent features of the passage are the north and north-westerly striking gravens extending from the Muertos Trough in the South to the Puerto Rico Trench in the north. The Mona Canyon graven seems to be the source of the 1918 earthquake (M=7.5) which, in conjunction with a tsunami that flooded the coastline, caused widespread destruction in the north-western region of Puerto Rico. The earthquake was probably caused by vertical displacements of the faults bounding the Canyon (Reid and Taber, 1918).

The seismicity along the Muertos Trough is low compared to that of the Puerto Rico Trench. The Muertos Trough is located approximately 75 km south of Puerto Rico. It extends from south of the Dominican Republic to near the St. Croix Ridge. This structure is likely to be a subduction zone where the northern margin of the Venezuelan Basin moves underneath Puerto Rico. This may indicate that Puerto Rico is a smaller plate or block separating the larger plates (McCann, 1984). Major quakes with a long repeat time are likely to occur on the slope south of Puerto Rico. Contrary to the eastern region where any fault rupturing during an event is of limited length (McCann, 1984), the western and central parts of the insular shelf's



Place Names and general bathymetry of northeastern Caribbean

Fig. 5

From McCann 1984

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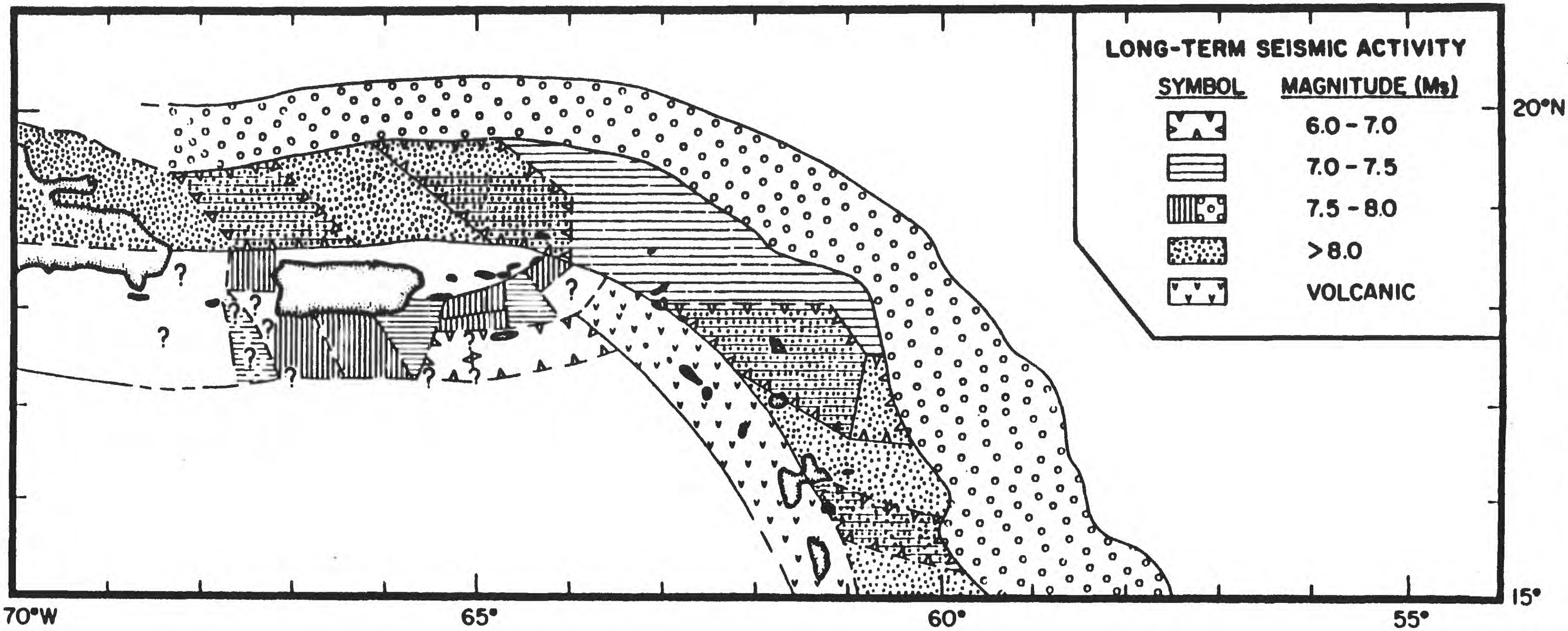
southern slope are likely to generate major earthquakes (M-7-8) because the tectonic blocks are bounded by long faults.

The Anegada Passage, lying 50 km east of Puerto Rico, consists of several basins and ridges that separate St. Croix from the Puerto Rico-Virgin Islands platform. Complex geologic features are present around the Virgin Islands and St. Croix basins. Faults in the northern wall of the Virgin Islands Basin are a likely source of strong shocks (M=7-8). The large earthquake of 1867 presumably originated along the northern flank of the Virgin Islands Basin (Reid and Taber, 1919). Although McCann's (1984) work concludes that the major earthquake hazard comes, not from great earthquakes to the north, but from major ones occurring closer to the land, this author concludes that the major earthquake hazard to the Metropolitan Area of San Juan comes from the Puerto Rico Trench to the north for the following reasons:

a) The San Juan metropolitan area is closer to the Puerto Rico Trench (approx. 60 km.) than to the Anegada Passage (approx. 100 km.) or the Mona Canyon (approx. 120 km.)

b) Following McCann, the frequency of great seismic events in the Puerto Rico Trench may not be different from that of major events originating from faults closer to the land. Thus, closer epicentral distance and great events with the same frequency of major ones closer to the island expose the metropolitan area of San Juan to a higher hazard from this zone.

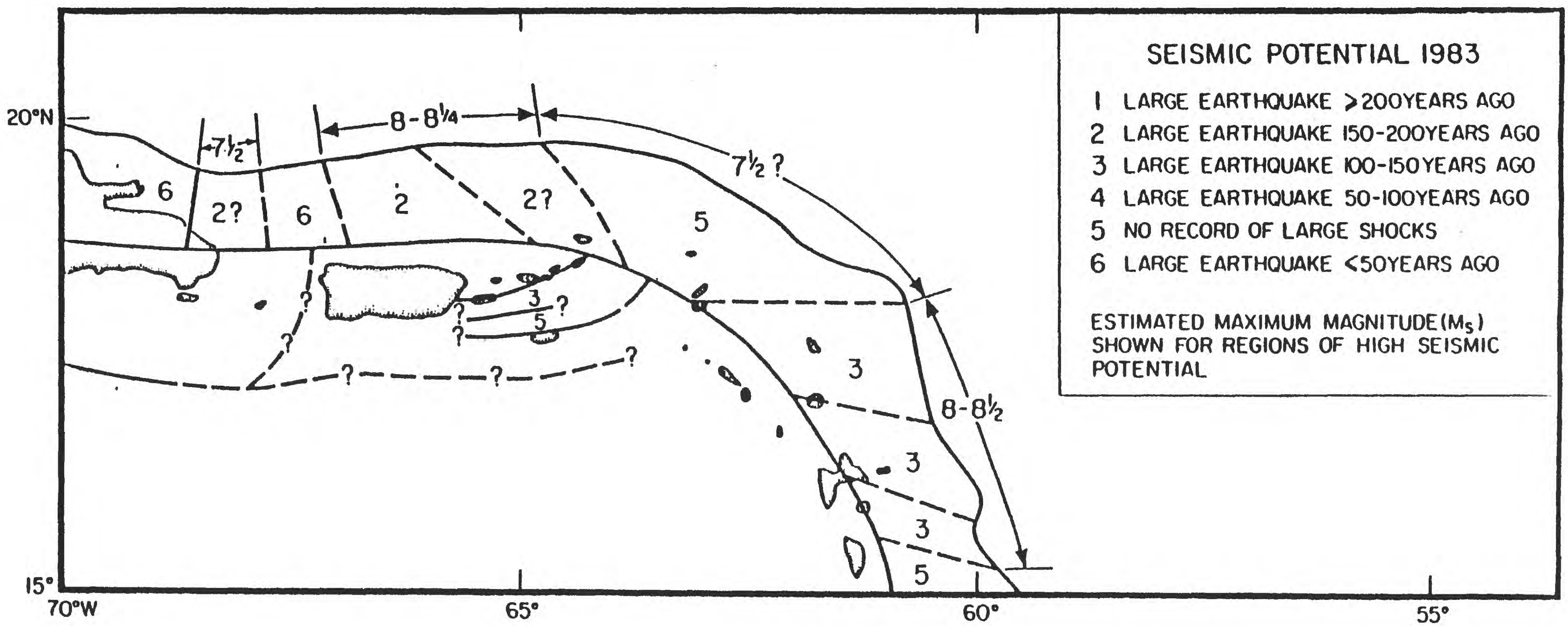
c) The portion of the Puerto Rico Trench north of San Juan is a zone of little seismicity likely to experience maximun magnitudes about 8.8.25 perhaps every 200 years (minimun value) (McCann 1984 Fig 6).



Estimate of long-term activity of shallow focus along the Caribbean - North American plate boundary

Fig. 6

From McCann and Sykes, 1984



Estimate of Seismic Potential for the northeastern Caribbean

Fig. 7

From McCann 1984

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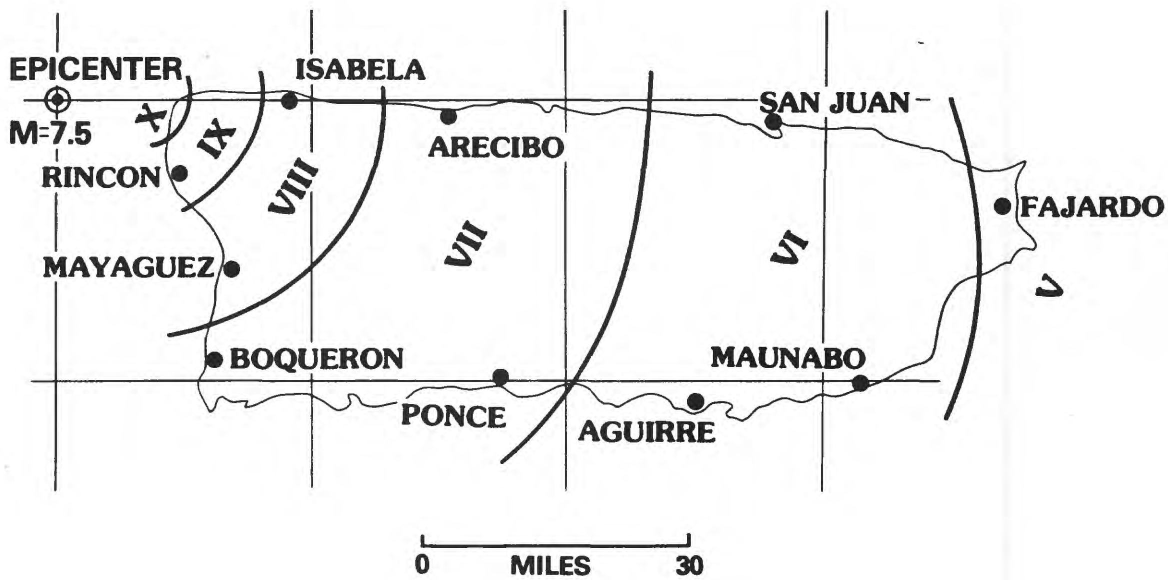
Attenuation

The appropiate estimation of earthquake energy attenuation is a fundamental part of the seismic vulnerability analysis because energy attenuation, as determined by path parameters, determines ground motion intensity on a regional scale. The lack of strong ground motion records and the limited usefulness of attenuation rates from other geographical areas require the use of isoseismal maps from past earthquake events in the area. The critical data contained in an isoseismal map are the values of maximun intensities reported at various locations either in Modified Mercalli or Rossi-Forel intensity scales. These values are plotted on an iso-intensity contour map. The isoseismal map for the earthquake of October 11, 1918 and November 18, 1867 are shown in fig.8 and 9 respectively. The contours can be deceiving because isoseismal maps typically represent intensity values reported at sites underlaid by alluvium or unconsolidated materials. Because these sites undergo more intense ground motion than sites underlaid by rock, attenuation functions derived from an isoseismal map without regard for the local site geology may overestimate ground motion at the site of interest (Hays, 1980).

The regional earthquake intensity attenuation used in this study is presented in fig 10. Differences of up to 1 on the Modified Mercalli intensity scale occurred between sites located in good and poor foundation conditions during the October 11, 1918 earthquake. This shows the effect of local ground conditions on earthquake ground motions. These intensity attenuation relations are equivalent to a reduction of 2 orders of magnitude at an epicentral distance of 120 kilometers. This relation is consistent with that shown for the July 7, 1970 earthquake in Figure 11 (Capacete, 1972).



From Western Geophysical Research Inc.

Figure 8. Isoseismal Map of the October 11, 1910 Puerto Rico Earthquake. (Intensitites are Rossi-Forel).

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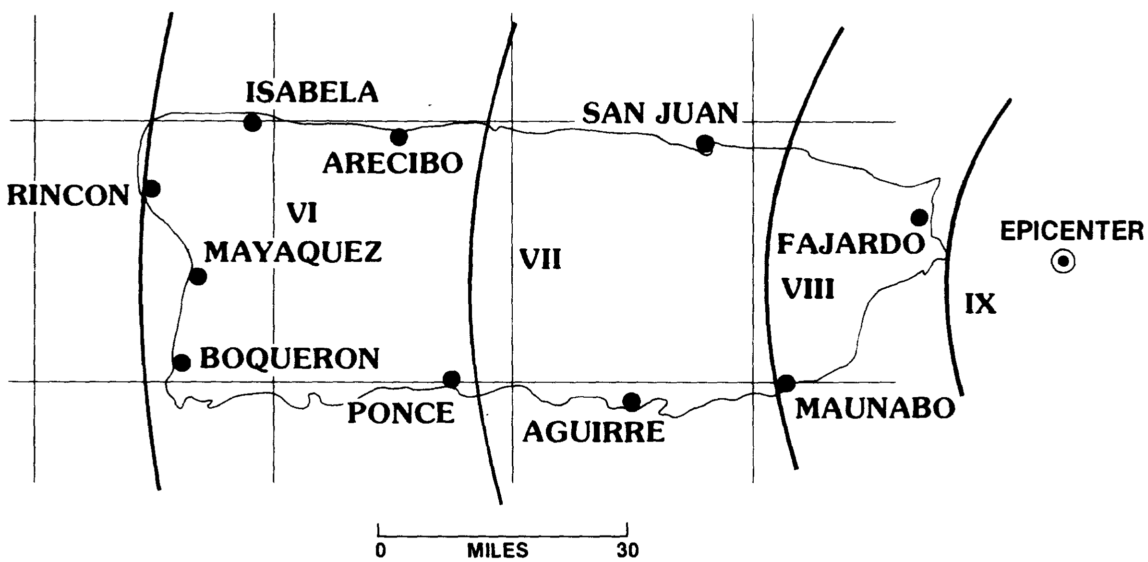

Figure 9. Isoseismal Map of the November 18, 1867 Virgin Island Earthquake (Intensities are Rossi-Forel).

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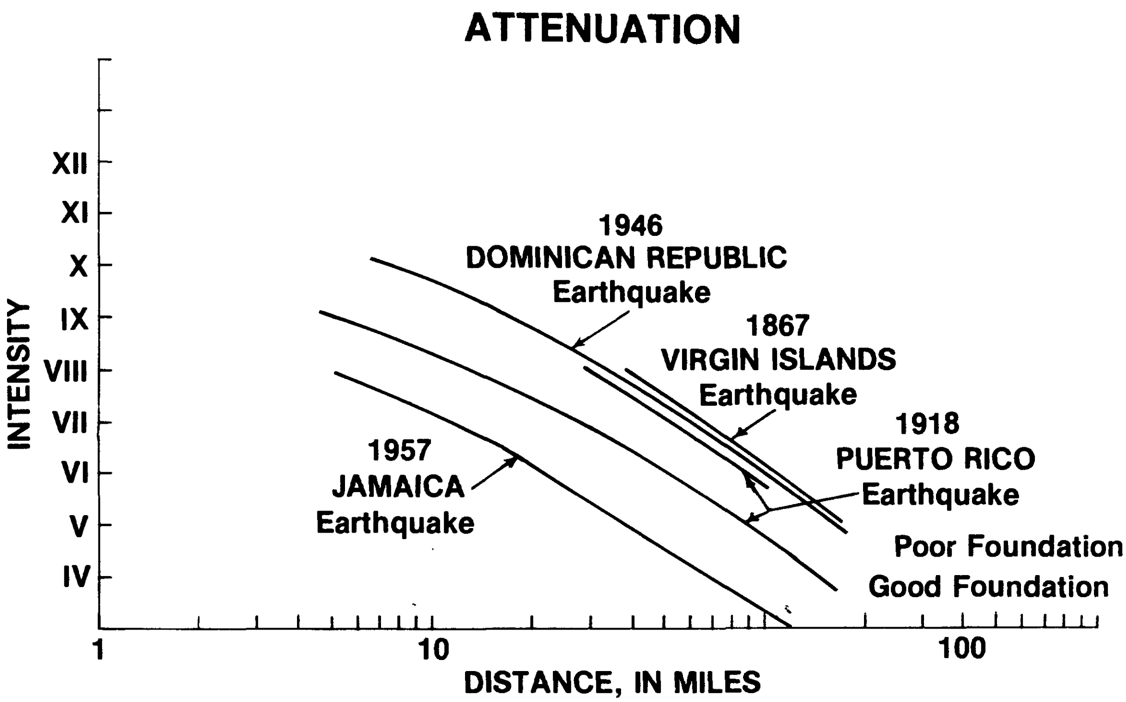

Figure 10. Regional earthquake intensity attenuation.

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. Prince William Sound Earthquake of 1964   M = 8.3
Δ Puerto Rico Earthquake of July 7, 1970,   M = 5.8   Δ = 120 miles

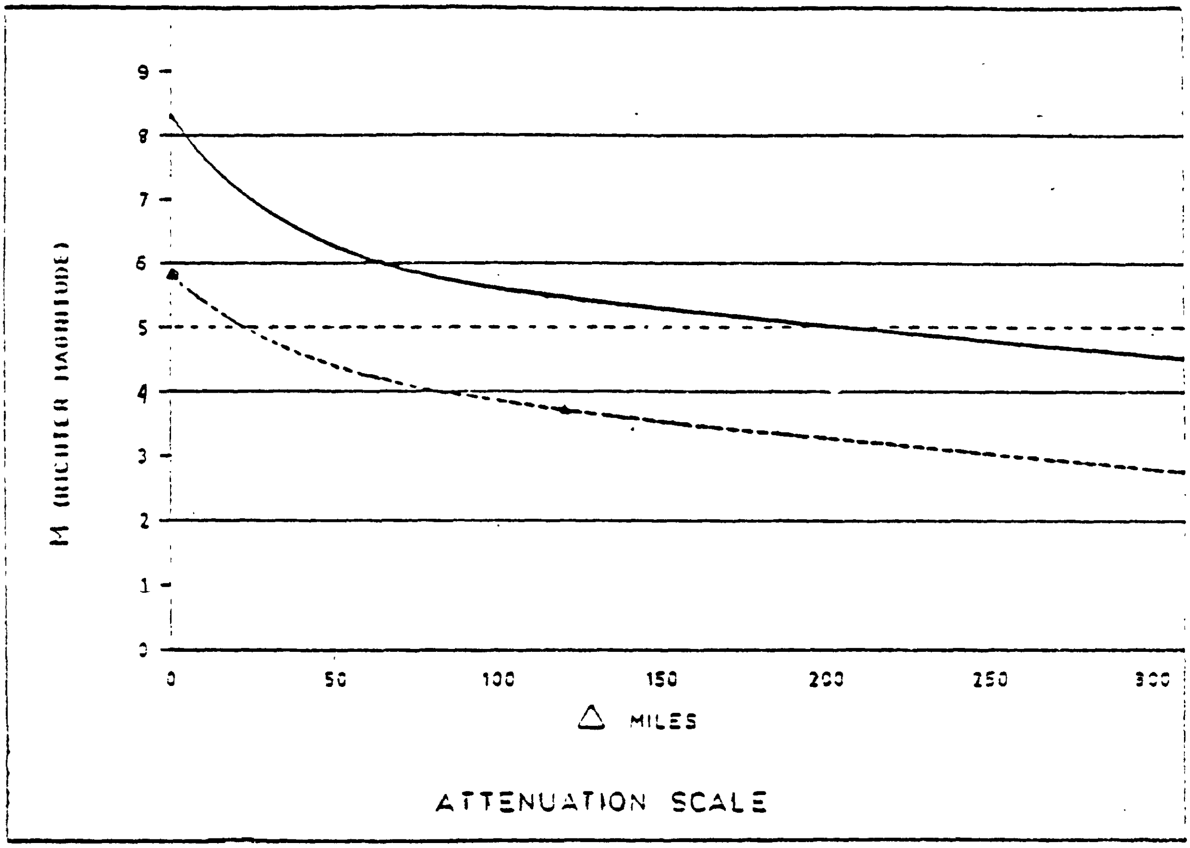

Figure 11. Earthquake attenuation curve for the July 7, 1970, earthquake (From Capacete, 1971).

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Selection of Earthquake Hazard Level

A probabilistic approach that incorporates judgement of the researcher is used in the selection of the earthquake hazard level for this study. An earthquake recurrence analysis prepared by the Department of Natural Resources (personal communication: Anselmo De Portu) using a catalogue of all instrumentally located earthquakes within 330 kms. of San Juan between 1915 and 1983 shows the one hundred year earthquake to be of an order of magnitude 8 on the Richter Scale (Appendix I). This is approximately the same order of magnitude as the largest earthquake in the historic record (8.0-8.25) (table 1). While great earthquakes ($M \leq 7.75$) will occasionally occur in the Puerto Rico Trench 50 to 100 kms to the north of the Island, the historic record and regional tectonic framework suggest that major shocks (M-7-7.5) occur on intraplate faults close to the Island just as frequently (McCann, 1984). These events (1867, 1918) did not cause serious damage in San Juan, but on the east and northwest coasts. The 1867 and 1918 earthquakes generated intensities equivalent to VI and V to VI at San Juan and Río Piedras. The historic record indicates that San Juan has experienced an intensity VIII to IX only once-- the 1787 earthquake. On the other hand, the Island as a whole, over a period of 450 years, had been subjected to one earthquake of intensity VIII or IX and to intensities VII to VIII five times (der Kiureghian and Ang, 1975). Thus, in terms of intensity, the island of Puerto Rico experiences on the average an MM intensity of VIII once every hundred years. Return periods in terms of intensity are presented below.

Return period in years	MMI	Estimated maximum acceleration
50	VII	.15
90	VII-VIII	.18
100	VII-VIII	.19
200	VIII	.25
450	VIII-IX	.33
500	IX	.35

(der Kiureghian and Ang, 1975).

Different criteria can be used to select a particular earthquake hazard level. Return periods of 500 years (maximum credible earthquake), of 100 years (widely used in flood plain management), and 50 years (approximate structure life in some areas) have been suggested for use in earthquake risk analysis (Rice, 1983). The maximum credible earthquake focuses on lower probability events with return periods of 300 years or more. The most probable earthquake considers a shorter return period of 100 years. Introducing conservatism in the selection of the maximum possible earthquake that can damage San Juan requires the selection of the maximum historical earthquake (Slemmons, 1982) (8-8.25) and moving it the closest credible epicentral distance to the study area (approx. 60 kms.). Such earthquakes will produce maximum intensities of X to XI, causing very severe to total damage in the San Juan metropolitan area. Its return period greatly exceeds the useful life of most building structures. A more realistic estimate is obtained by selecting a smaller but more frequent earthquake capable of causing significant damage. In addition, the damage pattern of the selected hazard level should exceed the threshold for most secondary geologic hazards. In this way, damages produced by higher levels of ground motion will change proportionally but not areally, permitting the estimation of likely damages for different hazard levels.

The 100 year earthquake, capable of producing an estimated MM intensity of VIII, fits the above requisites. Such intensity is felt in Puerto Rico (on the average) once every 100 years. Although San Juan experienced a similar intensity only once, conservatism dictates the use of the maximun intensity felt in Puerto Rico every 100 years.

Thus the selected hazard level is MM intensity VIII. Such intensity can be caused by an earthquake Richter magnitude 8 with epicenter 120 km north of San Juan.

TABLE I

Most destructive earthquakes felt on Puerto Rico

Date	Estimated Maximun Intensity	Description
1524-1528	VI	The Añasco house of Juan Ponce de León and other strong buildings were destroye The shock was felt strongest in the nortl from Mayaguez to Añasco
1615 Sept. 8	VI	The earthquake and hurricane did much damage and caused great suffering in Puerto Rico. Epicenter probably in or near Santo Domingo. Many aftershocks during the next 40 days.
1717	VII	Very strong and damaging earthquake. The San Felipe Church in Arecibo was complet ly ruined. The 100 year old parish hous in San German was destroyed.
1740	VI	The earthquake totally destroyed the Guadalupe Church in Ponce.
1787	VIII-IX	A violent earthquake felt over the entir Island. Many churches and chapels destroyed. In San Juan great damage was do to the forts of el Morro and San Cristob as well as to the docks and the Cathedra
1844	VI	Severe earthquake of 30 seconds duration The origin may have been north of Puerto Rico. Several houses and some public buildings were demolished or cracked. In San Juan nearly all stone houses were cracked.

Date		Estimated Maximun Intensity	Description
1846	November 28	VI	Felt throughout the Island, Epicenter probably in the Mona Passage. More intense in the northwestern part of Puerto Rico.
1867	November 18	VII-VIII	This was the great Virgin Islands earthquake that caused very great damage, specially in the eastern part of Puerto Rico. The shock was followed by a severe tsunami
1875	December 8	VI	Strong earthquake knocked down some chimneys at sugar mills and damage was reported in Arecibo and Ponce.
1906	September 27	V-VI	Heavy double shock with epicenter north of Puerto Rico. In San Juan objects were overturned and people were frightened and confused, but material damage was not done.
1918	October 11	VIII	Disastrous earthquake accompanied by tsunami. Very great damage to the west coast of Puerto Rico. Epicenter in the Mona Canyon northwest of Mayaguez
1946	August 4	VI	Strong earthquake with epicenter in the Dominican Republic caused general alarm and fear. No loss of life or serious property damage.
1946	August 8	VI	Strong earthquake of short duration accompanied by tsunami affected mostly the west coast. People terribly frightened, but no significant damage was done

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Geology

The San Juan metropolitan area lies on the northern flank of a thick sequence of highly deformed and faulted early Cretaceus to Early Terciary volcanic and sedimentary rock. Mid-Terciary epiclastic and limestone sequences rest over the deformed volcanic core. Late Terciary and Quaternary unconsolidated to semiconsolidated terrigenous materials overlie most of the Mid-Terciary formations and portions of the volcanic core (fig.12). The geology and the stratigraphic summary of the metropolitan area of San Juan appear in fig 13 .

Three physiographic regions are present in the study area: the interior volcanic upland province, the northern Karst province, and the Coastal Plain province (fig.14). These provinces are characterized by a unique combination of relief, landforms, and geology.

The interior upland shows the effects of fluvial erosion over a complex sequence of volcanic and sedimentary deposits of Cretaceous and Early Terciary age. The Cretaceous rocks were formed during a period when volcanism and sedimentation were dominant geological processes. The lower Cretaceous rocks consist primarily of lava, lava breccia, tuff and tuffaceous breccia with some thin bedded sandstone, siltstone, and limestone. When exposed they are thickly weathered. Upper Cretaceous rocks consist of tuffaceous sandstone, siltstone, breccia, conglomerate, lava, tuff, and some pure and impure limestone lenses. When exposed they, too, are deeply weathered (Briggs and Akers, 1965; Briggs, 1964). The collision of the Caribbean Plate with the North American Plate by the end of the Mesozoic gave rise to the "Caribbean Orogeny" (Malfait et al., 1972). At the end of orogeny (Middle Eocene), most Cretaceous

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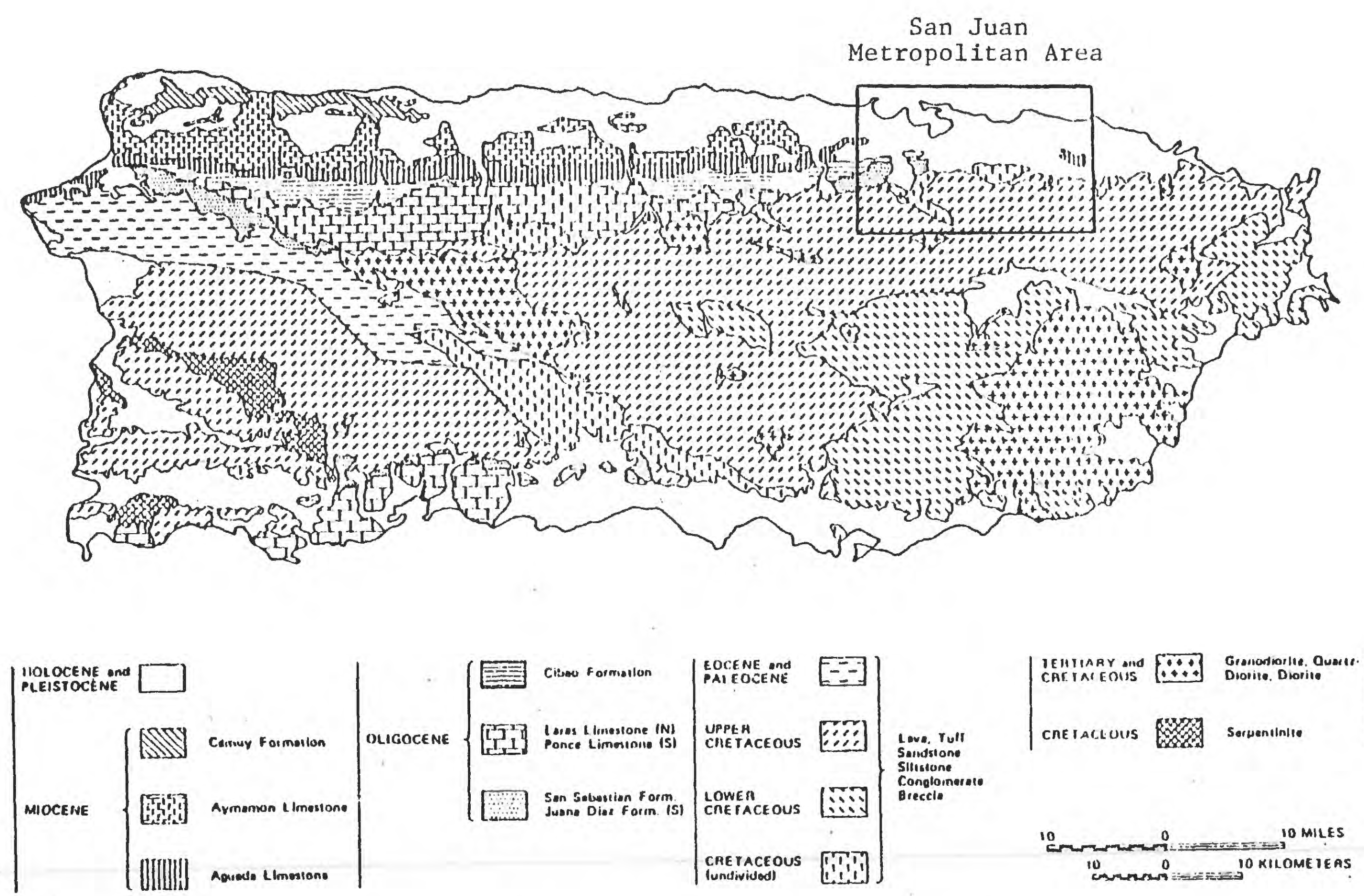

Simplified geologic map of Puerto Rico

Fig. 12

From Beinroth (1969)

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Stratigraphic summary of the San Juan area, Puerto Rico

General designation	Stratigraphic unit	Brief description	Approx. thickness (feet)	Age
	Made-ground and fill.			
	Beachrock pavement	Carbonate-cemented beach sand. Some iron-oxide cementation		Recent
	Recent littoral deposits	Beach sand and associated sand aprons; foredunes		
	Floodplain alluvium	Mostly silt and clay		
	Bay mud	Soft black mucky silt and clay		
	Pleistocene littoral deposits	Reef rock, eolianite, and paleosols		Pleistocene
	Santurce sand	Quartz sand and white clayey sand		Pleistocene and Pliocene (?)
	Older alluvium	Red silty to sandy clay		
	—— Unconformity ——			
Middle Tertiary sequence.	Aymamón limestone	Thick-bedded, light-colored, dense limestone	950+	early Miocene
	Aguada formation	Friable sandstone, clay, and concretionary limestone	325	
	—— Unconformity ——			
	Intrusive igneous rocks	Diabase, granodiorite porphyry, and augite andesite porphyry		early Eocene (?) or late Paleocene (?).
	Fajardo formation	Light-colored ashy siltstone, siliceous siltstone and chert, interfingering graywacke, conglomerate, and impure limestone	3,000+	
	Figuera volcanics	Hornblende andesite breccia, minor flow, limestone member at base	>3,000	
	—Local unconformity—			
Older complex	Trujillo Alto limestone	Fossiliferous medium-bedded to massive limestone	900−	Late Cretaceous
	Monacillo formation	Graywacke and conglomerate; commonly red or purple	900	
	Frailes formation (Leprocomio limestone member and La Muda limestone member).	Massive lapilli tuff with some stratified shale, siltstone, and graywacke. Leprocomio limestone member at top and La Muda limestone member near base.	2,350	
	Tortugas andesite	Augite andesite breccia and flows	1,300	
	Guaynabo formation	Graywacke, conglomerate, and shale	4,500	Late Cretaceous (?)
	Hato Puerco tuff	Massive volcanics and metavolcanics and some stratified ash	(?)	

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Stratigraphic summary of the metropolitan area of San Juan

Fig. 13

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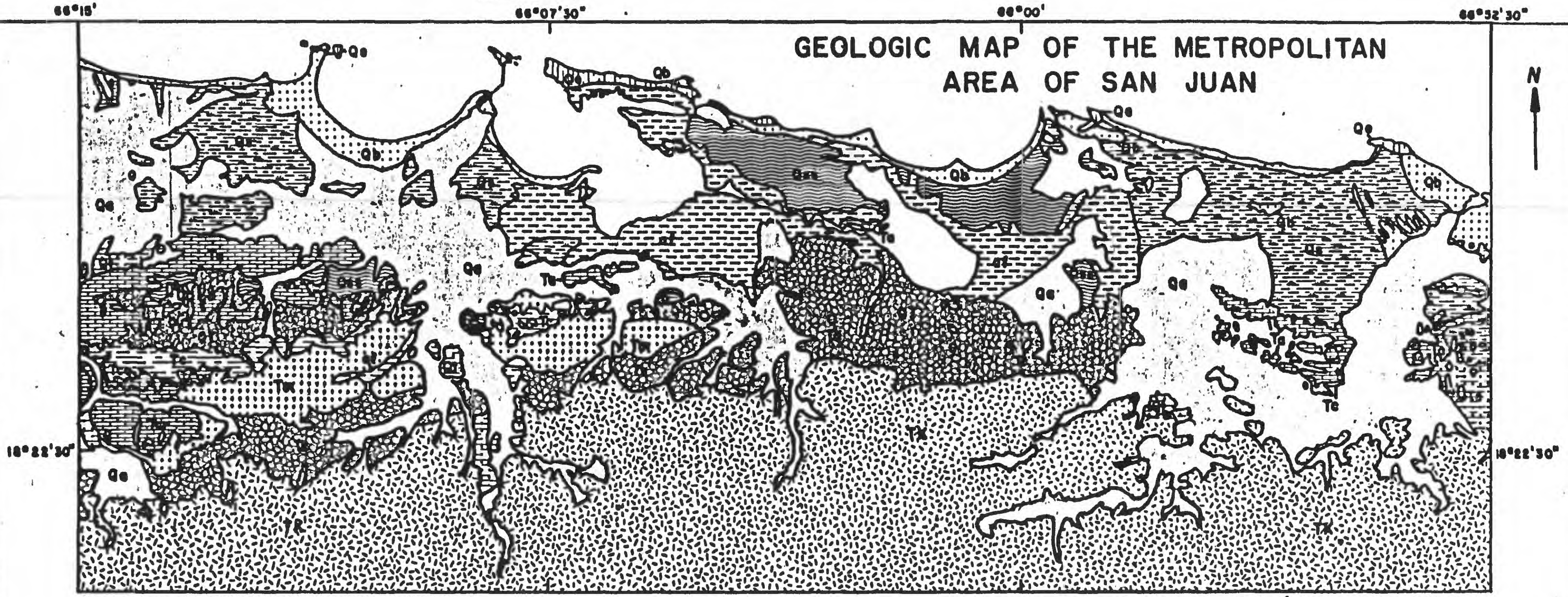
GEOLOGIC MAP OF THE METROPOLITAN AREA OF SAN JUAN



Adapted from GEOLOGIC MAP AND SECTION IN NORTHERN PUERTO RICO, U.S.G.S., 1978

Prepared for José Molinelli by Thelia Vove
April 1988

DESCRIPTION OF MAP UNITS

af — ARTIFICIAL FILL (HOLOCENE) - Sand, limestone, and volcanic rock

Qa — ALLUVIUM (HOLOCENE) - Sand, clay, gravel, and cobbles

Ql — LANDSLIDE DEPOSITS (HOLOCENE) - Masses of soil, calcareous clay, and limestone rubble

Qb — BEACH DEPOSITS (HOLOCENE) - Sand composed of rounded grains of quartz, volcanic rock, and shell

Qs — SWAMP DEPOSITS (HOLOCENE) - Sandy muds and clayey sand

Qe — EOLIANITE (PLEISTOCENE) - Friable to consolidated calcareous eolian sandstone composed of fine to coarse grains of shell fragments and quartz

Qss — SILICA SAND DEPOSITS (PLEISTOCENE) - Very pure fine to very fine quartz sand

Qt — TERRACE DEPOSITS (PLEISTOCENE) - Clay, sand, and gravel deposits

QT — UNDIFFERENTIATED SURFICIAL DEPOSITS (QUATERNARY AND TERTIARY?) - Clay, sandy clay, and sand deposits

Ta — AYMAMÓN LIMESTONE (MIOCENE) - White to very pale orange, locally pale-yellow and grayish-pink, very pure fossiliferous limestone and AGUADA LIMESTONE (MIOCENE) - Thick layers of very pale orange to pink hard calcarenite alternating with chalky and rubbly limestone

Tc — CIBAO FORMATION (MIOCENE AND OLIGOCENE) - Typical beds of calcareous clay, earthy limestone, and marl, very pale orange to pale yellowish-orange and moderate-orange-pink

Quebrada Arenas Limestone Member - Very finely crystalline to dense, very pale orange to pale-greyish-orange limestone

Río Indio Limestone Member - Compact, but chalky, fragmental limestone, mostly pale-yellowish-orange

Tm — MUCARABONES SAND (MIOCENE AND OLIGOCENE) - Greyish-orange to yellowish-brown highly crossbedded medium to coarse calcareous sand

Ts — SAN SEBASTIÁN FORMATION (OLIGOCENE) - Reddish-brown and gray sandy carbonaceous clay

TK — UNDIFFERENTIATED LOWER TERTIARY AND CRETACEOUS FORMATIONS - Limestone, tuffaceous sandstone and siltstone, tuff breccia, lava, granodiorite and quartz diorite

SCALE

0 — 5 — 10 KILOMETERS

0 — 2.5 — 5 MILES

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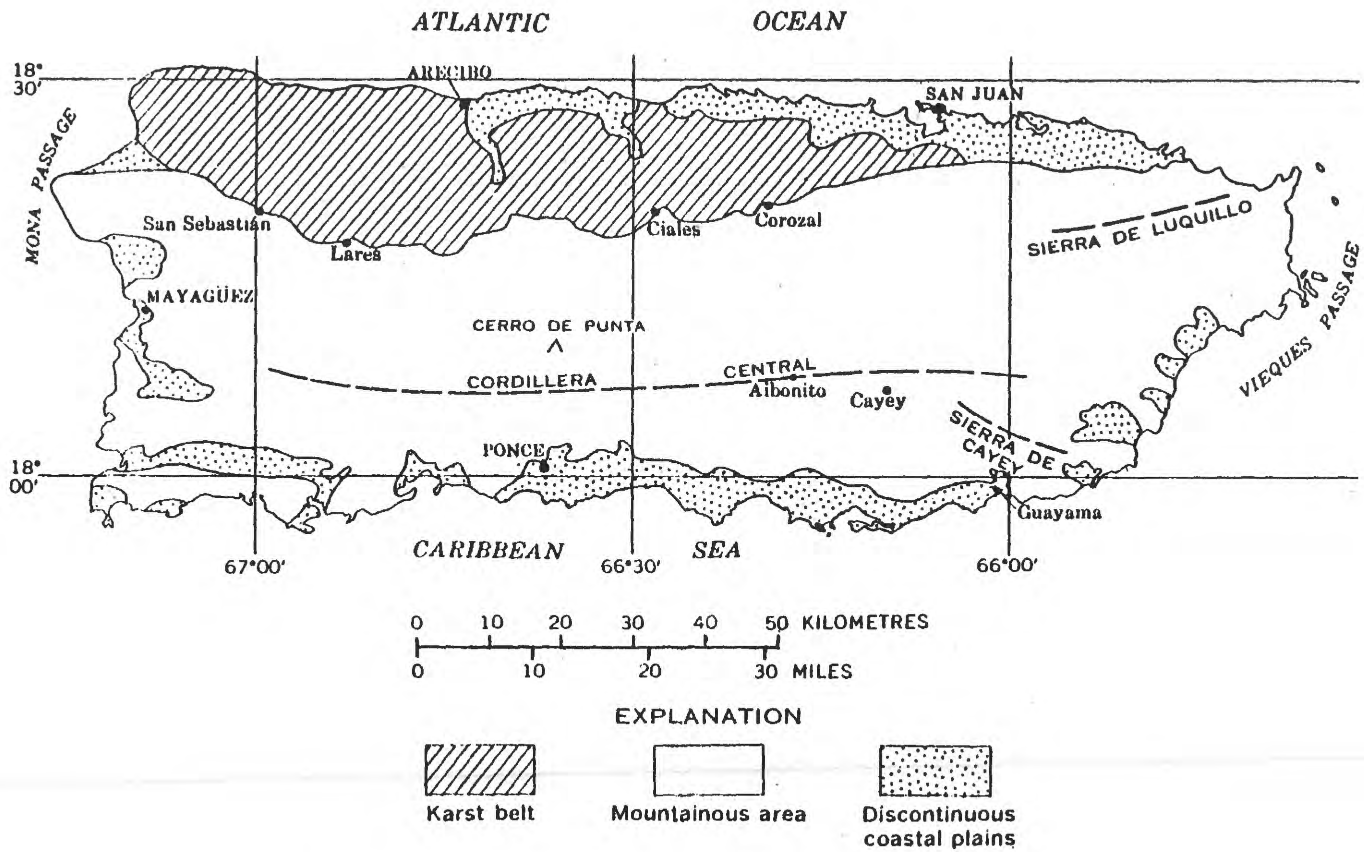

Map from Puerto Rico showing principal physiographic divisions

From Monroe (1976)

Fig. 14

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and Early Terciary rocks had been faulted, folded and intruded. Early Terciary rocks were formed during a mountain building period. Both intrusive and extrusive igneous activity were the dominant geologic processes.

Intrusive rocks emplaced during the orogeny are mainly granodiorite, quartz-diorite, diorite and some minor quartz porphyry, gabbro, and amphibolite. Associated with the intrusives are zones of hydrothermal alteration and contact methamorphism (Hildebrand, F.A., 1961).

Paleocene and Eocene deposits consist of siltstone, sandstone, conglomerate, lava, and tuff. They are locally deeply weathered.

The northern Karst province in the study area consists essentially of the following formations; San Sebastian, Cibao, Aguada, and Aymamon (Monroe, 1973, 1976, 1977, 1980, Pease and Monroe 1977). The San Sebastian formation is at the base of the Mid-Terciary sequence, lying unconformably over Cretaceous volcanics and sedimentaries. The formation is heterogeneous and contains clayey sand, lenses of sandy clay, pebbles, and cobbles. South of San Juan it grades upward into thin bedded, fine sand and mottled clay. The thickness is greater than 40 meters. The Cibao formation consists of an argillaceous marl, chalky limestone, and thin beds of sand and clay. Outcropping members are Miranda sand, upper member, and Quebrada Arenas and Rio Indio limestone. The Aguada formation consists of alternating beds of indurated calcarenite and clayey to chalky limestone. Its thickness ranges from 70 to 35 meters. Comformably overlying the Aguada is the Aymamon limestone formation consisting of massive to thickly bedded very pure fossiliferous limestone (Monroe, 1980, 1973). Sinkhole formation is a potential hazard in the Aymamon and Aguada limestones formations.

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The Coastal plain province consists of Late Terciary and Quaternary deposits. Late Terciary sequences include older alluvial deposits, high terrace deposits, alluvial fans (Hato Rey Formation), alluvium and river terrace deposits, silica sands, beach deposits, swamps, eolianite, and artificial fill.

Older alluvial deposits, high terrace deposits, and alluvial fans consist of varying proportions of clay, silt, and sand, mainly red or mottled red. The material is deeply weathered, stiff, and hard. Most of the non-quartz components are altered into clays. They are unrelated to present stream alluviation.

Holocene alluvium and river terrace deposits of Pleistocene age consist of sand, clay, and sandy clay. Beds of sand containing gravel are present at the sides of the Río Grande de Loíza, Río Grande de Bayamón, and Río La Plata. Thickness is variable, but as much as 20 meters has been penetrated in the Bayamón and San Juan quadrangle areas, possibly as great as 100 meters at the sides of the Río Grande de Loíza .

Silica sands of Holocene to Pleistocene age consist of very pure quartz sand 99% silica but locally containing organic matter.

The deposits grade downward into compact, ferruginous sand,mapped as blanket deposits,having a thickness ranging from 1 to 4 meters. In Santurce it was named Santurce sand (Kaye 1959). The outcrop of the Santurce sand is generally a loose, very well-sorted, medium grain, almost pure sand. It grades downward into the Older Alluvium where the cohesive nature of the clay binder imparts a great,dry strength. Erratic variations in the density of sand occurs with depth.

Beach deposits consist of sand composed largely of fine quartz mixed with minor quantities of shell and volcanic rock fragments on beaches and abandoned beach ridges in the Carolina quadrangle area. Deposits are generally medium to course sand in other zones. Thickness varies from 1 to 5 meters but may reach more than 13 meters in the Luis Muñoz Marín Airport area (Kaye, 1959). Beach rock is commonly present in the intertidal zone due to sand cementation.

Eolianites are cemented dunes consisting of sand and clayey sand, friable to consolidated, crossbedded, calcareous, eolian sandstone composed of fine to course grains of shell fragments and quartz. The maximum thickness ranges from 20 to 30 meters.

Together with beach and eolianite deposits of Holocene age, swamp deposits dominate the northern portions of the study area. They consist of sandy muck and clayey sand generally underlaid by peat formed in mangrove swamps. The peat is very compressible, generally 10 meters thick. Peat is the weakest foundation soil in the area.

Artificial fill has been placed over swamps, sections of the San Juan Bay, and in valleys to provide foundation for housing and industrial development. Fill material generally consists of sand, limestone, and volcanic rock. More than one third of the bay shoreline has been filled or dredged, mostly after 1940.

Ground Shaking

Ground shaking is by far the most important earthquake induced geologic hazard in the metropolitan area of San Juan. It is caused by the sudden release of elastic strain energy stored in the rocks. This process (faulting) generates different waves that propagate from the rupture zone. Two classes of waves are generated: body and surface waves.

Body waves consist of compressional (P) and shear (S) waves. They traverse the Earth's interior with different velocities and motions. Surface waves are Love and Rayleigh waves that travel more slowly than body waves. Body waves are mainly high frequency vibrations that are likely to make low buildings resonate. Surface waves cause mainly low frequency vibrations more efficient in making tall buildings vibrate. When buildings cannot resist earthquake vibrations generated by these waves, damage occurs (Hays , 1981).

It has long been recognized that different locations at essentially the same epicentral distance experience large variations in the distribution of damage due to the influence of local geologic and soil conditions on ground motion. Soil conditions such as thickness, water content, physical properties of the unconsolidated material, bedrock topography, geometry of the unconsolidated deposits and underlying rock, among others, can modify the ground surface motions by changing the amplitude and frequency content of the motion. Amplification of ground motion in a period range that coincides with the natural period of vibration of the structure explains the distribution of damage (Hays , 1980). Shorter period waves oscillate in the same frequency range as lower buildings, affecting such structures close to the epicenter. Longer period waves, which oscillate in the same frequency range as taller buildings, travel farther and can affect such buildings at relatively great distances from the epicenter. This is a potentially serious hazard in the metropolitan

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area of San Juan because tall buildings can resonate with higher period waves generated by relatively distant earthquakes offshore.

Local soil conditions modify the seismic input by generating maximum accelerations at lower periods for stiff soils where short height structures are likely to suffer more damage. In soft soils maximum accelerations occur at higher periods where taller structures are subjected to the worst conditions.

In general, areas underlaid by thick deposits of uncompacted artificial fill, by soft, water saturated mud, or by unconsolidated stream sediment,shake longer and harder than areas underlaid by bedrock (Brown and Kockelman, 1983). During the October 11, 1918 earthquake, the La Playa sector of Ponce was more severely shaken than the higher part of the city. Humacao suffered far more than other towns in the same area because it was built upon the alluvium. The greatest damage was registered in Aguada and Añasco, both located on alluvial deposits, while Rincón, built on bedrock and closer to the epicenter than Añasco, suffered much less damage (Reid and Taber, 1919).

Three main deposits are mapped in terms of ground shaking hazard. The lowest hazard is assigned to rock outcrops, high terrace, alluvial fan, older alluvial, and blanket deposits. Rock outcrops include Cretaceous and Early Terciary volcanic and sedimentary rocks; Middle Terciary formations such as Cibao, Aymamon, Aguada and San Sebastian, and eolianites. The rest are semiconsolidated deposits of Pleistocene and Miocene age characterized by being stiff, hard, and compact. Depth ranges from less than 10 meters in Carolina to more than 100 meters in San Juan and less than 50 meters in Bayamón (Monroe, 1973, 1977; Pease and Monroe, 1977). Diagenesis has resulted in a material that behaves much like bedrock.

All zones of moderate to high ground shaking hazard include all alluvial deposits of Holocene age and some terrace deposits of Pleistocene age. The deposits are present in the floodplains of Río Bayamón, Río Piedras, and Río Grande de Loíza. In Carolina the sand, clay, and sandy clay beds are up to 100 meters thick. Beds of sand, clay, and sandy clay exceed 20 meters in San Juan and Bayamón. These zones are much more vulnerable to ground shaking than the "stiff" clays but are considered, in general, less vulnerable than artificial fills placed over swamp and lagoonal deposits.

Fill materials have been shown to behave very poorly during earthquakes (Munich Re,1973). Extensive filling of mangrove swamp (Fig. 15) with fill material ranging from rock and sand, to soft, black, mucky clays dredged from the bottom of San Juan Bay after 1940, have created potentially unstable conditions. Manmade fills consisting of materials ranging from silt to sandy gravel have failed during earthquakes due to liquefaction of the basal zone of the fills themselves or in natural foundation materials underlying the fills (Keefer, 1984). In fact, flow failures carried away large sections of the port facilities at Seward, Wittier and Valdez, Alaska during the 1964 Prince William Sound Earthquake. Ground shaking induced failures caused the sinking of Port Royal in Jamaica 1692. Although the conditions where these events took place are not exactly the same as those present in San Juan Bay, the possibility of ground failure of portions of the artificial fill surrounding the Bay during a large earthquake cannot be discarded. The presence of relatively deep fill materials over swamp deposits and very high water tables place these areas under a combined high ground failure and ground shaking hazard. Ground shaking damages result

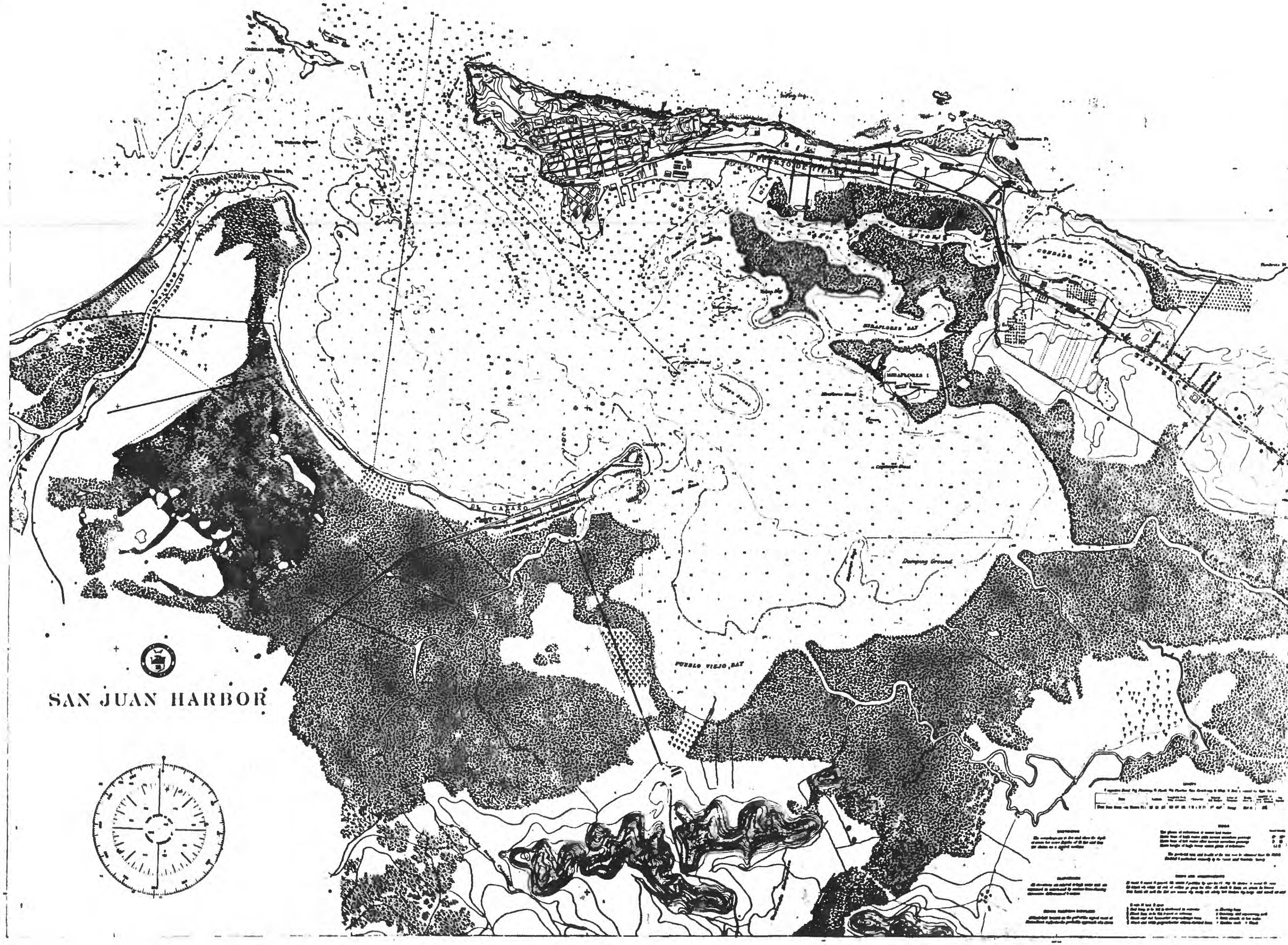

Figure 15. Map of San Juan Harbor, Porto Rico, (source: U.S. Department of Commerce, Washington, D.C., Scale: 1 inch = 1/3 statute mile).

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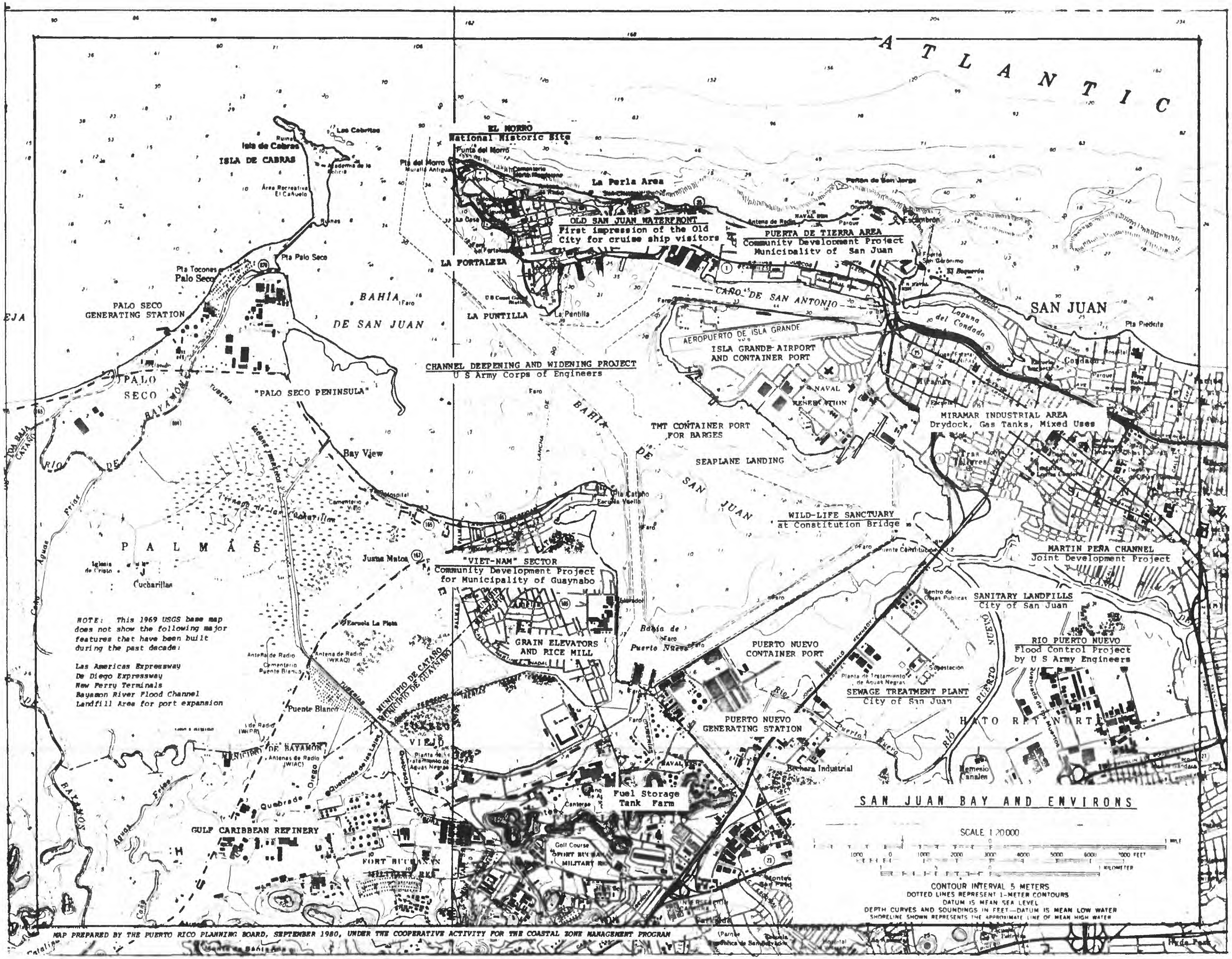
ATLANTIC
EL MORRO
National Historic Site
OLD SAN JUAN WATERFRONT
First impression of the Old City for cruise ship visitors
La Perla Area
PUERTA DE TIERRA AREA
Community Development Project
Municipality of San Juan
LA PORTALEZA
LA PUNTILLA
SAN JUAN
Laguna del Condado
CAÑO DE SAN ANTONIO
AEROPUERTO DE ISLA GRANDE
ISLA GRANDE AIRPORT AND CONTAINER PORT
NAVAL RESERVATION
ISLA DE CABRAS
Isla de Cabras
Pta Palo Seco
Palo Seco
PALO SECO GENERATING STATION
BAHIA DE SAN JUAN
"PALO SECO PENINSULA"
CHANNEL DEEPENING AND WIDENING PROJECT
U S Army Corps of Engineers
MIRAMAR INDUSTRIAL AREA
Drydock, Gas Tanks, Mixed Uses
TMT CONTAINER PORT FOR BARGES
SEAPLANE LANDING
WILD-LIFE SANCTUARY
at Constitution Bridge
MARTIN PEÑA CHANNEL
Joint Development Project
SANITARY LANDFILLS
City of San Juan
RIO PUERTO NUEVO
Flood Control Project
by U S Army Engineers
"VIET-NAM" SECTOR
Community Development Project
for Municipality of Guaynabo
GRAIN ELEVATORS AND RICE MILL
PUERTO NUEVO CONTAINER PORT
SEWAGE TREATMENT PLANT
City of San Juan
PUERTO NUEVO GENERATING STATION
HATO REY NORTE
Fuel Storage Tank Farm
GULF CARIBBEAN REFINERY
PALMAS
Bay View
Cucharillas
MUNICIPIO DE CATAÑO
MUNICIPIO DE GUAYNABO
MUNICIPIO DE BAYAMON
NOTE: This 1969 USGS base map does not show the following major features that have been built during the past decade:
Las Americas Expressway
De Diego Expressway
New Ferry Terminals
Bayamon River Flood Channel
Landfill Area for port expansion
SAN JUAN BAY AND ENVIRONS
SCALE 1:20 000
CONTOUR INTERVAL 5 METERS
DOTTED LINES REPRESENT 1-METER CONTOURS
DATUM IS MEAN SEA LEVEL
DEPTH CURVES AND SOUNDINGS IN FEET—DATUM IS MEAN LOW WATER
SHORELINE SHOWN REPRESENTS THE APPROXIMATE LINE OF MEAN HIGH WATER
MAP PREPARED BY THE PUERTO RICO PLANNING BOARD, SEPTEMBER 1980, UNDER THE COOPERATIVE ACTIVITY FOR THE COASTAL ZONE MANAGEMENT PROGRAM

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from the interaction of ground motion with the building structure. Ground motion characteristics are mainly determined by the depth of the focus, its magnitude, attenuation, and local ground response. The most important of these factors have been discussed earlier in this report. Building damageability depends mainly on the building ordinances and their effectiveness, design and construction technology, type of building structure, and location.

In Puerto Rico, building regulations containing lateral force provisions for earthquake went into effect in September, 1954. Prior to that date buildings were constructed using individual standards selected by each builder; but a building code alone is no guarantee of an adequate building performance during an earthquake. Other factors such as the experience of the designers, material quality, quality of workmanship, and supervision affect damageability. Steinbrugge (1962), during an inspection of several buildings in the metropolitan area of San Juan, found that in many buildings earthquake provisions and workmanship requirements were not effectively policed by the Puerto Rico Planning Board. Design errors and poor workmanship were commonly found even in the larger buildings.

Today, potentially serious deficiences are present in the actual building code. Leandro Rodríguez (1984) emphasizes that the present building code does not consider ductility, does not address soil structure interaction, does not consider the importance of the structure (for example the same design criteria are used for hospital and for a one-family house), and does not recommend earthquake resistant designs for underground lifeline structures. Thus, in spite of the building regulations, a significant number of structures in the metropolitan area are not likely to resist earthquake loadings adequately. Fortunately, the Seismic Committee of the Colegio de Ingenieros, Arquitectos y Agrimesores has submitted to the Puerto Rican Building Permits and Regulation Administration an updated proposal for the design of earthquake resistant structures in Puerto Rico.

Damage assesment of ground shaking hazard follows the procedures recommended by Rice (1983). Most of the information presented below originates from this source. The methodology considers only damage to buildings. Other facilities such as plants, dams, lifelines, etc., are outside its scope. Damage assesment is obtained by overlaying a building inventory map on the hazard map.

The structure response for different types of buildings, ground motion, and soil condition is based on past earthquake experience. The predicted damage is expressed as percent loss or damage ratio. This widely used parameter represents the ratio of the cost of repair to the replacement cost. For individual buildings, damage ratios beyond .5 are considered total losses. Since damage ratios of .3 already correspond to severe damage states, damage ratios typically vary from 0 to .3, increasingly rapidly to 1. The damage ratio for different building types are presented in figure 16.

The dominant type of building structure in the metropolitan area of San Juan is shear wall with seismic design (estructuras a base de muros de corte con diseño sísmico) . Damage ratios for other types of structures are shown in fig. 17. Areas of low ground shaking amplification (B-1) correspond to a MMl of VIII. In areas with moderate to very high ground motion amplification (B-2), damage ratios were raised .75 intensity (MMI). In areas with high ground motion amplification (B-3) damage ratios were raised 1.0 intensity (MMI). Damage ratios for ground shaking, liquefaction and landslides are shown in table 2 .

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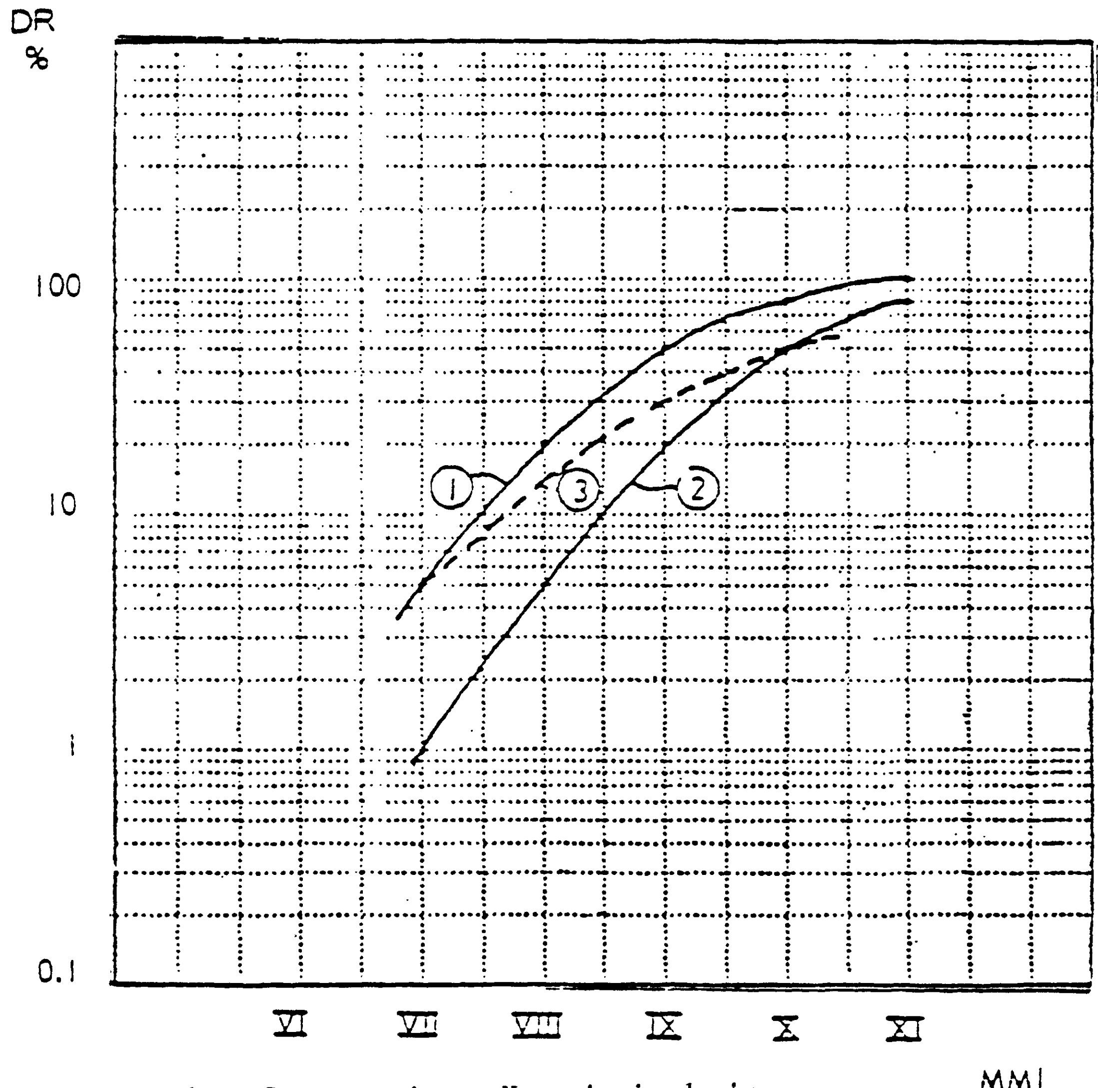

1 Modern Construction. No seismic design.
2 Modern Construction. Seismic design.
3 Average damage ratio used by Muchener Ruckversicherungs-Gesellschaft (verbally communicated to Sauter and Shah, 1978).

Figure 16. Average damageability for "modern construction" taken from Sauter and Shah, 1978; originally from "Guatemala 1976 Earthquake of the Carribbean Plant," Muchener Ruckversicherungs-Gesellschaft, Munich.

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TABLE 2

Generalized Damage Ratio Estimates

Hazard Zone	% Area	Damage Ratio
A-1	2	.35
	98	.05
A-2	5	.35
	95	.05
A-3	10	.35
	90	.07
B-1	100	.05
B-2	90	.15
	10	.35
B-3	20	.35
	80	.20
C-1	2	.10
	98	.05
C-2	5	.10
	95	.05
C-3	15	.10
	85	.05

Hazard zones are shown in the Earthquake-Induced Geologic Hazard Map included with this study.

Liquefaction

Although landslide is chiefly a hillside process, earthquakes can also cause ground failure in the lowland due to the process of liquefaction. When cohesionless water-saturated materials are subjected to earthquake vibrations, the tendency to compact is accompanied by an increase in pore water pressure in the soil due to load transfer from soil particles to pore water. Drainage can occur, but if restricted, pore water pressure can rise to an amount equal to the weight of the column of soil above the soil layer. Under this condition the soil may suffer great deformations and behave like a fluid rather than like a solid for a short period of time. Any structures, fills, and embankments located on liquifying soil will undergo deformations. These can be caused by lateral spreads, flow failures, and by the loss of bearing strength. In addition, ground settlement and sand boils can occur. The settlement of sand is principally caused by the horizontal shear component of motion. Lee and Albasia (1974) found that vertical settlements from drainage effects may be as much as 3% of the height of the affected soil layer. If sands are saturated, ground subsidence might be expected from soil compaction and water drainage at stresses less than required to induce complete liquefaction. The volumetric settlements from pore water pressure lower than that causing liquefaction are generally less than 1%.

Geologic conditions favoring liquefaction are: 1) a potentially liquefiable bed or lense of porous, well-sorted sand, 2) water saturation of intergranular pore spaces in the bed or lens , 3) confinement of pore water by impermeable layers above and below the liquefiable bed, and 4) proximity of the liquefiable bed to the surface (50 feet or less).

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Liquefaction occurs mainly where sands and silts have been deposited during the last 10,000 years and where ground water is within 10 meters of the surface. Generally, the younger and looser the sediment and the higher the water table, the more susceptible the soil is to liquefaction. In Puerto Rico, liquefaction was observed in the lowlands of Rincón during the October 11, 1918 earthquake. Water, bringing up sand, issued from cracks. The same phenomenum was observed in Añasco, but here the water brought up black sand. Liquefaction was reported in sandy, saturated alluvial materials in areas where the earthquake intensity (Rossi-Forel) was greater than VII (Reid and Taber, 1918). Massive water drainage from alluvial soils increased stream discharge for days after the earthquake.

Three major factors are conducive to liquefaction: ground shaking, a shallow water table, and sandy materials. In terms of ground shaking, the selected hazard level of MMI VIII is capable of generating cyclic stresses strong enough to cause liquefaction in the study area. The predominant minimum intensity for coherent slides and lateral spreads and flows is MMI VII. The lowest intensity reported is MMI V (Keefer, 1984). Thus, the study area will experience an MMI of 1 to 2 above the predominant minimum liquefaction threshold. Shallow water tables and sand deposits coincide in river channels, dunes, beach deposits. deltas, silica sand deposits, flood plains, and other topographic lowlands. In these areas the water table is usually less than two meters deep and rarely exceeds five meters.

Areas susceptible to liquefaction are mapped according to geomorphic setting, landforms, types and age of geologic deposits,and water table depth These factors are used to estimate areas of high, moderate, and low susceptibility. In large scale mapping, more refined methods based on boring

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logs and standard penetration tests (techniques developed by Seed and Idriss, 1971, and Seed, 1979) may be used to determine liquefaction potential. Included in areas of moderate to high susceptibility are Holocene beach deposits composed of sand consisting of grains of quartz, volcanic rock and shells. Thickness ranges from one to five meters. A second area is found in the Carolina quadrangle where fine to medium sands are present on beaches, coastal dunes, and abandoned beach ridges. It is usually not thicker than ten meters, and the water table is less than two meters. Areas of high susceptibility include the very fine and loose sands of Cangrejos Arriba with a thickness ranging from one to four meters and a high ground water table. Within these zones the ground failure potential is high in areas lacking lateral confinement, differentially loaded, loose sand deposits, or gentle slopes. Areas of low to moderate susceptibility include the older deposits of Holocene-Pleistocene age composed of almost pure silica sands derived from ferruginous sand by leaching. Loose sands are present on the surface. The degree of compaction increases irregularly with depth. Kaye (1958) noted the following features: 1) Great uniformity of sorting of the sand material 2) Lack of carbonate cementing material 3) High dry strength, imparted by clay, that acts as a binder 4) Erratic variation in the density of the sand with depth. Zones of low susceptibility are older Pleistocene silica sand deposits in the Bayamón quadrangle. They are one to four meters thick, and the water tables are generally deeper than in younger deposits. The liquefaction potential is not exclusive of beach and silica sand deposits, but a very high potential is locally present in river channels, deltas, uncompacted fills, and lagoonal and

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flood plain deposits less than 500 years old. Due to map scale limitations these areas are not mapped independently. Swamp and lagoonal deposits (hydraquents) are extensive in the study area and were mapped separately as zones with high liquefaction potential. Recent flood plain deposits are vulnerable where the alluvium is composed of cohesionless materials such as silt, silty sand, or fine grained sand. Most of the alluvium in the study area is composed of clay, sandy clay, and sand. Liquefaction induced flow failures and lateral spreading toward river channels are likely to occur where saturated sand lenses are present. Lateral spreading of flood plain deposits toward river channels destroyed more than 200 bridges during the 1964 Alaska earthquake. They are particularly destructive to pipelines and water mains, a factor which impeded the effort to fight the fire that ignited during the San Francisco earthquake (Hays, 1981). During the 1918 earthquake the Aguadilla water supply pipe over Rio Culebrinas was ruptured by compression when the concrete piers supporting the pipe moved more than 2 meters towards each other across the stream (Reid and Taber, 1918).

Liquefaction damage assesment requires the mapping of potentially susceptible sedimentary materials (table 3), the estimation of the percent area affected by liquefaciton, and the estimation of the damage ratio. Liquefaction mapping criteria have been presented above. The estimation of the percent area affected by liquefaction is done by adapting the procedures proposed by Rice (1983) based on the topographic and geologic conditions, soil profile characteristics, level of earthquake shaking, and liquefaction potential assesment using Seed's (1969) criterium. The resercher's subjective judgement is critical in the evaluation, specially when detailed data is not available.

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The percentage of area affected by liquefaction and the corresponding damage ratio for a magnitude 6 earthquake is shown in figure 17. The selected earthquake hazard level (MMI VIII) approximately corresponds to a peak ground acceleration of .2g. and an earthquake Richter Magnitude 6 (fig. 19). The percent area affected by liquefaction is 17 percent and the damage ratio is .35 according to fig. 18. Because portions of the areas mapped under moderate to high potential have higher blow counts (for example, indurated sand and beach rock) the percent area affected by liquefaction is overestimated. A conservative estimate of the percent area affected by liquefaction based on this researcher's judgement assigns 10 percent to areas of moderate to high susceptibility, and 2 percent to areas mapped under low susceptibility. These estimates can be improved by examining specific site profile characteristics and Standard Penetration Test results throughout potentially liquefiable deposits.

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TABLE ~~2~~

Estimated Susceptibility of Sedimentary deposits to Liquefaction During Strong Seismic Shaking

From Youd and Perkins 1978

Type of Deposit (1)	General Distribution of Cohesionless Sediments in Deposits (2)	Likelihood That Cohesionless Sediments, When Saturated, Would Be Susceptible to Liquefaction (by Age of Deposit)			
		<500 yr (3)	Holocene (4)	Pleistocene (5)	Pre-Pleistocene (6)
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	-	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low

(continued)

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Type of Deposit (1)	General Distribution of Cohesionless Sediments in Deposits (2)	Likelihood That Cohesionless Sediments, When Saturated, Would Be Susceptible to Liquefaction (by Age of Deposit)			
		<500 yr (3)	Holocene (4)	Pleistocene (5)	Pre-Pleistocene (6)
(a) Continental Deposits (cont'd)					
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach					
• High wave energy	Widespread	Moderate	Low	Very low	Very low
• Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial					
Uncompacted fill	Variable	Very high	-	-	-
Compacted fill	Variable	Low	-	-	-

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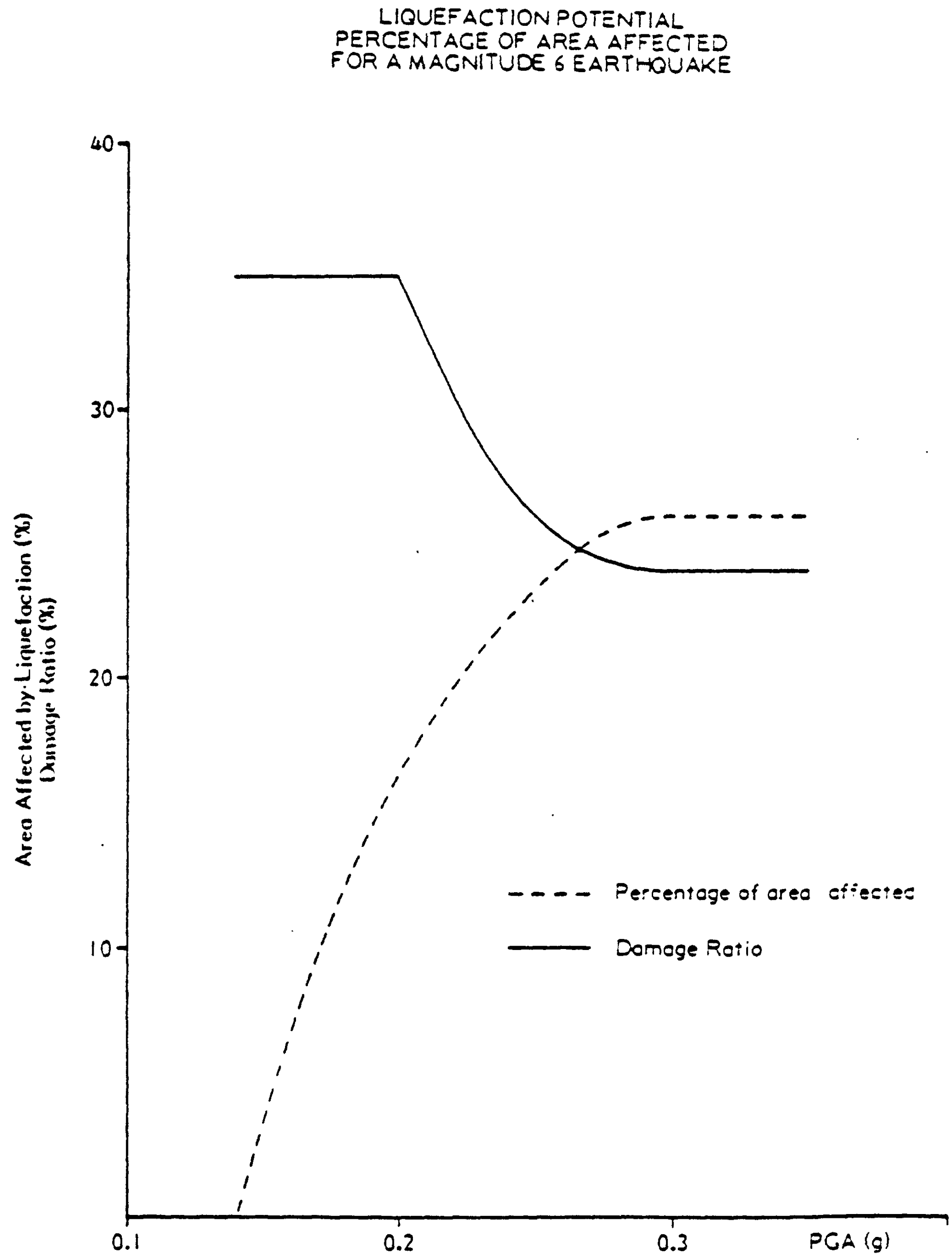

Figure 17

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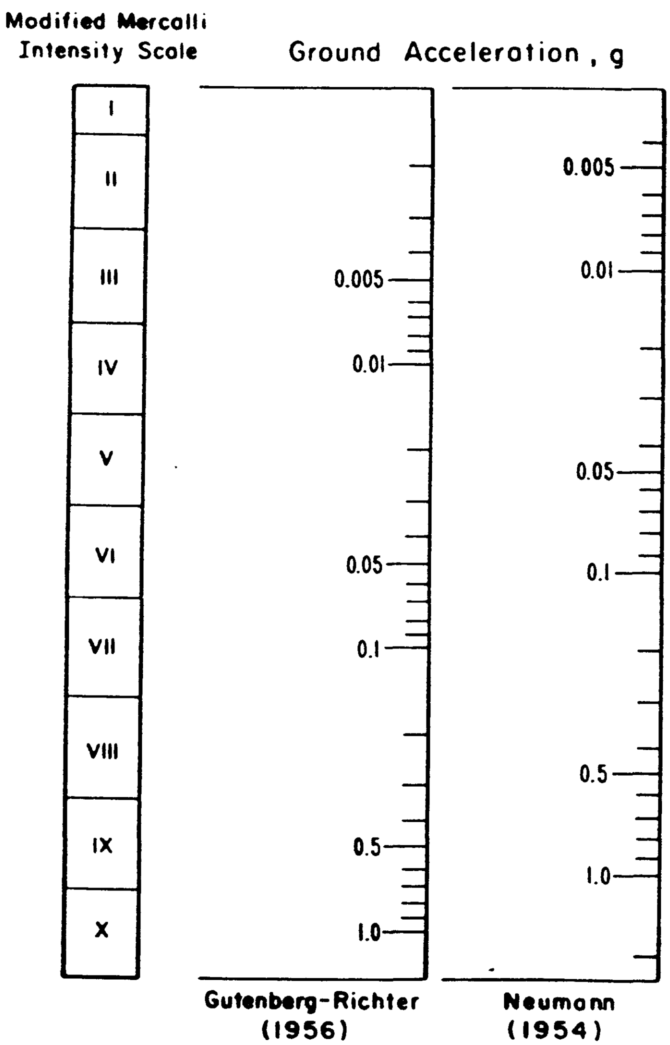

Figure 19. Intensity and acceleration relations proposed by Neumann, and Gutenberg and Richter (From Hays, 1980).

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Landslides

The term landslide, as used in this report, refers to all types of slope movements including falls, flows, slides, and topples.

Two main features that control earthquake induced landslides are slope inclination and the types and characteristics of the geologic materials beneath the slope. Ground motion can trigger landslides when slopes are subjected to repeated loadings consisting of irregular pulses that weaken and eventually loosen rock and soil materials forcing them down the slope. Keefer (1984) studied the relationship between earthquake magnitude and areas affected by landslides, and the epicentral distance and Modified Mercalli Intensity at which different landslides occur. Areas affected by landslides show a strong correlation with magnitude. Generally, landslides are caused by events greater than M. 4.0. The selected hazard level can trigger landslides over an area up to 100,000 km^2. This is extensive enough to cover the whole island of Puerto Rico, assuming the epicenter of the selected hazard level is along the southern wall of the Puerto Rico Trench. In addition, the epicentral distance from the study area is closer than the minimum distance of 200 kilometers required to experience all types of ground failure. At a given epicentral distance, different areas experience different intensities. The selected hazard level will produce an MM intensity of VIII to IX (deep alluvium), a value up to 2 intensities above the predominant minimum seismic shaking intensities required to trigger disrupted slides and falls (MMI VI) and coherent slides, lateral

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spreads, and flows (MMI VII). Thus, ground motion in San Juan, given the areal, epicentral, and intensity characteristics of the selected hazard level, is strong enough to cause landslides, especially in steep slope areas and near weak geologic materials. The mapping of areas susceptible to landsliding takes into consideration slope inclination as a primary factor affecting slope stability. In general, steep slope areas are chief sites of instability mainly through their control of the downslope component of the weight of slope material. However, the degree of stability depends considerably on the geologic material underlying the slope. Granular non-organic soils with little cohesion and low frictional strength are the most susceptible to failure. In addition, highly fractured or jointed rock, or rock which displays any other type of discontinuity, especially if planes are open, is susceptible to failure (Rice, 1983). Degree of susceptibility to landslides is mapped as high, moderate to high and low.

Zones of high susceptibility in the study area include those areas where geologic formations are characterized by a high landslide incidence due to steep slopes in vulnerable material, and the presence of a weak geologic stata below more resistant ones. Consequently, the Cibao - Aguada and San Sebastian terciary formations, and the Mucarabones sand are areas of high susceptibility. The first two formations show a high incidence of landslides extending along a considerable portion of their outcrop from Aguadilla to the southwestern portion of the San Juan metropotilitan area. The geologic contact along steep scarpments where the Aguada formation rests on clay and sandy clay of the the Cibao formation is potentially unstable. In similar humid, tropical, geomorphic environments earthquakes have triggered rotational slumps involving failure of incompetent, plastic strata beneath limestone (Simonett, 1967). Large landslides occur where the thick clayey beds of the San Sebastian formation beneath the Lares limestone are exposed along a scarpment that extends from Corozal to the west coast (Monroe, 1964). Although the Lares limestone is not present in the study area due to its eastern grading into the Mucarabones sand, steep portions of the clayey and pebbly San Sebastian and the Mucarabones sand are mapped as highly susceptible area.

Areas mapped as moderate to high susceptibility are located mainly at the southern portion of the San Juan metropolitan area where the interior mountainous uplands begin. Slope inclinations range from 12 to 32 degrees but do not show any significant incidence of landsliding except along steep-sided excavations, such as roadcuts (Molinelli, 1983). Soils are mostly Inceptisols, characterized by shallow depth (40 cm.) over slightly weathered bedrock, and Ultisols, moderately deep to deep soils (1.5 m. deep) (Soil Survey, San Juan). When dry, the high clay content of these residual soils imparts a high cohesive stability to the slopes, greatly reducing their vulnerability to the probable earthquake. On the other hand, protracted periods of rain can saturate the soils, increasing the pore water pressure, reducing the shear strength, and increasing the shear stress with the weight of the water. Under these conditions, the probable earthquake can trigger a large amount of debris, earth flow, and slides. In humid, tropical, geomorphic environments similar to those mapped as moderate susceptibility, the percentage area that has failed during an earthquake of similar magnitude as the probable earthquake ranges from 25 to 40 percent (Simonett, 1967; Pain, 1972).

Areas mapped as low susceptibility include nearly flat slope zones (less than 10 degrees inclination) and very stable rock outcrops. Included in this mapping unit are the low relief portions of the San Sebastian formation and Rio Piedras siltstone, the Guaynabo formation, the Guaracanal Andesite, and the Frailes formation. Most of these areas are presently urbanized, a process that has further leveled the topography. There is little likelihood of significant downslope movement, except along excavations. The rock outcrops included within this unit are the Aymamón and Aguada limestone formations and eolianites. In spite of steep slopes, limestones, along with other formations of Terciary age, are considered the most stable rock in Puerto Rico (Monroe, 1979). Case hardening by solution and immediate redeposition in situ stabilize the slope (Monroe, 1976). Eolianites are very stable except where undermining has taken place due to mechanical and chemical weathering associated with wave action.

Not all slopes with landslide potential will actually fail at the selected hazard level. To estimate the expected percentage area of slope failure, criteria that reflect engineering judgement based on geological data and past earthquake experience (Rice, 1983) are incorporated. A conservative estimate of percentage area of failure assigns a value of 2 to 15 percent to areas of low, moderate, and high susceptibility (Fig.20) These values can more than double if the earthquake occurs after a protacted period of rain when the shear strength of the soil is lower. Landslide damage assessment assumes that for a given landslide potential, the percentage of area affected is the same as the percentage of buildings that suffer landslide induced damage. In addition, damage ratios (percent loss) are shifted arbitrarily by .5 intensity (Rice 1983).

LANDSLIDE POTENTIAL
PERCENTAGE OF AREA AFFECTED BY
LANDSLIDE VERSUS PGA
FOR THREE LANDSLIDE POTENTIALS

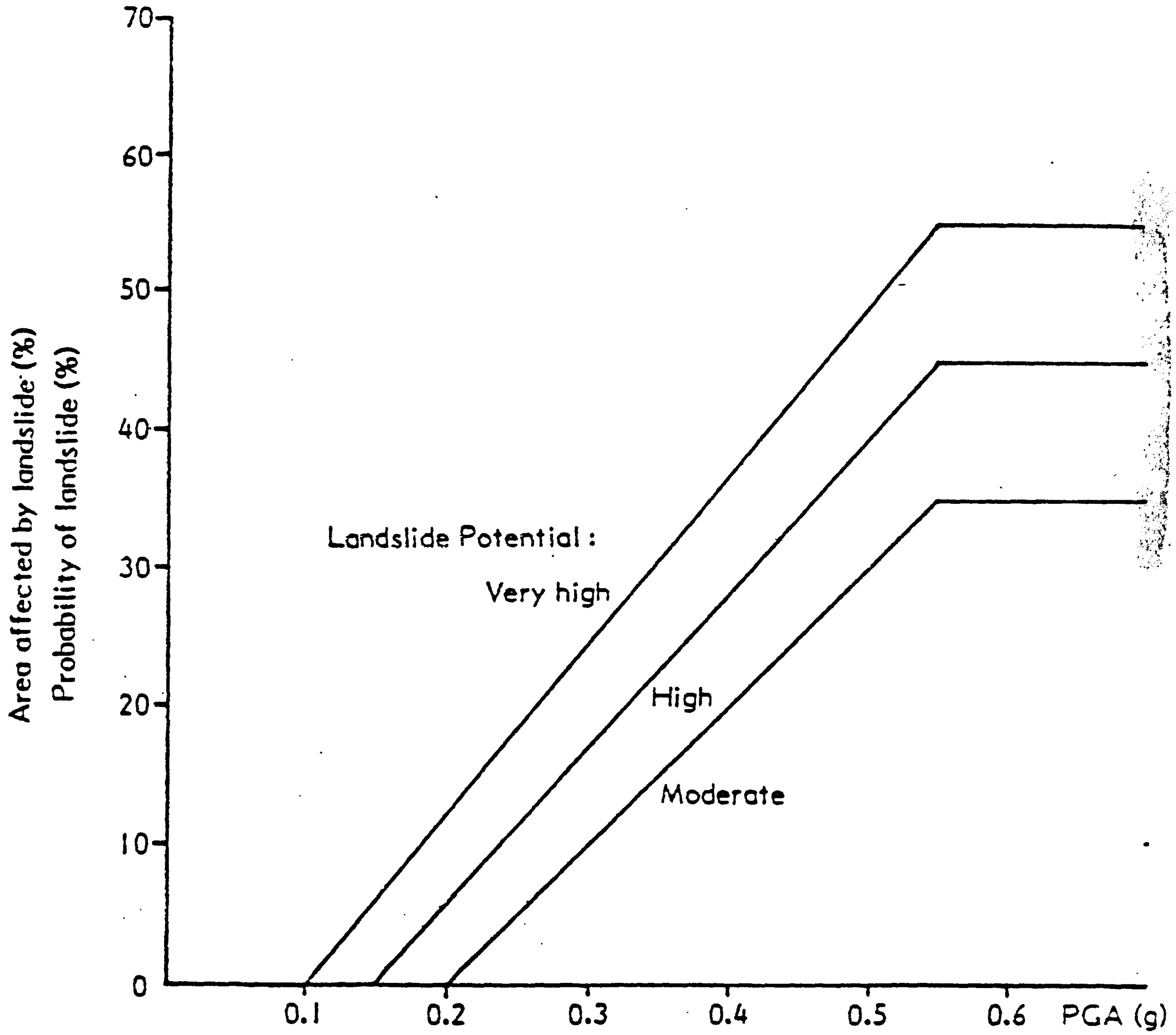

Figure 19

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COMMONWEALTH OF PUERTO RICO
DEPARTMENT OF NATURAL RESOURCES

PLANNING RESOURCES AREA

GENERALIZED EARTHQUAKE INDUCED GEOLOGIC HAZARDS MAP FOR THE SAN JUAN METROPOLITAN AREA

	GROUND MOTION AMPLIFICATION	LIQUEFACTION POTENTIAL	GROUND FAILURE POTENTIAL
A - I	NOT SIGNIFICANT	LOW	VERY LOW
A - 2	NOT SIGNIFICANT	LOW TO MODERATE	LOW
A - 3	NOT SIGNIFICANT-TO LOW	MODERATE TO HIGH	HIGH-WHERE THE MATERIALS ARE NOT LATERALLY CONFINED AND MODERATELY SLOPING
A - 3 - S	HIGH	HIGH-IN SAND COVERED LAGOONAL DEPOSITS	HIGH-IN SAND COVERED LAGOONAL DEPOSITS
B - I	NOT SIGNIFICANT	NONE	VERY LOW
B - 2	MODERATE TO VERY HIGH	HIGH-SPECIALLY WHERE THE MATERIALS ARE NOT LATERALLY CONFINED	HIGH-ALONG RIVER BANKS SLUMP, FLOWS AND LATERAL SPREADS
B - 3	HIGH	HIGH-SPECIALLY IN THE LOOSE SANDS LAGOONAL DEPOSITS	HIGH-SLUMPS-FLOWS AND LATERAL SPREADS
C - I	NOT SIGNIFICANT	NONE	LOW
C - 2	NOT SIGNIFICANT	NONE	MODERATE TO HIGH
C - 3	NOT SIGNIFICANT	NONE	HIGH

SCALE 1: 40,000

0 .5 1 KM

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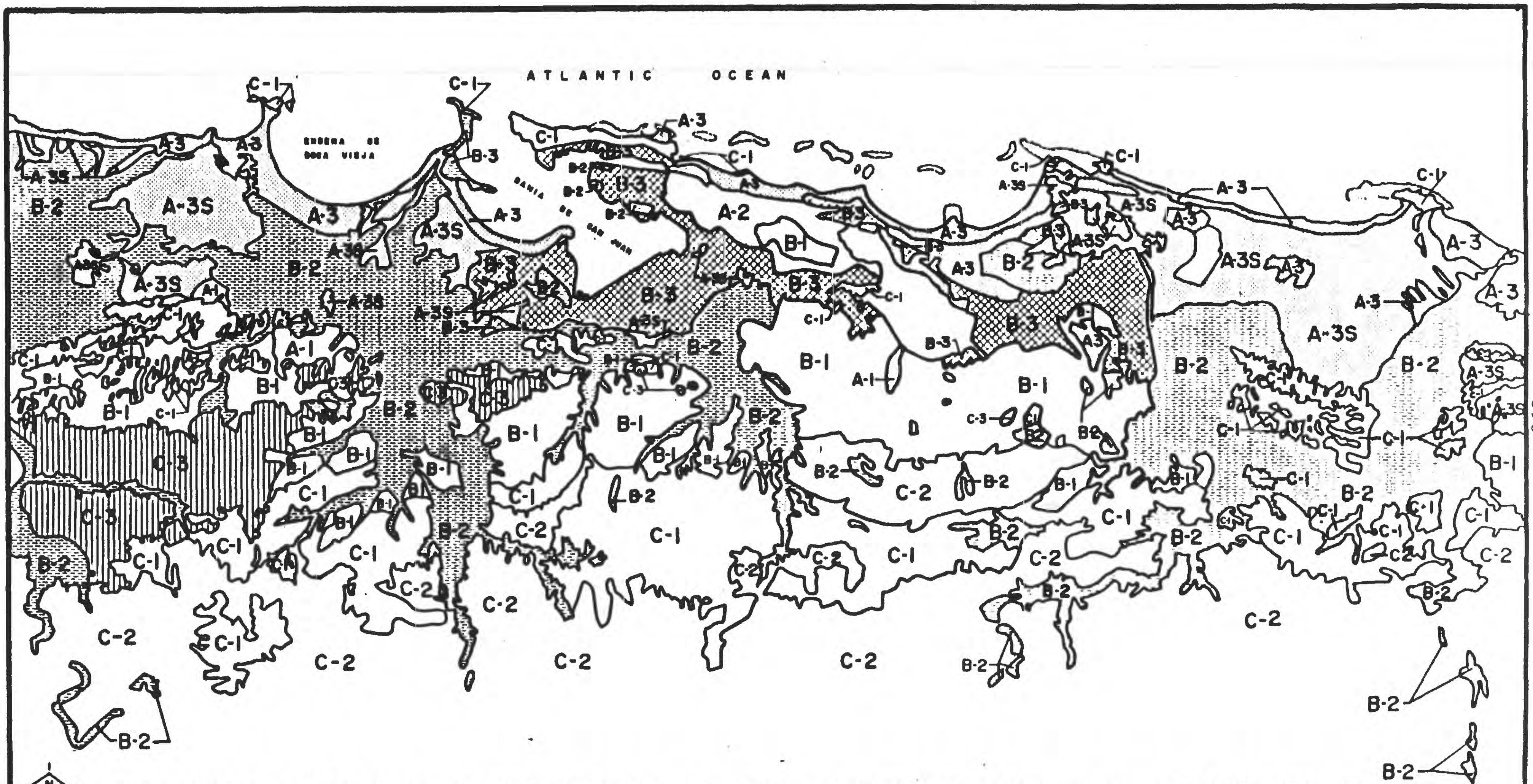

GENERALIZED EARTHQUAKE INDUCED GEOLOGIC HAZARDS MAP FOR THE SAN JUAN METROPOLITAN AREA

CONSULTANT: JOSE MOLINELLI FREYTES Ph.D. DWNG by VICTOR DAVILA OSORIO

Summary and Conclusions

The tectonic setting and regional seismicity of the northeastern Caribbean expose the island of Puerto Rico to a high seismic hazard. Large magnitude events in 1918 (est. magnitude 7.5), 1867 (est. magnitude 7.5 - 7.75), and 1787 (est. magnitude 8.-8.25) caused hundred of deaths and millions of dollars in material losses. Similar events will occur in the future. Off-shore faults in the Puerto Rico Trench, Mona Passage-Mona Canyon area, Anegada Passage, and the Muertos Trough are the most important potential earthquake sources in the Puerto Rico area. The Puerto Rico Trench, approximately 60 km. north of the metropolitan area of San Juan, poses the greatest hazard to the study area due to its proximity and high seismic potential (est. magnitude 8.8.25). On the basis of earthquake magnitude and intensity recurrence, regional attenuation and this researcher's judgement, the selected earthquake hazard level (most probable earthquake) for the risk analysis corresponds to a Modified Mercalli intensity VIII. This value is used as the basis for damage estimation.

The geology and geomorphology of the study area were defined as a preliminary step to mapping earthquake-induced geologic hazards. Three hazards were defined for the study area; ground shaking, landslides, and liquefaction. A map depicting hazard zones was prepared showing three levels of susceptibily for each hazard. Damage ratio was estimated for each zone adapting the procedures recommended by the Rice Center for earthquake risk analysis. The most important geologic hazards in the metropolitan area of San Juan are ground shaking, liquefaction and landslides. The analysis concludes that the most vulnerable areas are the

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artificial fills placed over swamp deposits around San Juan Bay, Caño Martín Peña and Laguna San José and the alluvial deposits in the floodplains of Río Grande de Loíza, Río Piedras and Río Bayamón. Both areas are exposed to a high ground shaking and ground failure hazard. Located in these zones are important lifelines such as the Bahía de Puerto Nuevo thermoelectric plant, transmission lines, electric energy substations, water treatment plants, pumping stations, water mains, docks, airport facilities and vital expressways that link the capital with the rest of the Island.

Moderate to high liquefaction potential is present in the alluvial deposits of the floodplains of Río Grande de Loíza, Río Piedras and Río Bayamón and in the loose saturated sands near the coasts. Located in these zones are a large number of high rises and housing units, airport facilities, roads, water mains, pumping stations, and other lifelines.

Moderate to high landslide potential is present in the southern portion of the study area. Landslide damage potential in this zone varies with the antecedent moisture conditions of the hillslopes. An earthquake after a protracted period of rains can severely affect lifelines specially roads, where slope excavations, overloading, removal of lateral support, and other similar situations cause potentially unstable slope conditions.

It is recommended that earthquake mitigation strategies focus on high risk zones on the artificial fills surrounding the Bay and lagoons, the floodplains of Río Grande de Loíza, Río Bayamón and Río Piedras, and localized zones near the coast characterized by a moderate to high liquefaction potential. Site specific geotechnical studies should be conducted in areas of greater risk in order to assess the specific vulnerability.

Puerto Rico must prepare for a big earthquake. A significant portion of the residential, commercial and transportation infrastructure are located in hazardous zones. Today the potential damage that will be created by a large earthquake event is greater than ever before. This study is a step in the efforts to prepare the Island for such an event.

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STATUS OF PUERTO RICO BUILDING CODE
By

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University of Puerto Rico
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INTRODUCTION

Puerto Rico is earthquake country. For thousands of years earthquakes have been relatively frequent in occurrence in this region. The last destructive earthquake occurred on October 11, 1918. It took 116 lives and caused $4 million in damages. Most buildings were made of unreinforced masonry which accounted for a large number of failures, especially on the west coast.

Building regulations were enacted in 1956 as the result of Act 168 of May 4, 1949, which provided the adoption of such regulations. Seismic provisions at the time still stand to this date.

Part IV of the Building Regulations provides for . . . "such minimum requirements as are necessary to insure that the buildings will be designed to resist stresses due to horizontal forces caused by hurricane winds and by earthquakes." Only once, 1968, has the Puerto Rican Building Regulations been amended as the result of public hearings on May 20, 27, and July 3, 1966.

SEISMIC PROVISIONS FOR BUILDINGS

At present, according to the Code, . . . "All buildings shall be designed and constructed to stand stresses produced by lateral forces at the level of the roof and each floor, as well as at ground level, resulting from Seismic Motion."

The minimum earthquake forces are calculated by formula (a) below or as the results of tests on scale models.

$$V + K\ C\ W \ldots\ldots\ldots(a)$$

V = Base Shear

K = Coefficient dependent on the structural system used

$$C = \frac{0.225}{T^{1/3}} \ldots\ldots\ldots(b)$$

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$C = 0.05$ for one or two story buildings

W = Weight of the structure

$$T = \frac{0.05H}{D^{1/2}} \quad \text{........(c)}$$

H = Building height

D = Depth of building (perpendicular to lateral load)

The distribution of the total horizontal force is done according to formula (d) below. When $\frac{H}{D}$ is larger than five (5), a concentrated load equal to 10% of V is put at the top and the differences is distributed following formula (d).

$$F_x = \frac{V\, W_x h_x}{\sum_{i=1}^{n} W_i h_i} \quad \text{............(d)}$$

PROPOSED CHANGES

During three years, 1980 to 1983, a commission appointed by the President of the Institute of Engineers drafted a set of changes directed at improving the seismic design provisions of the Puerto Rico Building Regulations. The draft is still at rest at the Regulations and Permits Administration (ARPE by its Spanish name).

Engineers in private practice in Puerto Rico are aware of loop holes in present Building Regulations and provide in their designs the needed seismic load carrying capacity to modern buildings. This has not eliminated possible discrepancies in the designs of important buildings. There are cases where contractors, one known to this author, include by reference the present Building Regulations in their contracts resulting in adequate designs as per our present knowledge of seismic behavior of structures.

The proposed changes to the seismic design provisions of the Puerto Rico Building Regulations are based on the Uniform Building Code adapted to the Island.

Earthquake forces must be calculated by:

$V = Z\ I\ K\ C\ S\ W$..........(a')

$Z = 0.6$, Seismic Zone Coefficient for Puerto Rico

I = Occupancy Importance Factor; Min. = 1.0, Max. = 1.5

$C = \frac{1}{15T}$ for $T \leq 1$ Sec. ....(b')

$C = \frac{1}{15T^{2/3}}$ for $T > 1$ Sec.

value of C not to exceed 0.10

$T = 0.35\ h_n^{3/4}$ for steel frames

$T = 0.025\ h_n^{3/4}$ for concrete frames ....(c')

$T = \frac{0.05\ h_n}{D^{1/2}}$ for other buildings

S = Soil interaction factor which should not be less than 1.0

In the absence of a soil investigations S should be taken as 1.5.

*The Product CS need not exceed 0.14.

In the structures with a fundamental period of vibration in excess of 0.7 sec. a concentrated load equivalent to $F = 0.07TV$ is added on the tip of the structure. This need not to exceed 0.25 V. The difference in total lateral forces is to be distributed to each floor level.

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The proposed revisions also includes requirements for the P-Delta effects. Whenever the incremental factors, θ, exceed 0.10, the story drift, resisting moments, and shears should be increased correspondingly.

$$\theta = \frac{P_x D}{V_x h_{sx}}$$

CONCLUSION

The long awaited revisions to the seismic provisions of the Puerto Rico Building Regulations are badly needed. To keep the usefulness of many buildings after a major earthquake hits the Island, it is necessary to assess the risks of those designed under the present Regulations and bring them to meet the new proposed standards which are consistent with life and property preservation.

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3. Catalogue of earthquakes felt in Puerto Rico, 1772-1970.

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DRAFT PLANS FOR MAPPING OF GEOLOGIC HAZARDS TO MEET THE NEEDS OF LAND USE AND EMERGENCY RESPONSE PLANNING

by

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Puerto Rico Department of Natural Resources
San Juan, Puerto Rico

FOREWARD

These draft plans and recommendations were developed by the participants of the workshop on "Geologic Hazards in Puerto Rico." They are intended to serve as a guide for public officials, scientists, engineers, social scientists, and emergency managers. Representatives of these disciplines can use the plans and recommendations in several ways: 1) to evaluate their current research, mitigation, response and recovery programs, 2) to devise new programs and plans and, 3) to create a seismic safety policy in Puerto Rico.

Dr. William MacCann of Lamont Doherty Geological Observatory of Columbia University, Dr. Earl Brabb of USGS, and Dr. Alejandro Soto of the University of Puerto Rico (Mayaguez Campus) provided special assistance in the formulation of the draft plan and recommendations. The membership of the discussion group included:

Luis Biaggi	Puerto Rico Planning Board
Heriberto Capella Acevedo	Department of Education of Puerto Rico
Orlando Cordero	University of Puerto Rico
Benicio Correa Matos	Civil Defense of Bayamon
Anselmo De Portu	Department of Natural Resources (Recorder)
Juan A. Deliz	U.S. Geological Survey--San Juan
Bernardo Deschapelles	University of Puerto Rico
Samuel Diaz Santiago	Engineering Consultant
Rafael Esteva	Puerto Rico Planning Board
Ellis FebresSiliva	Puerto Rico Civil Defense
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Jose E. Hernandez	Center for Energy and Environment Research
Gilberto Isaac Valdes	Soil Engineering Office, Highway Authority
Rafael Jimenez Perez	Universty of Puerto Rico
Juan Luis Trias	U.S. Geological Survey, San Juan
Jussef M. Galib	Puerto Rico Chamber of Commerce
Jose Martinez Cruzado	University of Puerto Rico
Bill McCann	Lamont-Doherty Geological Observatory at Columbia University
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Andres Paiva Liendo	Soil Engineering Office, Highway Authority
Cesar Pujols	Soil Engineering Office, Highway Authority

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Nestor Ramirez	Public Building Authority
Leandro Rodriguez	University of Puerto Rico
Jesus Rodriguez	San Juan Department of Natural Resources
Emilio Rodriguez Esteban	Department of Commerce
Pedro Salicrup Rivera	Soil Engineering Office, Highway Authority
Ramon Santiago	U.S. Department of Agriculture
Alejandro E. Soto	University of Puerto Rico
Rodolfo Tardy Garcia	University of Puerto Rico
Thomas Thornton	U.S. Corps of Engineers
Heriberto Torres	U.S. Geological Survey--San Juan
Antonio Zaragoza Rodriguez	University of Puerto Rico

HIGHLIGHTS OF THE DISCUSSION

The group noted that a large amount of information published by USGS is presently available and can be used in the evaluation of geologic hazards. Specific comments included:

1) Most of the USGS Geologic Maps of Puerto Rico have been published or are on "open file." (Available from USGS Libraries in Reston, Virginia; Denver, Colorado; and Menlo Park, California.)

2) There is an urgent need to upgrade the information on landslides. Detailed maps at a scale 1:20,000 are needed. This effort could be accomplished by USGS and the University of Puerto Rico.

3) Fault inventory (land and sea) is incomplete and must be completed. This effort could be undertaken by USGS and Lamont-Doherty Geological Observatory.

4) Mapping the depth of the bedrock is critical and should be given a high priority because fundamental knowledge of the following factors are not well known in Puerto Rico: a) the relationship between the natural period of vibration of a specific structure and the dominant period of the soil under its foundation, and b) the effect of the local soil conditions underlying the building on the frequency content and duration of the vibrations induced in the building.

5) Identification of areas subject to liquefaction is a high priority task. Most of Puerto Rico's urban development is located in coastal areas with a relatively high water table. That is where major infrastructures are located, including airport and port facilities.

6) A preliminary study on liquefaction potential has been funded by the USGS for the San Juan Metropolitan Area and is due before the end of 1984.

7) Lamont-Doherty Geologic Observatory has specific interest in assisting Puerto Rico to improve its seismic information, especially with respect to: a) historical seismicity map, b) isoseismal maps, and c) upgrading its seismic network.

RECOMMENDED ACTIONS

Considering the availability of information, the discussion group recommended that the following actions be given high priority.

1) Produce a probabilistic map of the ground-shaking hazard for an exposure time of 50 years. Such a map would be consistent with the zoning map in Applied Technology Council's model building codes for other parts of the United States.

2) Update landslide inventory and identify areas that are potentially susceptible to landslides

3) Identify geologic hazards as well as other natural and for man made hazards in urban areas, quantifying the frequency of occurrence and the severity of effects.

4) Request the Puerto Rican Regulations and Permits Administration to assign a high priority to the review and amendment of the Puerto Rican building code with respect to seismic design and construction standards. The recommendations prepared by the local College of Engineers and Surveyors should be promptly evaluated and incorporated into the Building Code.

5) The Government of Puerto Rico should set forth its planning needs for mapping geologic hazards and should determine the extent to which the private sector is willing to provide part of the needed financial support.

6) A formal application should be submitted to the Federal Government (for example to FEMA or the USGS) for technical and financial aid for preparedness planning and hazard mitigation measures.

The activities identified above are designed to upgrade and refine knowledge of the spatial distribution of potential geologic hazards in urban areas. Presently a uniform standard is applied to seismic design, regardless of location on the Island. Accomplishment of these activities will ensure the achievement of the following rule of the "P's" and "S's:" "Proper preparedness planning seeks site specific surveys."

By the time of the next workshop on "Geological Hazards in Puerto Rico" (tentatively scheduled for April 1985) we believe that major accomplishments will have taken place and our knowledge of Puerto Rico's geologic hazards will be greatly enhanced.

DRAFT PLANS TO INCREASE COMMUNITY PREPAREDNESS

by

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and
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FOREWARD

These draft plans and recommendations were developed by the participants of the workshop on "Geologic Hazards in Puerto Rico." They are intended to serve as a guide for public officials, scientists, engineers, social scientists, and emergency managers. Representatives of these disciplines can use the plans and recommendations in several ways: 1) to evaluate their current research, mitigation, response and recovery programs, 2) to devise new programs and plans and, 3) to create a seismic safety policy in Puerto Rico.

The membership of the discussion group included:

Joyce B. Bagwell	Baptist College at Charleston
Andres Castillo Ortiz	Centro Unido de Detallistas de Puerto Rico
Jose A. Colon	National Weather Service Forecast Office
Charles Culver	National Bureau of Standards
Luz D. Diaz Rozado	State Civil Defense Agency
Jaime Fonseca	Cooperative de Seguros Multiples
Hildelisa Gonzalez	Department of Natural Resources
Paula Gori	U.S. Geological Survey
Olga Hernandez	University of Puerto Rico (Recorder)
Alba Martin	Department of Natural Resources
Jose Molinelli	University of Puerto Rico
Lizette Montaluo	Department of Natural Resources
Risa Palm	University of Colorado (Moderator)
Jesus Parrilla Calderon	University of Puerto Rico
Miguel Puig	Puerto Rico Telephone Company
Lourdes Rivera	Asociacion de Bancos de Puerto Rico
Ismael Roque	Continental Claim Services Inc.
Graciela Seijo	American Red Cross of Puerto Rico
Ismae Valazquez	Puerto Rico Telephone Company
Nara Zenoni	State Civil Defense Agency of Puerto Rico

HIGHLIGHTS OF THE DISCUSSION

The participants in the discussion group noted that there is a great need for community preparedness in Puerto Rico. However, the reality of the situation is that earthquakes are not the first priority problem; unemployment is.

The group pointed out the need to know how prediction of an earthquake and the actual occurrence of an earthquake might affect the economy of Puerto Rico (for example, the flow of money from companies as land values are decreased as a consequence of either the prediction or the actual event).

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The question of possible overemphasis on earthquake hazards was raised by the group. The potential negative impact of "overkill" based on imprecise data dictates that earthquake hazards be studied very carefully in Puerto Rico to build a credible and well documented scientific data base that can be used in community preparedness activities.

RECOMMENDATIONS

The discussion group concluded that the information available at the present time was adequate to undertake a number of activities that would enhance community preparedness. The group recommended that the following subjects be given high priority:

1) Provide information on preparedness and mitigation strategies to the people of Puerto Rico.

2) Inform corporate executives about earthquake hazards and risk in Puerto Rico.

3) Provide information to the public about earthquake hazards and risk and practical actions the community can take to increase their preparedness.

4) Using this workshop as a starting point, provide the press (and others) with: a) correct and timely information on earthquake hazards and risk in Puerto Rico, b) carefully designed scientific information on selected topics (such as the ground shaking hazard, tsunamis, liquefaction, building codes, etc.), and c) popular articles which can be used in a public educational campaign that would give answers to the following types of questions:

    a) What is the hazard and what caused the hazard?
    b) What to do after the hazardous event?
    c) How are communities organized to respond to a hazardous event?

5) Promote educational campaigns to increase awareness and personal preparedness for geologic hazards in Puerto Rico seeking sponsorship from: a) the Department of Education (for example, incorporate information about the nature of geologic hazards and what to do to mitigate their effects in the curriculum and textbooks), b) churches (for example, provide puppet shows, etc.), c) civil defense organizations, d) volunteer groups, and e) civic and professional organizations.

6) Information should be prepared for target audiences.

7) Inform the Puerto Rican Permits and Regulation Administration of the need and strong support for their approval of the new building code.

8) Promote educational campaigns seeking sponsorship by: a) hotels, b) industry, c) public utility companies, d) insurance companies, and e) local and Federal agencies

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DRAFT PLANS FOR IMPLEMENTATION OF LOSS REDUCTION MEASURES

by

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Menlo Park, California
and
Luis E. Biaggi
Puerto Rico Planning Board
Santurce, Puerto Rico

FOREWARD

These draft plans and recommendations were developed by the participants of the workshop on "Geologic Hazards in Puerto Rico." They are intended to serve as a guide for public officials, scientists, engineers, social scientists, and emergency managers. Representatives of these disciplines can use the plans and recommendations in several ways: 1) to evaluate their current research, mitigation, response and recovery programs, 2) to devise new programs and plans and, 3) to create a seismic safety policy in Puerto Rico.

The following participants attended this discussion group on implementation:

Luis E. Biaggi	Puerto Rico Planning Board (Recorder)
Earl Brabb	U.S. Geological Survey (Moderator)
Anselmo De Portu	Department of Natural Resources
Bernardo Deschapelles	University of Puerto Rico
Rafael Esteva	Puerto Rico Planning Board
Angel Figueroa	Puerto Rico Police
Esteban L. Fuertes	Citibank
Walter Hays	U.S. Geological Survey
Jose E. Hernandez	Center for Energy and Environmental Research
Gilberto Isasc Valdez	Highway Authority
Rafael Jimenez	University of Puerto Rico
Victor M. Marques	Highway Authority
Philip McIntire	Federal Emergency Management Agency
Stanley McIntosh	Federal Emergency Management Agency
Andres Paiva Paiva	Puerto Rico Highway Ing. Soils
Edgardo Pagan Anes	Highway Authority
Fernando L. Perez	Puerto Rico Electric Power Assoc.
Cesar Pujols	Highway Authority
Pedro Salcrup Rivera	Highway Authority
Heriberto Torres	U.S. Geological Survey, San Juan

HIGHLIGHTS OF THE DISCUSSION

The participants in this group discussed the following subjects:

1) The necessity to implement the amendments submitted to the Puerto Rican Administration of Permits and Regulations by a Seismic Committee of Engineers. These amendments will be reviewed in public hearings and after adoption they will be approved by the Puerto Rican Planning Board.

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2) The need for the Department of Natural Resources to gather all available information from public and private enterprises and make an economic cost analysis of the impact of geologic hazards in Puerto Rico (for example, the effects of landslides on subdivisions, housing, and roads). The possibility exists that an executive order may be needed to implement this study.

3) Utilization of advisory services on soils and geologic hazards offered by the Department of Soil Engineering at the Puerto Rican Highway Authority by all government agencies involved in construction and planning.

4) The need for geologic reports for critical and important facilities such as hospitals, schools, and lifelines.

5) The need and possible requirement for federal agencies who fund construction (such as Veterans Administration, Federal Housing Authority, and Farmers Home Administration) to obtain site geologic reports in hazardous areas (for example, those areas shown in red on the USGS landslide maps).

6 The importance of continuing education for architects and other disciplines not represented at the workshop.

RECOMMENDATION

The members of the discussion group approved the following declaration:

> "Whereas, the Seismic Committee of Engineers Association has submitted to the Puerto Rican Building Permits and Regulation Administration an updated proposal for the earthquake resistant design of structures in Puerto Rico. Whereas, after nine months after the document has been submitted no action has been taken. Therefore, the participants of the workshop on Geologic Hazards in Puerto Rico strongly recommended the need for urgent action in the evaluation and implementation of the aforementioned updated seismic code."

Implementation of this declaration would reduce losses from geologic hazards in Puerto Rico.

DRAFT PLANS TO ENHANCE INFORMATION TRANSFER AND PERSONAL PREPAREDNESS

by
William Kockelman
U.S. Geological Survey
Menlo Park, California
and
Leandro Rodriguez
University of Puerto Rico
Mayaguez, Puerto Rico

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These draft plans and recommendations were developed by the participants of the workshop on "Geologic Hazards in Puerto Rico." They are intended to serve as a guide for public officials, scientists, engineers, social scientists, and emergency managers. Representatives of these disciplines can use the plans and recommendations in several ways: 1) to evaluate their current research, mitigation, response and recovery programs, 2) to devise new programs and plans and, 3) to create a seismic safety policy in Puerto Rico.

The membership of the discussion group included:

Heriberto Capella Acevedo	Puerto Rico Department of Education
Walter Cedeno Rivera	Center for Energy and Environment Research
Jacobo Colon Gutierrez	Puerto Rico Department of Civil Defense
Orlando Cordero	University of Puerto Rico
Ellis S. Febres Silva	Puerto Rico Department of Civil Defense
Ralph M. Field	Ralph M. Field Associates, Inc.
Chalres Gover	Puerto Rico Highway Authority
Jorge Hidalgo	Hidalgo & Associates
Luis Humberto Vega	Office of the Commissioner of Insurance of Puerto Rico
William Kockelman	U.S. Geological Survey (Moderator)
Jose Martinez Cruzado	University of Puerto Rico
William McCann	Lamont-Doherty Geological Observatory at Columbia University
Wilfredo Melendez	Puerto Rico Department of Civil Defense
Antonio V. Munera	Puerto Rico Department of Civil Defense
Boris Oxman	Puerto Rico Department of Natural Resources of Puerto Rico
Robert D. Prince	LANT NAVFAC (Navy)
Nestor Ramirez	Public Building Authority
Leandro Rodiquez	Puerto Rico Department of Civil Engineering (Recorder)
Jesus Rodriguez	Puerto Rico Department of Natural Resources
Ramon Santiago	U.S. Department of Agriculture, Soil Conservation Service
Alejandro Soto	University of Puerto Rico
Rodolfo Tardy	University of Puerto Rico
William Vazquez	International Charter Mortgage Corporation of San Juan
Antonio Zargoza Rodriguez	University of Puerto Rico

HIGHLIGHTS OF THE DISCUSSION

Information transfer is a complex subject; therefore, the members of the discussion group spent a great deal of time identifying the primary steps in the process. The process can be represented as follows:

> The community (people and programs) require geologic hazards information (maps, reports, etc.). The process of transferring the information to users in the community (scientists, engineers, architects, social scientists, emergency managers, public officials) is controlled by constraints (political-legal, safety, physical, economic, social, technological) which must be eliminated or minimized by creative activities (partnerships, incentives, reduction of costs, development of technology for solving discrete components of the problem, optimization of decisions, etc.). The activities designed to transfer information require demonstration of their value (publications, workshops, etc.) for evaluation and promotion of acceptance (ordinances, legislation, etc.).

In addition the group also discussed the following subjects:

1) The need to allocate resources to support "geologic hazards crusaders" who will carry the message to decisionmakers about the threat and the options for mitigation.
2) The opportunties to educate builders, engineers, architects, the financial sector, and others.
3) The emerging challenge of rehabilitation of existing buildings and the opportunity to test various techniques.
5) Design and construction problems in Puerto Rico.
6) Implementation of reasonable seismic design provisions of the building code.
7) The need for a seismologist in Puerto Rico.
8) Organization of a Caribbean Basin Geologic Hazards Conference to share information, to build networks, and to continue the effort begun at this workshop.
9) Personal preparedness; i.e., those actions which individuals can take to make their home, work place, and their children's schools safer from geologic hazards.

RECOMMENDATION

The members of the discussion group recommended two priority actions:

1) The adoption of the new seismic design provisions of the Puerto Rican Building Code.

2) Every participant find extraordinary ways to enhance transfer of information on geologic hazards to various users. The goal is to make the process become "routine" and a model for other regions of the world to follow.

3) Every participant identify "zero cost" actions which they can take to make their home safer from earthquake hazards. These actions include bolting the house to the foundation, tying down the water heater, reinforcing bookcases so that they will not fall, etc.

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DRAFT PLANS FOR INCREASING AWARENESS OF GEOLOGIC HAZARDS

by

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FOREWARD

These draft plans and recommendations were developed by the participants of the workshop on "Geologic Hazards in Puerto Rico." They are intended to serve as a guide for public officials, scientists, engineers, social scientists, and emergency managers. Representatives of these disciplines can use the plans and recommendations in several ways: 1) to evaluate their current research, mitigation, response and recovery programs, 2) to devise new programs and plans and, 3) to create a seismic safety policy in Puerto Rico.

The membership of the discussion group included:

Joyce Bagwell	Baptist College at Charleston
Jane Bullock	Federal Emergency Management Agency (Moderator)
Carmen L. Burges	Department of Social Services
Luz Delia Díaz Rosado	State Civil Defense Agency
Walter Hays	U.S. Geological Survey
Bill Kockelman	U.S. Geological Survey
Philip McIntire	Federal Emergeny Management Agency (Recorder)
Antonio Munera	Puerto Rico Civil Defense
Boris L. Oxman	Department of Natural Resources
Miguel Pagan Mir	State Civil Defense
Fernando L. Perez	Puerto Rico Electrical Power Authority
Miguel Puig	Puerto Rico Telephone Company
Claire Rubin	George Washington University
Graciela Seijo	American Red Cross
Robert A. Shuman	Independent Insurance Broker
Mariano Vargas	Department of Natural Resources
Manuel Vazquez	Puerto Rico Electrical Power Authority
Nora E. Zenoni	State Civil Defense Agency
Hildelisa Gonzalez	Department of Natural Resources

HIGHLIGHTS OF THE DISCUSSION

The members of the discussion group identified a wide range of subjects in the context of increasing hazard awareness. They included:

1) The American Red Cross check-list for making homes, busineses, and industries safe from geologic hazards. The need to update this information and publish it in Spanish was noted.

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2) Ways to achieve effective radio and T.V. spot announcements on geologic hazards.

3) Forming partnerships with business and industry.

4) Mobilization of organizations such as the scouts and others and civil defense personnel to carry information about geologic hazards to the home.

5) The location of existing shelters that could be used in the event of a damaging earthquake. The questions of being self sufficient for 48 hours (for example, food supplies, procedures for making water safe, first aid, communications, etc) were addressed.

6) Evacuation procedures for buildings; potential limitations on use of roads and other transportation lifelines.

7) Education of of school children, beginning at the earliest levels, on geologic hazards. Earthquake drills.

8) Training of local civil defense organizations; earthquake exercises; evacuation exercises; formulation of multihazards emergency management concepts.

RECOMMENDATIONS

The group recommended the following actions:

1) Increasing hazard awareness is a team effort. Each member of the team (scientists, engineers, architects, social scientists, planners, emergency managers, and public officials) has a job to do that depends on the intergration and coordination of their activities and programs with others. Therefore, a priority effort is needed in Puerto Rico to continue the work that has already begun to increase the level of awareness of geologic hazards. Identification of leaders, "geologic hazards crusaders," and other resources to achieve the short- and long-term goals of hazard awareness should be undertaken immediately and continued throughout this decade.

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GLOSSARY OF TERMS USED IN EARTHQUAKE HAZARDS ASSESSMENTS

Accelerogram. The record from an accelerometer showing acceleration as a function of time. The peak acceleration is the largest value of acceleration on the accelerogram.

Acceptable Risk. A probability of occurrences of social or economic consequences due to earthquakes that is sufficiently low (for example in comparison to other natural or manmade risks) as to be judged by appropriate authorities to represent a realistic basis for determining design requirements for engineered structures, or for taking certain social or economic actions.

Active fault. A fault is active if, because of its present tectonic setting, it can undergo movement from time to time in the immediate geologic future. This active state exists independently of the geologists' ability to recognize it. Geologists have used a number of characteristics to identify active faults, such as historic seismicity or surface faulting, geologically recent displacement inferred from topography or stratigraphy, or physical connection with an active fault. However, not enough is known of the behavior of faults to assure identification of all active faults by such characteristics. Selection of the criteria used to identify active faults for a particular purpose must be influenced by the consequences of fault movement on the engineering structures involved.

Asthenosphere. The worldwide layer below the lithosphere which is marked by low seismic wave velocities. It is a soft layer, probably partially molten.

Attenuation law. A description of the average behavior of one or more characteristics of earthquake ground motion as a function of distance from the source of energy.

Attenuation. A decrease in seismic signal strength with distance which depends not only on geometrical spreading, but also may be related to the physical characteristics of the transmitting medium that cause absorption and scattering.

b-value. A parameter indicating the relative frequency of earthquakes of different sizes derived from historical seismicity data.

Capable fault. A fault along which future surface displacement is possible, especially during the lifetime of the engineering project under consideration.

Convection. A mechanism of heat transfer through a liquid in which hot material from the bottom rises because of its lesser density, while cool surface materials sinks.

Convergence Zone. A band along which moving plates collide and area is lost either by shortening and crustal thickening or subduction and destruction of crust. The site of volcanism, earthquakes, trenches, and mountain building.

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Design earthquake. A specification of the ground motion at a site based on integrated studies of historic seismicity and structural geology used for the earthquake-resistant design of a structure.

Design spectra. Spectra used in earthquake-resistant design which correlate with design earthquake ground motion values. Design spectra typically are smooth curves that take into account features peculiar to a geographic region and a particular site.

Design time history. One of a family of time histories used in earthquake-resistant design which produces a response spectrum enveloping the smooth design spectrum, for a selected value of damping.

Duration. A qualitative or quantitative description of the length of time during which ground motion at a site exhibits certain characteristics such as being equal to or exceeding a specified level of acceleration such as 0.05g.

Earthquake hazards. The probability that natural events accompanying an earthquake such as ground shaking, ground failure, surface faulting, tectonic deformation, and inundation, which may cause damage and loss of life, will occur at a site during a specified exposure time. See earthquake risk.

Earthquake risk. The probability that social or economic consequences of earthquakes, expressed in dollars or casualties, will equal or exceed specified values at a site during a specified exposure time.

Earthquake waves. Elastic waves (P, S, Love, Rayleigh) propagating in the Earth, set in motion by faulting of a portion of the Earth.

Effective peak acceleration. The peak ground acceleration after the ground-motion record has been filtered to remove the very high frequencies that have little or no influence upon structural response.

Elastic rebound theory. A theory of fault movement and earthquake generation that holds that faults remain lock while strain energy accumulates in the rock, and then suddenly slip and release this energy.

Epicenter. The point on the Earth's surface vertically above the point where the first fault rupture and the first earthquake motion occur.

Exceedance probability. The probability (for example, 10 percent) over some period of time that an event will generate a level of ground shaking greater than some specified level.

Exposure time. The period of time (for example, 50 years) that a structure is exposed to the earthquake threat. The exposure time is sometimes related to the design lifetime of the structure and is used in seismic risk calculations.

Fault. A fracture or fracture zone in the Earth along which displacement of the two sides relative to one another has occurred parallel to the fracture. See Active and Capable faults.

Focal depth. The vertical distance between the hypocenter and the Earth's surface in an earthquake.

Ground motion. A general term including all aspects of motion; for example, particle acceleration, velocity, or displacement; stress and strain; duration; and spectral content generated by a nuclear explosion, an earthquake, or another energy source.

Intensity. A numerical index describing the effects of an earthquake on the Earth's surface, on man, and on structures built by him. The scale in common use in the United States today is the Modified Mercalli scale of 1931 with intensity values indicated by Roman numerals from I to XII. The narrative descriptions of each intensity value are summarized below.

I. Not felt--or, except rarely under especially favorable circumstances. Under certain conditions, at and outside the boundary of the area in which a great shock is felt: sometimes birds and animals reported uneasy or disturbed; sometimes dizziness or nausea experienced; sometimes trees, structures, liquids, bodies of water, may sway--doors may swing, very slowly.

II. Felt indoors by few, especially on upper floors, or by sensitive, or nervous persons. Also, as in grade I, but often more noticeably: sometimes hanging objects may swing, especially when delicately suspended; sometimes trees, structures, liquids, bodies of water, may sway, doors may swing, very slowly; sometimes birds and animals reported uneasy or disturbed; sometimes dizziness or nausea experienced.

III. Felt indoors by several, motion usually rapid vibration. Sometimes not recognized to be an earthquake at first. Duration estimated in some cases. Vibration like that due to passing of light, or lightly loaded trucks, or heavy trucks some distance away. Hanging objects may swing slightly. Movements may be appreciable on upper levels of tall structures. Rocked standing motor cars slightly.

IV. Felt indoors by many, outdoors by few. Awakened few, especially light sleepers. Frightened no one, unless apprehensive from previous experience. Vibration like that due to passing of heavy or heavily loaded trucks. Sensation like heavy body of striking building or falling of heavy objects inside. Rattling of dishes, windows, doors; glassware and crockery clink or clash. Creaking of walls, frame, especially in the upper range of this grade. Hanging objects swung, in numerous instances. Disturbed liquids in open vessels slightly. Rocked standing motor cars noticeably.

V. Felt indoors by practially all, outdoors by many or most; outdoors direction estimated. Awakened many or most. Frightened few--slight excitement, a few ran outdoors. Buildings trembled throughout. Broke dishes and glassware to some extent. Cracked windows--in some cases, but not generally. Overturned vases, small or unstable objects, in many instances, with occasional fall. Hanging objects, doors, swing generally or considerably. Knocked pictures against

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walls, or swung them out of place. Opened, or closed, doors and shutters abruptly. Pendulum clocks stopped, started or ran fast, or slow. Move small objects, furnishings, the latter to slight extent. Spilled liquids in small amounts from well-filled open containers. Trees and bushes shaken slightly.

VI. Felt by all, indoors and outdoors. Frightened many, excitement general, some alarm, many ran outdoors. Awakened all. Persons made to move unsteadily. Trees and bushes shaken slightly to moderately. Liquid set in strong motion. Small bells rang--church, chapel, school, etc. Damage slight in poorly built buildings. Fall of plaster in small amount. Cracked plaster somewhat, especially fine cracks chimneys in some instances. Broke dishes, glassware, in considerable quantity, also some windows. Fall of knickknacks, books, pictures. Overturned furniture in many instances. Move furnishings of moderately heavy kind.

VII. Frightened all--general alarm, all ran outdoors. Some, or many, found it difficult to stand. Noticed by persons driving motor cars. Trees and bushes shaken moderately to strongly. Waves on ponds, lakes, and running water. Water turbid from mud stirred up. Incaving to some extent of sand or gravel stream banks. Rang large church bells, etc. Suspended objects made to quiver. Damage negligible in buildings of good design and construction, slight to moderate in well-built ordinary buildings, considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Cracked chimneys to considerable extent, walls to some extent. Fall of plaster in considerable to large amount, also some stucco. Broke numerous windows and furniture to some extent. Shook down loosened brickwork and tiles. Broke weak chimneys at the roof-line (sometimes damaging roofs). Fall of cornices from towers and high buildings. Dislodged bricks and stones. Overturned heavy furniture, with damage from breaking. Damage considerable to concrete irrigation ditches.

VIII. Fright general--alarm approaches panic. Disturbed persons driving motor cars. Trees shaken strongly--branches and trunks broken off, especially palm trees. Ejected sand and mud in small amounts. Changes: temporary, permanent; in flow of springs and wells; dry wells renewed flow; in temperature of spring and well waters. Damage slight in structures (brick) built especially to withstand earthquakes. Considerable in ordinary substantial buildings, partial collapse, racked, tumbled down, wooden houses in some cases; threw out panel walls in frame structures, broke off decayed piling. Fall of walls, cracked, broke, solid stone walls seriously. Wet ground to some extent, also ground on steep slopes. Twisting, fall, of chimneys, columns, monuments, also factory stacks, towers. Moved conspicuously, overturned, very heavy furniture.

IX. Panic general. Cracked ground conspicuously. Damage considerable in (masonry) buildings, some collapse in large part; or wholly shifted frame buildings off foundations, racked frames; serious to reservoirs; underground pipes sometimes broken.

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X. Cracked ground, especially when loose and wet, up to widths of several inches; fissures up to a yard in width ran parallel to canal and stream banks. Landslides considerable from river banks and steep coasts. Shifted sand and mud horizontally on beaches and flat land. Changes level of water in wells. Threw water on banks of canals, lakes, rivers, etc. Damage serious to dams, dikes, embankments. Severe to well-built wooden structures and bridges, some destroyed. Developed dangerous cracks in excellent brick walls. Destroyed most masonry and frame structures, also their foundations. Bent railroad rails slightly. Tore apart, or crushed endwise, pipelines buried in earth. Open cracks and broad wavy folds in cement pavements and asphalt road surfaces.

XI. Disturbances in ground many and widespread, varying with ground material. Broad fissures, earth slumps, and land slips in soft, wet ground. Ejected water in large amounts charged with sand and mud. Caused sea-waves ("tidal" waves) of significant magnitude. Damage severe to wood-frame structures, especially near shock centers. Great to dams, dikes, embankments often for long distances. Few, if any (masonry) structures, remained standing. Destroyed large well-built bridges by the wrecking of supporting piers or pillars. Affected yielding wooden bridges less. Bent railroad rails greatly, and thrust them endwise. Put pipelines buried in each completely out of service.

XII. Damage total--practically all works of construction damaged greatly or destroyed. Disturbances in ground great and varied, numerous shearing cracks. Landslides, falls of rock of significant character, slumping of river banks, etc., numerous and extensive. Wrenched loose, tore off, large rock masses. Fault slips in firm rock, with notable horizontal and vertical offset displacements. Water channels, surface and underground, disturbed and modified greatly. Dammed lakes, produced waterfalls, deflected rivers, etc. Waves seen on ground surfaces (actually seen, probably, in some cases). Distorted lines of sight and level. Threw objects upward into the air.

Liquefaction. Temporary transformation of unconsolidated materials into a fluid mass.

Lithosophere. The outer, rigid shell of the earth, situated above the asthenosphere containing the crust, continents, and plates.

Magnitude. A quantity characteristic of the total energy released by an earthquake, as contrasted to intensity that describes its effects at a particular place. Professor C. F. Richter devised the logarithmic scale for local magnitude (M_L) in 1935. Magnitude is expressed in terms of the motion that would be measured by a standard type of seismograph located 100 km from the epicenter of an earthquake. Several other magnitude scales in addition to M_L are in use; for example, body-wave magnitude (m_b) and surface-wave magnitude (M_s), which utilize body waves and surface waves, and local magnitude (M_L). The scale is open ended, but the largest known earthquake have had M_s magnitudes near 8.9.

Mantle. The main bulk of earth between the crust and core, ranging from depths of about 40 to 2900 kilometers.

Mid-oceanridge. Characteristic type of plate boundary occurring in a divergence zone, a site where two plates are being pulled apart and new oceanic lithosphere is being created.

Plate tectonics. The theory and study of plate formation, movement, interaction, and destruction.

Plate. One of the dozen or more segments of the lithosphere that are internally rigid and move independently over the interior, meeting in convergence zones and separating in divergence zones.

Region. A geographical area, surrounding and including the construction site, which is sufficiently large to contain all the geologic features related to the evaluation of earthquake hazards at the site.

Response spectrum. The peak response of a series of simple harmonic oscillators having different natural periods when subjected mathematically to a particular earthquake ground motion. The response spectrum may be plotted as a curve on tripartite logarithmic graph paper showing the variations of the peak spectral acceleration, displacement, and velocity of the oscillators as a function of vibration period and damping.

Return period. For ground shaking, return period denotes the average period of time or recurrence interval between events causing ground shaking that exceeds a particular level at a site; the reciprocal of annual probability of exceedance. A return period of 475 years means that, on the average, a particular level of ground motion will be exceeded once in 475 years.

Risk. See earthquake risk.

Rock. Any solid rock either at the surface or underlying soil having a shear-wave velocity 2,500 ft/sec (765 m/s) at small (0.0001 percent) strains.

Sea-floor spreading. The mechanism by which new sea floor crust is created at ridges in divergence zones and adjacent plates are moved apart to make room.

Seismic Microzoning. The division of a region into geographic areas having a similar relative response to a particular earthquake hazard (for example, ground shaking, surface fault rupture, etc.). Microzoning requires an integrated study of: 1) the frequency of earthquake occurrence in the region, 2) the source parameters and mechanics of faulting for historical and recent earthquakes affecting the region, 3) the filtering characteristics of the crust and mantle constituting the regional paths along which the seismic waves travel, and 4) the filtering characteristics of the near-surface column of rock and soil.

Seismic zone. A generally large area within which seismic design requirements for structures are uniform.

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Seismotectonic province. A geographic area characterized by similarity of geological structure and earthquake characteristics. The tectonic processes causing earthquakes have been identified in a seismotectonic province.

Source. The source of energy release causing an earthquake. The source is characterized by one or more variables, for example, magnitude stress drop, seismic moment. Regions can be divided into areas having spatially homogeneous source characteristics.

Strain. A quantity describing the exact deformation of each point in a body. Roughly the change in a dimension or volume divided by the original dimension or volume.

Stress. A quantity describing the forces acting on each part of a body in units of force per unit area.

Strong motion. Ground motion of sufficient amplitude to be of engineering interest in the evaluation of damage due to earthquakes or in earthquake-resistant design of structures.

Subduction zone. A dipping planar zone descending away from a trench and defined by high seismicity, interpreted as the shear zone between a sinking oceanic plate and an overriding plate.

Transform fault. A strike-slip fault connecting the ends of an offset in a mid-ocean ridge. Some pairs of plates slide past each other along transform faults.

Trench. A long and narrow deep trough in the sea floor; interpreted as marking the line along which a plate bends down into a subduction zone.

Triple junction. A point that is common to three plates and which must be the meeting place of three boundary features, such as convergence zones, divergence zones, or transform faults.

APENDICE A

GLOSARIO DE TERMINOS PARA ANALISIS PROBABILISTICO DE LOS RIESGOS Y PELIGROS SISMICOS

ACELERACION NOMINAL O DE DISEÑO - Una especificación de la aceleración del terreno en un emplazamiento, términos de un valor único, tales como máximo o rms; utilizados para el diseño resistente a los terremotos de una estructura (como base para derivar un espectro de diseño). Véase "Historia cronológica de diseño".

CARGA DE DISEÑO SISMICO - la representación prescrita (historia cronológica, espectro de respuestas o desplazamiento de la base estática equivalente) de un movimiento sísmico del terreno que se utilizará para el diseño de una estructura.

COEFICIENTE DE VARIACION - la razón de desviación estándar de la media.

CUADRADO MEDIO - valor esperado del cuadrado de la variable aleatoria. (El cuadrado medio menos el cuadrado de la media da la variancia de la variable aleatoria.)

DAÑO - cualquier pérdida económica o destrucción ocasionada por los terremotos.

DESVIACION ESTANDAR - la raíz cuadrada de la variancia de una variable aleatoria.

DURACION - una descripción cualitativa o cuantitativa de la duración de tiempo en el que el movimiento de tierra en un emplazamiento presenta ciertas características (perceptibilidad, temblores violentos, etc.).

EFECTOS DE CARGA DE DISEÑO SISMICO - las acciones (fuerzas axiales, deslizamientos o movimientos de flexión) y deformaciones inducidas en un sistema estructural debido a una representación específica (historia cronológica, espectro de respuestas o deslizamiento de la base) del movimiento de tierra de diseño sísmico.

ELEMENTOS SUJETOS A RIESGO - población, propiedades, actividades económicas, incluyendo los servicios públicos, etc., sujetos a riesgo en una determinada zona.

ESPECTRO DE DISEÑO - una serie de curvas para fines de diseño que proporcionan la velocidad de aceleración o desplazamiento (de ordinario, la aceleración absoluta, la velocidad relativa o el desplazamiento relativo de una masa en vibración) en función del período de vibración y amortiguación.

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ESPECTRO DE RESPUESTAS - una serie de curvas calculadas a partir de un acelerógrafo sísmico que proporciona valores de respuestas máximas de un oscilador lineal amortiguado, en función de su período de vibración y amortiguación.

ESPERADO - medio, promedio, previsto.

EVENTO NOMINAL O DE DISEÑO, EVENTO SISMICO NOMINAL O DE DISEÑO - una especificación de uno o más parámetros de fuentes de terremotos, y del lugar de la liberación de la energía con respecto al punto de interés; se utiliza para el diseño resistente a terremotos de una estructura.

EVENTO SISMICO - la liberación abrupta de energía en la litosfera terreste que ocasiona un terremoto.

EXPOSICION - La pérdida económica posible para todas las estructuras o algunas de ellas como resultado de uno o más terremotos en una región. Este término se refiere, de ordinario, al valor asegurado de las estructuras que mantiene uno o más aseguradores. Véase "Valor en riesgo".

FALLA ACTIVA - una falla que, tomando como base la evidencia histórica, sismológica o geológica, tiene una elevada probabilidad de producir un terremoto (Alternativa: una falla que puede producir un terremoto dentro de un período de tiempo de exposición especificado, dadas las hipótesis adoptadas para un análisis específico del riesgo sísmico).

HISTORIA CRONOLOGICA DE DISEÑO - la variación con el tiempo de movimiento de tierra (por ejemplo, la aceleración del terreno o velocidad o desplazamiento) en un lugar; se utiliza para el diseño resistente a terremotos de una estructura. Véase "Aceleración nominal o de diseño".

INDICE DE ACTIVIDAD SISMICA - el número medio por unidad de tiempo de terremotos con características específicas (por ejemplo, magnitud $\geq$ 6) que se origina en una falla o zona determinada.

INTENSIDAD - una medida cualitativa o cuantitativa de la gravedad de un movimiento sísmico de tierra en un emplazamiento específico (por ejemplo, intensidad Mercalli Modificada, intensidad Rossi-Forel, intensidad Espectral Houser, intensidad Arias, aceleración máxima, etc.).

INTERVALO MEDIO DE INCIDENCIA, INTERVALO DE INCIDENCIA PROMEDIO - el tiempo promedio entre terremotos o eventos de falla con características específicas (por ejemplo, una magnitud de $\geq$ 6) en una región específica o en una zona de falla específica.

LEY DE ATENUACION - una descripción del comportamiento de una característica del movimiento de tierra de un terremoto en función de la distancia de la fuente de energía.

LIMITE SUPERIOR - Véase "máximo posible".

MAXIMO - el valor mayor logrado por una variable durante un tiempo de exposición especificado. Véase "Valor máximo".

MAXIMO CREIBLE
MAXIMO ESPERABLE
MAXIMO PREVISTO
MAXIMO PROBABLE

Estos términos se utilizan para especificar el valor máximo de una variable, por ejemplo, la magnitud de un terremoto, que pudiera esperarse que ocurra razonablemente. <u>En opinión del Comité, son términos equívocos y no se recomienda su uso</u>. (El U.S. Geological Survey y algunos individuos y empresas definen el terremoto máximo creíble como "el terremoto mayor que puede esperarse que ocurra razonablemente". La Oficina de Reclamación, el Primer Grupo de Trabajo Interministerial (septiembre de 1978), definió el terremoto máximo creíble como "el terremoto que ocasionaría el movimiento de tierra vibratorio más agudo capaz de ser producido en el emplazamiento dentro del actual marco tectónico conocido". Es un evento que pueden apoyar todos los datos geológicos y sismológicos conocidos. El USGS define el terremoto máximo esperable o previsto como "el mayor terremoto que puede esperarse razonablemente que ocurra". El terremoto máximo probable es definido a veces como el peor terremoto histórico. Como alternativa, es definido como el terremoto que se reproduce periódicamente cada 100 años o un terremoto que según la determinación probabilística de incidencia ocurrirá durante la vida de la estructura).

MAXIMO POSIBLE - el valor máximo posible para una variable. Sigue a una hipótesis explícita de que no son posibles valores más grandes, o implícitamente a hipótesis en el sentido de que las variables o funciones relacionadas son limitadas en su alcance. El valor máximo posible puede expresarse determinísta o probabilísticamente.

MICROZONA SISMICA - una zona generalmente pequeña dentro de la que los requisitos de diseño sísmico para las estructuras son uniformes. Las microzonas sísmicas pueden presentar la amplificación relativa del movimiento del terreno debido a condiciones locales del suelo sin especificar los niveles absolutos de movimiento o peligro sísmico.

MICROZONACION SISMICA, MICROZONIFICACION SISMICA - el proceso de determinar la peligrosidad sísmica absoluta o relativa en muchos emplazamientos, tomando en cuenta los efectos de la amplificación geológica y topográfica del movimiento y de las microzonas sísmicas. Como alternativa, la microzonación es un proceso para identificar características geológicas, sismológicas, hidrológicas y geotécnicas detalladas del emplazamiento en una región específica e incorporarlas en la planificación del uso de la tierra y el diseño de estructuras seguras a fin de reducir el daño a la vida humana y la propiedad como resultado de los terremotos.

MOVIMIENTO DE TIERRA ESPERADO - el valor medio de una o más características del movimiento de tierra en un emplazamiento para un terremoto dado (movimiento medio del terreno).

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PELIGRO GEOLOGICO - un proceso geológico (por ejemplo, corrimiento de tierra, suelos en licuefacción, falla activa) que durante un terremoto u otro evento natural puede producir efectos adversos sobre las estructuras.

PELIGRO SISMICO - cualquier fenómeno físico (por ejemplo, temblor de tierra, falla de tierra) asociado con un terremoto que puede producir efectos adversos sobre las actividades del hombre.

PERDIDA - cualquier consecuencia social o económica adversa ocasionada por uno o más terremotos.

PERIODO DE RETORNO MEDIO - el tiempo promedio entre incidencias de movimientos de tierra con características específicas (por ejemplo, aceleración horizontal máxima $\geq$ 0,1 g) en un emplazamiento. (Igual a la inversa de la probabilidad anual de superación).

PROBABILIDAD DE SUPERACION - la probabilidad de que un nivel específico de movimiento de tierra o consecuencias sociales o económicas específicas de los terremotos sean superados en el emplazamiento en una región durante un tiempo de exposición específico.

RAIZ CUADRADA MEDIA (rms) - raíz cuadrada del valor cuadrado medio de una variable aleatoria.

RIESGO ACEPTABLE - probabilidad de consecuencias sociales o económicas debidas a terremotos que es suficientemente baja (por ejemplo, en comparación con otros riesgos naturales o creados por el hombre), para que las autoridades pertinentes juzguen que representan un análisis pragmático para determinar requisitos de diseño para estructuras de ingeniería o para adoptar ciertas medidas sociales o económicas.

RIESGO SISMICO - la probabilidad de que las consecuencias sociales o económicas de los terremotos sean iguales o superen valores específicos en un emplazamiento, en varios emplazamientos o en una zona durante un período de exposición específico.

TERREMOTO - un movimiento o vibración repentino de la tierra ocasionado por la liberación abrupta de energía en la litosfera terrestre. El movimiento de las ondas puede oscilar entre violento en algunos lugares e imperceptible en otros.

TERREMOTO NOMINAL O DE DISEÑO - una especificación del movimiento sísmico de tierra en un emplazamiento; se utiliza para el diseño resistente a los terremotos de una estructura.

TIEMPO DE EXPOSICION - el período cronológico de interés para cálculos de riesgos sísmicos, cálculos de la peligrosidad sísmica o diseño de estructuras. Para las estructuras, el tiempo de exposición se selecciona a menudo de forma que sea igual a la vida de diseño de la estructura.

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VALOR B - un parámetro que indica la frecuencia relativa de incidencia de terremotos de distintas magnitudes. Es la pendiente de una línea recta que indica la frecuencia absoluta o relativa (trazada logarítmicamente) frente a la magnitud del terremoto o intensidad meisosísmica Mercalli Modificada. (El valor B indica la pendiente de la relación de periodicidad Gutenberg-Richter).

VALOR EN RIESGO - la pérdida económica posible (asegurada o no) a todas las estructuras o cierto juego de estructuras como resultado de uno o más terremotos en una región. Véase "Exposición".

VALOR TOPE O MAXIMO - el valor máximo de una variable dependiente del tiempo durante un terremoto.

VARIABLE DE FUENTE - una variable que describe una característica física (por ejemplo, magnitud, descenso en esfuerzo, momento sísmico, desplazamiento) de la fuente de liberación de la energía que ocasiona un terremoto.

VARIANCIA - la desviación media al cuadrado de una variable aleatoria de su valor promedio.

VULNERABILIDAD - el grado de pérdida a un elemento dado sujeto a riesgo, o una serie de esos elementos, como resultado de un terremoto de una determinada magnitud o intensidad, que de ordinario se expresa en una escala de 0 (sin daño) a 10 (pérdida total).

ZONA DE DISEÑO SISMICO - zona sísmica.

ZONA SISMOGENAS - un término anticuado. Véase "Zona sismogénica" y "Zona sismotectónica".

ZONA DE RIESGOS SISMICOS - un término anticuado. Véase "Zonas sísmicas".

ZONA SISMICA - una zona generalmente grande dentro de la cual los requisitos de diseño sísmico para las estructuras son constantes.

ZONA SISMOGENICA, PROVINCIA SISMOGENICA - una representación planar de un ambiente de tres dimensiones en la litosfera terrestre en el que se infiere que los terremotos tendrán un origen tectónico análogo. Una zona sismogénica puede representar una falla en la litosfera terrestre. Véase "Zona sismotectónica".

ZONA SISMOTECTONICA, PROVINCIA SISMOTECTONICA - una zona sismogénica en la que se han identificado los procesos tectónicos que ocasionan los terremotos. Estas zonas son, de ordinario, zonas de falla.

ZONACION SISMICA, ZONIFICACION SISMICA - el proceso de determinar la peligrosidad sísmica en muchos emplazamientos para fines de delineación de zonas sísmicas.

ZONACION SISMOGENICA - el proceso de delinear regiones que tienen un carácter tectónico y geológico casi homogéneo, para los fines de trazar zonas sismogénicas. Los procedimientos específicos utilizados dependen de las hipótesis y modelos matemáticos empleados en el análisis de riesgo sísmico y el análisis de peligrosidad sísmica.

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ESTADO LIBRE ASOCIADO DE PUERTO RICO
ADMINISTRACION DE REGLAMENTOS Y PERMISOS
Santurce, Puerto Rico

RESUMEN DE REVISIONES PROPUESTAS AL
REGLAMENTO DE EDIFICACION
(REGLAMENTO PLANIFICACION NUM. 7)

ARTICULO IV-A-0 CARGAS MINIMAS PARA EL CALCULO DE LAS ESTRUCTURAS

Los artículos sobre cargas mínimas se revisan completamente. En la revisión se utiliza material proveniente del Reglamento de Edificación actual, del "Uniform Building Code" (UBC, edición 1982) y de la revisión realizada por la Comisión sobre Terremoto del Colegio de Ingenieros y Agrimensores de Puerto Rico.

En algunos de los artículos se organiza el material en forma más lógica y fácil de usar. A continuación se resumen los cambios para estos artículos sobre cargas mínimas.

ARTICULO IV-A-1.0 GENERAL

En este artículo se modifica el párrafo del Código actual y se añaden dos secciones para referirse a factores de carga y combinaciones de carga.

ARTICULO IV-A-2.0 CARGAS FIJAS

Este artículo se queda básicamente igual a como está en el Código vigente aunque se reduce la tabla de pesos de materiales para eliminar algunos materiales que ya no se usan en Puerto Rico.

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ARTICULO IV-A-3.0 CARGAS ACCIDENTALES

En este artículo se cambia el orden de presentación para hacerlo más lógico. Además, se subdivide en tres secciones. Una de éstas se refiere a cargas de piso, otra a cargas de techo y una sobre disposiciones de reducción de cargas accidentales.

Las cargas mínimas para pisos y techo no se cambian, pero ahora se reglamenta para que se consideren distribuciones críticas de ellas. Además, se especifica que en techos especiales se use una carga diferente a la mínima de 20 libras por pie cuadrado. También se hace referencia en esta sección del problema de acumulación de agua en los techos.

Las especificaciones sobre reducción de cargas accidentales se deja igual a las del Código vigente por considerarla adecuada, más práctica y fácil de automatizar que la recomendada por el UBC.

ARTICULO IV-A-4.0 CARGAS ESPECIALES

Esta sección se dejó igual a la del Código vigente.

ARTICULO IV-A-5.0 FUERZAS LATERALES

Sección IV-A-5.1 General - Esta sección se modifica para especificar claramente que se considere el momento horizontal causado por todo tipo de carga horizontal. Se incluyen unas especificaciones sobre vuelco y otras sobre los anclajes que

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deben existir entre techo y paredes o columnas y entre paredes o columnas y cimientos.

También se reduce el incremento en la resistencia del subsuelo de un cien a un treinta y tres porciento cuando se consideran cargas de viento o terremoto. Esta reducción es a tono con las recomendaciones del "UBC. Uniform Building Code".

Sección IV-A-5.2 Resistencia a Vientos Huracanados

La sección se cambia por completo. Siguiendo las normas del UBC como guía se crean unas especificaciones a tono con nuestra situación.

Primero se escoge y se especifica una velocidad básica de diseño basada en un estudio estadístico realizado por el "National Oceanographic Administration (NOAA)". Reconociendo que Puerto Rico ha sufrido y puede sufrir huracanes de mayor intensidad se incluye un factor de sobrecarga a estructuras livianas por considerar que estas, debido a su poco peso y redundancia, no podrían resistir en forma segura las presiones que pueden generarse durante un huracán de gran intensidad. Por otra parte estamos seguros que, dado su peso y configuración estructural, un edificio multipiso podría resistir en forma segura esta situación.

Se proveen dos métodos de análisis, uno para estructuras livianas y otro para estructuras multipisos no livianas. Se considera que en estructuras multipisos se obtiene un análisis realista aplicando la presión en la proyección de superficies verticales y horizontales. Sin embargo, se considera que esto

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no es suficientemente preciso para estructuras livianas y se provee para aplicar las cargas en las diferentes superficies de la estructura.

En la fórmula para computar las presiones de diseño se provee un factor de importancia para tomar en consideración que hay estructuras tales como hospitales, escuelas, estaciones de bomberos y otras que son esenciales en caso de un desastre natural por lo cual deben tener un factor de seguridad mayor.

Al igual que en la reglamentación vigente se provee para variación de presión con altura, la variación recomendada es de acuerdo con el UBC. Además, se provee para dos tipos de exposición de la estructura. El caso más severo es aquel en que la estructura está localizada en un lugar abierto mientras que el segundo tipo corresponde a estructuras que están protegidas por bosques, irregularidades del terreno o edificios.

Se proveen coeficientes de presiones más severos para elementos y componentes de estructuras reconociendo que estos pueden estar sometidos a presiones mayores que la presión normal de diseño.

ARTICULO IV-A-5.3 RESISTENCIA A TERREMOTOS

La sección de fuerzas laterales inducidas por terremoto fue reemplazada completamente siguiendo las recomendaciones de la Comisión de Terremotos del Colegio de Ingenieros y Agrimensores de Puerto Rico. Los cambios fundamentales son:

a. La ecuación para determinar las fuerzas mínimas de terremotos para las estructuras incluye un factor de zona igual a 0.6 para toda la isla, un factor de importancia para las edificaciones vitales que deben continuar en uso luego de un sismo, y un factor de suelos que considera las condiciones del subsuelo en la respuesta del edificio. La ecuación para el coeficiente c usado actualmente en las normas vigentes fue también actualizado así como la ecuación para la determinación del período de vibración para edificios donde las cargas laterales se resisten con un sistema estructural de pórticos.

b. Para edificios esbeltos donde el período de vibración es mayor de 0.7 segundos, se especifica que una proporción de las fuerzas mínimas deberán aplicarse como una carga concentrada en el tope del edificio.

c. Se incluyen provisiones para considerar el efecto P-Delta cuando en edificaciones esbeltas y flexibles el coeficiente de estabilidad sea menor de 10%.

d. Se permite el uso de fuerzas y distribuciones alternas a las recomendadas en las normas propuestas siempre y cuando se conduzca un análisis dinámico para las estructuras utilizando los movimientos de tierra denominados como el registro de Hollywood y el registro de Taft.

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e. El Administrador de la Administración de Reglamentos y Permisos podrá requerir la instalación de acelerógrafos en edificios de más de 15 pisos de altura.

ARTICULO IV-M-0 CONSTRUCCIONES DE MADERA

Los artículos sobre construcción de madera fueron cambiados completamente y los nuevos corresponden básicamente a traducciones hechas del Código del UBC. Algunos artículos del UBC, al igual que algunas de las Tablas, fueron eliminados por corresponder a materiales que no son comunmente usados en Puerto Rico y detalles que no corresponden al Reglamento de Edificación.

Algunas secciones se modificaron considerablemente al traducirlas para obtener más claridad en el texto.

Se proveen unas reglamentaciones completas que viabilizan el diseño en madera.

ARTICULOS IV-M-1, IV-M-2 y IV-M-3

En estos artículos se definen términos y notaciones y se hacen algunos requerimientos de carácter general.

ARTICULO IV-M-4 ESFUERZOS

En este artículo se especifican los esfuerzos unitarios permitidos de diseño, en forma tabulada, para los diferentes tipos de madera comunmente usados en Puerto Rico. Además, se especifican los factores de ajuste de estos esfuerzos unitarios permitidos para las diferentes circunstancias que los afectan. Por ejemplo, se especifican factores de ajuste para duración de carga, esbeltez, tamaño, forma, temperatura, etc.

ARTICULO IV-M-5.0 IDENTIFICACION

En el artículo se requiere que toda madera que tenga un uso estructural esté identificada debidamente lo cual es necesario para poder realizar una inspección efectiva en construcciones de madera.

ARTICULO IV-M-6.0 DISEÑO DE ELEMENTOS HORIZONTALES

Este artículo provee todas las especificaciones relacionadas con el diseño de elementos horizontales. Se provee además para el diseño en flexión y cortante del elemento y para la compresión perpendicular a la fibra. Se proveen también especificaciones para el diseño de conexiones y de elementos con muescas.

ARTICULO IV-M-7.0 DISEÑO DE COLUMNAS

En este artículo se clasifican las columnas y se especifica la razón de esbeltez permitida para diferentes tipos de elementos actuando en compresión.

ARTICULO IV-M-8.0 CARGAS EN FLEXION O AXIAL COMBINADAS

El artículo provee las fórmulas y especificaciones para el diseño de diferentes tipos de elementos de madera sometidas a cargas axiales y en flexión.

ARTICULO IV-M-9.0 COMPRESION EN ANGULO AL GRANO

La fórmula de Hankinson para computar esfuerzos en compresión permitidos en un ángulo al grano se provee en este artículo.

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ARTICULO IV-M-10-0 CONECTORES Y AMARRES DE MADERA

En este artículo se incluyen las especificaciones para el diseño de conexiones de madera con pernos, tornillos y clavos. Se incluyen Tablas en la cual se indican las cargas permitidas en cada caso.

ARTICULO IV-M-11.0 y IV-M-12.0

Los artículos hacen referencia a las normas que rigen el diseño de miembros de madera estructural laminada pegada y de madera laminada pegada contrachapada "(plywood)".

ARTICULO IV-M-13.0 DIAFRAGMAS DE MADERA

Se proveen en este artículo especificaciones y tablas para el diseño de diafragmas de madera. Se provee también para el diseño de diafragmas de madera laminada (plywood).

ARTICULO IV-M-14.0 MADERA LAMINADA CON MAMPOSTERIA O CONCRETO

Este artículo provee las especificaciones que rigen el uso de madera combinada con mampostería o concreto.

ARTICULO IV-M-15.0 REQUERIMIENTOS GENERALES DE CONSTRUCCION

Este artículo incluye requerimientos generales de construcción para obtener una estructura en madera segura y duradera. Especificaciones sobre protección contra el deterioro y la polilla se incluyen en detalle. Estas especificaciones, entre otras cosas, tiene requerimientos sobre contenido de humedad, separación entre el terreno y la madera, ventilación debajo del piso, etc.

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ARTICULO IV-F - CONSTRUCCIONES EN MAMPOSTERIA

Esta sección del Reglamento de Edificación existente fue revisada completamente usando como base las recomendaciones del "Uniform Building Code (UBC, edición 1982)" y las recomendaciones del Código de Mampostería de Hormigón (ACI-531-79) preparados por el Instituto Americano del Hormigón (ACI). En resumen, se elimina del código existente los tipos de construcciones de mampostería que no se utilizan comunmente en Puerto Rico así como aquellos tipos de construcciones que presentan riesgos mayores en caso de actividades sísmicas significativos. Los cambios fundamentales se concentran en las siguientes áreas:

a. Materiales - se eliminan materiales de construcción en mampostería que no se utilizan comunmente en Puerto Rico. Se establecen además, las especificaciones mínimas que deben satisfacer los materiales usados en construcciones de mampostería en Puerto Rico.

b. Resistencia última de la mampostería - esta nueva sección le provee al usario varios métodos para determinar la resistencia última de la mampostería de tal forma que se pueda realizar el diseño, especificación y construcción de la misma en forma controlada. En base a la resistencia última determinada en esta sección se especifican los esfuerzos permisibles de diseño en secciones subsiguientes.

c. Las construcciones de mampostería se dividen en dos tipos principales, a saber: mampostería reforzada y mampostería sin reforzar. La sección de mampostería sin reforzar específica los esfuerzos de trabajo en compresión, cortante o

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tensión, permitidos para la mampostería de unidades huecas o sólidas. Se indican además, los valores de fuerzas en cortante permitidas en tornillos de anclaje incrustrados en unidades de mampostería sin reforzar.

Para la construcción en mampostería reforzada se indican los esfuerzos de trabajo permitidos para compresión y cortante así como el módulo de elasticidad, módulo de rigidez y esfuerzos de contacto, en unidades de mampostería sólidos o huecos. Se especifica además, los esfuerzos de trabajo en tensión y compresión permitidos en el acero de refuerzo. Esta sección le provee guías específicas diseñada para el cómputo de los esfuerzos en flexión, cortante y de esfuerzos combinados axiales y de flexión. También se especifica el método para determinar el esfuerzo para cortante necesario así como los requerimientos para anclajes y empalmes del refuerzo.

d. Se incluye una sección para los muros de mampostería parcialmente reforzada o sin reforzar donde se indican los esfuerzos permisibles, los espesores de muros, y las razones de altura o espesor permitidas.

ARTICULO IV-S - DISPOSICIONES ESPECIALES PARA EL DISEÑO SISMICO DE ESTRUCTURAS DE HORMIGON ARMADO

Esta sección, que se incluye por primera vez en el Reglamento de Edificación de Puerto Rico, detalla los requisitos especiales para el diseño sismo-resistente de estructuras de hormigón armado necesarias para que la estructura pueda sostener deformaciones inelásticas considerables sin que ocurra el colapso de la misma. Estos requisitos deben proveerse para todas aquellas estructuras de hormigón armado diseñadas por el Código de Construcción del Instituto Americano del Hormigón (ACI-318-83) que se incluirá por referencia en la sección IV-G - Construcción de Hormigón Armado. Debemos observar que esta sección reemplaza el Apéndice A del ACI-318-83. Las secciones sobresalientes de esta sección son:

a. Armazones dúctiles - se indican los detalles de refuerzo longitudinal y de refuerzo de cortante requeridos por vigas y columnas de armazones dúctiles que deben sostener deformaciones inelásticas excesivas. Estas provisiones especifican las dimensiones mínimas de la sección, los porcientos de acero máximos permitidos en la sección, la continuidad del acero positivo y negativo, los anclajes y largos de empalmes requeridos para el refuerzo longitudinal y los refuerzos de cortante especiales requeridos en las vigas y columnas para asegurar que no ocurran fallas prematuras por tensión diagonal. Se indica además los requerimientos y consideraciones de diseño para las conexiones de vigas y columnas para que ésta resista adecuadamente las fuerzas inducidas por las vigas y columnas.

b. Muros de corte - se indican los requerimientos de cargas para los cuales se deben diseñar los muros de corte. En esta sección se indica además, los esfuerzos permitidos para el cortante máximo en el hormigón y los porcientos mínimos de refuerzo

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longitudinal y de cortante. Se especifica también los porcientos máximos de acero permitidos para las vigas de acoplamiento de paredes acopladas.

ARTICULO IV-T DISPOSICIONES ESPECIALES PARA CONSTRUCCIONES DE ACERO

La sección IV-T detalla los requisitos especiales para el diseño sismo-resistente de estructuras en acero. Al igual que la sección IV-S, se incluye por primera vez en el Reglamento de Edificación con el propósito de implementar el Código de Estructuras de Acero (American Institute of Steel Construction, edición, 1978) referida en la sección IV-I. El material incluído en esta sección fue traducido directamente de las recomendaciones finales de la Comisión de Terremotos del Colegio de Ingenieros y Agrimensores de Puerto Rico. Los puntos sobresalientes de esta sección son:

a. Juntas de armazones - se requiere que en las juntas de armazones de acero se provean placas de acero que le den continuidad a las alas de los elementos que se unen.

b. Conexiones de vigas - las conexiones de vigas a columnas deben resistir el cortante último desarrollado por los momentos plásticos ocurridos en las articulaciones de las vigas.

c. Inestabilidad local - las alas de los elementos deben satisfacer las razones de ancho a espesor indicadas en el reglamento para evitar fallas prematuras del elemento por inestabilidades locales.

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d. Pruebas no destructivas - se requieren pruebas no destructivas para las soldaduras de los elementos primarios para asegurarse que no tienen fallas internas que impidan el desarrollo de la capacidad última de la conexión.

e. Arriostramiento laterales - se especifican largos máximos de arriostramiento de tal forma que se pueda resistir los desplazamientos laterales y torsionales máximos sin que ocurran inestabilidades locales.

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