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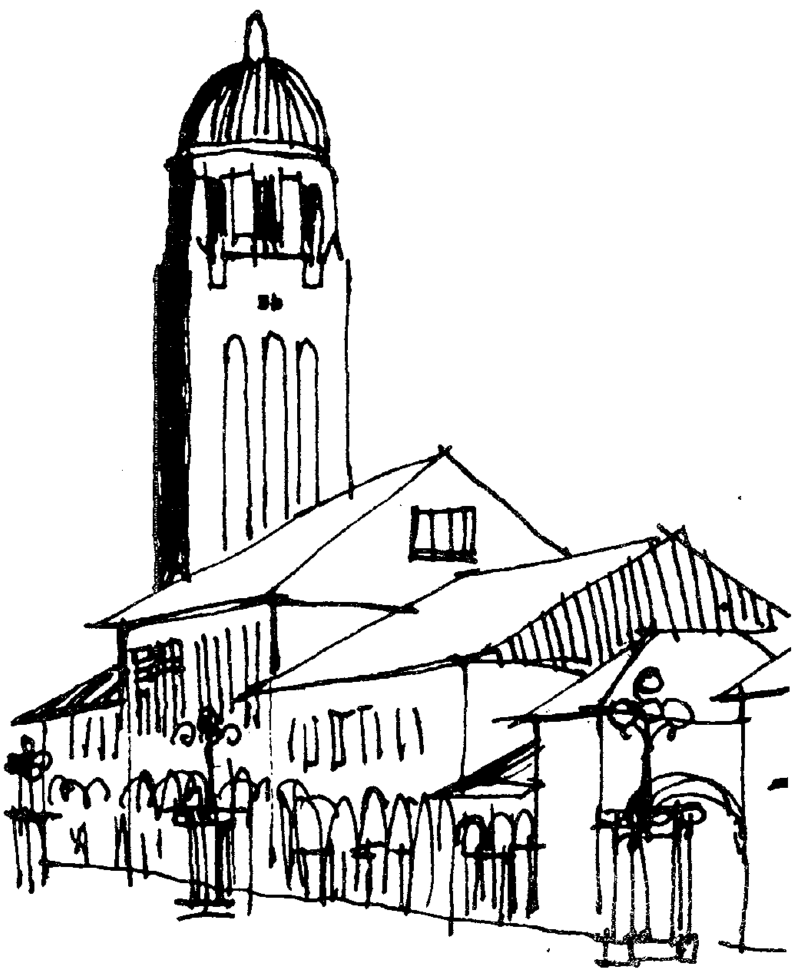
The John A. Blume Earthquake Engineering Center

Department of Civil Engineering
Stanford University

A STUDY OF SEISMIC RISK FOR NICARAGUA
Part I



by
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16. Abstract (Limit: 200 words)
The geological setting for the country in general and the Managua area in particular is described. The data base for past seismic events was extensively studied. Limitations of the data and approximations are discussed. Seismic recurrence for ten line sources and three area sources was developed. Based on the assumption of the Poisson occurrence of seismic events, probabilities of exceeding different magnitude levels as functions of time for different regions were derived. Using Esteva's attenuation relationship, isoacceleration maps for the country were constructed. Eleven cities in Nicaragua were considered in this mapping process. Cumulative distribution functions of peak ground accelerations for 20 and 50 years were established. This is shown to be one way of presenting seismic risk for Nicaragua. Based on isoacceleration maps, the Acceleration Zone Graphs (AZG) were developed for the eleven cities. A method for determining load levels for consistent risk for the whole country is discussed and suggested. It is proposed that charts such as AZG be used for seismic zoning of Nicaragua. Ground acceleration values from AZG were employed to set the level of the design spectra for structural damage prevention and condemnation control. A design methodology is proposed based on ultimate strength and loads resulting from the above inelastic design spectra.

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Any opinions, findings, conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the National Science Foundation.

SYMBOLS AND DEFINITIONS

a	=	Fixed Acceleration
A_g, A	=	Effective Ground Acceleration
AZG	=	Acceleration Zone Charts
C	=	Subscript for Condemnation Threshold
CAP	=	Subscript for Structure Capacity
CON	=	Subscript for Condemnation Capacity
CTSD	=	Condemnation Threshold Spectrum for Structure Deformation Determination
D	=	Subscript for Damage Threshold
D_g	=	Effective Ground Displacement
DAF	=	Dynamic Amplification Factor for Spectral Shape
DAM	=	Subscript for Damage Capacity
DEM	=	Subscript for Earthquake Demand
DES	=	Subscript for Member Design Level
DTSS	=	Damage Threshold Spectrum for Member Strength Determination
E	=	Expected Value
K_G	=	Confidence Limit Contribution due to Use Group
K_T	=	Confidence Limit Contribution due to structural system type
$(K_G+K_T)\sigma$	=	Confidence Limit above the mean DAF
L	=	Structure Life in years
m	=	Fixed Richter Magnitude
M	=	Richter Magnitude
M_b	=	Body Wave Magnitude

M_S = Surface Wave Magnitude

MMI = Modified Mercalli Intensity

n = Number of events

N(M) = Number of earthquakes above Richter Magnitude M

N'(M) = Normalized N(M)

p = Probability of success (Bernouilli Trials)

P = Probability

R = Reliability = 1-P

RP = Return Period

$S_a(\beta,T)$ = A basic acceleration response spectrum ordinate for a system with damping β, and period T

SRSS = Square Root of Sum of Squared Modal Responses

t = Fixed Period of Time

V = Lateral Load due to Earthquake Ground Motion

V_g = Effective Ground Velocity

α β = Regression Coefficient

α' = Normalized α

β = Damping Ratio

β' = Modified Damping Ratio

β_F = Damping due to structure-foundation interaction

β_S = Damping due to structural system type

Δ = Deformation

δ = Mean Rate of Occurrence (Poisson Law)

μ = Ductility Ratio

μ' = Modified Ductility Ratio

σ = Standard Deviation of the DAF

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CHAPTER I

INTRODUCTION

On December 23, 1972, three strong earthquake tremors struck the city of Managua. Even though of "moderate" magnitude, this event caused thousands of deaths, many more injuries, an untold amount of economic hardship and disruption of a way of life. It is very hard if not impossible to translate all these losses into quantitative economic terms. However, such a catastrophe does remind us of the devastation and far-reaching consequences of major earthquake events.

The rebuilding efforts which follow such events bring up many questions. Are existing design requirements adequate? What level of future risk is acceptable? How should the acceptable risk level be translated into acceptable design parameters? Should similar land uses be permitted in the future for areas which suffered major damage? These and numerous other questions become especially relevant after a significant damaging event. The decision process which leads to answers for these questions is a complex mixture of political expediency, engineering knowhow, socioeconomic optimization and proper understanding of the overall parameters involved in the decision making process. In times when no significant earthquake events have taken place, the decision making process goes on at a notably slow rate, while decisions immediately after significant events are often based on expediency and, at times, on irrational analysis. This leads to decisions which might well be considered inadequate in the light of rational decisions made on the basis of long-term perspective.

This report is the result of a seismic risk study conducted at Stanford and supported by Banco Central de Nicaragua and the National Science Foundation grant GI 39122. The total seismic risk analysis of Nicaragua is done in two parts. This report is Part I of the study. In general, Part I is associated with the future probably seismic loading determination of Nicaragua and how that loading can be used to determine future damage potential and "insurance risk." Suggestions regarding seismic zoning of the country are also presented in this part. Similarly, the relationships between seismic loading information and design provisions is discussed. Part II is a continuation of Part I, and in general is associated with probabilistic response spectra analysis, probabilistic seismic exposure of different classes of structures and the seismic structural response. A decision analysis of associated risks of loss of life, injury and economic loss will be performed in Part II. A simplified equivalent design procedure will be developed, based on the general concepts and findings of the more detailed response spectrum method. This simplified approach is intended to be applicable for a majority of ordinary regular structures.

It should be emphasized that the work presented in this report is to provide a base for planning and decision making in Nicaragua. The project results provide professionals in Nicaragua with tools and procedures to make seismic risk analysis. A single recommendation today does not appear practical to fit all future circumstances. Hence, major effort is focused on presenting methodology and procedures that can be used by participating organizations in decision making processes.

Finally, it should be kept in mind that the work and results presented here depend on the available data base and information. The reliability of results are at best as good as the reliability of the data on which the results are based. It is very easy to attack and criticize any work from the point of view of data reliability. However, it is very difficult to obtain long-range reliable data. We have used the best available information through various organizations and researchers. The forecasts and predictions are based on those data. However, if in the future more reliable data are available, the model can easily accommodate the inclusion of new information and update the results. Further discussion on this topic will be presented in Chapter 3. At this time, the authors of this report feel that the results presented here represent the "best available" estimates of the future forecasts.

The report is organized in eight chapters and six appendices. Chapter 2 deals with the geologic setting of Nicaragua in general and Managua in particular. In this chapter, the geologic hazards and their implications are pointed out. Chapter 3 gives the discussion on available data. This chapter should be carefully read because it points out, in detail, the shortcomings of the available seismic data and how those shortcomings are treated in the present work. Chapter 4 develops the future forecasting models based on past data, and presents iso-acceleration maps for the country in general and selected cities in particular. In Chapter 5, the concept of seismic zoning is presented.

Charts relating risk level, economic life of structures, return period and the corresponding loading levels are presented in that chapter. Chapter 6 deals with future damage potential prediction and presents some thoughts on insurance risk. Chapter 7 gives the relationship between seismic zoning, group and use of structures and the needed design provisions. Chapter 7 should be viewed as an introduction to part II of the current study in which further design provision development will be presented. Chapter 8 gives summary and conclusion for part I of the research project and introduces to the reader part II of the study.

In reading this report, a casual reader can start with Chapter 4 and see the forecasting on future seismic loading. A planner can start with Chapter 5 to see the seismic zoning of the country. A structural engineer should read Chapters 4, 5, 6, and 7. In conclusion, it should be emphasized that this is a report on seismic risk analysis. As the name implies, there are many uncertainties and there is always a chance that nature will have the last say.

CHAPTER II

REGIONAL GEOLOGIC SETTING

Relation to Plate Tectonics

Nicaragua lies on the Circumpacific "Ring of Fire" which dominates the tectonics of the Pacific Ocean region. The city of Managua lies on the western edge of the Caribbean Plate. In the parlance of the new global plate tectonics, the Caribbean Plate is apparently being underridden by both the Cocos Plate, to the west, and the Atlantic Plate, to the east. Volcanic arcs and grabens, or long, linear depressions in the earth's crust, are characteristic of the intersection of many plates. Managua lies within such a graben, the Nicaragua Depression, and also within a volcanic arc.

Another characteristic of plate intersections are ocean trenches. In this case, such a trench is the Middle America Trench, which marks the depression of the Cocos Plate below the Caribbean Plate. The trench is 4-5 km deep, extends west of the Central American Coast from Mexico to Costa Rica, and runs sub-parallel to the arc-shaped chain of andesitic stratovolcanoes.

Marking the descent of the Cocos Plate is a zone of friction, generally termed the Benioff Zone. This zone is marked by numerous earthquakes, extending several hundred kilometers into the Earth's interior. The 1972 earthquakes, however, did not occur along this zone, as they were much shallower. They were probably related to relatively shallow adjustment to accumulating crustal strain within the southwest part of the Caribbean Plate.

Geology of the Nicaragua Depression

The outstanding feature of the Pacific Coastal Region of Nicaragua is the Nicaragua Depression (also called the Nicaragua Graben, or Central Valley). Bounded on the northeast by a long, straight, en echelon fault, or the Boundary Fault, the Depression extends from the Gulf of Fonseca, to the north, to near Limon, Costa Rica, in the south (figures 2-1, 2-2, and 2-3). The western boundary is placed by some workers beneath the Pacific Ocean, by others along the Mateare Fault, a semi-continuous fault zone. Downfaulting, which began at the beginning of the Quaternary, about 1,000,000 years ago, has continued to the present.

The Boundary Fault, although at depth probably a normal (block above the fault moves relatively downward) fault, locally suggests right-lateral, or strike slip (opposite block moves laterally to the right) movement. It is locally buried by volcanic rocks. This fault is more regular than the Mateare; it is traceable along the entire length of the Depression. The graben floor is tilted away from the fault, suggesting that either normal movement is minor, or that it is less recent than on the Mateare Fault. In addition, much of the vertical movement could have been taken on several, sub-parallel, en echelon faults.

The Mateare Fault is less clearly expressed in all places. Where prominent, it is a normal fault, displaying a scarp with a

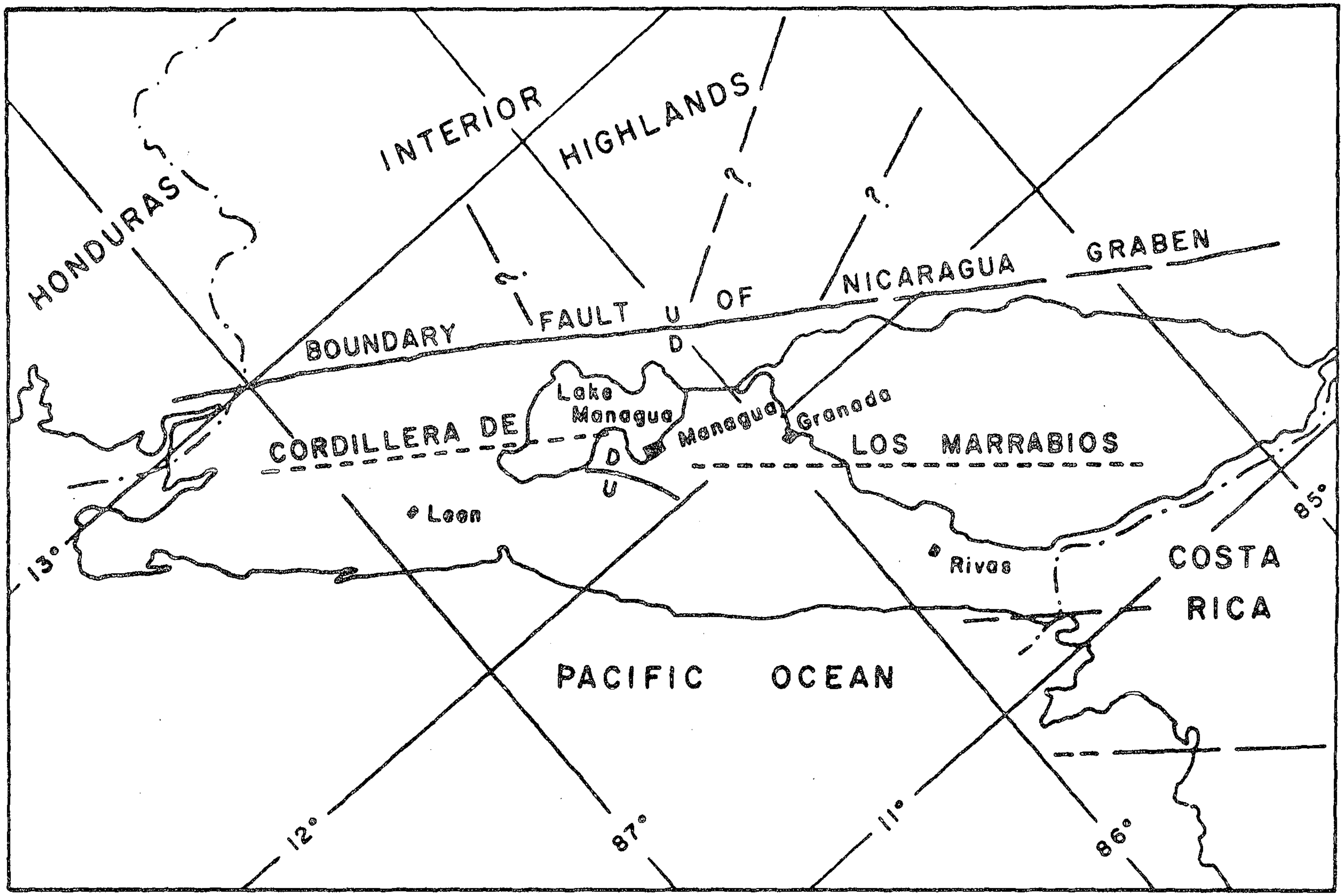

Figure 2-1. Sketch of part of Nicaragua showing the most conspicuous tectonic features. The Cordillera de Marrabios is a line of volcanoes, offset in the vicinity of Managua. The valley containing Lake Managua and Lake Nicaragua is called the Nicaragua Graben. The boundary fault of the graben bounds the Atlantic side of the tectonically active area.

From Saint-Amand, 1973

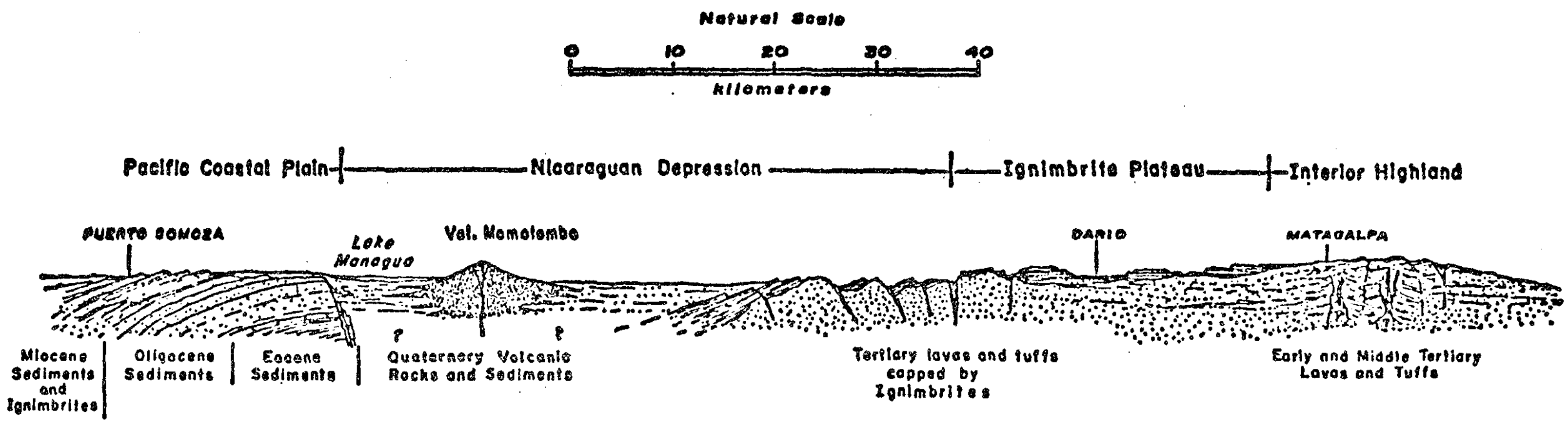

Fig. 2-2 Generalized geologic section across western Nicaragua.
After McBirney, A, and Williams, H, 1965

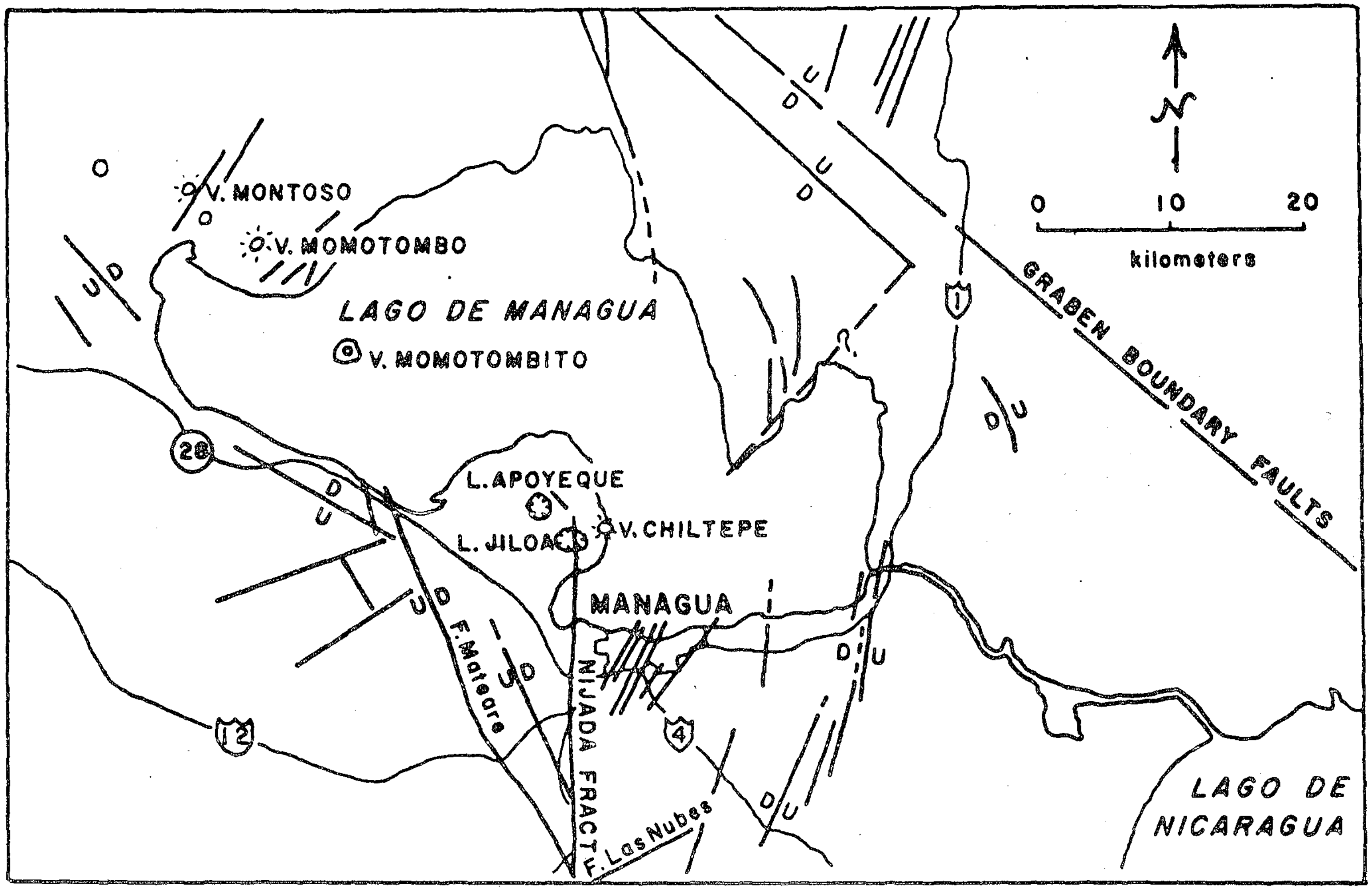

Figure 2-3-Local Faulting in the Managua Area. The boundary fault of the graben is some 40 miles to the northeast. The Falla Mateare is a normal fault with some lateral movement of undetermined sense. The Fractura de Nejapa is a line of explosion pits, craters, collapse structures and aberrant stream channels arranged along an extensional zone. Between Lago Nicaragua and the line of faults between the airport and Rio Tipitapa, extensive alluviation from the Masaya Complex may hide the surface expression of some faults. Data is not available to me at this time to delineate faults on the northside of Lago Managua in the same detail as on the south side. Note that the fracture pattern strongly suggests a rudely circular area of subsidence across the valley.
From Saint-Amand, 1973

maximum height of 1000 meters along the Sierras de Carazo. The uplift is recent, as evidenced by the slight erosion of the upland surface, despite easily eroded volcanic rocks and high rainfall. Matumoto and Latham calculate the total vertical movement along the western edge of the graben as 2.4 km by adding the thickness of accumulated sediments in the graben (1.4 km) to the vertical drop (scarp) (1 km). (See reference 11.)

The Depression contains a thick accumulation of alluvium, lake sediments, deeply weathered volcanic ash, and some volcanic flow deposits, aggregating, as stated, to a total thickness in excess of 1400 m. The basement rocks are unknown, being neither penetrated by wells nor ejected from volcanoes. The isolated hills of Tertiary volcanics are largely buried, although more exposed to the northeast. This suggests a thicker sedimentary accumulation to the southwest, giving further evidence of greater subsidence in this part of the graben.

Geology of Managua Region

The city of Managua sits atop a succession of volcanic rocks and sediments aggregating at least 1000 meters in thickness. The section is probably typical of the entire Nicaragua Depression, although specific units lense out and are replaced by other units elsewhere in the graben.

To a depth of at least 200 m, the section is a relatively homogeneous and predominantly volcanic sequence of lapilli-sized, angular scoria, or cinder deposits, with interbedded, thin ash deposits. These are derived either from Masaya Crater, 22 km distant, or the line of volcanoes to the west. Firm and relatively well-lithified (consolidated) volcanic mudflow deposits are also common. These are thick and firm enough to be used as building stones. The scoria is extremely porous, permeable, and features a low bulk density. It demonstrates good stability under static loads, and stands in near-vertical slopes if undisturbed, but is not stable under dynamic loading (see reference 14).

Some authors emphasize sedimentary rocks, especially lake sediments, in the sequence. A more exact determination of the nature of the rocks would aid in predictions of seismic wave propagation, especially the attenuation of the waves, in general, and the effect of the rocks on accelerations, in particular.

West of Managua, relatively dense lava flows and vent debris are associated with pyroclastics similar to those underlying the city. Less damage occurred here during the 1972 earthquake, but this is probably because of the greater distance from the earthquake epicenter (see reference 14).

Volcanism

The entire Nicaragua Depression is either an active or potentially active area. Managua lies atop volcanics and volcanically derived sediments that have been deposited in the recent geologic past. Masaya Crater, centered 22 km distant from Managua, has been active in historic times (see reference 12), and some of the volcanic deposits underlying the city have been traced to this same volcanoe. Managua lies within an apparent right-lateral offset of a line of volcanoes, the

Cordillera de los Marrabios. The reason for this offset is unclear.

Soils

The soils of Managua are, on the whole, relatively similar throughout the city (Figures 2-4, 2-5). They "consist mainly of volcanic deposits of cohesionless silts, sand and gravels ranging from loose to well-consolidated and having various degrees of cementation" (see reference 17). The soils occur in well-defined layers of from a few to several hundred centimeters thickness. However, thicknesses, as well as degree of compaction, are somewhat variable even at individual sites. The first "rock-like" material occurs at variable depths. It is called "cantera," or "volcanic sandstone," but in reality is a volcanic tuff agglomerate (see reference 17).

Water Levels

The water table is generally 10-30 m below the ground, and 19 m in the city center. Near Lago de Managua (Lake Managua) it reaches to within 3 m of the surface. For most of the city, it is too deep to be of significance in the design or location of foundations (see references 17, 14).

Faulting

The faults which pass through Managua are members of a system of faults which scar much of the Nicaragua Depression. The faults show some normal, or vertical movement. In general, faults in the western part of the city show movement down to the east, whereas faults to the east demonstrate the opposite. Thus, a shallow composite graben

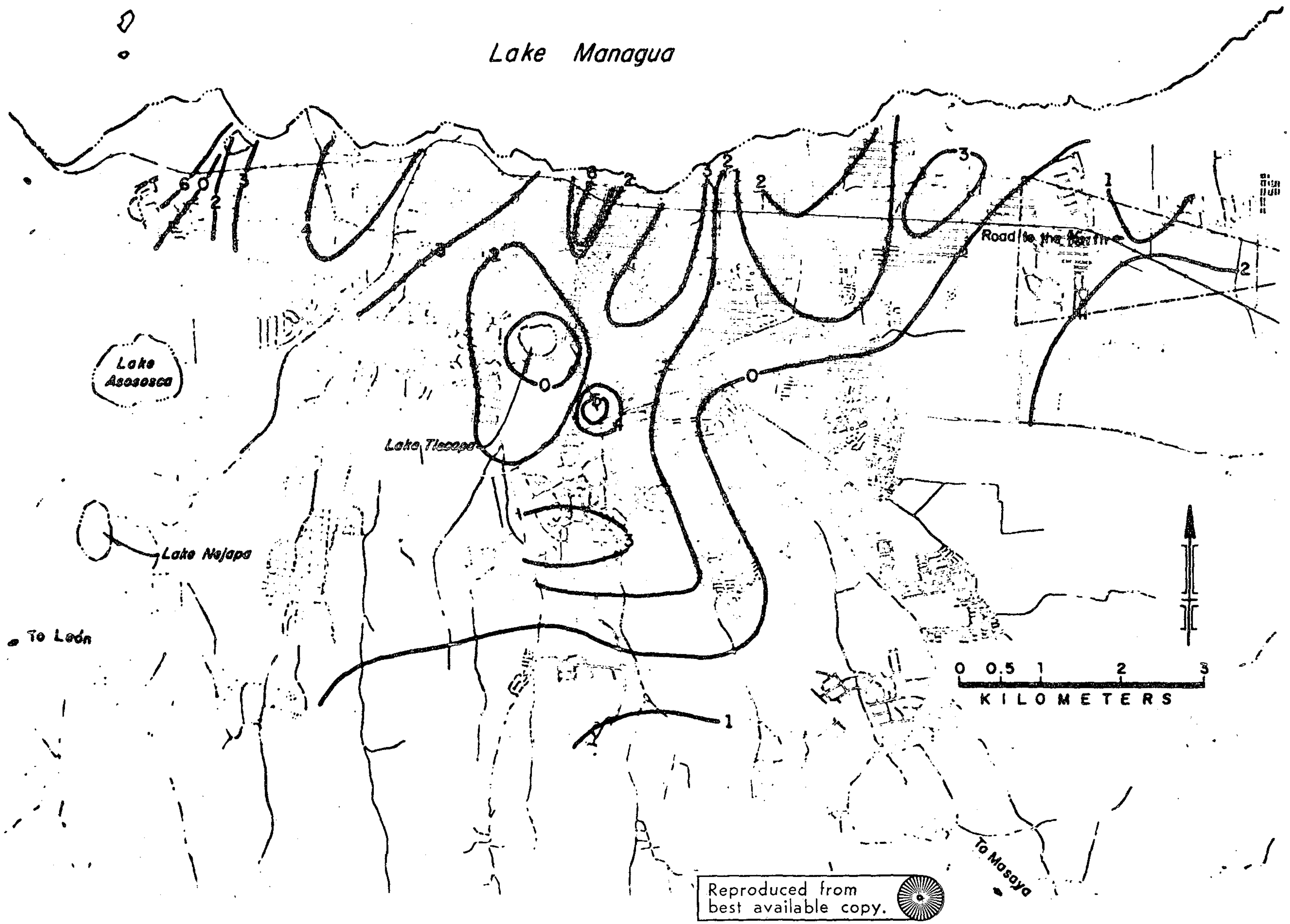

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Fig.2-4 Contour lines of depth of loose surface deposits ($N < 10$)

From Faccioli, et al, 1973

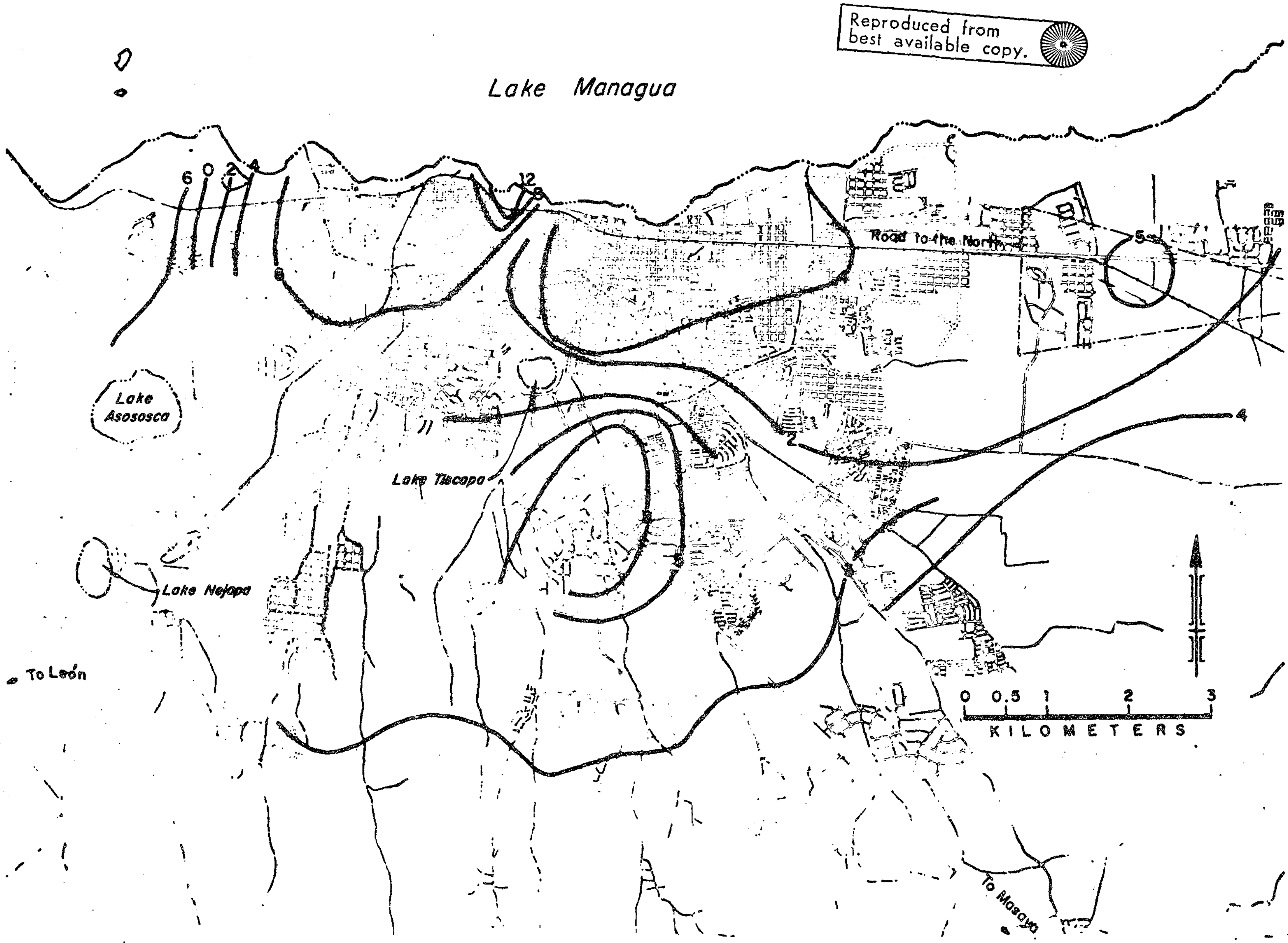

Fig. 2-5 Contour lines of depth of intermediate soils ($10 < N < 50$) Fro
From Faccioli, et al, 1973

is being formed. Movement in the 1972 earthquake is in general agreement with older fault displacements.

One of the purposes of the geologic study was to describe and interpret the pattern of faulting within the city, as an aid to seismic zoning. The task is extremely difficult, however, because there is no agreement on the location of the faults in Managua. As many as 10 faults have been mapped (Figures 2-6, 2-7); at least 4 suffered offset in the 1972 earthquake (Figure 2-8). One other, the poorly defined Stadium Fault, was offset in the 1931 earthquake. Although we will address ourselves to some general remarks in relation to these faults, final discussions must wait until thorough trenching and mapping, currently underway, is completed and made available.

In conclusion, the Nicaragua Depression is a currently active downfaulted block, bounded on one side by an active fault, and on the other by a potentially active fault. The pile of sediments and volcanics underlying the Depression total more than 1000 m in thickness. Active volcanics cover part of its surface.

A few important points relating to seismic zoning follow:

(1) Each of the 10 "faults" is a zone, rather than a line of movement. Whether the "fault" has moved in the recent past is not of significance in Managua, as each zone of fracturing could suffer either more fracturing or actual offset in a future earthquake. Thus, fracture zones from the 1972 earthquake should be considered in the same light as faults where displacement actually occurred.

(2) The distance a structure should be placed from a known

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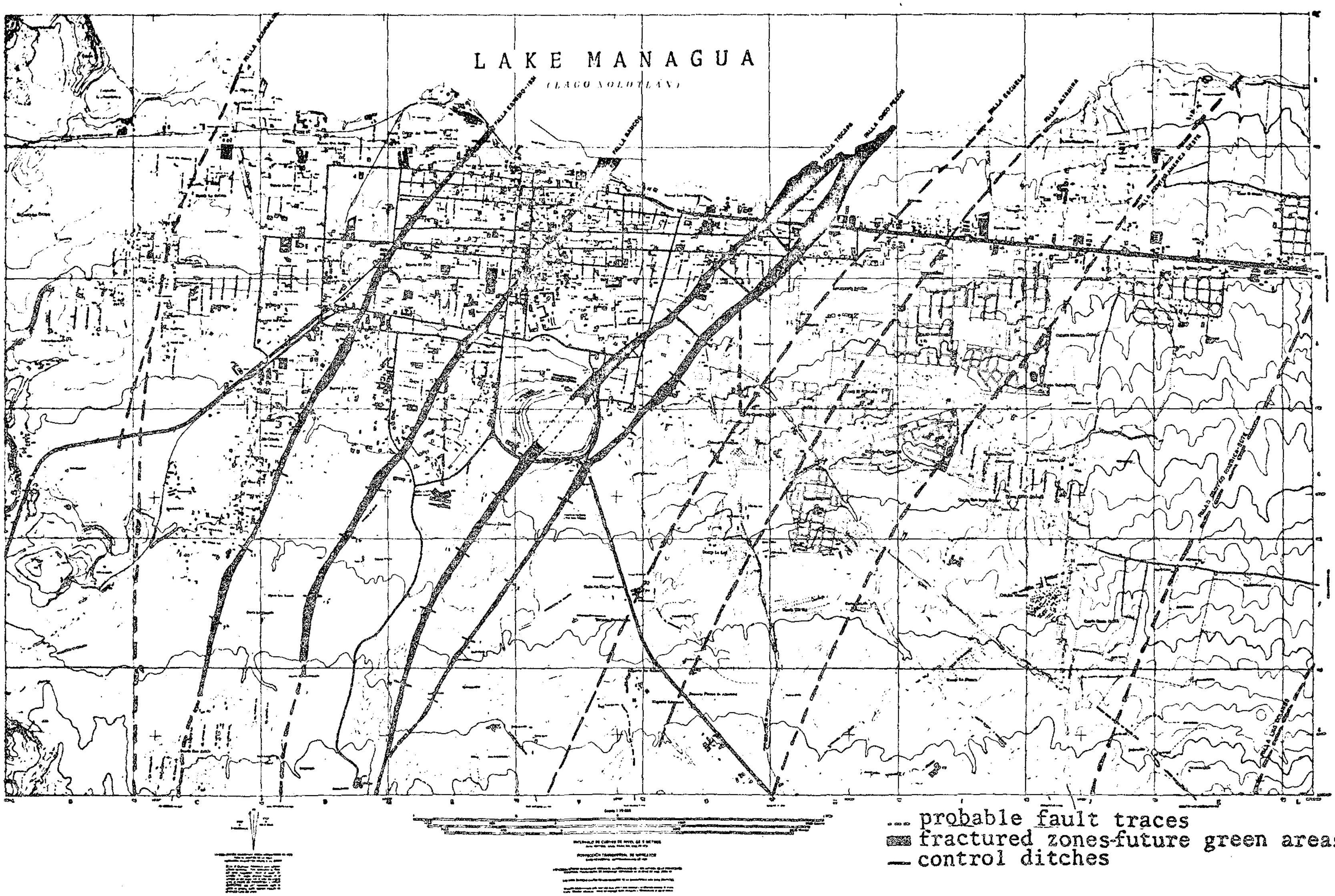


Figure 2-6 Map of Managua showing faults as located by the group of Mexican geologists

From Caldera, 1973

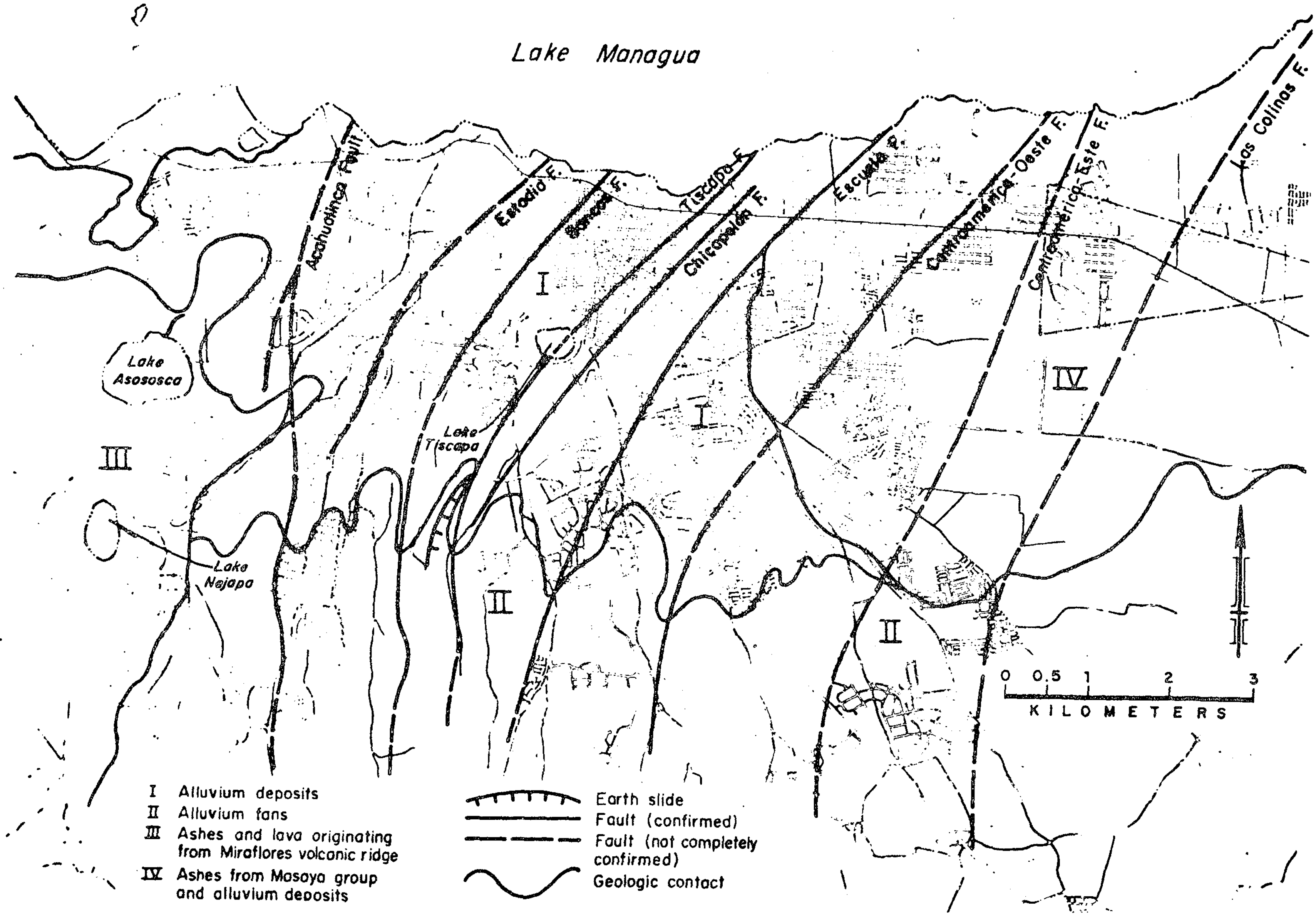


Fig. 2-7 Map of faults and surface geologic zonation of Managua area
From Faccioli, et al, 1973

fault is open to considerable debate. It has been observed that faults tend to rupture repeatedly along nearly the same line, although many recent scarps are located within much wider fault zones, and in alluvium, the fault rupture could easily occur outward from the previously "recognized" breakage. It is our opinion that vital structures, such as hospitals, police and fire stations, etc., should be located at least 100 feet from any fault zone or individual rupture. In a major fault zone, such as the San Andreas Fault in California, movement can be expected to occur along the same trace as the previous movement, but in a region such as Managua, it is our opinion that this would not necessarily be the case.

(3) The type of material on which the structure rests, and the degree of saturation of the material, are extremely important. However, in the case of Managua, these parameters are relatively insignificant, because of the nearness of seismic activity. The only exceptions would be saturated fill and lake sediments. Structures built on these materials faired especially poorly in the 1972 earthquake.

(4) The definition of an "active" fault is not agreed upon. Many geologists use 40,000 years since the last movement as the criterion of "active." This, of course, is often difficult to determine. In addition, the valid argument that even 10,000 years, or less, is economically impractical when considering structures with an anticipated lifetime of less than 100 years. It is our opinion that strategically important and large, public structures should not be located over defined faults in the city of Managua, unless the most conservative

(40,000 years) definition of "active" is used. This would apply to the 10 faults and fracture zones discussed earlier, as well as possibly some buried faults. As a practical manner for the city of Managua, any faults that have not displaced the "White Pumice" unit are probably no longer active.

(5) It is unlikely that a better location for the city could easily be found within the Nicaragua Depression, as the graben is cut by numerous faults. On a random movement from the city, the chances for an earthquake would be equal or greater, and the chances for actual surface rupture would be less, the same, or greater. To find a lesser chance for rupture would require an exhaustive and time-consuming study, which would have to locate an almost fault-free site.

Each of the identified surface or near-surface "faults" within the city should be considered potentially active. As apparently only one fault ruptured in the 1931 earthquake, and at least 4 other faults moved in the 1972 earthquake, it is likely that still other faults could rupture in a future earthquake. From a geologic hazard point of view, where fault rupture and fault displacement are of main concern, critical facilities should not be built within 100 feet of the fault zone or individual rupture. Wallace (see reference 20) suggested one zoning scheme (fig. 2-8) based on strictly geologic hazard. For resistance to seismic vibrational loading, the following chapters will develop criteria for design and zoning requirements. It should be kept in mind that the location of rupture and fault zones within Managua is under study and the future zoning for such geologic hazards should incorporate results of that study.

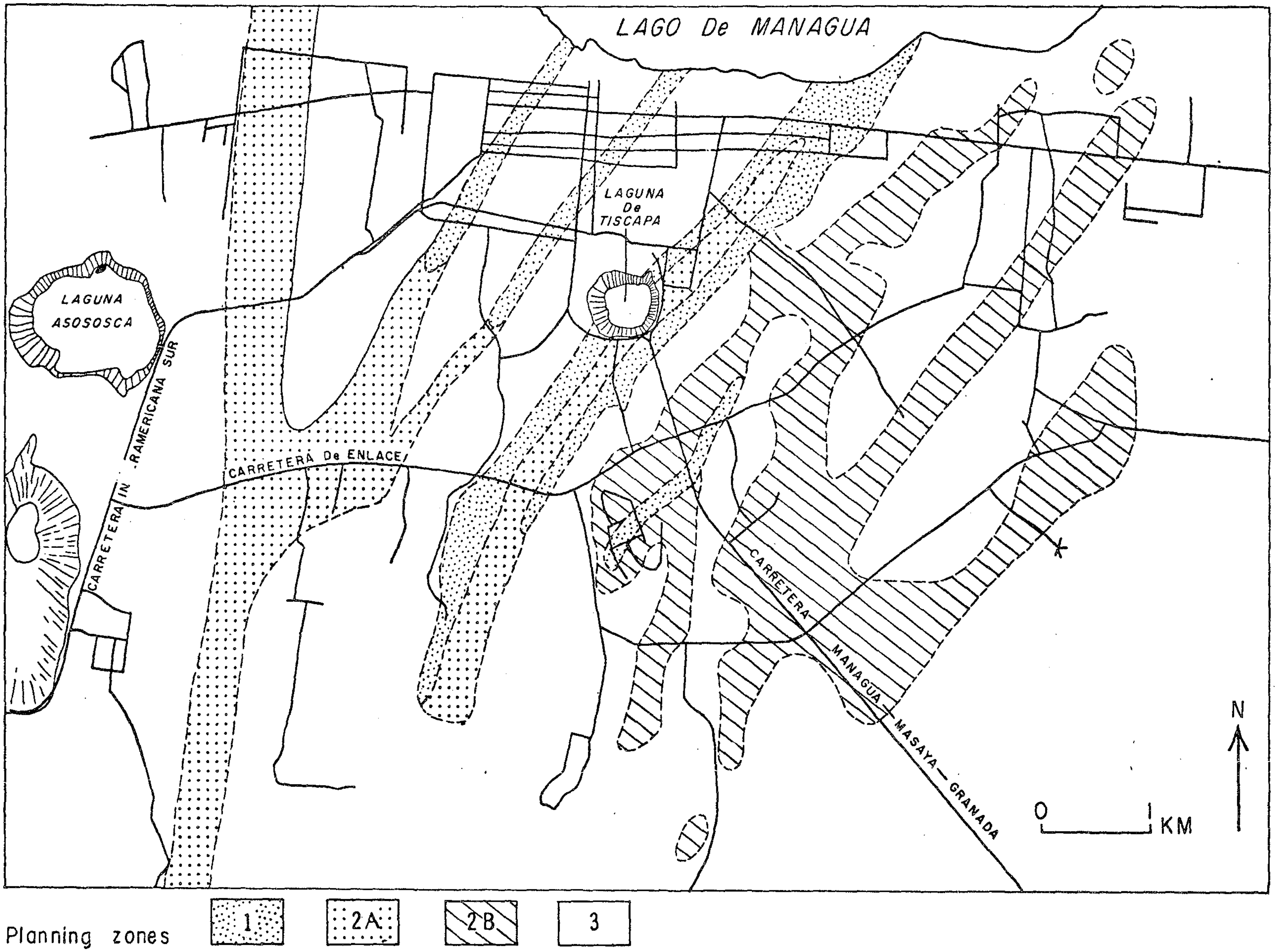


Figure 2-8 Map of Managua, Nicaragua, showing planning zones for the reduction of hazards from surface faulting
From Wallace, 1973

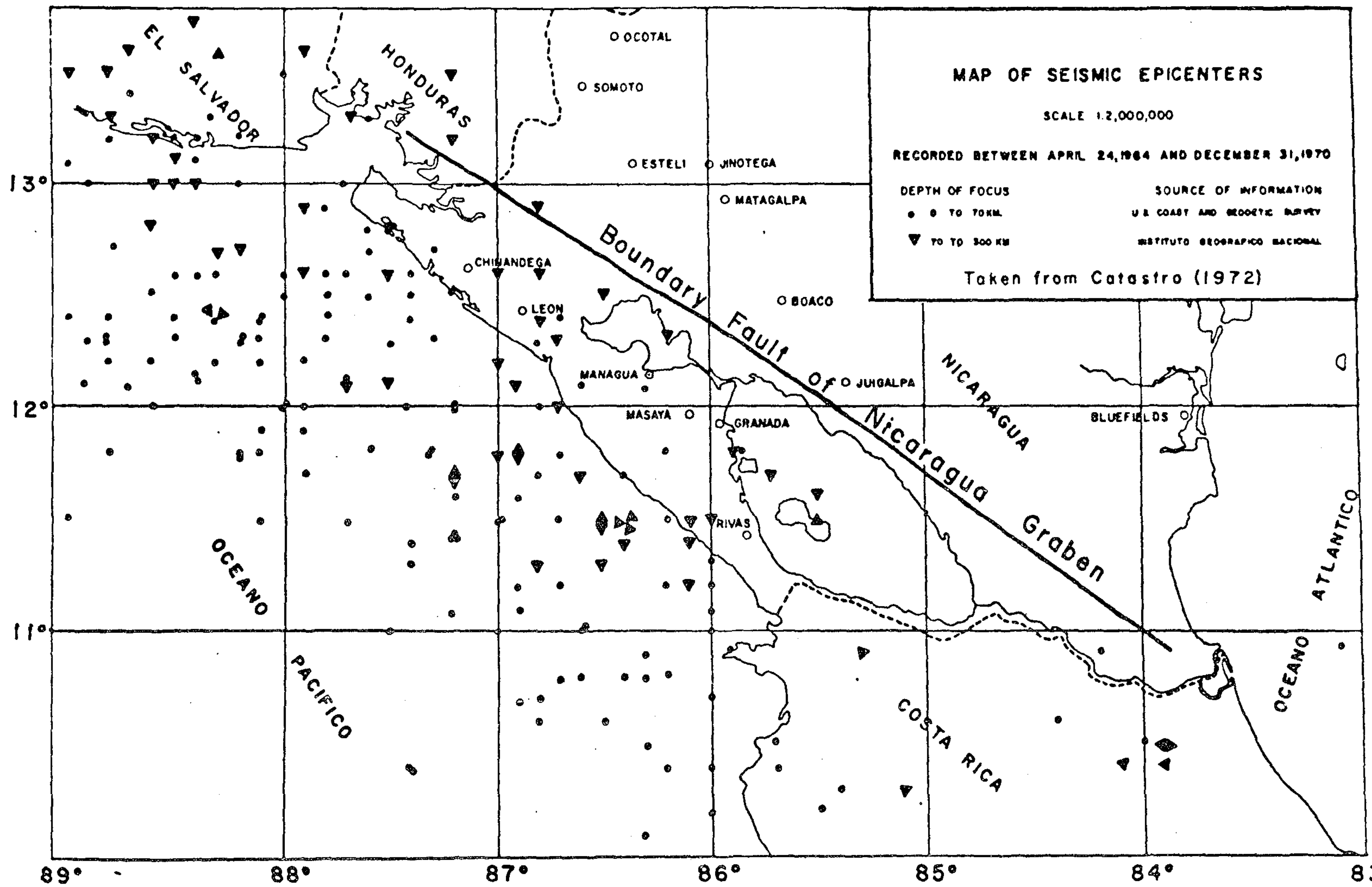


Figure 2-9-Seismic Map of Part of Nicaragua. The epicenters shown are from 1964 to 1970 as calculated by the U.S. Coast and Geodetic Survey. It is important to note that only one appears north of the boundary fault in Nicaragua and that one is deeper than 70 kms. The most active area lies offshore. It must be realized that although an area may appear to be earthquake free for a long time that large earthquakes may occur in areas thought to be devoid of them because of a total lack of activity in the area. Nonetheless, by picking the areas shown to be free of earthquakes for a protracted period, one is indeed reducing the overall hazard.
From Saint-Amand, 1973

CHAPTER III

SEISMIC DATA BASE

Introduction

In Chapter 2, we discussed general geology of Nicaragua. We also discussed the geologic hazards that planners and builders should consider. One major informational parameter needed in any future planning of a facility in a seismic region is the amount of shaking or vibration that this facility will have to undergo during its economic life. In other words, one has to consider the future seismic dynamic loads for which the structures should be designed. Such information helps in seismic zoning of a region. There are various parameters used in the literature to represent the seismic loading. They are:

(1) Richter Magnitude (M);

(2) Modified Mercalli Intensity (MMI);

(3) Peak ground acceleration (PGA);

(4) Spectral Intensity;

(5) Root Mean Square (RMS) acceleration, velocity, or displacement.

However, the most commonly used loading parameters are the Richter Magnitude, the Modified Mercalli Intensity, and the peak ground acceleration. As for the Richter Magnitude, the loading information is in the form of overall energy release of a seismic event. It does not explicitly represent a loading at a given site some distance away from the source of energy release.

The Modified Mercalli Intensity scale represents the effect of an earthquake at a given site. It is a subjective scale of damage at a site. Thus, for a given seismic event in a region, various sites experience different intensities. In general, intensity decreases with distance. Appendix 2 gives the MM Intensity scale. Future forecasting of MM intensities for a given region help in determining the future damage potential and hence insurance risk. However, for structural engineering and design purposes, this parameter is not as useful as the peak ground acceleration. The most commonly and conveniently used parameter is the peak ground acceleration. In this work, peak ground acceleration (PGA) will be used to represent the seismic load level. For frequency content, normalized design spectra will be developed for different parts of the country. (See Chapter 7.)

To estimate probabilistically these peak ground acceleration levels throughout the country in some time frame, we need to get information regarding past seismic events. In particular, we need to get the following information:

(1) Epicentral locations of past seismic events;

(2) Time of occurrence;

(3) Magnitude associated with each occurrence;

(4) Depth of hypocenter;

(5) Acceleration records associated with the above occurrences at different sites;

(6) If possible, information on how energy (or peak ground

acceleration) attenuates from source of energy release to any site away from the source.

As for items 5 and 6 above, not much data is available for Central America in general and Nicaragua in particular. We will discuss these two items in detail in the next two chapters. As for the information on epicentral locations, time of occurrence, magnitude of occurrence and depth of hypocenter, the basic data were obtained from the National Earthquake Information Center, U.S. Department of Commerce, National Oceanic & Atmospheric Agency, National Earthquake Information Center, Boulder. The information obtained from this source contained all events from 1900 to 1973. Another source that was consulted for earthquakes before 1900 was the "Catalog of Nicaraguan Earthquakes 1520-1973" by David J. Leeds of Dames & Moore, Los Angeles. The list of references at the end of this report gives other sources used to develop the total data base (see references 21, 22, 23, and 24).

Before we go into the discussion on the use and analysis of available data, certain observations should be made regarding the type, amount and reliability of the data base used for the current research.

All the data sources have one common shortcoming. That is, the frequency of recorded earthquakes increases with time. It is very realistic to assume that the seismic phenomenon in Nicaragua has not drastically changed in the past few hundred years and that it will remain the same for a few hundred years more. However, the number of seismic events recorded in general increases with each year. Also, only major events were recorded in the 16th, 17th, and 18th centuries.

This gives a bias to the data because in recent years, all events, big and small, are recorded, whereas old records only show large events. This nonhomogeneity in data reliability is a "fact of life" and we can not get away from it. Another problem is in the events which were not recorded as to time, place and magnitude, but only conveyed through church records or through word-of-mouth. How can one incorporate such information quantitatively?

Fortunately, for well-codified methods and structures, the structural performance and consequences in general follow the pattern shown in figure 3-1. The horizontal axis represents structural performance such as, say, deformation, whereas the vertical axis represents the consequences of those performances. The performance can be tied in with the seismic demand (or loading). Thus, a 10 percent variation about the mean demand value D can be represented by D_1 and D_2. Corresponding to this variation, the performance variations could be 10% about the mean performance P, represented by P_1 and P_2. However, this 10 percent variation in loading estimate may result in only a slight variation on the consequence side. This ability of a well designed structure based on a well designed code helps in overcoming the uncertainty in the loading parameter such as peak ground acceleration. Further discussion on this aspect will be presented in Chapter 7 of this report. Also, inclusion or exclusion of some unrecorded historical events in the past does not change the estimated values of the loading parameters substantially, because the estimates are based on a large collection of data to begin with.

Due to above mentioned considerations, the authors of this

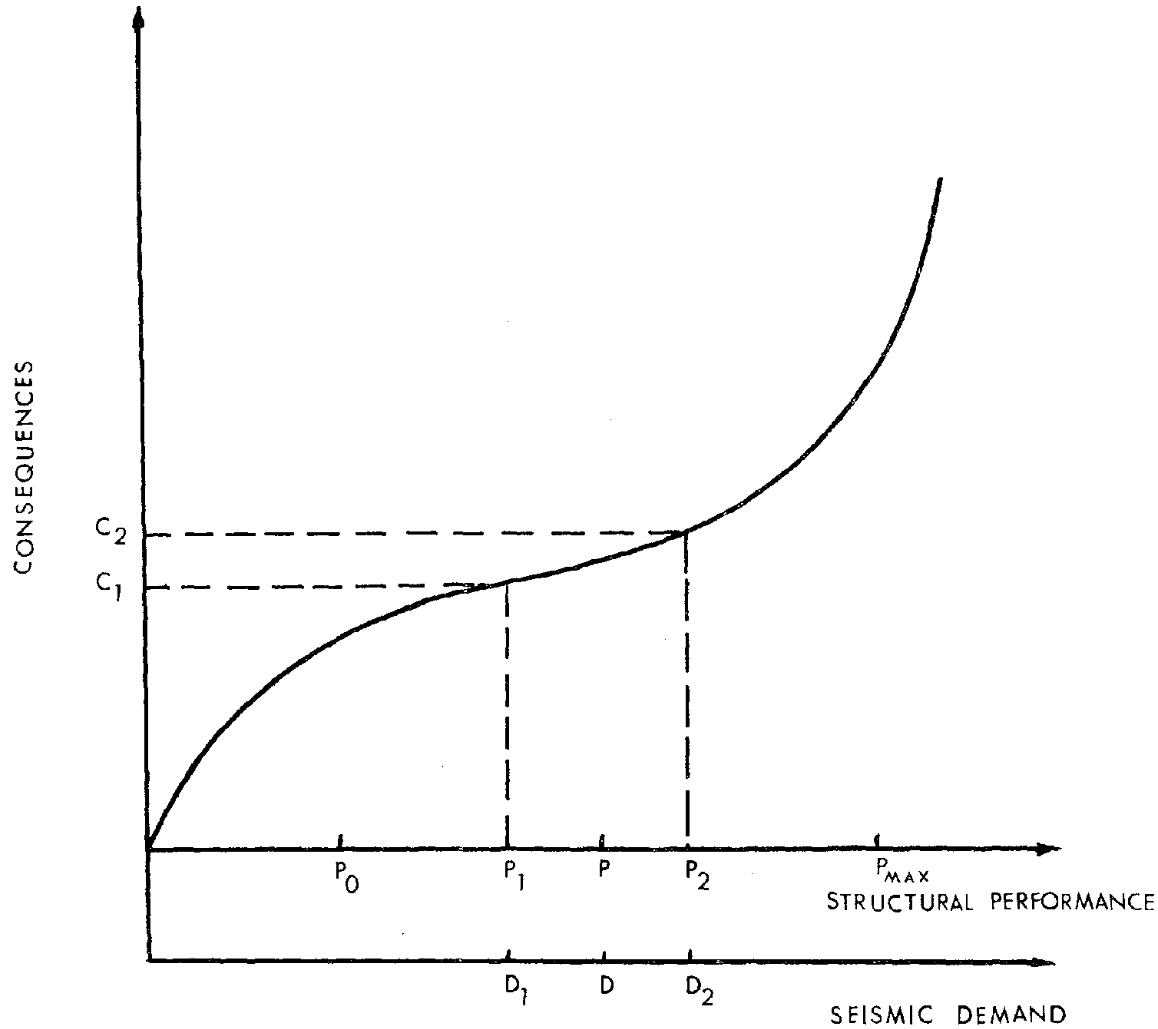

CONSEQUENCES
C2
C1
P0
P1
P
P2
PMAX
STRUCTURAL PERFORMANCE
D1
D
D2
SEISMIC DEMAND

FIGURE 3-1

report feel that the seismic loading estimates, based on probabilistic analysis, are realistic and representative of the future seismic loadings.

Data Analysis

As mentioned previously, two main sources of information were considered. The NEIC-NOAA data file covering the period from January 1900 to August 1973 constituted the primary source of information and is referred to hereafter as Source 1. The Catalogue of Nicaraguan Earthquakes, 1520-1973, by David J. Leeds, is referred to as Source 2. It was used to obtain:

. data about earthquakes associated with volcanic activity along the Cordillera de los Marrabios (1850-1973);

. data about earthquakes not reported in the NEIC-NOAA file (1900-1973);

. additional information about events incompletely documented in the NEIC-NOAA file (1900-1973).

The time period of data gathering is thus 73 years for the whole country and 123 years for earthquakes associated with volcanic activity along the Cordillera de los Marrabios.

In spite of the complementarity of the two sources, a large number of events remained insufficiently documented to be used as such in the analysis. Rather than disregarding these events, the missing information was generated using a Monte Carlo Process supplemented by judgment. It is felt that the total analysis benefits more than suffers from such an additional input.

The following remarks are valid for both sources:

. No critical study was made regarding the validity of the information and the reliability of the data.

. Whenever information as basic as epicentral location or magnitude were missing, the event was disregarded.

. Events with Richter Magnitude smaller than 3.0 were not considered.

Source 1

When complete, the information contained in this source includes for a given event: time of occurrence, epicentral location (degree), depth of hypocenter (km), and magnitude. The magnitude is in terms of one of the following:

(1) CGS M_b average (body wave magnitude)

(2) CGS M_s average (surface wave magnitude)

(3) Richter Magnitude M.

The acceleration attenuation relationships used in Chapter 4 are based on the Richter Magnitude. Hence, when missing, this information was generated from M_b or M_s. It is known that for a given part of the world, the Richter Magnitude and CGS M_b are linearly related such that

$$M = a + b\ M_b \qquad 3\text{-}1$$

In order to determine the coefficients a and b, a regression analysis was run for all the earthquakes of which M and M_b were known using the total data of Central America. The Richter Magnitude was then obtained by substituting the value of M_b in equation 3-1.

Whenever data on depth of hypocenter were not available, a depth was assigned, as will be explained later in the chapter.

From Source 1, 419 events contained complete information; they are plotted in Chart 1 and shown as a function of depth in Table 3-1.

Source 2

When complete, the information contained in this source includes for a given event: time of occurrence, epicenter location (degree), depth, Richter Magnitude and sometimes a short description of the seismic event. The depth is either expressed in km or by a letter symbol N (0 - 60 km) or I (70 - 200 km). In the same way, the Richter Magnitude is either expressed by its numerical value or by a letter symbol, as follows:

B - $7 \leq M \leq 7.7$

C - $6 \leq M \leq 6.9$

D - $5.3 \leq M \leq 5.9$

E - $M < 5.3$

Through a simulation process, all the events taken from Source 2 were assigned a numerical Richter Magnitude from letter magnitude.

Hence, an additional 196 events were obtained (including events from Source 1 with partial information), distributed as follows:

43 events associated with volcanic activity and with shallow hypocenters N (0 - 60 km).

40 events with shallow hypocenters N (0 - 60 km).

3 events with deep hypocenters I (70 - 200 km).

63 events with no data on depth.

47 events with numerical data on depth (km).

Table 3-1

Data from Source 1, Sorted According to Depth of Hypocenter

(Total Events 421)

Number of Earthquakes	Depth Range (kms.)
8	0- 9
9	10- 19
12	20- 29
159	30- 39
35	40- 49
32	50- 59
34	60- 69
32	70- 79
18	80- 89
14	90- 99
13	100-109
9	110-119
3	120-129
7	130-139
3	140-149
7	150-159
3	160-169
6	170-179
3	180-189
5	190-199
9	200-215

The 466 earthquakes with complete data (419 from Source 1 and 47 from Source 2), were plotted as a function of depth. Using those plots together with epicenter location, magnitude value, partial information on depth and judgment, the 156 remaining events were assigned appropriate depths. This led to a total data of 615 events ranging from 5 to 215 km in depth and from 3.0 to 7.7 in magnitude. Appendix 3 gives the listing of those earthquakes; in Chart 2-7 they are plotted as a function of depth.

From these charts, the general seismic pattern of Nicaragua can be divided into the following regions:

(1) The Benioff Zone dipping North East toward the Nicaraguan coast. This zone is marked by numerous earthquakes covering the whole range of magnitude (larger as depth increases) and extending several hundred kilometers into the earth's interior. The shallow earthquakes ($\pm$ 30 km) due to this source are from 30 to 100 km away from the coast. As the epicenters get closer to the coast, the hypocenters get deeper. Hence, under Managua the hypocenters of earthquakes situated on the Benioff Zone are very deep (100 - 200 km).

(2) In contrast, for the local seismic sources, such as the ones identified under Managua (Figures 2-3 and 2-6, Chapter 2), the hypocenters are shallow (5 - 30 km). In magnitude scale, these sources do not generate major earthquakes such as those on the Benioff Zone. However, due to their shallowness and nearness to populated areas, they have caused extensive damage and loss of life in past history. The

December 23, 1972 event was due to the local source under Managua. Appendix I gives details regarding this source of seismic activity.

(3) The line of volcanoes from Northwest to Southeast (Cordillera de los Marrabios) represents sources of future seismic activities. Volcanic eruptions are seldom by themselves sources of seismic activity, and in the past various earthquakes have been recorded preceding volcanic eruptions. For this reason earthquakes "associated" with volcanic activity were treated in the model (Chapter 4) as shallow tectonic earthquakes.

(4) Two shallow (± 30 km) seismic regions, one more or less coinciding with the Pacific seashore between Lake Managua and the Costa Rica border, the other one in the Gulf of Fonseca.

(5) The Atlantic coast of low seismicity.

Source Location and Seismicity

Based on the above observations, the total number of events was divided into 13 seismic sources: Ten of these are line sources and three are area sources. Table 3-2 shows these 13 sources, the number of events and the depth range of each source.

Appendix 3 gives a listing of the earthquakes included in each source. Line sources were located by fitting a line through the data using regression analysis. For area sources, the centroid was obtained from the data and the radius taken as the distance from the centroid to the most distant epicenter in the source.

The depth of each source was computed as an average hypocentral

Table 3-2

Seismic Sources for Nicaragua

Source	Number of Events	Name of Source	Depth (kms.)
1 Line	159	Benioff	5 - 39
2 Line	186	Benioff & Costa Rica	40 - 79
3 Line	72	Benioff	80 - 109
4 Line	31	Benioff	110 - 159
5 Line	41	Benioff	160 - 215
6 Line	23	"Costa Rica"	5 - 39
7 Line	11	Atlantic	All Depths
8 Line	12	Pacific Coast Line	33
9 Line	57	Line of Volcanoes	33
10 Line	57	Line of Volcanoes	33
11 Area	5	Managua Area	5
12 Area	8	Gulf of Fonseca	33
13 Area	10	Costa Rica Area	80 - 109

depth of all the earthquakes included in the source. Earthquakes with no or limited depth information were not included in this averaging process. However, they were considered in determining the location and the seismicity of the source. Charts 2 through 7 show the source locations and depths. The recurrence relationship for each individual source was obtained by fitting a regression line of the following form:

$$\ln_e N(M) = \alpha + \beta M \qquad 3\text{-}2$$

$N(M)$ = Number of events above magnitude M

M = Richter Magnitude

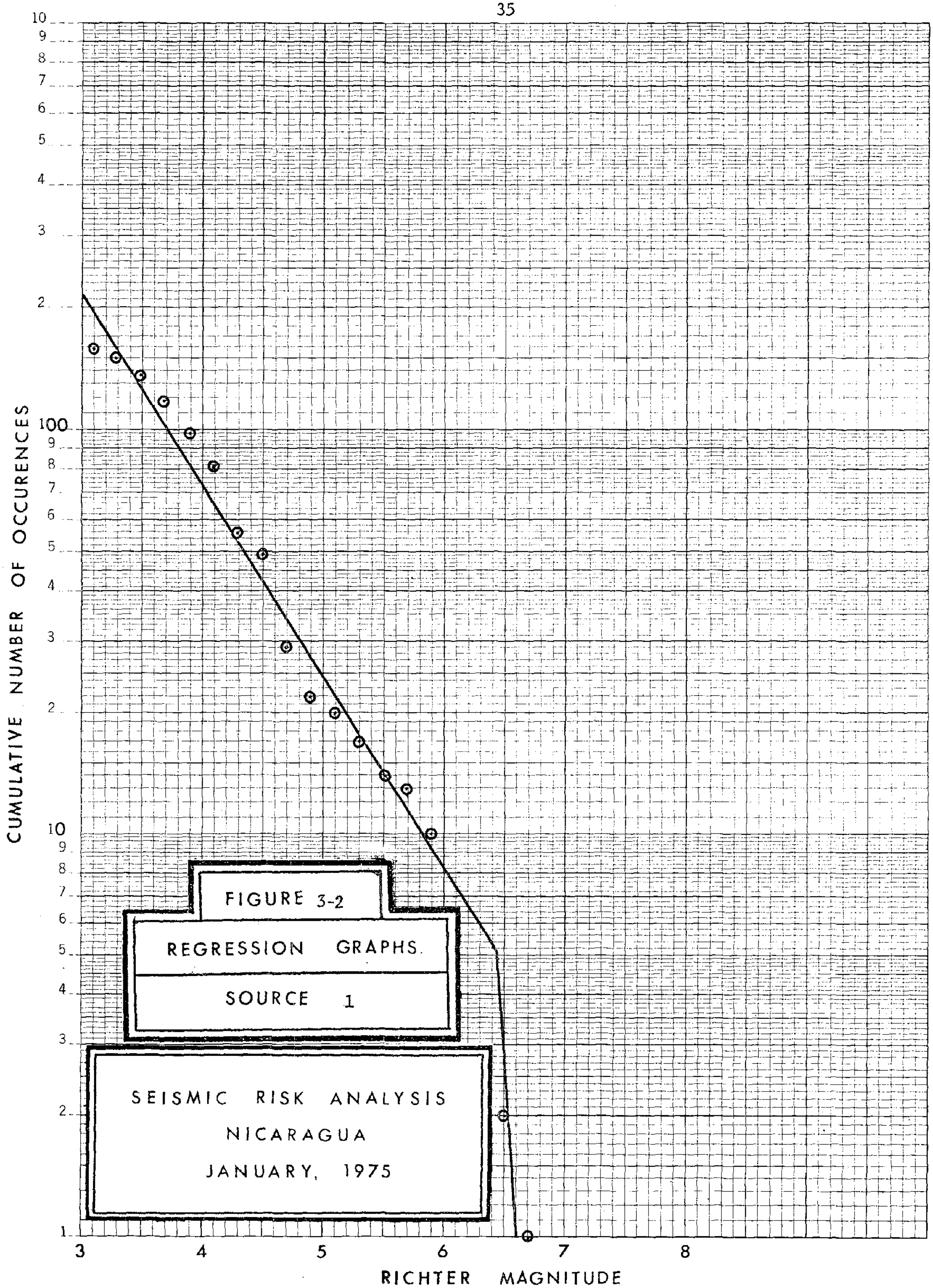
α and β are regression constants.

α is a measure of the number of events above magnitude 0 for a given source and β is a measure of the seismic severity for a given source. The larger the negative value of β, the smaller the seismic severity. For many sources, a single regression line gave erroneous results because the interpolation of the line beyond the range of data indicated unreasonably high magnitude occurrences. For such cases, two regression lines were fitted to the data, and a geologically consistent upper magnitude value was used for cutoff. (See Figures 3-2 through 3-13.) Table 3-3 gives a summary of α' and β values for each source and the magnitude cutoff point corresponding to $\ln N(M) = 0.1$.

$$\text{Let } N'(M) = \frac{N(M)}{AT} \text{ for area source} \qquad 3\text{-}3$$

$$= \frac{N(M)}{LT} \text{ for line source}$$

where L = length of the line source



CUMULATIVE NUMBER OF OCCURENCES
RICHTER MAGNITUDE
FIGURE 3-2
REGRESSION GRAPHS
SOURCE 1
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

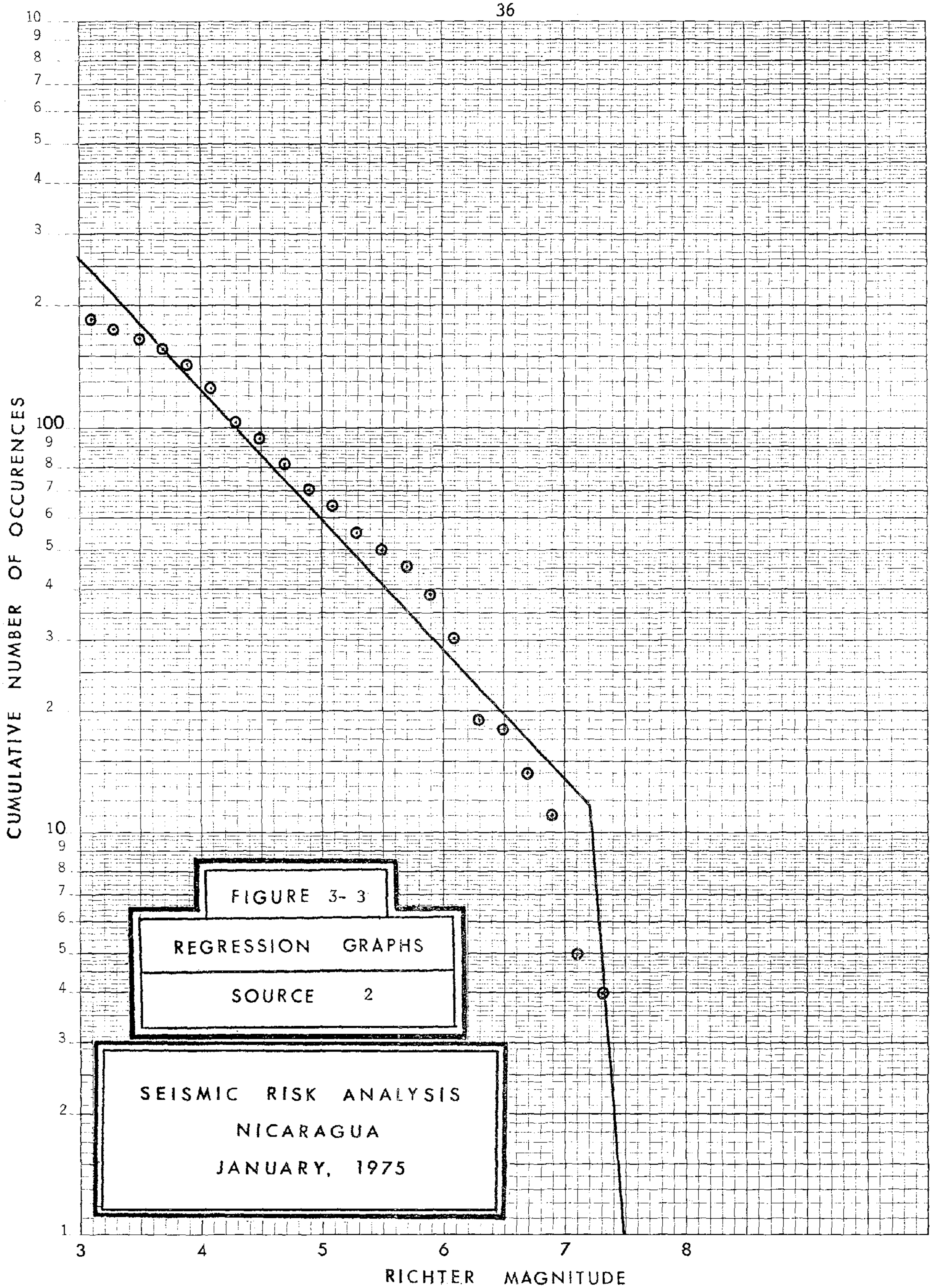
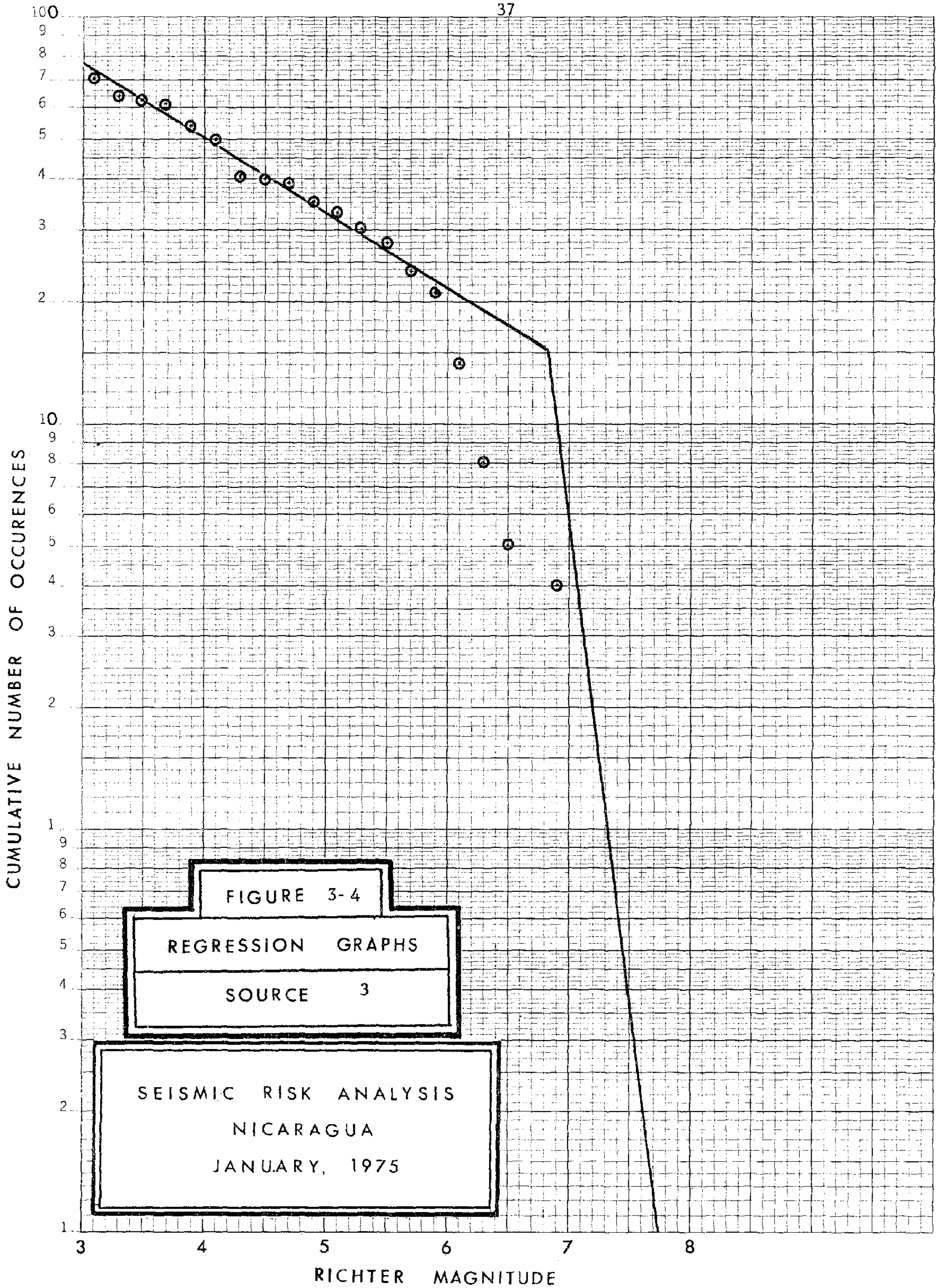


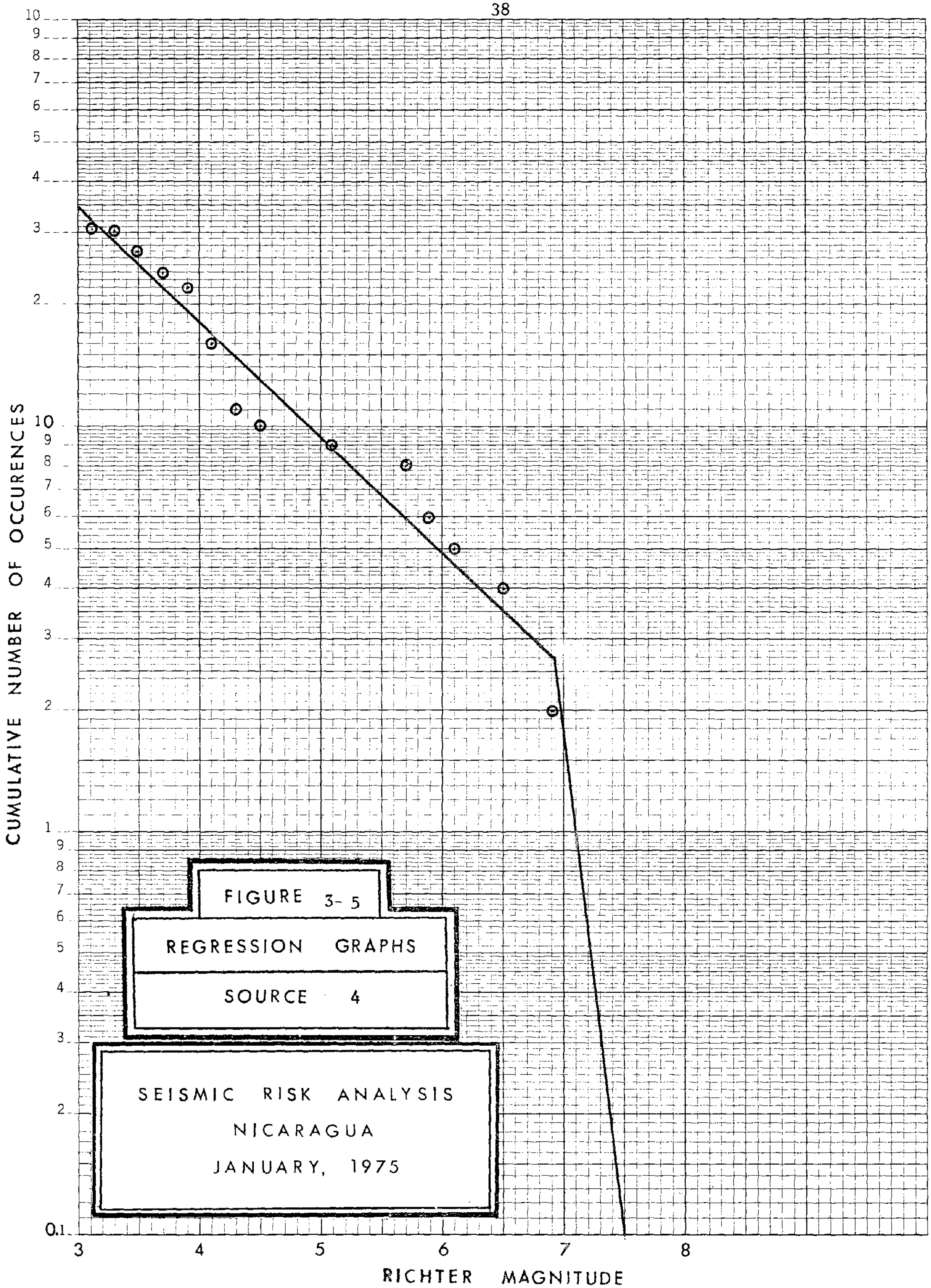
FIGURE 3- 3

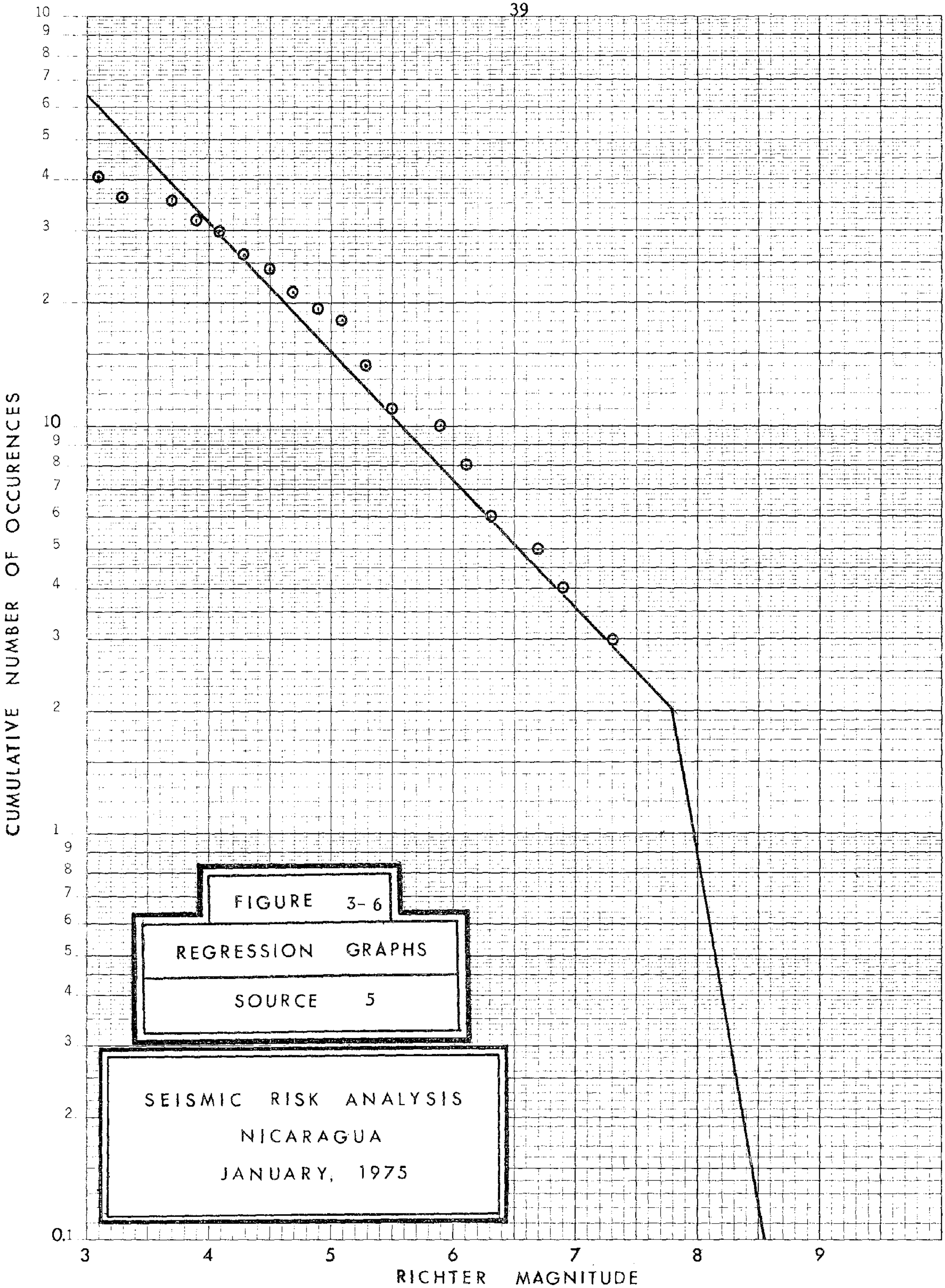
REGRESSION GRAPHS

SOURCE 2

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975


CUMULATIVE NUMBER OF OCCURENCES
RICHTER MAGNITUDE
100
10
1
3
4
5
6
7
8
FIGURE 3-4
REGRESSION GRAPHS
SOURCE 3
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975


CUMULATIVE NUMBER OF OCCURENCES
RICHTER MAGNITUDE
FIGURE 3- 5
REGRESSION GRAPHS
SOURCE 4
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975


CUMULATIVE NUMBER OF OCCURENCES
RICHTER MAGNITUDE
10
9
8
7
6
5
4
3
2
1
0.1
3
4
5
6
7
8
9
FIGURE 3-6
REGRESSION GRAPHS
SOURCE 5
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

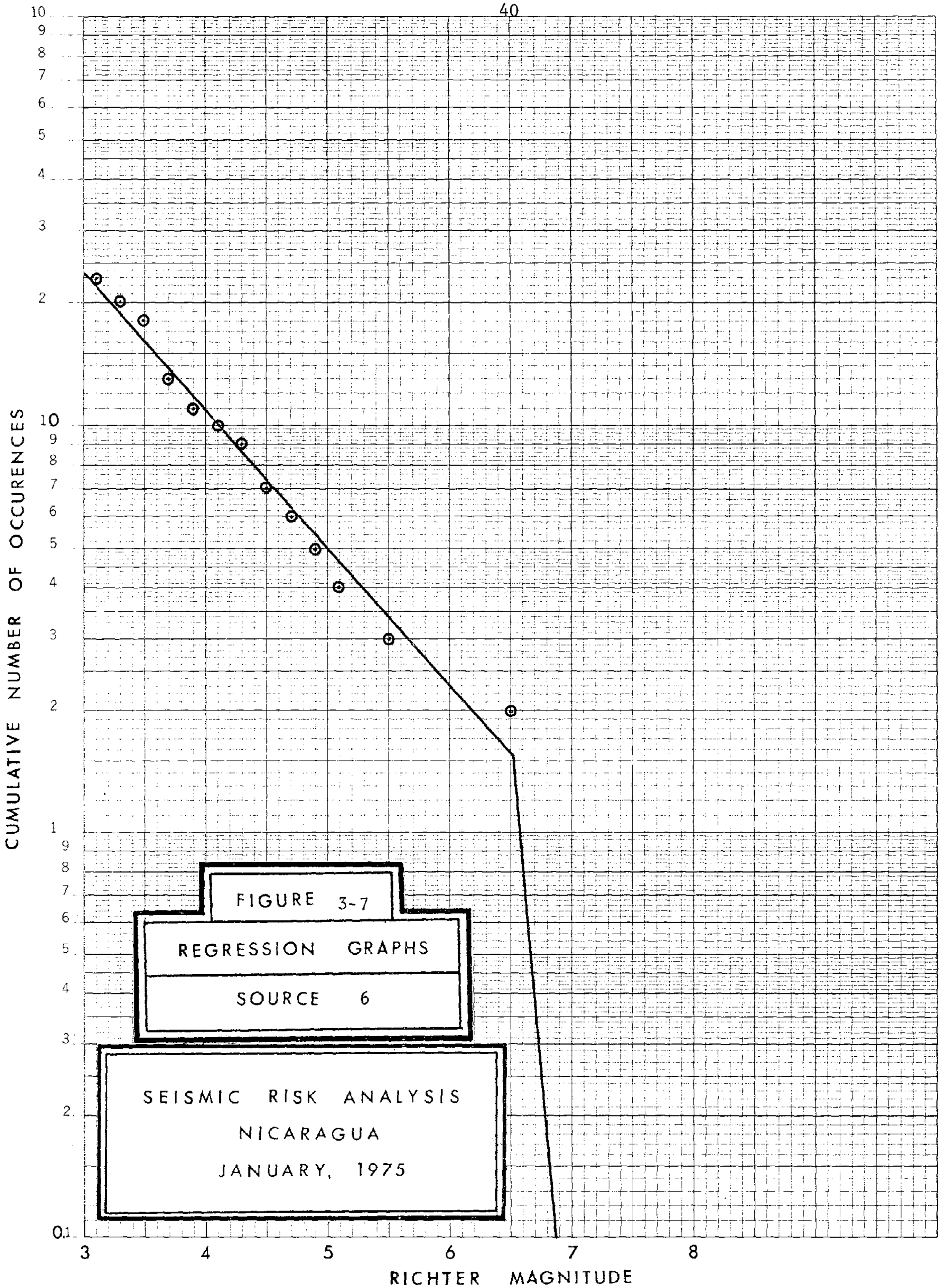


FIGURE 3-7

REGRESSION GRAPHS

SOURCE 6

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

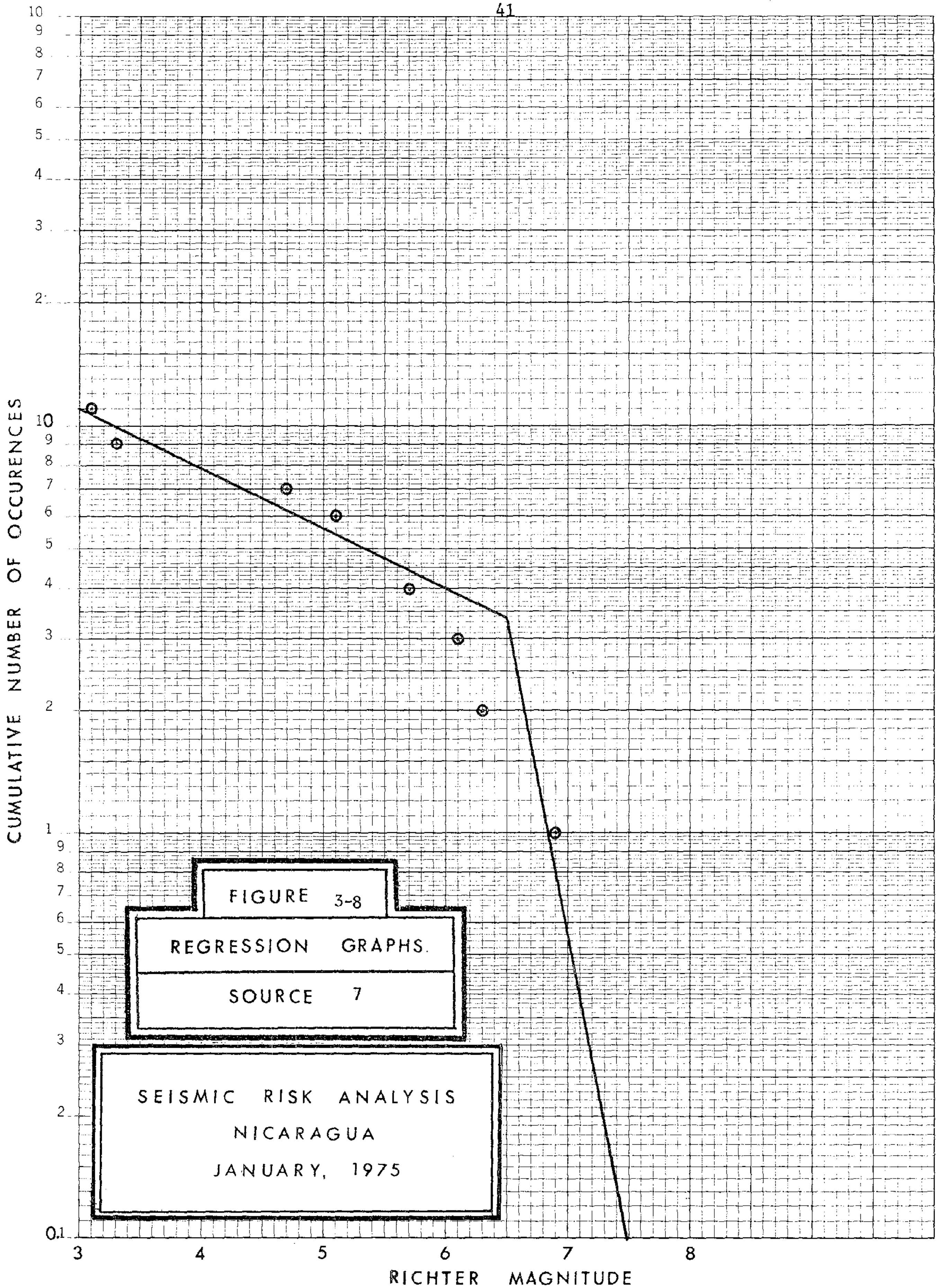


FIGURE 3-8

REGRESSION GRAPHS.

SOURCE 7

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

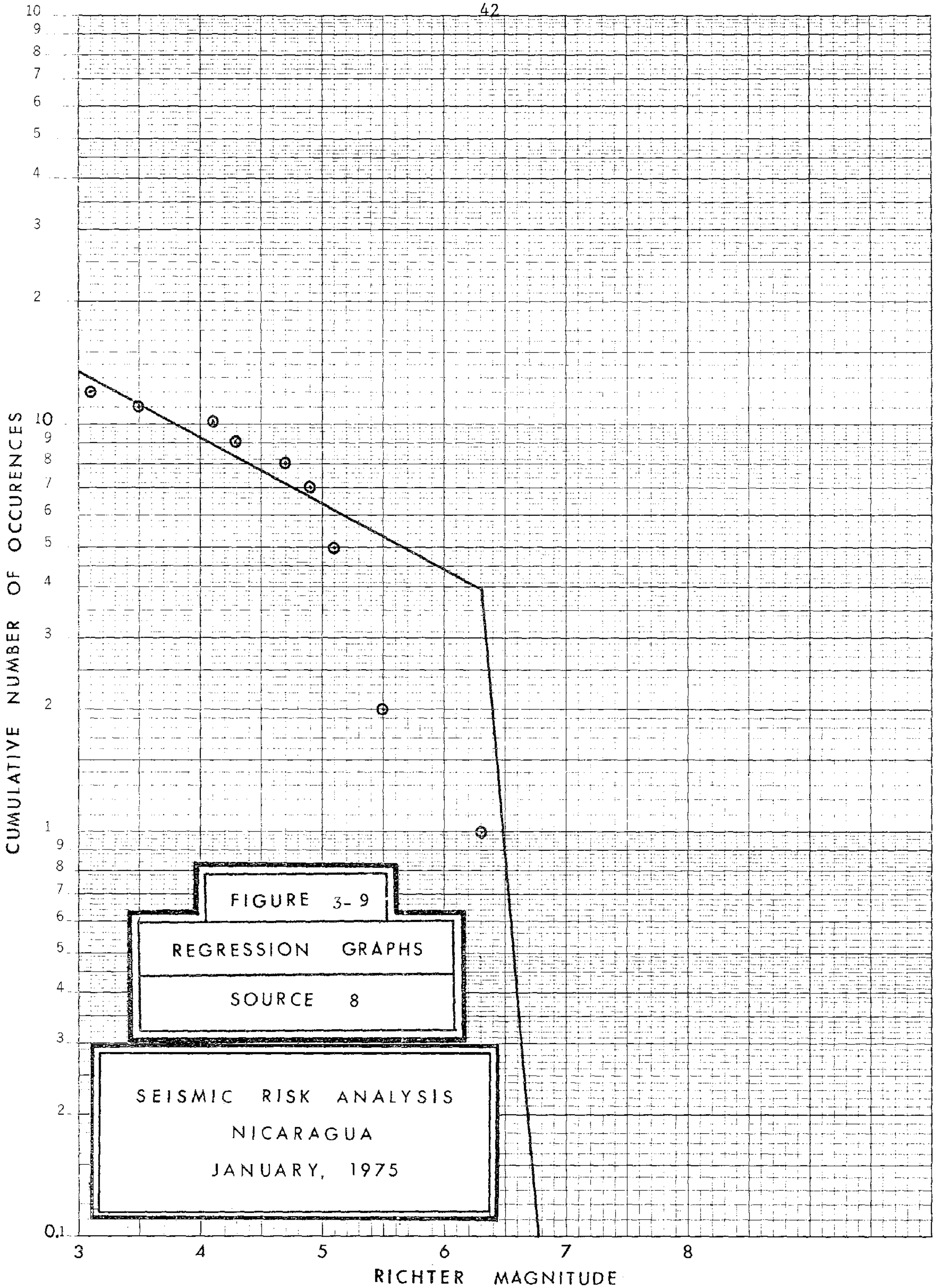
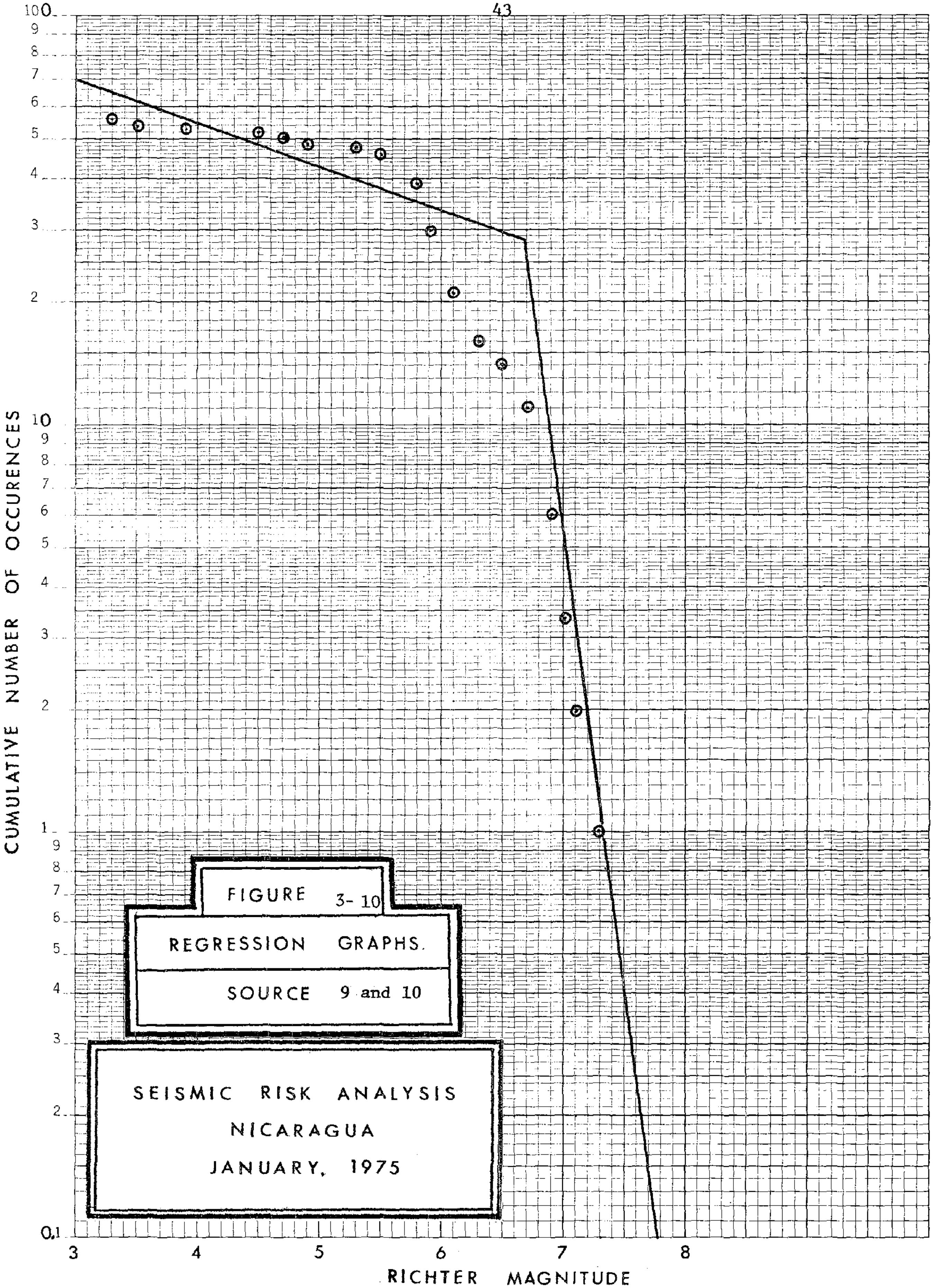


FIGURE 3-9

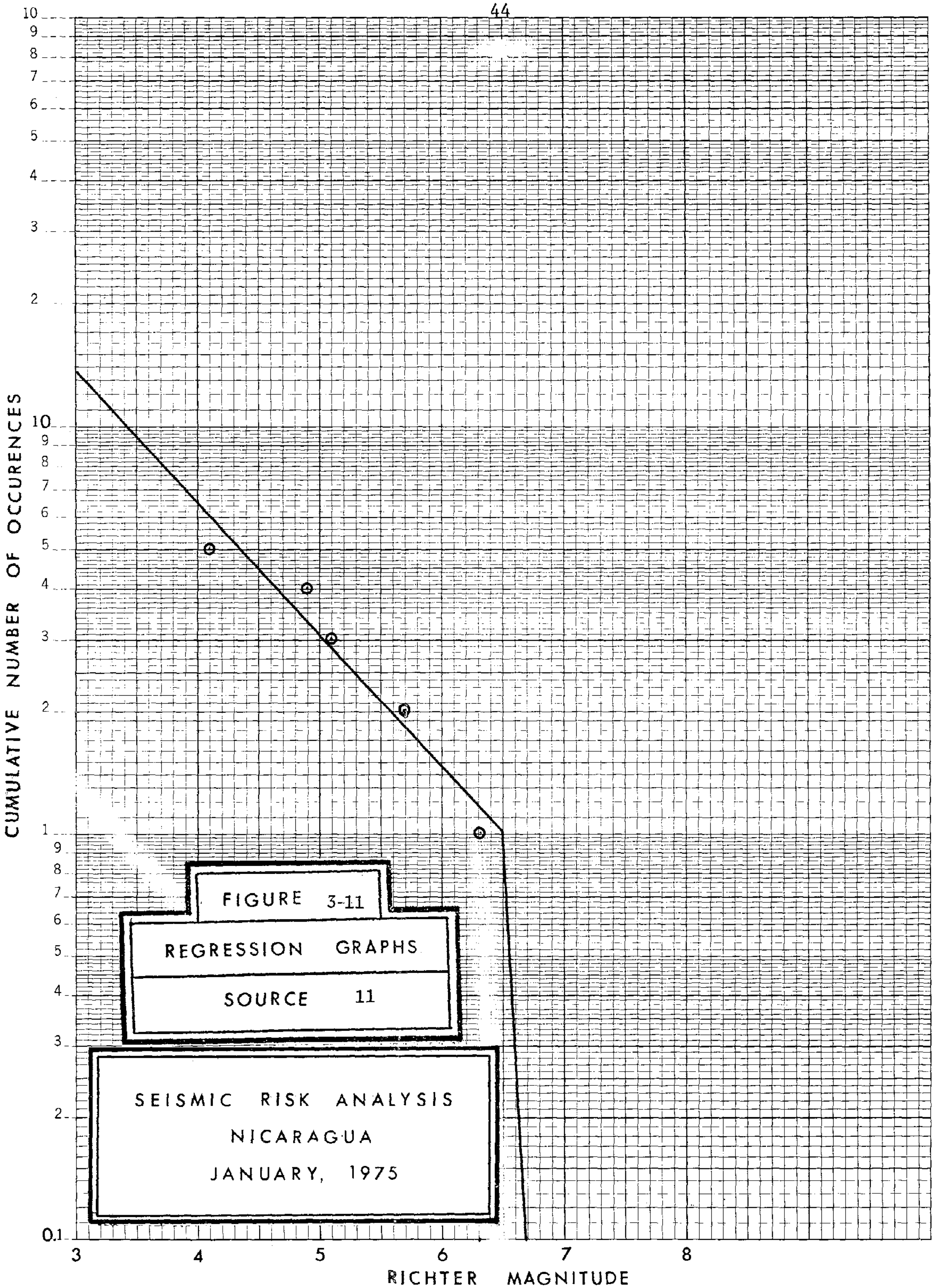
REGRESSION GRAPHS

SOURCE 8

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975



CUMULATIVE NUMBER OF OCCURENCES
100
10
1
0.1
3
4
5
6
7
8
RICHTER MAGNITUDE
FIGURE 3- 10
REGRESSION GRAPHS.
SOURCE 9 and 10
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975


CUMULATIVE NUMBER OF OCCURENCES
RICHTER MAGNITUDE
10
9
8
7
6
5
4
3
2
10
1
0.1
3
4
5
6
7
8
FIGURE 3-11
REGRESSION GRAPHS
SOURCE 11
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

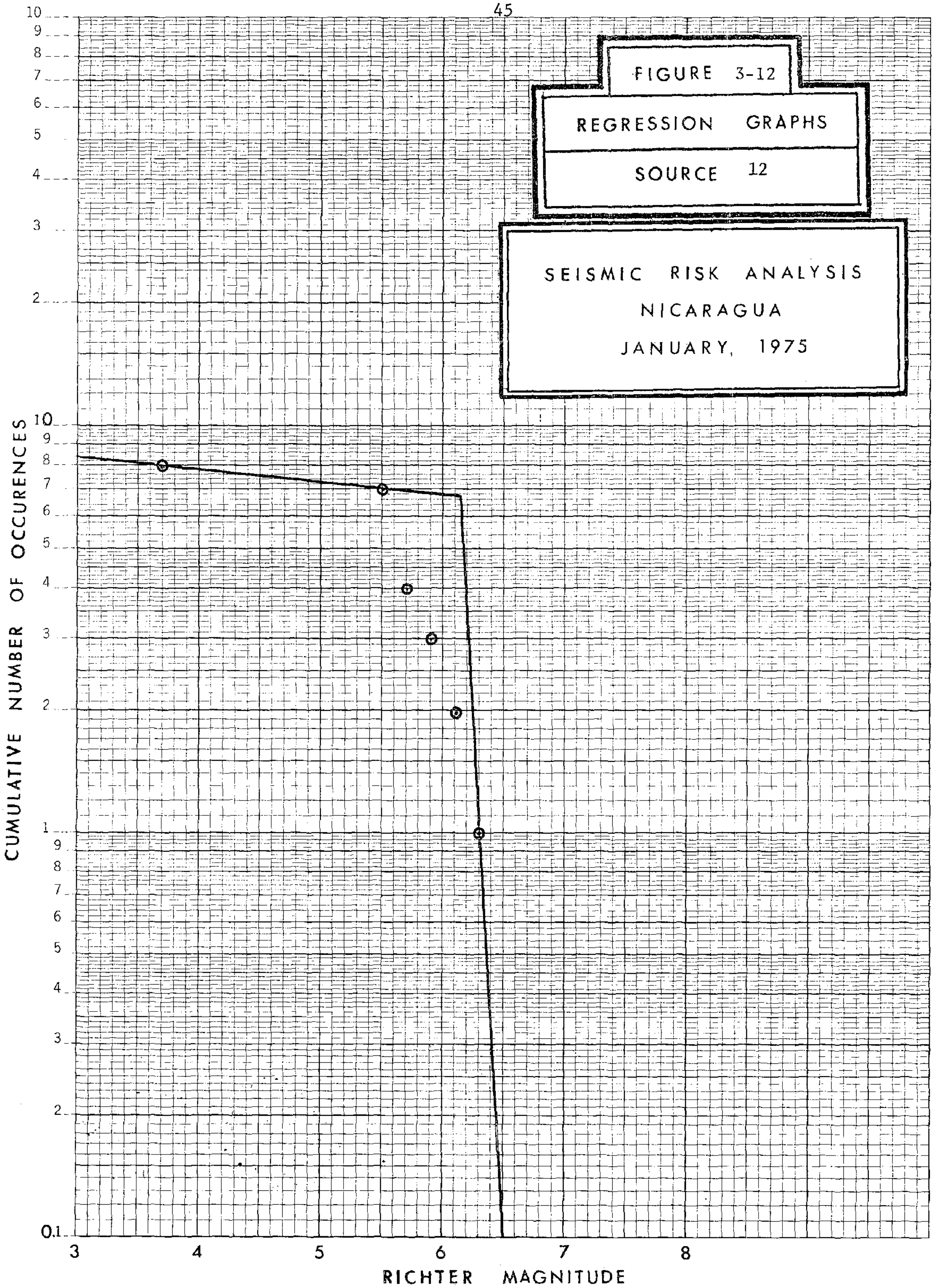


FIGURE 3-12
REGRESSION GRAPHS
SOURCE 12
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
CUMULATIVE NUMBER OF OCCURENCES
RICHTER MAGNITUDE
10
1
0.1
3
4
5
6
7
8

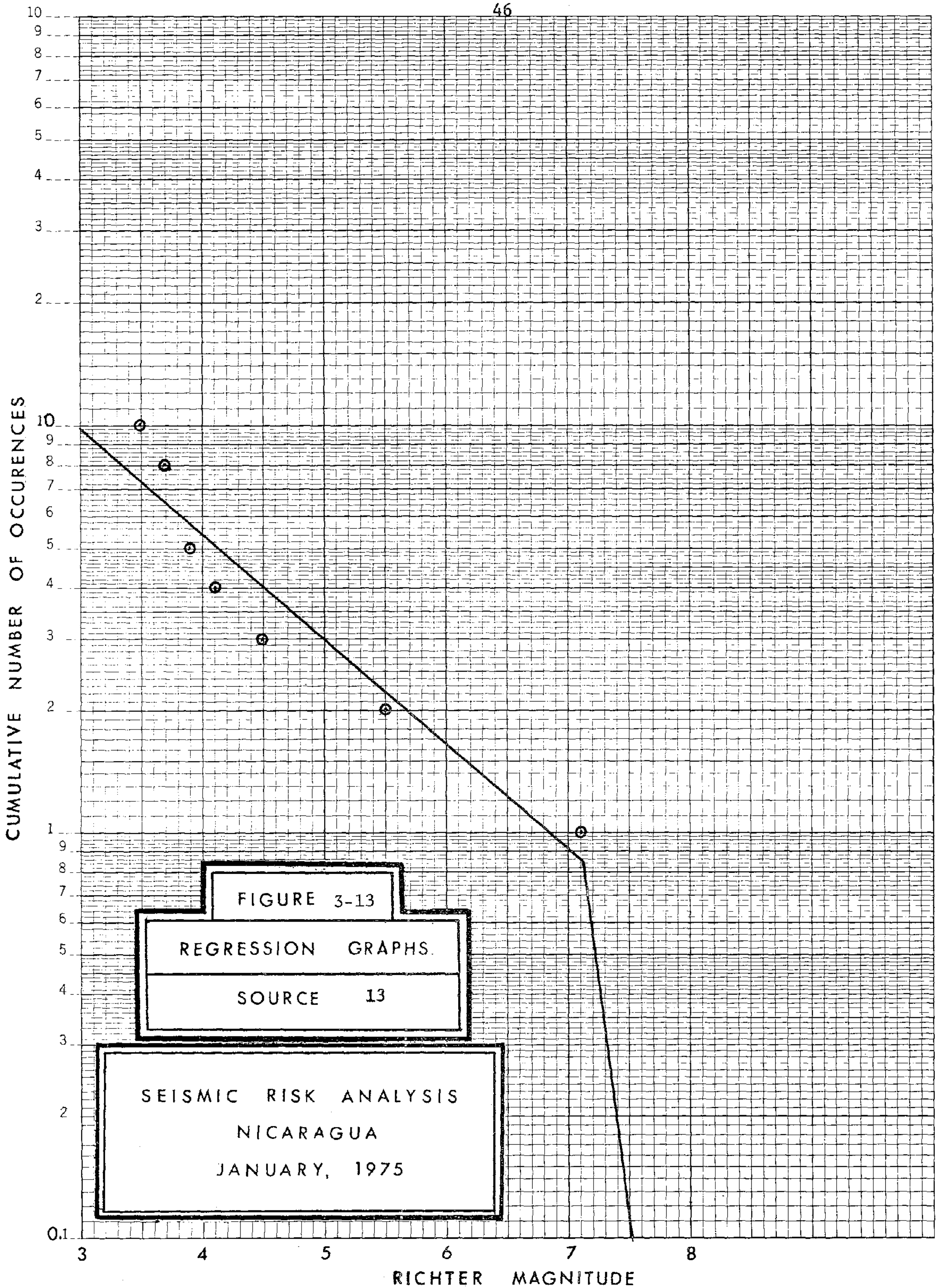


FIGURE 3-13

REGRESSION GRAPHS

SOURCE 13

A = Area of the area source

T = time for which data was obtained

$N'(M)$ = Normalized mean number of events above magnitude M for unit-time (1 year) and unit-area or unit length.

Then

$$\ln N'(M) = \alpha' + \beta M \qquad 3\text{-}4$$

where $\alpha' = \alpha - \ln(AT)$ for area source

$= \alpha - \ln(LT)$ for line source.

Table 3-3 shows values of α', β and the upper cutoff magnitude as described previously. The table gives values of α' and β in terms of degrees of latitude and longitude. These relationships will be used to develop the forecasting model in Chapter 4.

Limitations

In conclusion, it can be said that there are limitations to the use of available data for the Nicaragua region. These limitations are given below.

1. 24% of the data contain incomplete information regarding the depth. This information was added from either judgment or by correlating the event with other data where the depth information was available.
2. 32% of the data have magnitude defined by a symbol. Numerical value of magnitude for these cases was obtained through simulation.

Table 3-3

Source	α_1'	β_1	α_2'	β_2	Cutoff
1.	2.58	-1.09	24.00	-4.55	6.8
2.	1.49	-0.74	62.80	-9.21	7.8
3.	-0.38	-0.42	3.60	-5.75	7.7
4.	-0.39	-0.65	26.50	-4.55	7.5
5.	0.33	-0.72	36.20	-5.27	8.5
6	0.42	-0.77	46.50	-7.82	6.9
7.	-2.13	-0.33	18.60	-3.53	7.5
8.	-0.89	-0.37	43.10	-7.57	6.8
9.	-4.71	-0.24	34.20	-5.43	7.8
10.	-4.71	-0.24	34.20	-5.43	7.8
11.	3.17	-0.74	79.15	-12.4	6.7
12.	0.14	-0.07	79.90	-13.04	6.5
13.	-0.66	-0.59	34.60	- 5.54	7.5

3. The reliability of the total data base was not evaluated.

   (1) Some information was from church and historical records.

   (ii) Distribution of information over the country is biased. Populated areas have better records than sparsely populated areas. (No population $\rightarrow$ no records.)

   (iii) Epicentral location could be in error due to lack of a good grid of recording system. It is hoped that the recording network presently installed by the Nicaraguan authorities, the U.S.G.S. and private organizations in Nicaragua will help in increasing the understanding of attenuation relationships and the accuracy of epicentral locations in the future. Such calibration may help in relocating the past events. (See reference 4 by Dewey.)

It is felt that the work done by Dewey (see reference 4) and others in calibrating the epicentral locations through the ESSO refinery record does not have sufficient experimental evidence as yet. Hence, no hypocenters are moved based on Dewey's work. (One exception is the 1931 earthquake-stadium fault.) It should be emphasized that as additional data become available to give more reliable information on epicentral locations, the methodology presented in this research project will be able to modify the results accordingly.

CHAPTER IV

PROBABILISTIC SEISMIC LOADING -- ISO-ACCELERATION MAPPING OF NICARAGUA

Introduction

In Chapter 3, we discussed the data base, the limitations of the available information and the approximations made in using the seismic data of Nicaragua. We also presented the recurrence relationship associated with all the seismic sources for the region. These recurrence relationships give us the mean number of events of magnitude greater than M due to a given source and time period. If the mean number of events above a specified magnitude M is normalized with respect to time and length of the source for line source or area of the source for area source, we get the normalized recurrence relationship. This normalized relationship gives the quantitative statistical seismic recurrence formula for each source. This, however, represents the past seismic history of the region. In developing understanding on the seismic risk for Nicaragua, we need the future forecasting of events. Based on the past data, the future forecasting can be done by means of two widely used statistical models. These models are:

(1) Poisson Model.

(2) Markov Model.

The Poisson Model assumes that major seismic events are spatially and temporally independent. This has been observed to be true for the southern California region. (See reference 25.) The Markov Model assumes memory in two successive seismic events. Thus, occurrence or non-occurrence of an earthquake this year effects the occurrence or non-occurrence of an event next year. Even though this model conforms with the so-called elastic rebound theory, it has been observed that for events with interarrival times of more than 10 years, the Markov Model gives similar results to the Poisson Model. References 26 and 27 are two good examples of using Poisson and Markov Chain Models. In this study, the Poisson Model is used because of its simplicity, its widespread use in literature, and because the results it gives are very similar to results arising from more complex models such as the Markov Chain Model.

Poisson Model of Seismic Occurrences

As mentioned in the previous paragraph, earthquake occurrences can be modeled using the Poisson probability law. For earthquake events to follow the Poisson Model, the following assumptions must be valid:

(1) Earthquakes are spatially independent;

(2) Earthquakes are temporally independent;

(3) Probability that two seismic events will take place at the same place and at the same instant of time approaches zero.

These assumptions are necessary for the formulation of the Poisson Model. The first assumption implies that occurrence or nonoccurrence of a seismic event at one site does not affect the occurrence or non-occurrence of another seismic event at some other site. The second assumption implies that the seismic events do not have memory in time. A Markovian assumption of one-step memory in time may be a better assumption, but as mentioned previously, this assumption for large events does not introduce major errors (see reference 25). The third assumption implies that for a small time interval, Δt, more than one seismic event cannot occur. This is a very realistic and good assumption which fits the physical phenomenon.

In its most general form, the Poisson law can be written as

$$P_n(t) = \frac{e^{-\lambda t} (\lambda t)^n}{n!} \qquad 4\text{-}1$$

where $P_n(t)$ = Probability of having n events in time period t.

n = Number of events.

λ = Mean rate of occurrence per unit of time.

In Chapter 3, we have seen how, using recurrence relationships, we can obtain the mean number of occurrences above Magnitude M for a given source. This relationship in its general form can be written as

$$N(M) = \phi\ (M, A, T) \qquad 4\text{-}2$$

where $N(M)$ = Number of occurrences above Richter Magnitude M.

M = Richter Magnitude.

A = Source characteristic (area for area source, length for line source).

T = Time period of data base.

As mentioned in Chapter 3, a log-linear recurrence relationship is assumed for all sources. Also, for each source, the relationship is bi-linear (two lines described by α_1, β_1 and α_2, β_2). (See Table 3-3 of Chapter 3.) Thus, for a given source, the two lines describing the recurrence relationship are given by:

$$\begin{aligned} \ln N'(M) &= \alpha_1' + \beta_1 M \qquad & 0 \leq M \leq M_1 \\ \ln N'(M) &= \alpha_2' + \beta_2 M \qquad & M_1 \leq M \leq M_2 \end{aligned} \tag{4-3}$$

where M_1 is the magnitude at which the two recurrence lines intersect (see, for example, fig. 3-2)

M_2 is the upper cutoff magnitude for a given source (see Table 3-3, Chapter 3).

Thus, depending on the source and the value of M, the mean number of events above Magnitude M for a unit area for area source, a unit-length for line source, and a unit-time is given by:

$$N'(M) = \exp\,[\alpha_i' + \beta_i M] \tag{4-4}$$

Thus, from equation 4-1

$$P_n(t) = \frac{\exp\,[-\exp\,(\alpha_i' + \beta_i M)t]\,[\exp\,(\alpha_i' + \beta_i M)t]^n}{n!} \tag{4-5}$$

Note that in equation 4.5 above, λ is replaced by $N'(M)$. Equation 4-5 gives the probability of observing n events above magnitude M in time period t, based on the seismic history of a given source.

Source Mechanisms

Three different types of sources can be used to represent the seismicity of any location. They are point, line, and area sources. All three source mechanisms will be discussed for generality and completeness, although only the line and area sources were considered for the Nicaragua region.

a. Point Source

For this type of source, all occurrences (past and future) take place at one point. The recurrence relationship can be normalized with respect to time T as follows:

$$N'(M) = \frac{N(M)}{T} \qquad 4\text{-}6$$

and $\ln N'(M) = \alpha' + \beta M$ --- 4-3 repeated.

Substituting the value of N'(M) in the Poisson law of equation 4-1, we get:

$$P_n (M > m, t) = \frac{\exp[-N'(m)\, t]\, [N'(m)\, t]^n}{n!} \qquad 4\text{-}7$$

where the notation

$P_n (M > m, t)$ gives the probability that there will be n events of Richter magnitude greater than m in time period t.

For engineering purposes, we are usually interested in determining the probability of at least one event greater than m in time period t. This probability is given by

P (at least one event of Richter Magnitude $M > m$ in time period t)

$$= 1 - P \text{ (no earthquake of magnitude } M > m \text{ in time } t).$$

Hence, from equation 4-7,

P (at least one event of Magnitude $M > m$ in time t)

$$= 1 - \exp\,[-N'(m)t]. \qquad 4\text{-}8$$

b. <u>Line Source</u>

For a line source, it is assumed that epicenters lie along a linear fault. For a line source of length L (fault length L) and the data base for a time period T, the recurrence relationship of Chapter 3 and equation 3-2 can be normalized to:

$$N'(M) = \frac{N(M)}{LT} \quad \text{--- Equation 3-3 repeated}$$

and

$$\ln N'(M) = \alpha' + \beta M \text{ --- Equation 3-4 repeated.}$$

Thus, the Poisson law of equation 4-1 can be written as

$$P_n(M > m,\ t) = \frac{\exp\,[-N'(m)t]\ [N'(m)t]^n}{n!}$$

where $N'(m)$ for line source is normalized with respect to length of the fault and time period T. Again, for determining the probability of at least one event of magnitude greater than m for a future time period t is given by,

P (at least one earthquake of $M > m$ in time t)

$$= 1 - P_o\ (M > m,\ t)$$

$$= 1 - \exp\,[-N'(m)t]$$

which is a similar expression to equation 4-8 except that the interpretation of $N'(m)$ is different.

c. Area Source

When the past earthquake epicenters do not lie on a line (i.e. along a given fault line) but are scattered over a region, the seismic source should be considered as an area source. The area source could be a full circle or any section of a circle where epicenters are scattered. In this case, the recurrence relationship is normalized with respect to area A and the time of data base T.

$$N'(M) = \frac{N(M)}{AT} \quad \text{--- Eq. 3-3 repeated}$$

and $\ln\ N'(M) = \alpha' + \beta M$ --- Eq. 3-4 repeated.

Thus, the probability of at least one event due to this area source above magnitude m in time period t is given by:

$$P\ (\text{at least one } M > m \text{ in time } t) = 1 - \exp\,[-N'(m)t].$$

Again, this expression is similar to equation 4 8 for a point source, and also for a line source. However, in each case the normalized $N'(M)$ has a different interpretation.

Peak Ground Acceleration at a Site

As we mentioned in Chapter 3, the most commonly used parameter to describe the seismic loading at a given site is the peak ground acceleration (PGA, usually denoted by A). In the previous section, we obtained the probability of exceeding a magnitude level in time t by using a Poisson odel. (The probability distribution represented by Eq. 4 8 gives information only on Richter magnitude.) For design purposes, we wish to know the loading at a site, away from the epicenter. Modified Mercalli Intensity (MMI, see Appendix 2) peak ground acceleration, spectral acceleration, and several other parameters have been used to represent the loading at a given site. To obtain probabilistic information about peak ground acceleration at a site, we have to know the following parameters:

1. Probabilistic information on Richter Magnitude for a source as a function of future time t.
2. Distance of the site from the source.
3. Attenuation of peak ground acceleration from source to site.

We have already determined the first parameter in the previous section. Various attenuation formulae are available which give relationship between the Richter Magnitude M, the epicentral distance or the hypocentral distance, and the peak ground acceleration. The most commonly used relationship is of the form given by

$$A = \frac{b_1 \exp (b_2 M)}{(R_h + b_4)^{b_3}} \qquad 4\text{-}9$$

where A = Peak Ground Acceleration (PGA) in cm/sec^2.

R_h = Hypocentral distance from source to site (in kms.).

M = Richter Magnitude.

b_1, b_2, b_3, and b_4 are constants depending on the region.

Since there is not a close grid of seismographs in Nicaragua, not much information is available on attenuation of accelerations in that part of the world. However, various values of b_1, b_2, b_3 and b_4 are available for other parts of the world. One such relationship that has been adopted for this work is the one developed by Esteva. The attenuation constants are given by

$$b_1 = 5{,}000; \qquad b_3 = 2.0;$$

$$b_2 = 0.8; \qquad b_4 = 40$$

Thus, the attenuation relationship becomes

$$A = \frac{5{,}000 \exp (0.8 M)}{(R_h + 40)^2} \qquad 4\text{-}10$$

Figure 4-1 shows the behavior of Eq. 4-10 for different Richter magnitudes and hypocentral distances. This equation was correlated with the ESSO Refinery data for the 1972 December earthquake and also with the aftershocks. The correlation of the curve of Eq. 4-10 with the data is quite reasonable; consequently, the attenuation relationship given by Eq. 4-10 is used in this study. It should be pointed out that the installation of many new instruments will help in calibrating the attenuation relationship for Nicaragua in the future. When that is done, the results presented in this study can be readily modified.

We have seen that three types of seismic sources are possible. Due to each of these seismic sources, the peak ground acceleration at a site in a probabilistic sense can be determined.

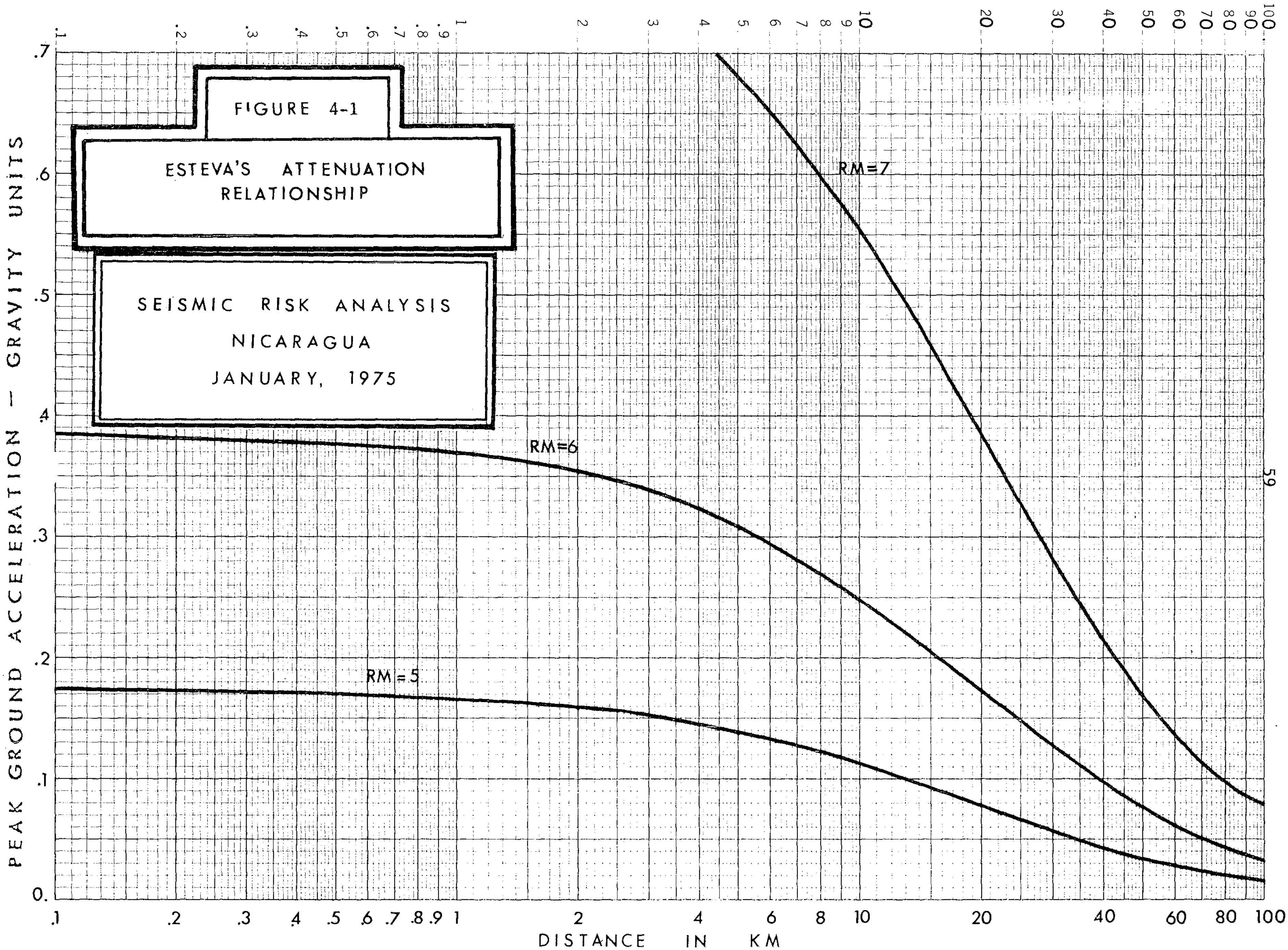


FIGURE 4-1
ESTEVA'S ATTENUATION RELATIONSHIP
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
RM=7
RM=6
RM=5
PEAK GROUND ACCELERATION – GRAVITY UNITS
DISTANCE IN KM

a. Point Source

For a point source shown in Figure 4-2, we can derive the following expressions:

$$P\ (M > m,\ t) = \text{Probability of at least one event greater than } m \text{ in time } t.$$

$$= 1 - \exp\ [-N'(m)t].$$

$$\text{but } N'(M) = \exp\ [\alpha' + \beta M].$$

Thus,

$$P(M > m,\ t) = 1 - \exp\ [-\exp\ (\alpha' + \beta m)t] \qquad 4\text{-}11$$

To determine the probability distribution on peak ground acceleration A, we have

$$P\ [A > a] = P\ \left[\frac{b_1 \exp\ (b_2\ M)}{(R_h + b_4)^{b_3}} > a \right]$$

$$= P\ \left[M > \ln\ \left\{\frac{a}{b_1}\ (R_h + b_4)^{b_3}\right\}^{\frac{1}{b_2}}\right] \qquad 4\text{-}12$$

Using equation 4-11 in 4-12, we get

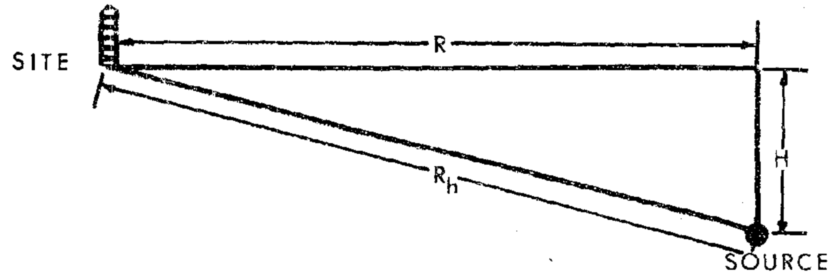
$$P\ [A > a] = 1 - \exp\ \left\{-e^{\alpha'} \left(\frac{a}{b_1}\right)^{\beta/b_2} (R_h + b_4)^{\frac{\beta/b_3}{b_2}}\ t\right\} \qquad 4\text{-}13$$

Denoting

$$\gamma = e^{\alpha'}$$

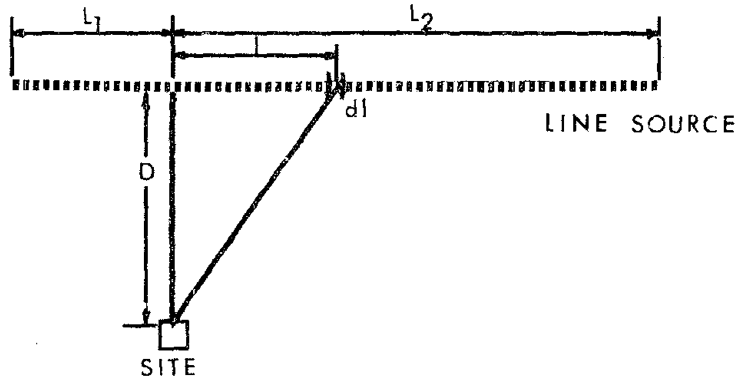
$$\delta = \beta/b_2$$

$$\text{and } \rho = \frac{\beta\ b_3}{b_2} = \delta b_3$$

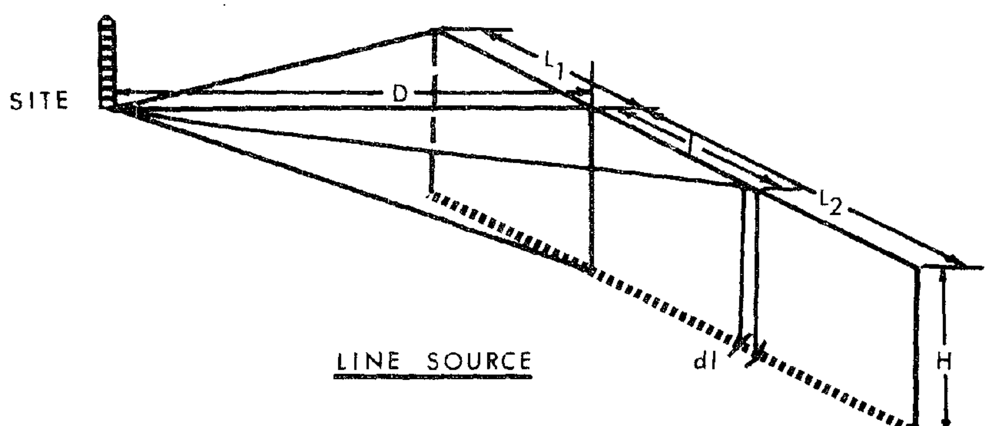


POINT SOURCE

FIGURE 4-2



TOP VIEW



LINE SOURCE

FIGURE 4-3

we get

$$P\ [A > a] = 1 - \exp\ \{-\gamma\ (\frac{a}{b_1})^{\delta}\ (R_h + b_4)^{\rho}\ t\} \qquad 4\text{-}14$$

b. Line Source

Most of the earthquake epicenters around the world are generally located around the major fault systems. Thus, the usual case of epicenters falling along a line gives rise to the so-called line source. The line source can be divided into κ small segments of length dl. Each one of these segments can be treated as a point source. Summing the effects of all such segments, as $dl \to o$, gives the probability of a peak ground acceleration A exceeding a value a due to a fault line source of length L.

For a point source, we have seen that

$$P\ [A > a] = 1\text{-}\exp\ \{-\ \gamma\ (\frac{a}{b_1})^{\delta}\ (R_h + b_4)^{\rho}\ t\}$$

then $$P\ [A < a] = \exp\ \{-\gamma(\frac{a}{b_1})^{\delta}\ (R_h + b_4)^{\rho}\ t\}\ .$$

Thus, for an element i of the line source, we have

$$P_i\ [A < a] = \exp\ \{-\ \gamma\ (\frac{a}{b_1})^{\delta}\ (R_{h_i} + b_4)^{\rho}\ d\ell_i\ t\ \}$$

From Fig. 4-3,

$$R_h = (d^2 + \ell^2 + h^2)^{1/2}$$

where ℓ is the distance of element under consideration from the perpendicular on the line source.

Thus,

$$P_i\ [A < a] = \exp\ \{- \gamma(\frac{a}{b_1})^{\delta}\ [(d^2 + \ell_j^{\ 2} + h^2)^{1/2} + b_4]^{\rho}\ d\ell_i\ t\}$$

From the basic assumption of spatial independence of occurrences, we get

$$P\ [A < a] = \lim_{\substack{d\ell_i \to o \\ K \to \infty}} \prod_{i=1}^{K} P_i\ [A < a].$$

$$= \lim_{\substack{d\ell_i \to o \\ K \to \infty}} \exp\ \{- \gamma(\frac{a}{b_1})^{\delta} \sum_{i=1}^{K} [(d^2 + \ell_i^{\ 2} + h^2)^{1/2} + b_4]^{\rho}\ d\ell_i t\}$$

$$P\ [A < a] = \exp\ \{(- \gamma(\frac{a}{b_1})^{\delta}\ t \int_{\ell_1}^{\ell_2} [(d^2+\ell^2+h^2)^{1/2} + b_4]^{\rho}\ d\ell\} \qquad 4\text{-}15$$

Alternatively,

$$P\ [A > a] = 1 - \exp\ \{- \gamma\ (\frac{a}{b_1})^{\delta}\ t \int_{\ell_1}^{\ell_2} [(d^2+\ell^2+h^2)^{1/2} + b_4]^{\rho}\ d\ell\} \qquad 4\text{-}16$$

Expressions 4-15 and 4-16 provide the probabilities of peak ground acceleration A due to a line source located some distance away and whose seismicity is available in terms of α' and β and also for which the attenuation relationship of the form of Eq. 4-9 is available.

c. Area Source

Peak ground acceleration due to an area source at a site can be obtained in a manner similar to that for line source. In many parts of the world, including Nicaragua, there are regions where the epicenters are not only located along a line but are scattered all over the region. This may be due to the existence of numerous faults criss-crossing the region or due to errors in estimating epicentral locations. In any case, there are places where point or line sources may not fit the scatter of epicentral locations. For such cases, area source should be used to determine probabilistic loadings at a site. Figure 4-4 shows schematically the area source geometry.

Consider an elemental area $dA_i = R_i dR_i d\Theta_i$. With this elemental area as a source, the probability that the Richter Magnitude M will be less than m in time t is given by

$$P_i \; [M < m, \; t] = \exp \; [-N'(m) \; dA_i t]$$

$$= \exp \; [-\exp \; (\alpha' + \beta m) \; tR_i dR_i d\Theta_i]$$

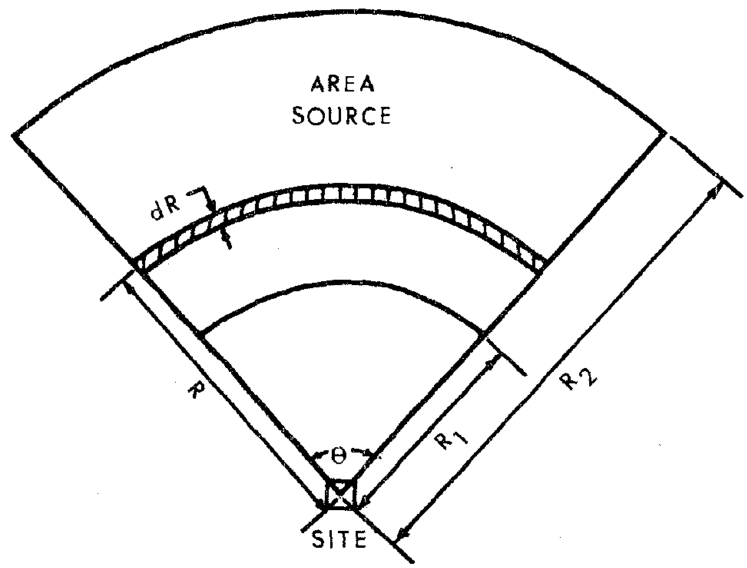
or

$$P_i [A < a, t] = \exp \; [-e^{\alpha'} (\frac{a}{b_1})^{\beta/b_2} (\sqrt{R_i^2 + h^2} + b_4)^{\frac{\beta b_3}{b_2}} \; tR_i dR_i d\Theta_i]$$

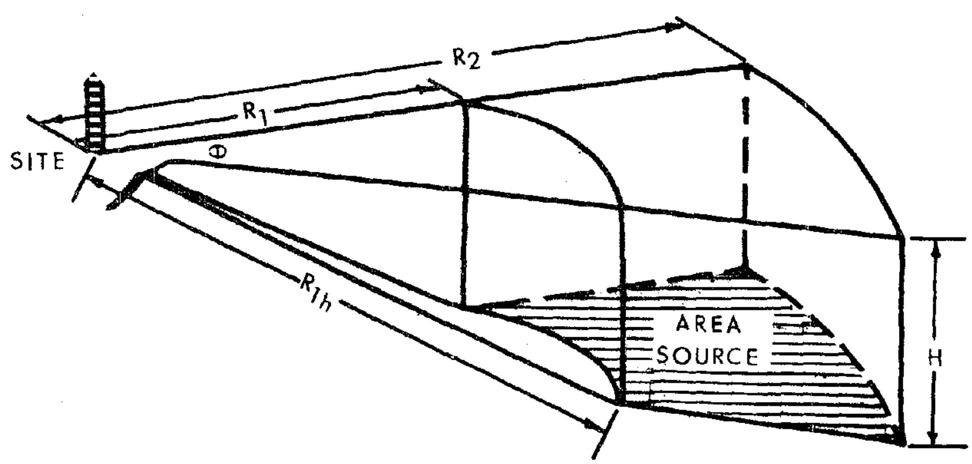
Summing the effect of all elemental areas, we get

$$P \; [A < a, \; t] = \lim_{\substack{dR_i \to 0 \\ d\Theta_i \to 0}} \; P_i \; (A < a, \; t)$$

Hence,



TOP VIEW



AREA SOURSE

FIGURE 4-4

$$P\,[A < a,\ t] = \exp\ \{-e^{+\alpha'}\ (\frac{a}{b_1})^{\beta/b_2}\ t\int_0^{\Theta_o} d\Theta \int_{R_1}^{R_2} (\sqrt{R^2+h^2} + b_4)^{\frac{\beta b_3}{b_2}}\ RdR\}$$

$$= \exp\ \{-e^{\alpha'}\ (\frac{a}{b_1})^{\beta/b_2}\ t\Theta\int_{R_1}^{R_2} (\sqrt{R^2+h^2} + b_4)^{\frac{\beta b_3}{b_2}}\ RdR\}$$

Let $\gamma = e^{\alpha'}$

$\delta = \beta/b_2$

$\rho = \frac{\beta b_3}{b_2}$ as before

and $R_h = \sqrt{R^2+h^2}$

Then

$$P\,[A < a,\ t] = \exp\ \{ - \gamma(\frac{a}{b_1})^{\delta}\ t\Theta \int_{R_1}^{R_2} (R_h + b_4)^{\rho}\ RdR\ \} \qquad 4.17$$

and

$$P\,[A > a,\ t] = 1 - \exp\ \{ - \gamma\ (\frac{a}{b_1})^{\delta}\ t\Theta\int_{R_1}^{R_2} (R_h + b_4)^{\rho}\ RdR\} \quad 4.18$$

Equations 4-17 and 4-18 provide the probability distribution of peak ground acceleration A at a site due to a generalized area source shown in Figure 4-4.

In general, a site is usually surrounded by any or all of the above three sources discussed in this section. The probabilistic loading due to such a case can be obtained by the following expression.

Let there be NP point sources

NL line sources

NA area sources

The probability distribution of peak ground acceleration at a site is then given by

$$P\,[A > a,\, t] = 1 - \exp\Big\{ - \sum_{i=1}^{NP} \gamma_i \left(\frac{a}{b_1}\right)^{\delta_i} t\,(R_{h_i} + b_4)^{\rho_i}$$

$$- \sum_{j=1}^{NL} \gamma_j \left(\frac{a}{b_1}\right)^{\delta_j} t \int_{\ell_{1j}}^{\ell_{2j}} [(d_j{}^2+\ell^2+h_j{}^2)^{1/2} + b_4]^{\rho_j} d\ell$$

$$- \sum_{k=1}^{NA} \gamma_k \left(\frac{a}{b_1}\right)^{\delta_k} t\,\Theta_k \int_{R_{1k}}^{R_{2k}} (R_{h_k} + b_4)^{\rho_k} R dR \Big\} \qquad 4\text{-}19$$

In equation 4-19, summation over i is for all point sources, that over j is for all line sources, and over k is for all area sources.

As we have seen in Chapter 3, there are ten line sources and three area sources that we have formulated for the Nicaragua region, based on past data. Any part of the country is affected by these sources, depending upon the proximity of the site to the source location.

Iso-Acceleration Maps for Nicaragua

Equation 4-19 can be used to determine the probability distribution function of peak ground acceleration as a function of time and space. For example, at a given site, the probability of $A > a$ increases with time. In other words, the longer the "exposure time," the greater the probability that the peak ground acceleration will exceed some level a. If we take the country as a whole and determine accelerations at different locations for a specific time period t (exposure time) and specific probability of $A < a$, we can obtain lines of equal accelerations. These lines of equal accelerationsfor a specific probability of non-exceedence and exposure time are called "Iso-Acceleration" lines. The maps representing iso-acceleration lines are called the iso-acceleration maps. These iso-acceleration maps are a form of seismic zoning maps. From these maps, for a given reliability or risk, one can determine the loading parameter (peak ground acceleration) for the seismic design of a structure. Detailed methodology describing the use of these maps for structural design will be presented in Chapter 7 of this report and also in the Part II report of the total study.

Charts 8 through 13 show the iso-acceleration maps for Nicaragua for a time period of 50 years and 20 years. For each time period the iso-acceleration maps are drawn for three risk levels. The risk level is defined as the probability that the peak ground acceleration will be exceeded during the exposure time (or economic lifetime) of the facility under consideration.

In addition to the iso-acceleration maps for the whole country, the following cities are studied in detail.

1. Managua
2. Leon
3. Granada
4. Masaya
5. Chinandega
6. Matagalpa
7. Esteli
8. San Carlos
9. Rivas
10. Juigalpa
11. Bluefields.

Figures 4-5 through 4-26 show the cumulative distribution function of the peak ground acceleration for each of the cities. Again, results are presented for the exposure time of 20 years and 50 years. Thus, as an example, for Leon, there is approximately 53% chance that the peak ground acceleration in 20 years of exposure time will not exceed 0.20 g (see Figure 4-8). The corresponding value for a 50 year exposure time for the same city is 21%. Thus, the probability of exceeding 0.20 g in 20 years is 47%, whereas it goes up to 79% in 50 years. The implications of these probability values and the corresponding acceleration values will be discussed in Chapters 5, 6, and 7.

When we compare the cumulative distribution plots for different cities, we can see the relative seismicity in terms of peak ground acceleration for each city. In conclusion, it can be said that one method of representing seismic risk is by means of the iso-acceleration

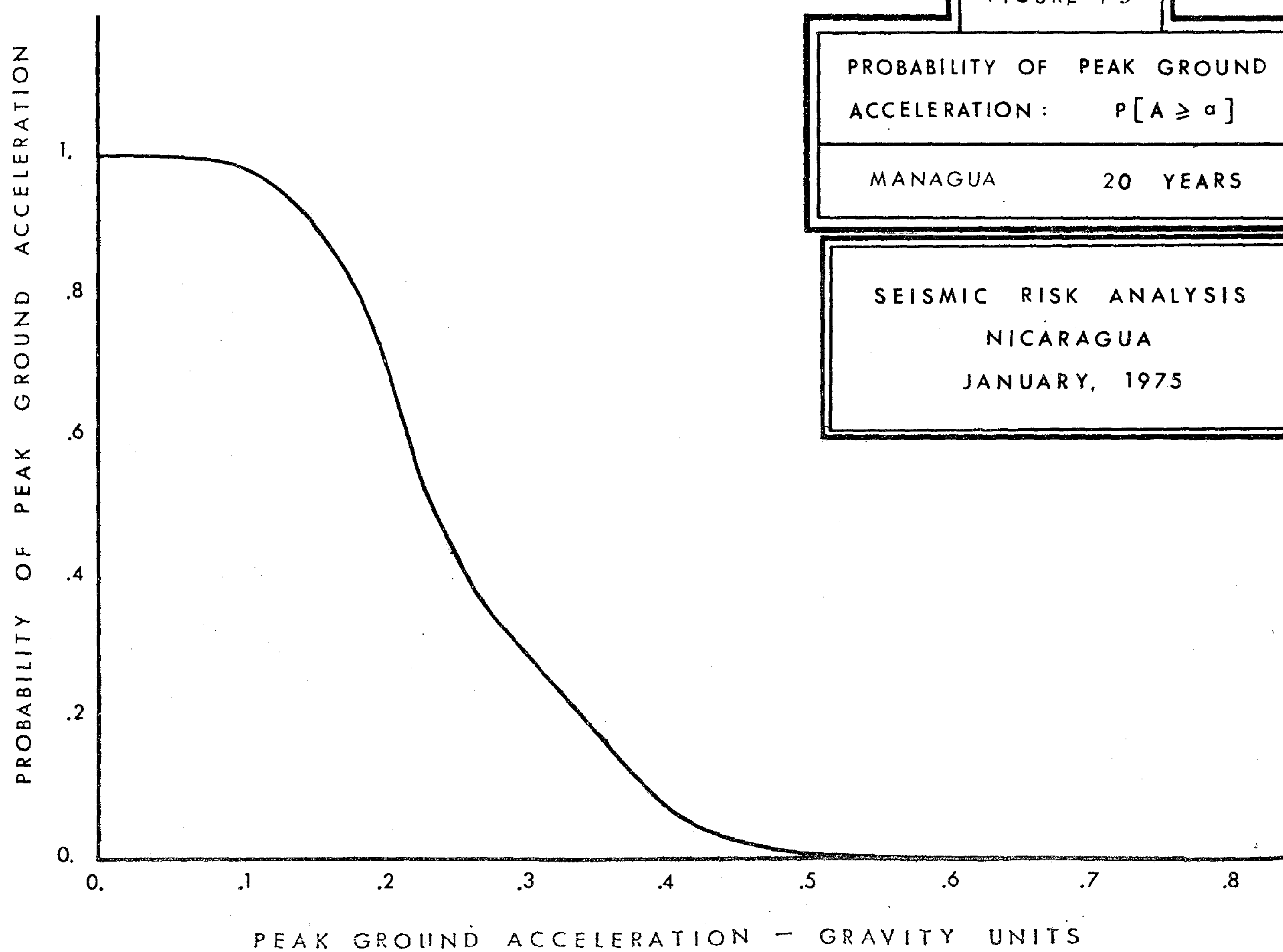


FIGURE 4-5

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

MANAGUA 20 YEARS

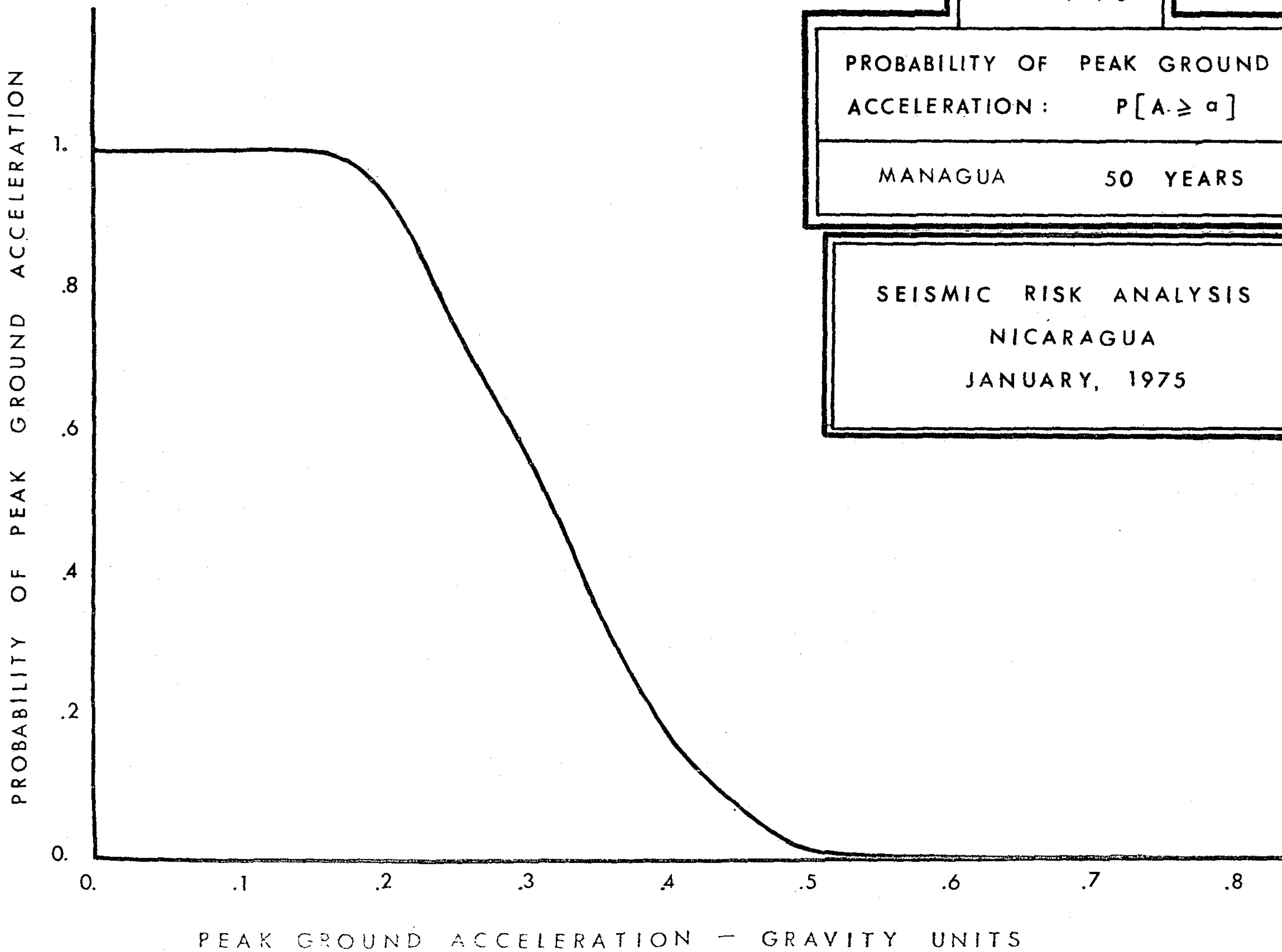
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

FIGURE 4-6

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

MANAGUA 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975



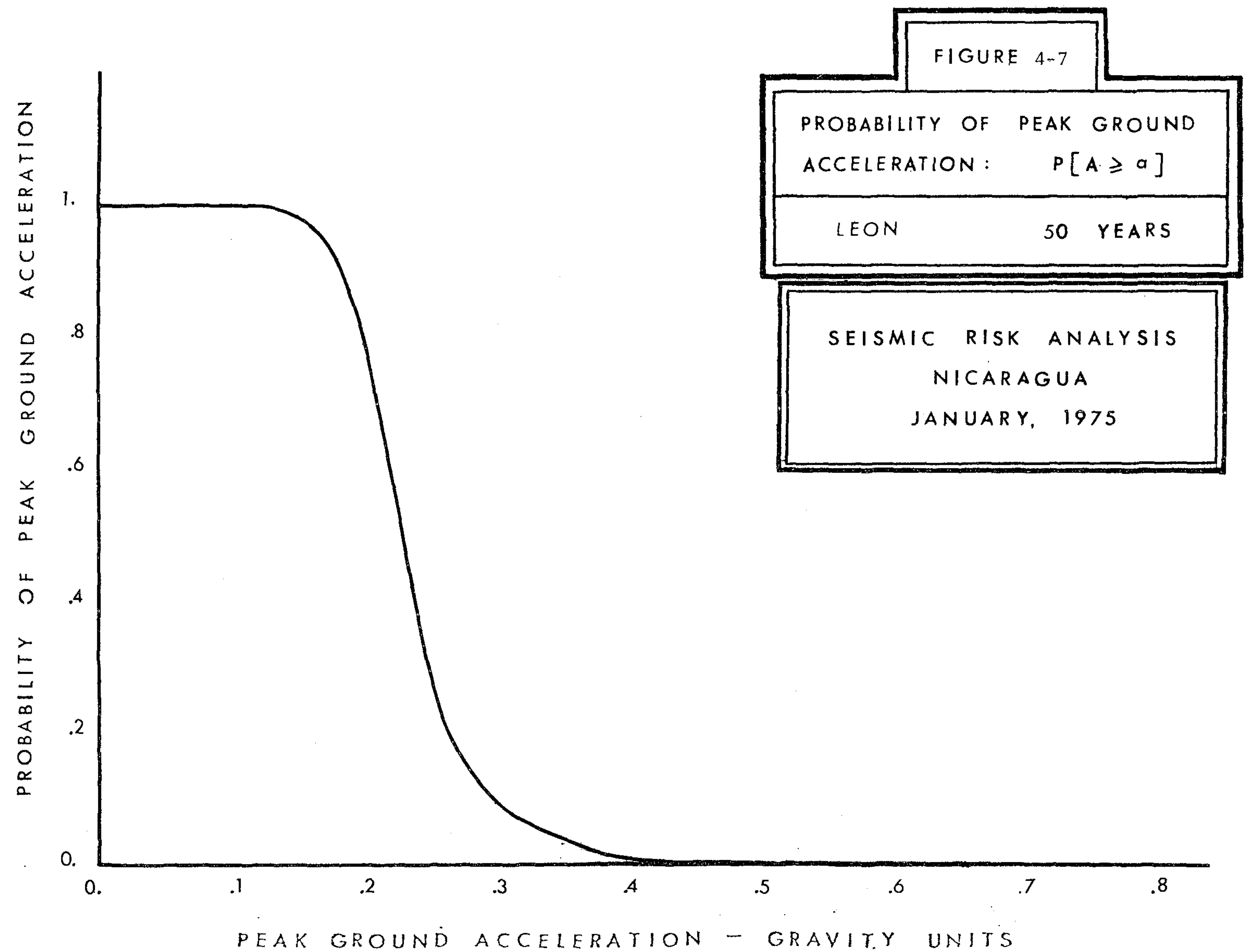


FIGURE 4-7
PROBABILITY OF PEAK GROUND ACCELERATION : P[A ≥ a]
LEON 50 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
.8
.6
.4
.2
0.
0.
.1
.2
.3
.4
.5
.6
.7
.8
PEAK GROUND ACCELERATION — GRAVITY UNITS

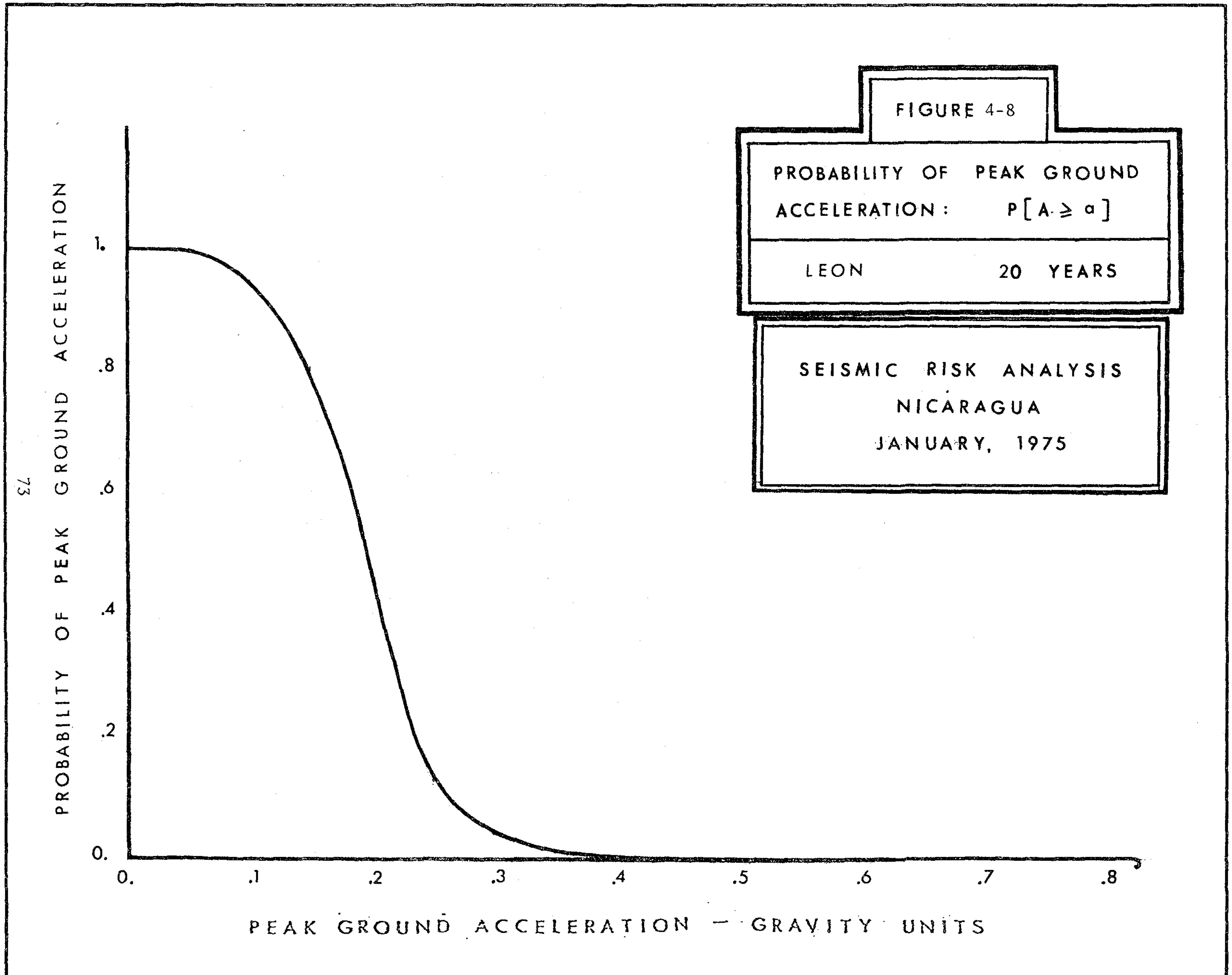


FIGURE 4-8
PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]
LEON
20 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
.8
.6
.4
.2
0.
0.
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PEAK GROUND ACCELERATION — GRAVITY UNITS

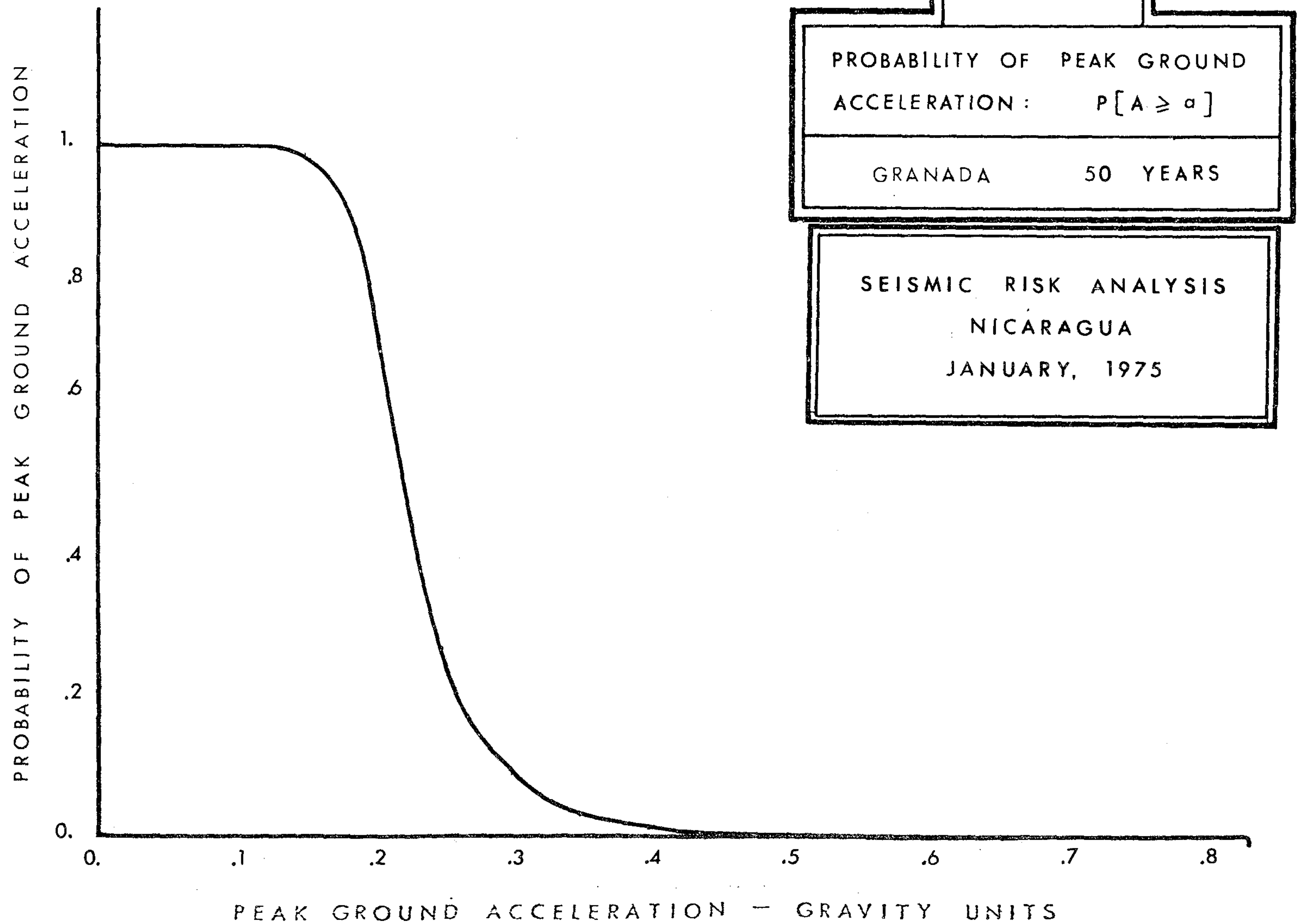

FIGURE 4-9

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

GRANADA 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

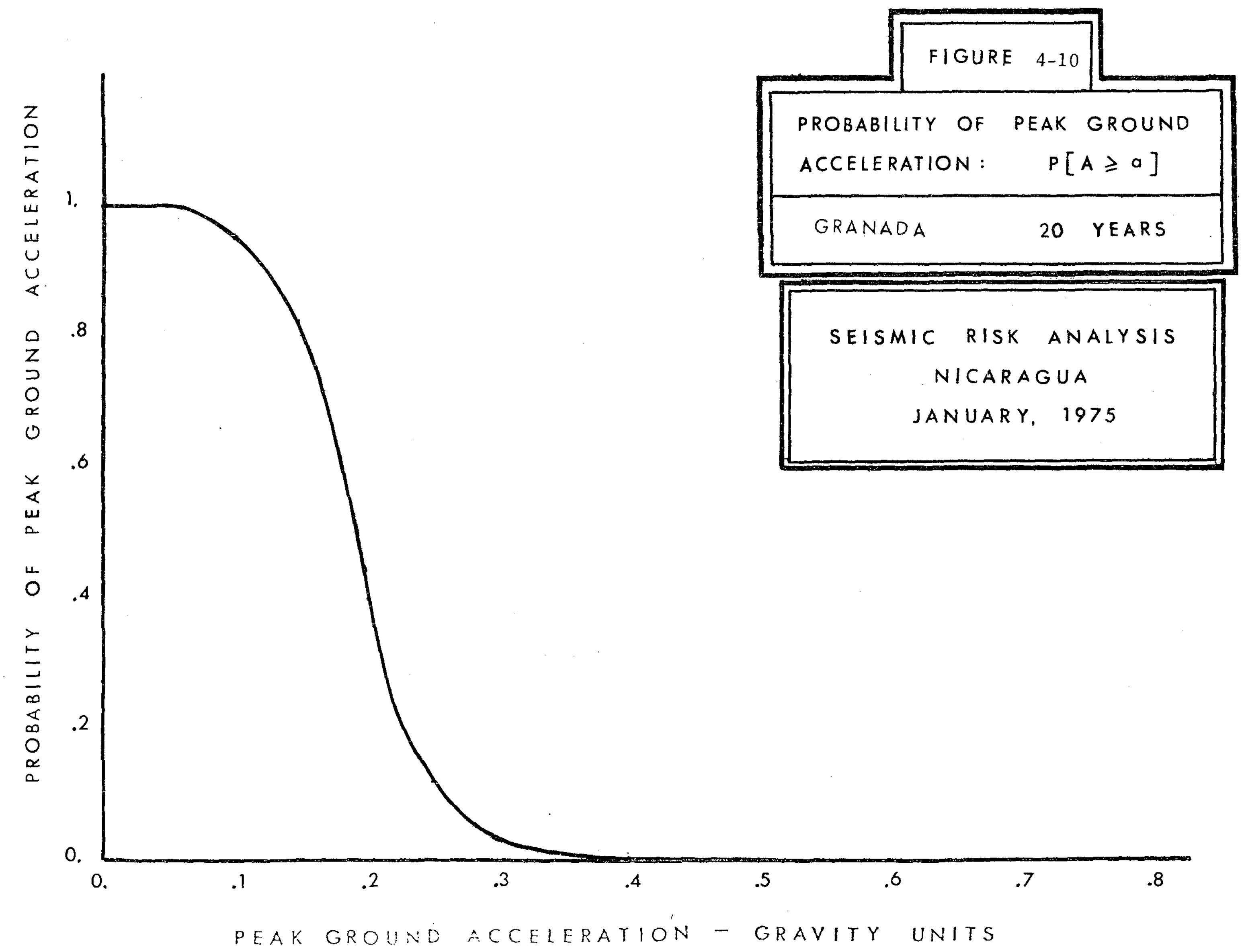
FIGURE 4-10
PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]
GRANADA 20 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
.8
.6
.4
.2
0.
0.
.1
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.3
.4
.5
.6
.7
.8
PEAK GROUND ACCELERATION - GRAVITY UNITS

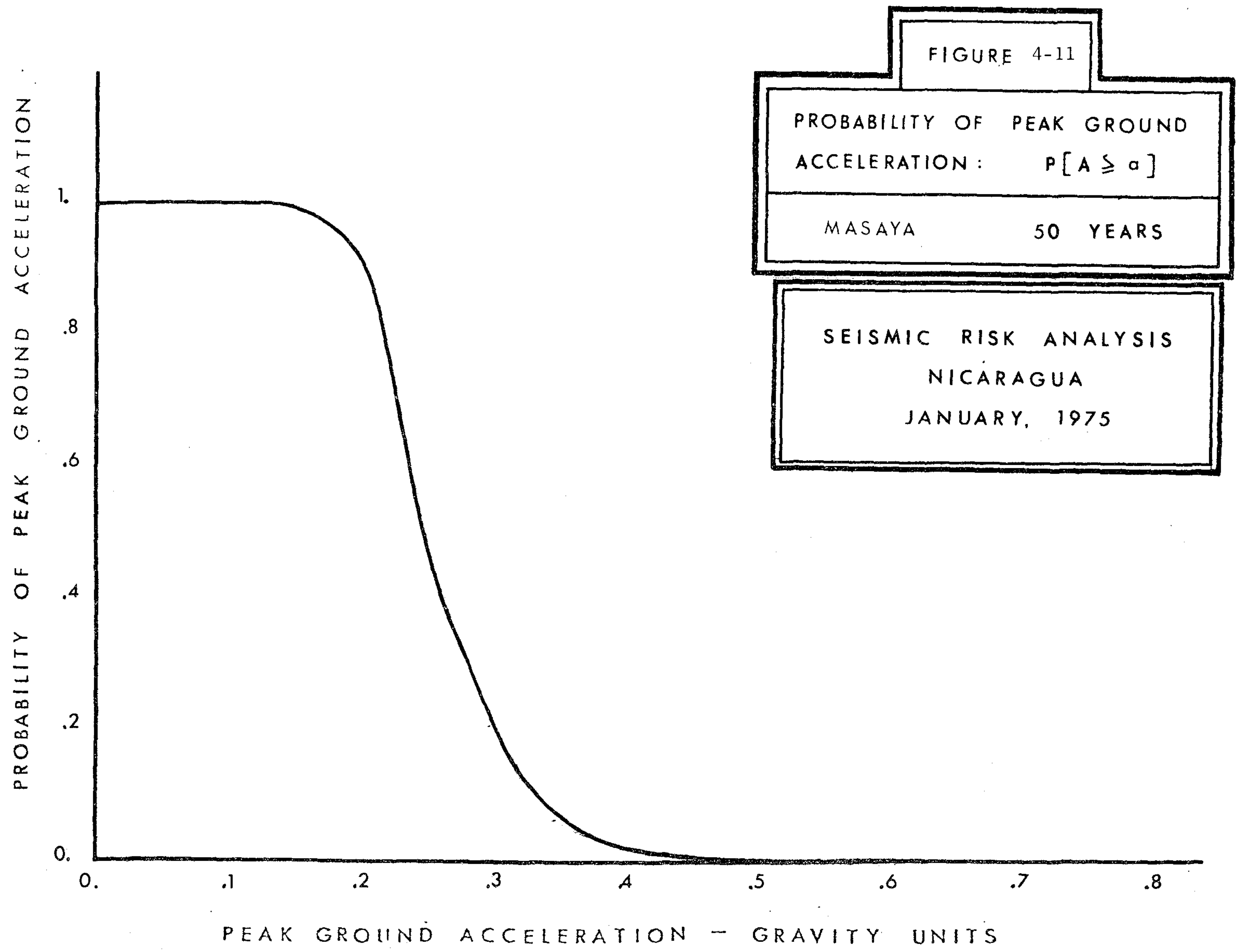
FIGURE 4-11
PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]
MASAYA 50 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
.8
.6
.4
.2
0.
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PEAK GROUND ACCELERATION – GRAVITY UNITS

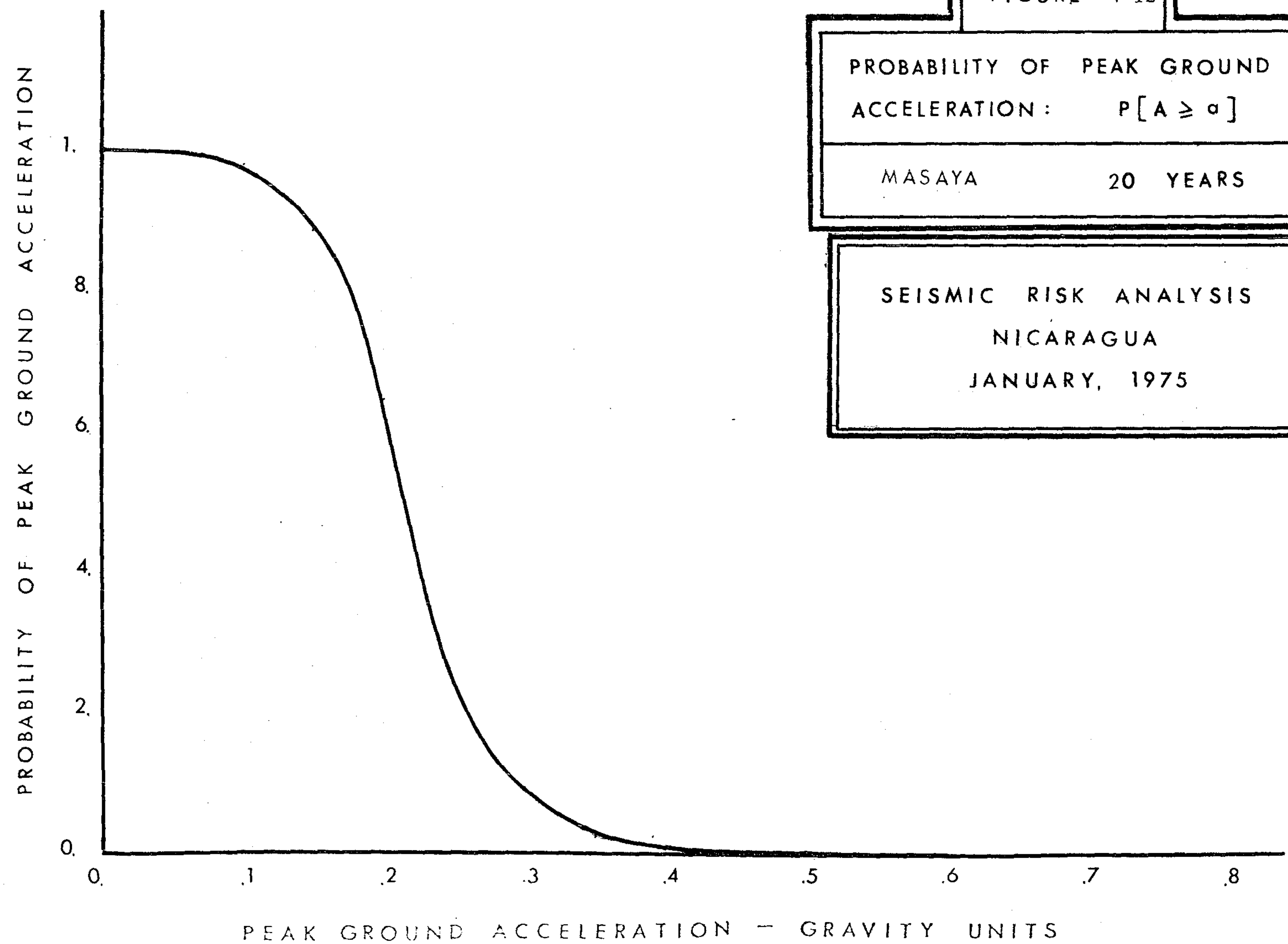


FIGURE 4-12

PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]

MASAYA 20 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

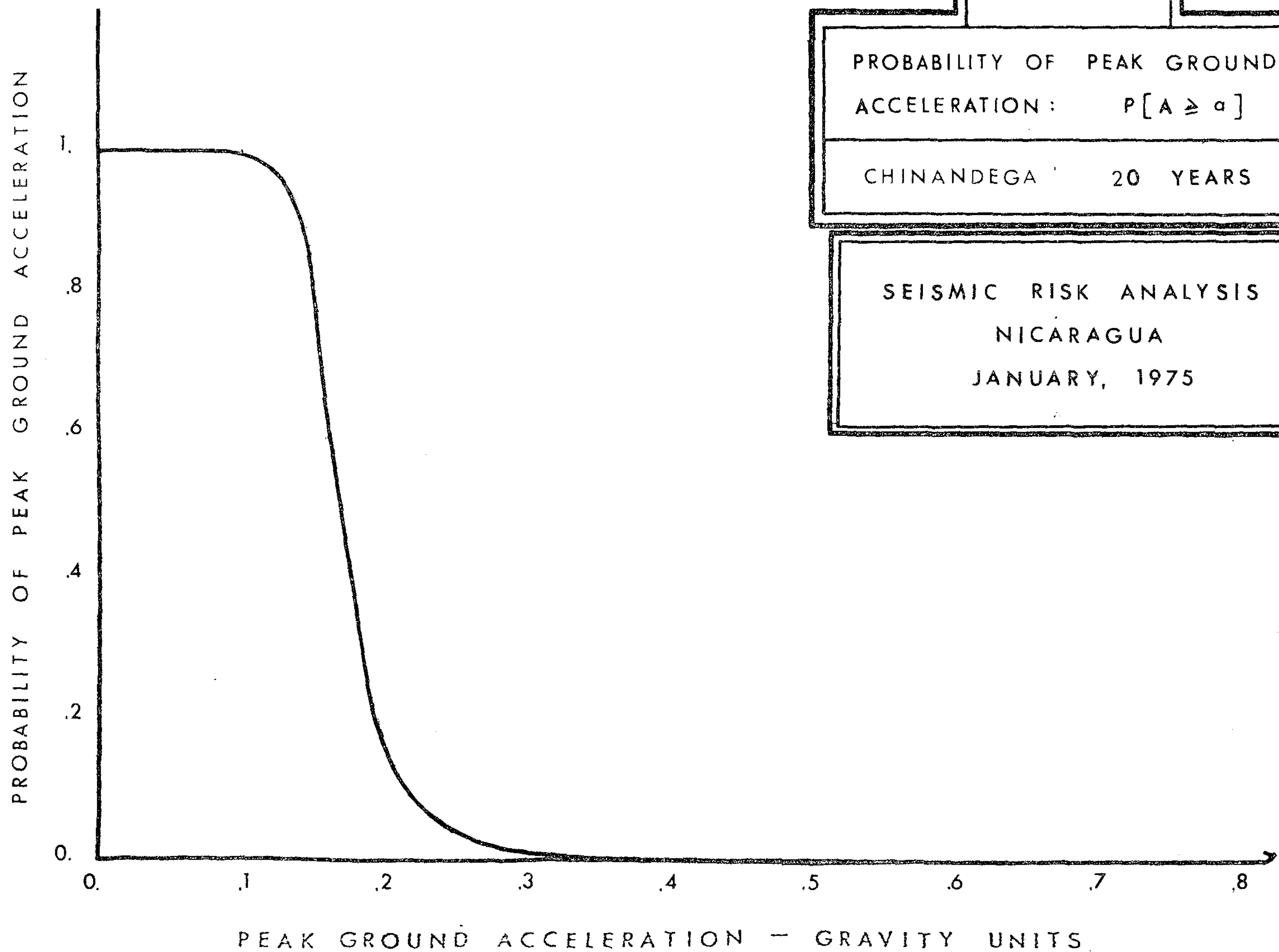


FIGURE 4-13

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

CHINANDEGA 20 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

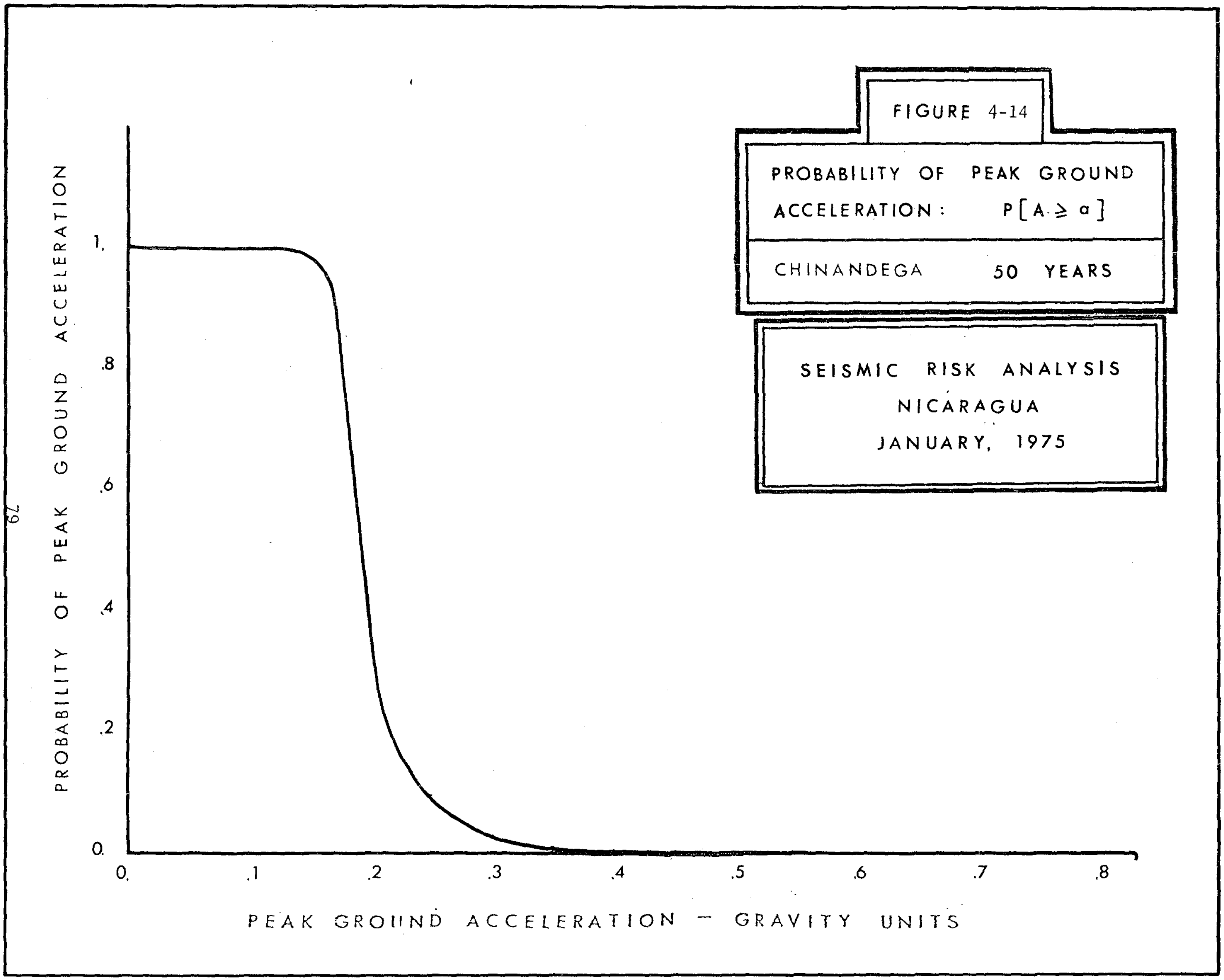


FIGURE 4-14
PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]
CHINANDEGA 50 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
.8
.6
.4
.2
0.
0.
.1
.2
.3
.4
.5
.6
.7
.8
PEAK GROUND ACCELERATION — GRAVITY UNITS

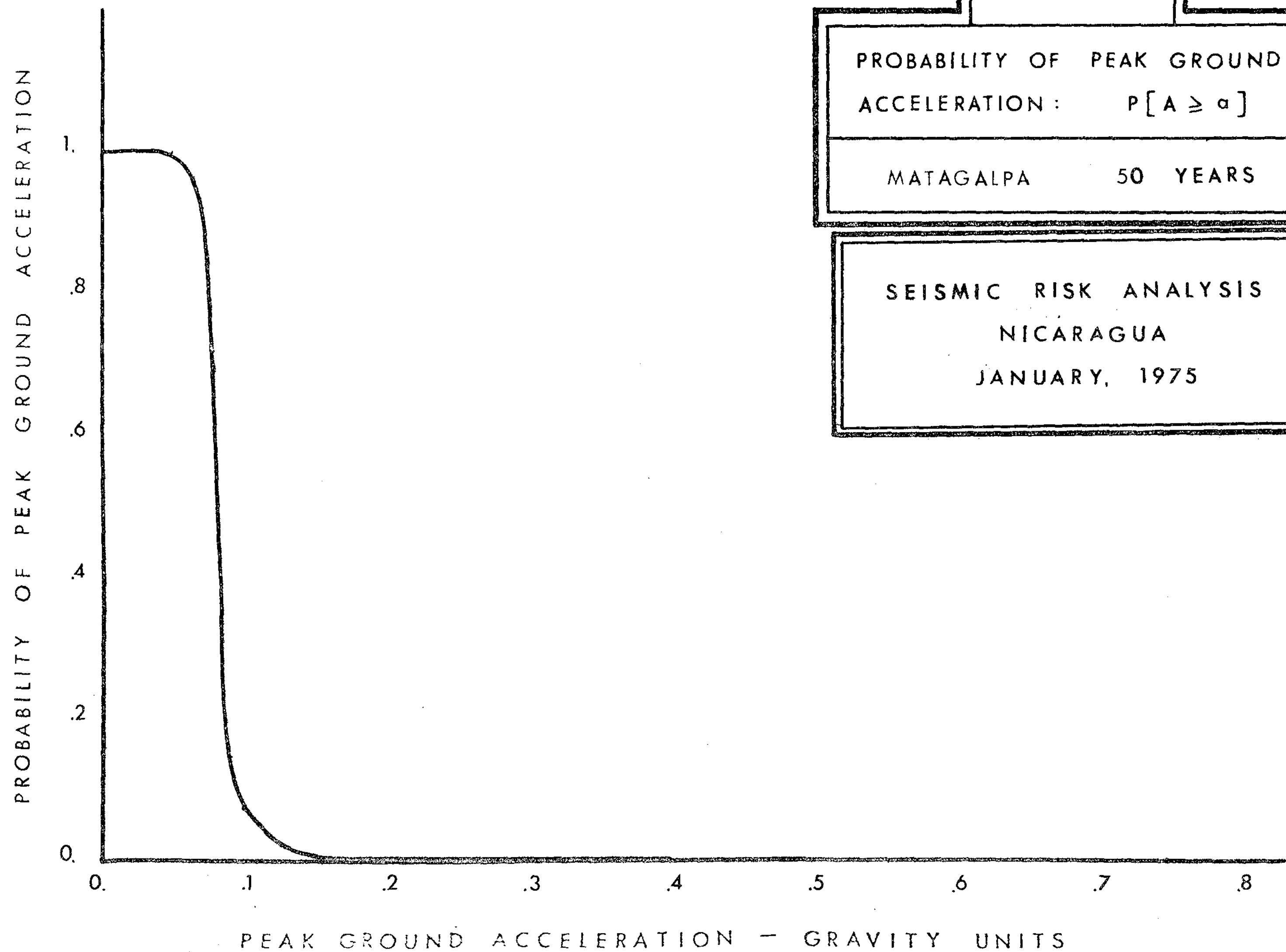


FIGURE 4-15

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

MATAGALPA 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

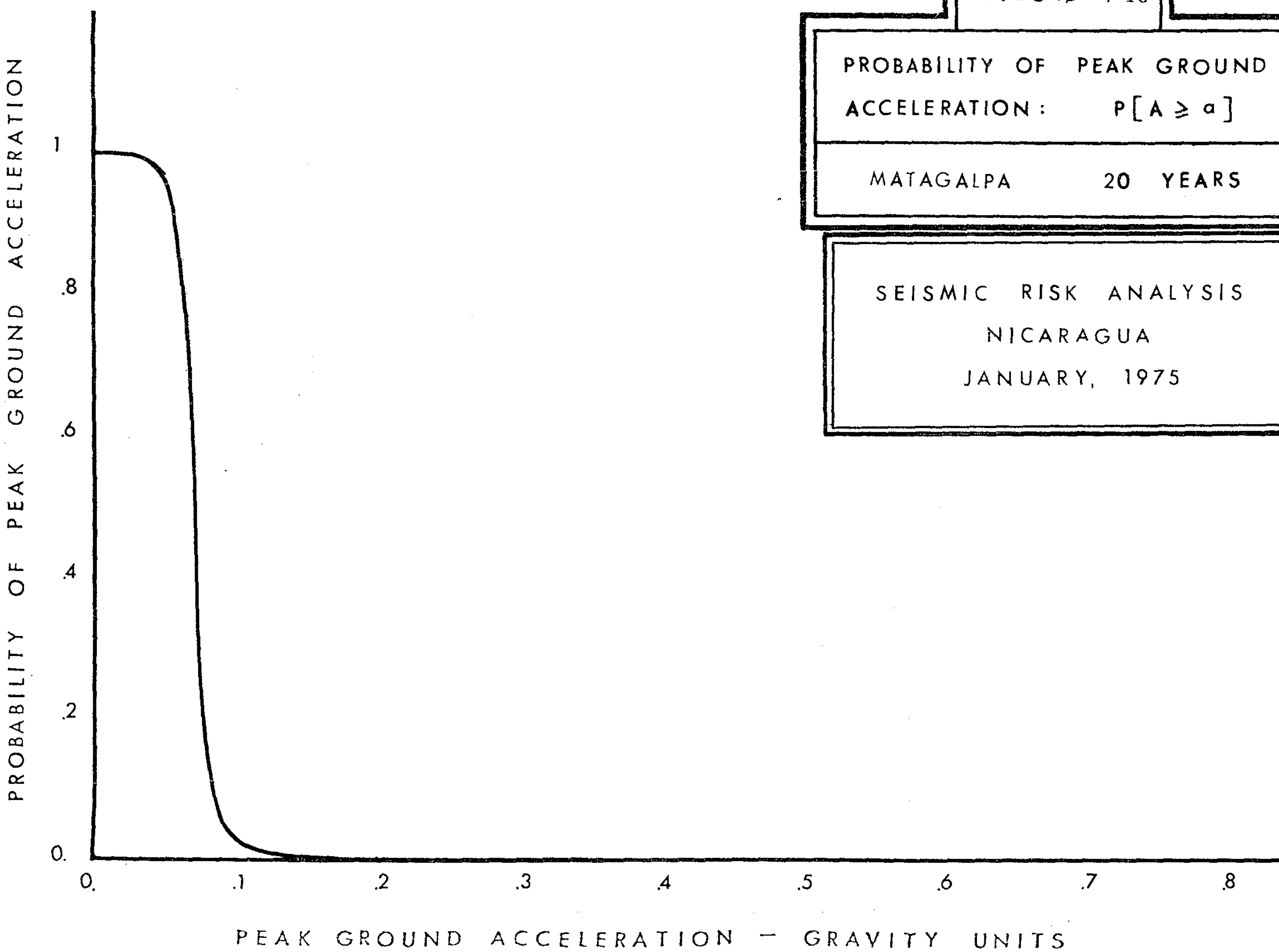


FIGURE 4-16

PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]

MATAGALPA 20 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

FIGURE 4-17

PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]

ESTELI 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

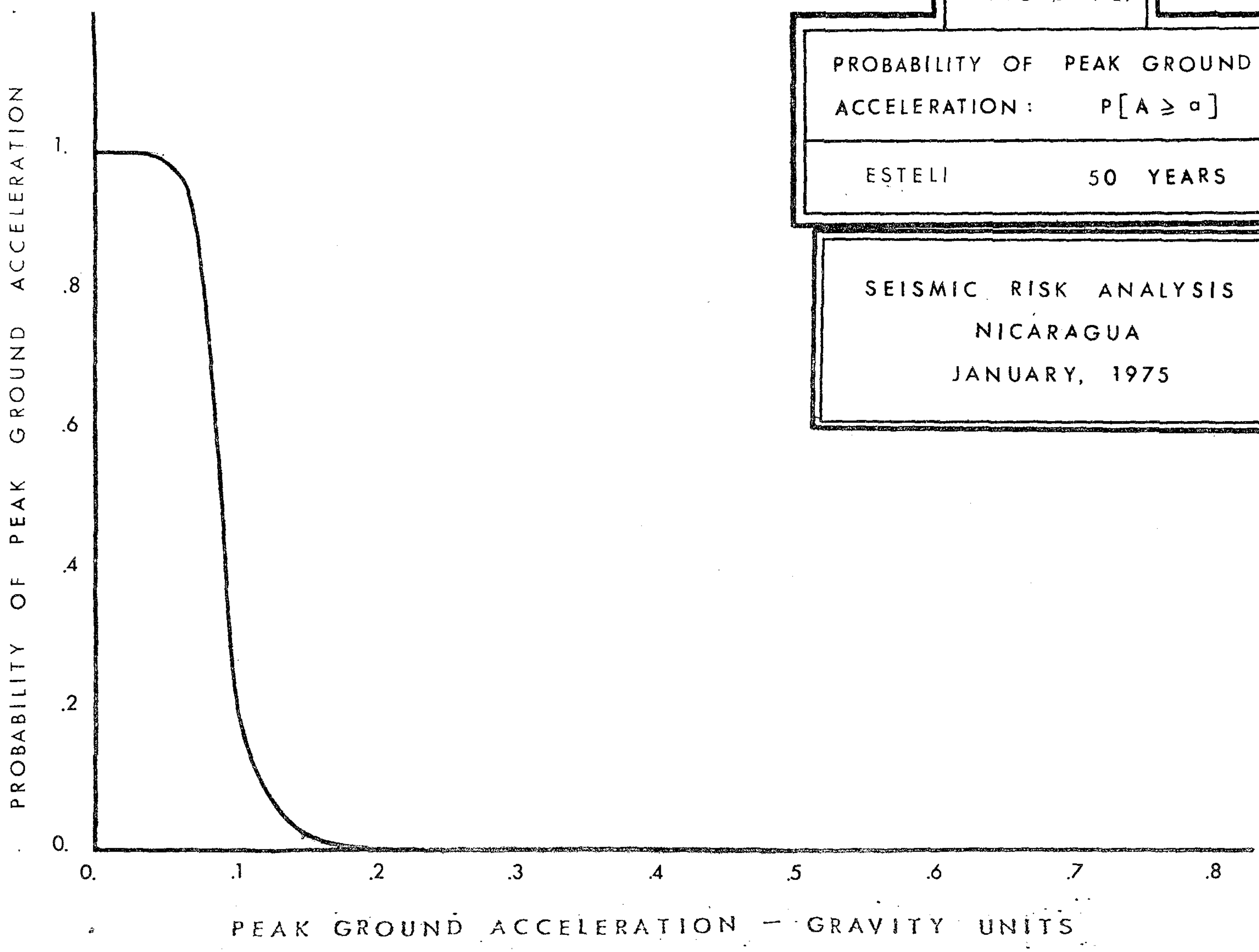
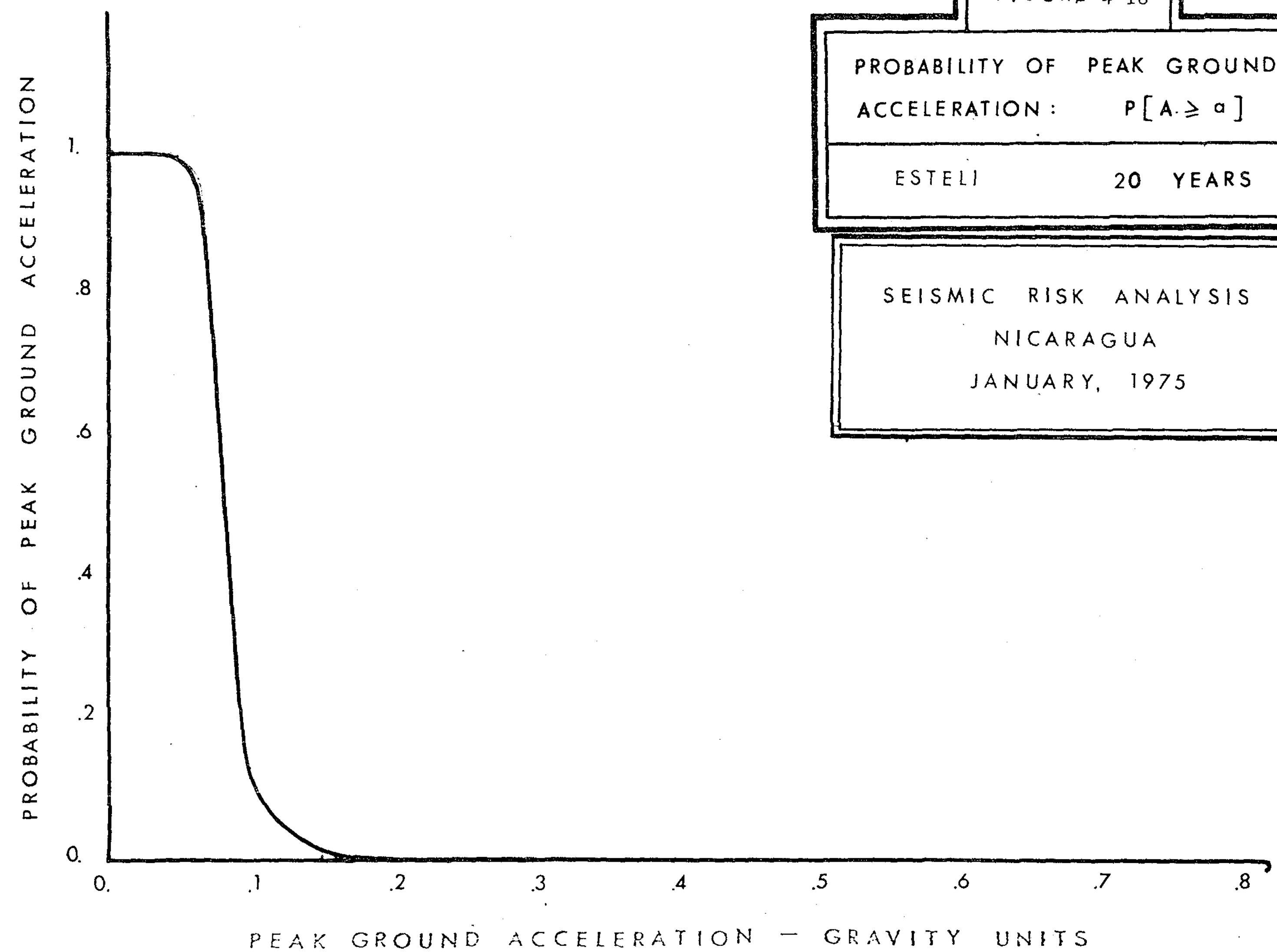


FIGURE 4-18

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

ESTELI 20 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975



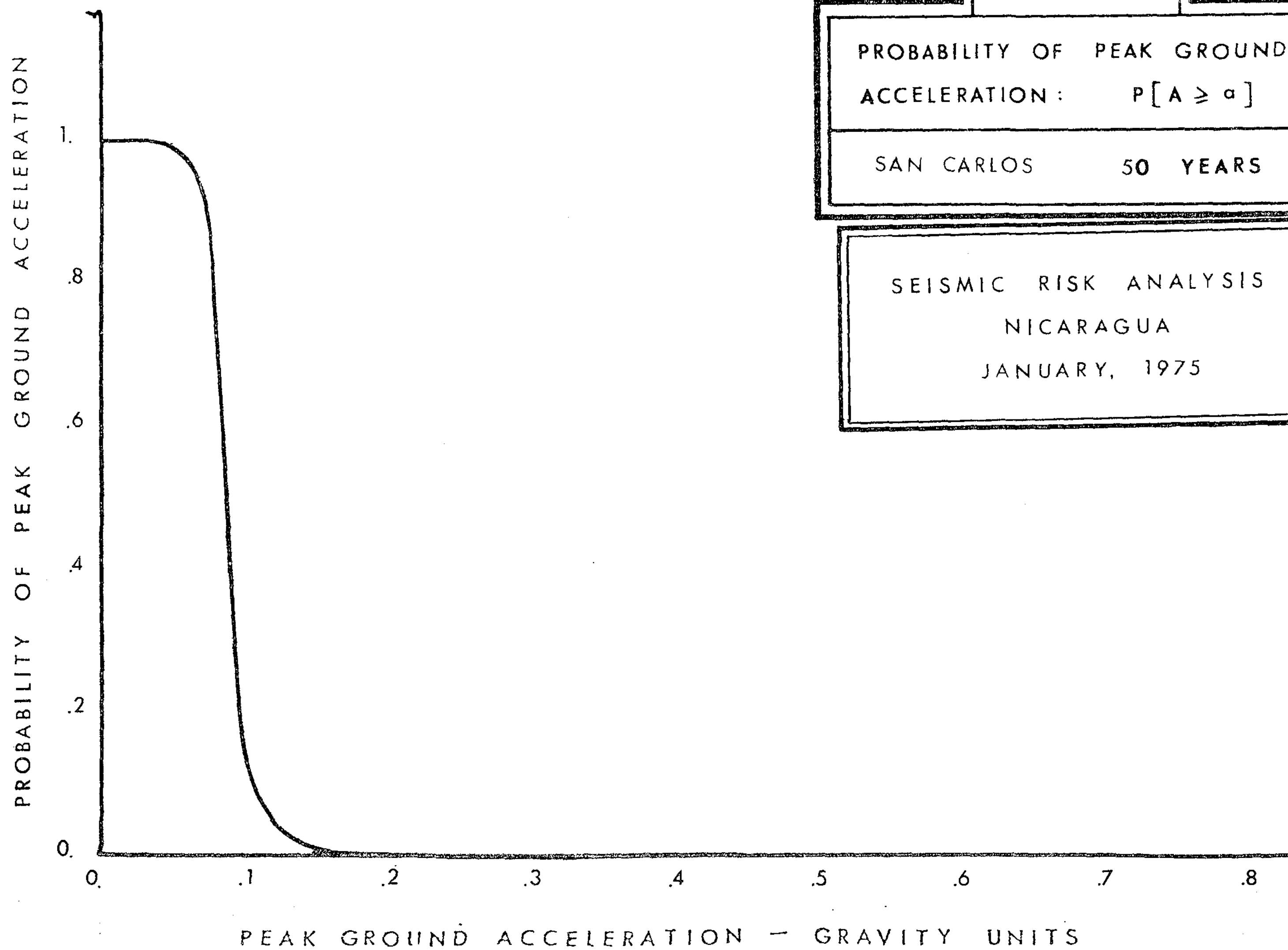


FIGURE 4-19

PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]

SAN CARLOS **50 YEARS**

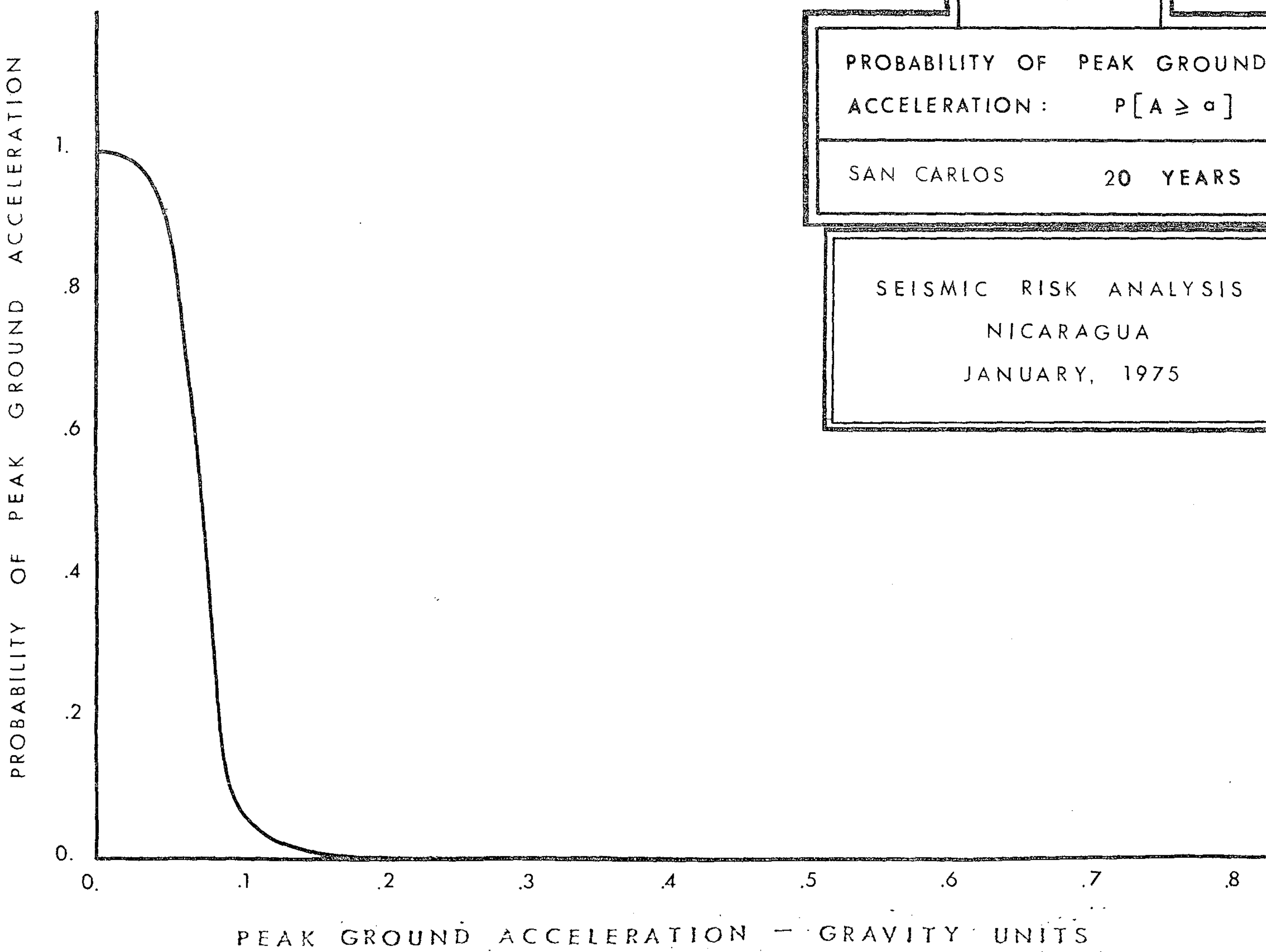
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

FIGURE 4-20

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

SAN CARLOS 20 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975



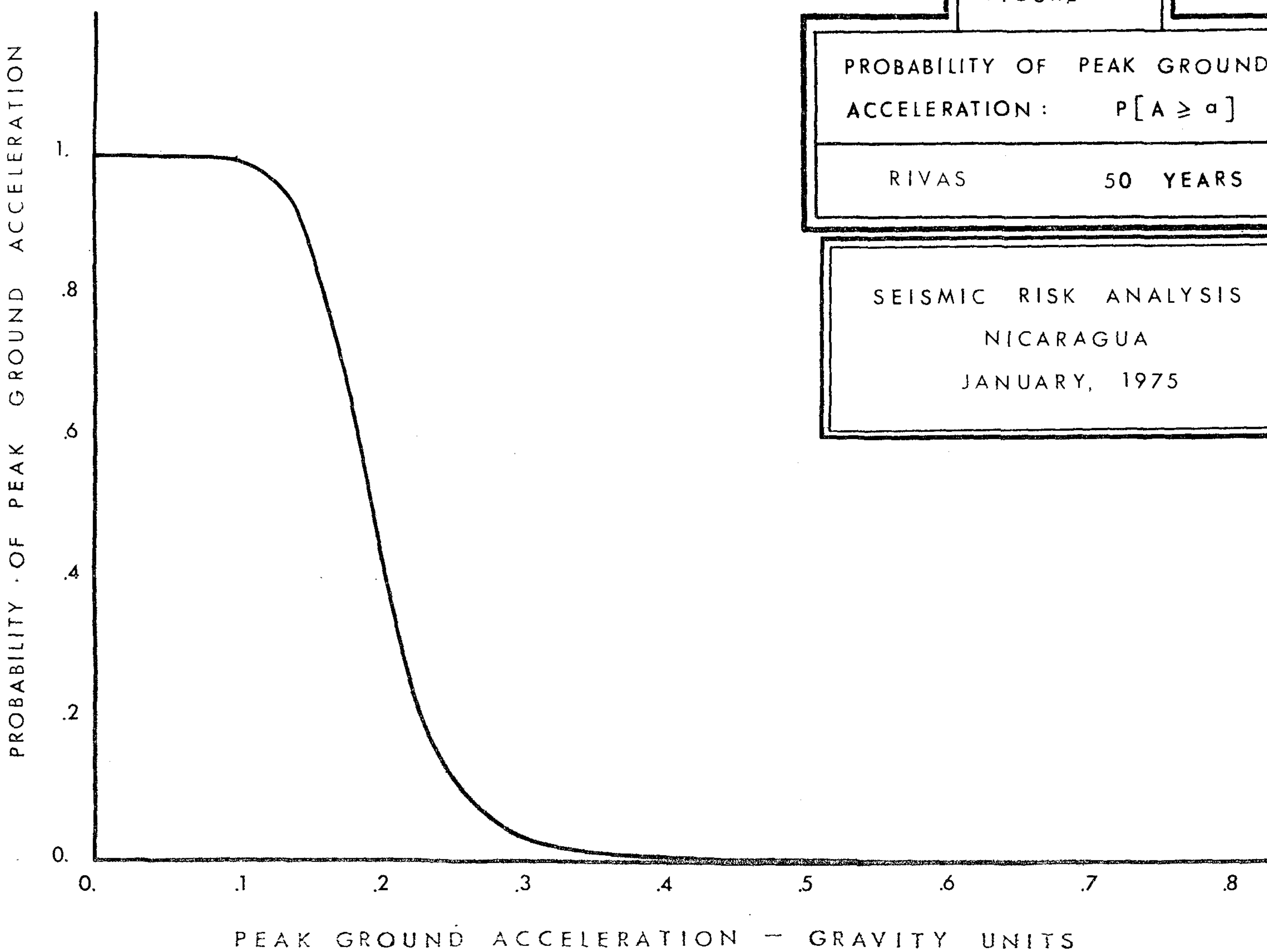


FIGURE 4-21

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

RIVAS 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

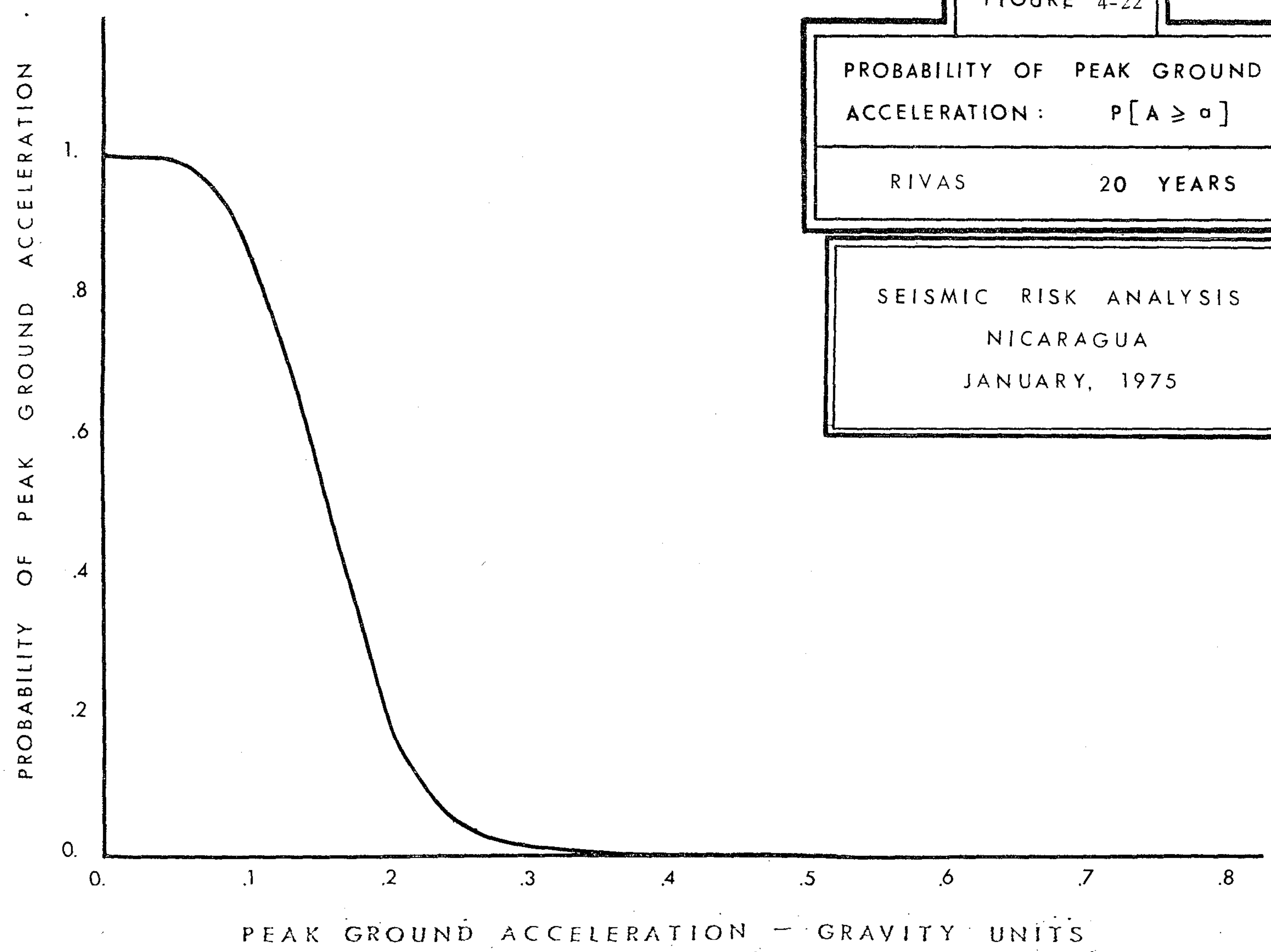


FIGURE 4-22

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

RIVAS 20 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

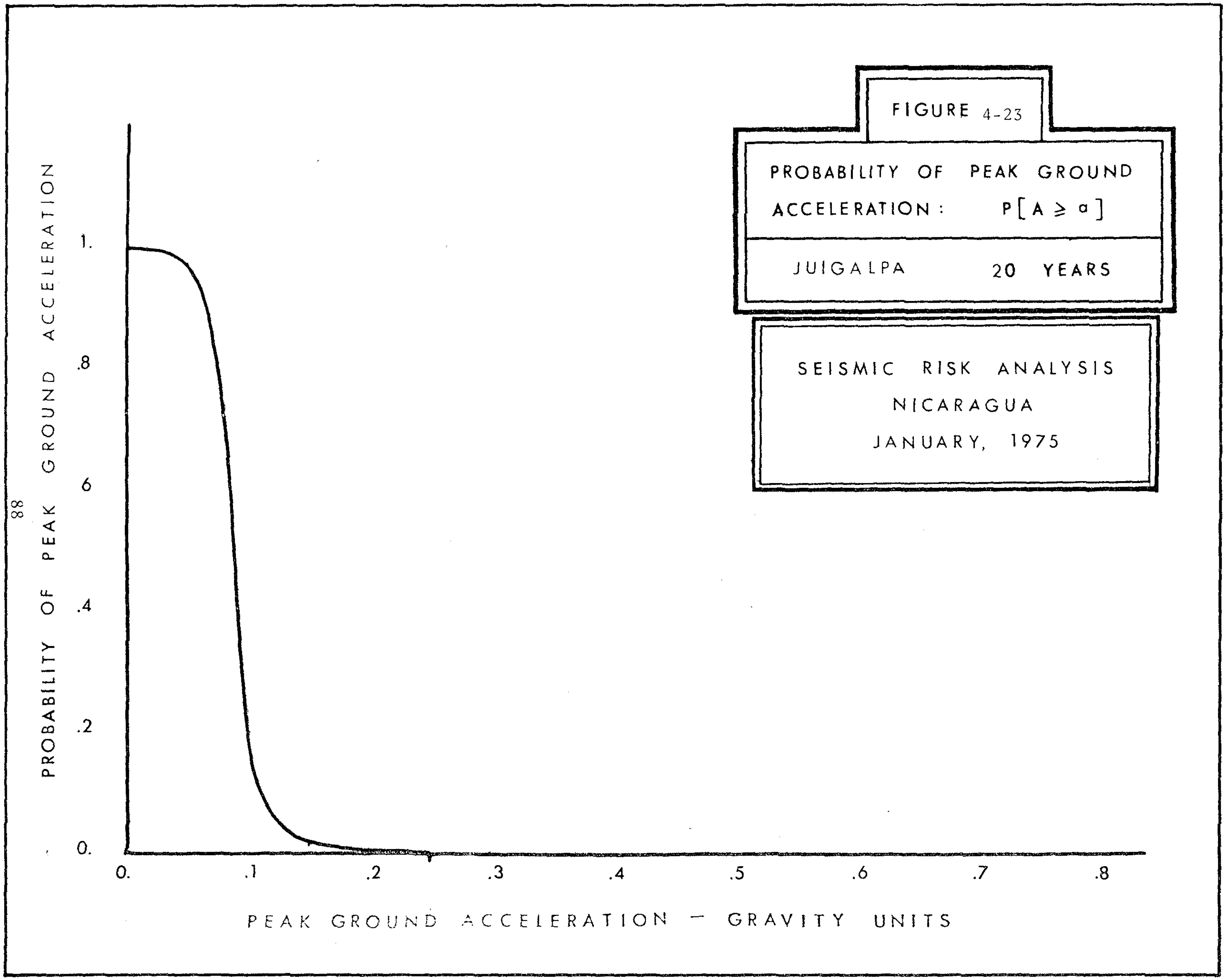


FIGURE 4-23
PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]
JUIGALPA 20 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
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6
.4
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.8
PEAK GROUND ACCELERATION — GRAVITY UNITS

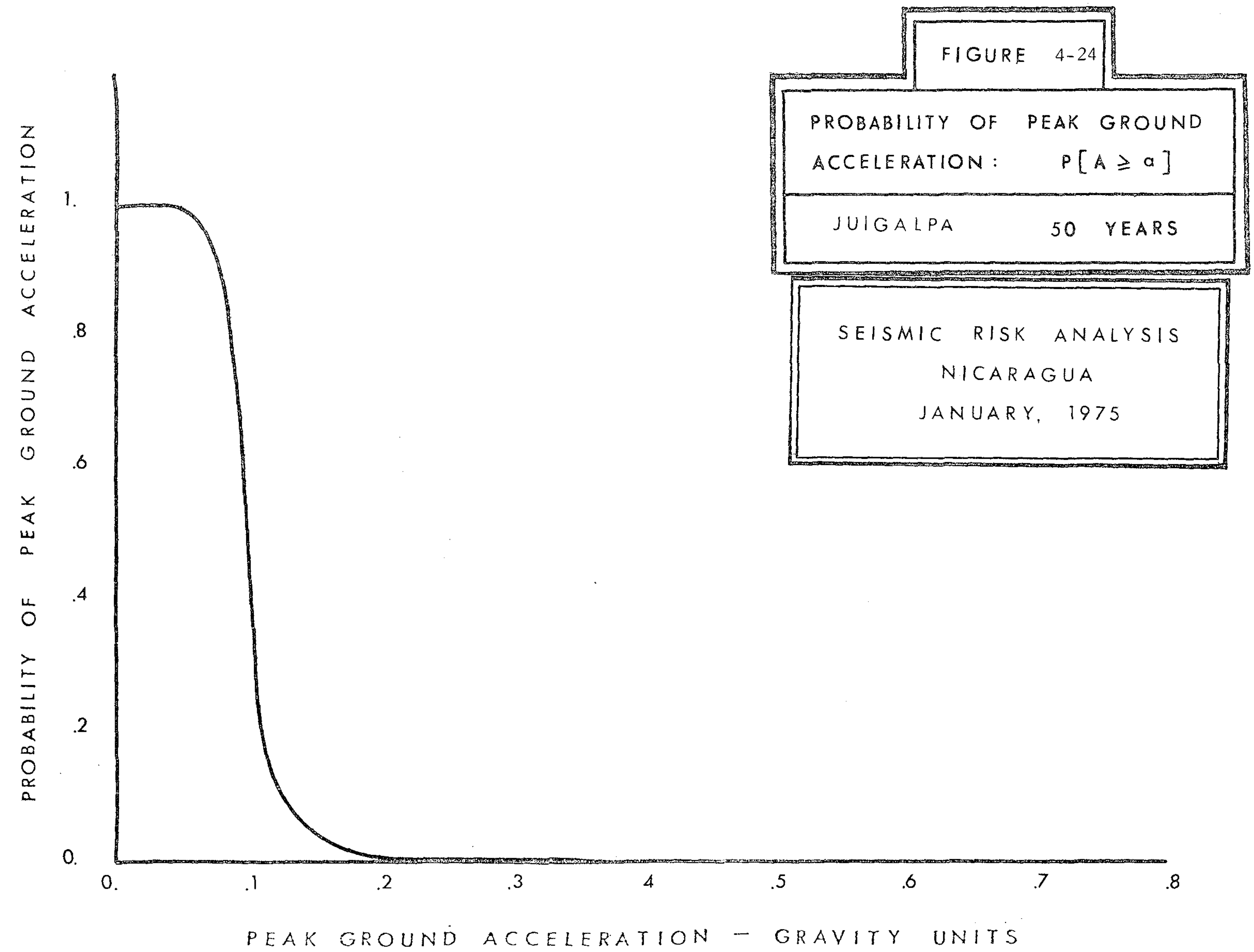
FIGURE 4-24
PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]
JUIGALPA 50 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
.8
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PEAK GROUND ACCELERATION — GRAVITY UNITS

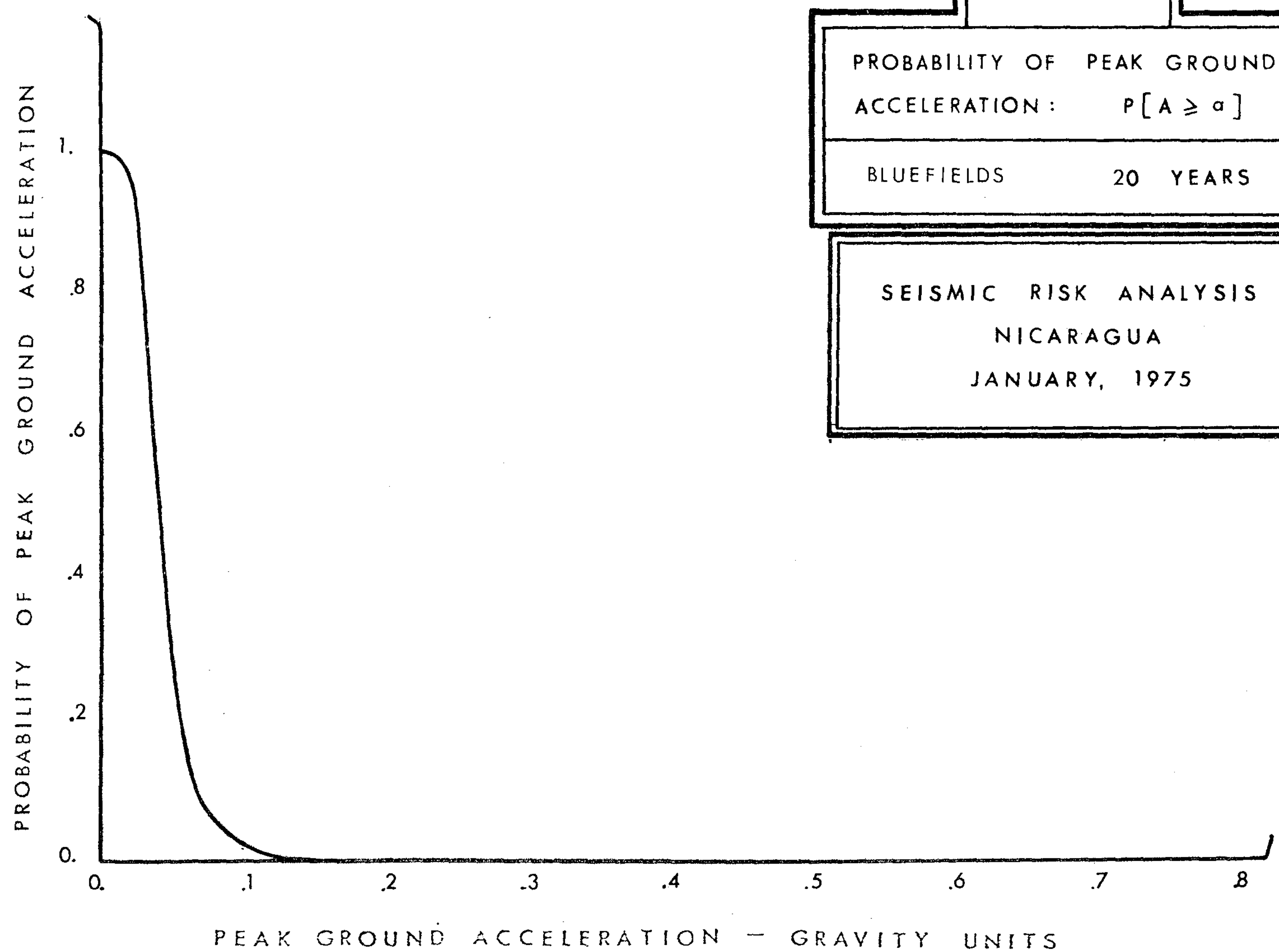

FIGURE 4-25

PROBABILITY OF PEAK GROUND ACCELERATION: $P[A \geq a]$

BLUEFIELDS 20 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

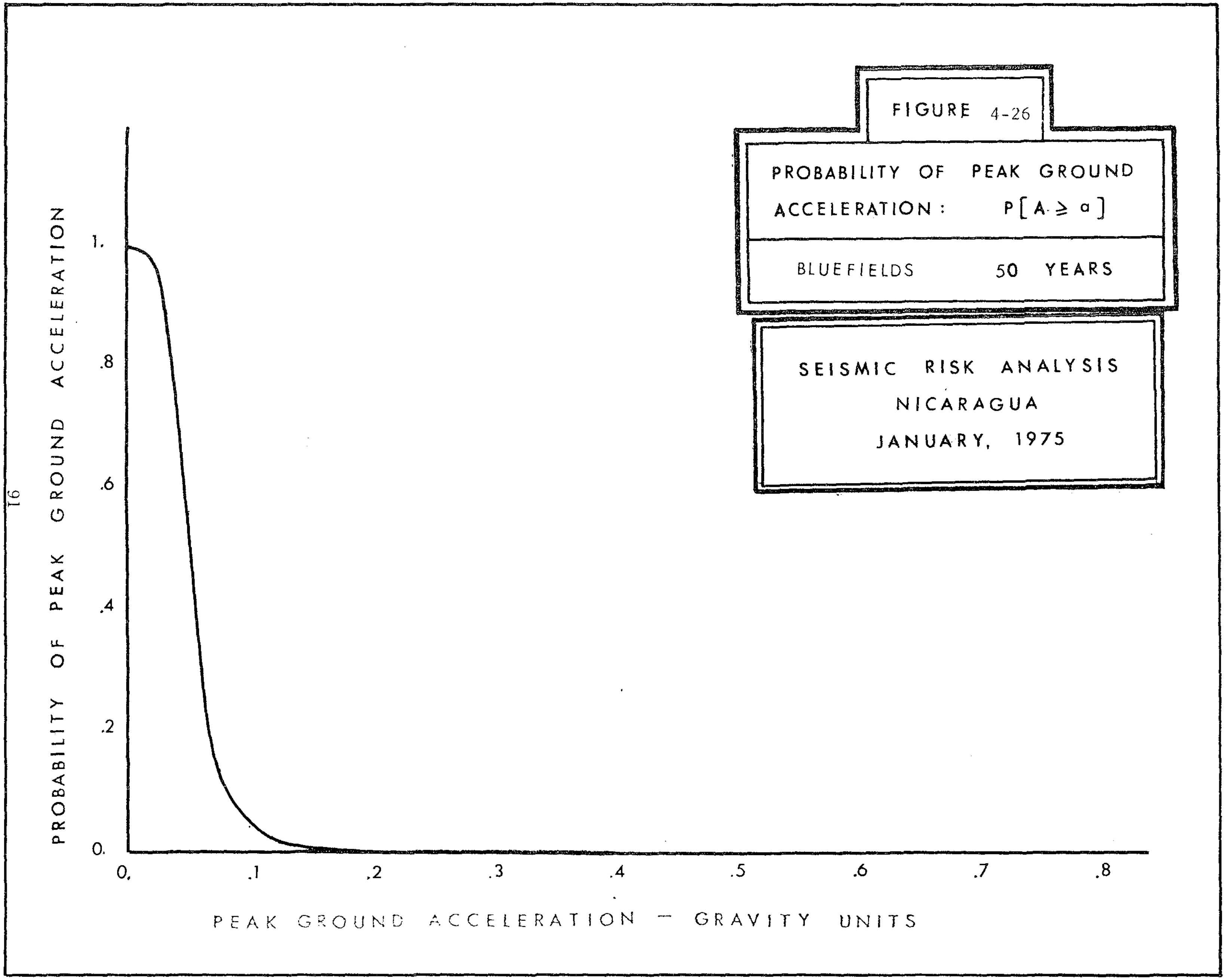
FIGURE 4-26
PROBABILITY OF PEAK GROUND ACCELERATION: P[A ≥ a]
BLUEFIELDS 50 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY OF PEAK GROUND ACCELERATION
1.
.8
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PEAK GROUND ACCELERATION — GRAVITY UNITS

maps and the other method of representing seismic risk is by means of cumulative distribution function plots of Figures 4-5 through 4-26.

The engineering interpretation of these results will be presented in the next three chapters. It should also be pointed out that the iso-acceleration maps and any zoning based on such maps only represent macro characteristics. The macrozoning of the country should be modified with site-specific micro characteristics to microzone a given region. In that case, the local geotechnical and geological features (such as those discussed in Chapter 2) should be incorporated together with the macro characteristics presented in this chapter.

CHAPTER V

SEISMIC RISK ZONING

Concept of Return Period and Acceleration Zone Graphs (AZG)

In deriving the probabilistic loading at a given site as a function of time, we have assumed that the forecasting process is Poisson. This process implies that the events are independent in time and space. Using this assumption and an appropriate attenuation relationship, we developed the iso-acceleration maps for the country. For a given city, we also developed the cumulative distribution function of the peak ground acceleration A, as mentioned in Chapter 4. Consider the cumulative distribution of peak ground acceleration in Managua for an exposure time of 20 years. (See Figure 4-5.)

Then $$P_{20}\ (A > 0.20g) = 0.73 \qquad 5\text{-}1$$

Equation 5-1 can be interpreted in the following way: "For Managua, there is a 73% chance that during the next 20 years, the peak ground acceleration of 0.20g will be exceeded at least once."

Thus, there is a 27% chance that for Managua, 0.20g peak ground acceleration will not be exceeded a single time.

Hence,

$$P\ (\text{Zero exceedence of } 0.20g \text{ in 20 years}) = 0.27.$$

From the Binomial Probability Law, we know that for independent trials with probability of success p at each trial, the probability of r successes in n trials is given by

$$p_n(r) = \binom{n}{r} p^r (1-p)^{n-r} \qquad 5\text{-}2$$

where

$$r = 0, 1, \ldots n; \quad n = r, r+1, r+2, \ldots$$

and

$$\binom{n}{r} = \frac{n!}{r!\,(n-r)!}$$

Let each trial be a one-year duration for which we are observing the level of peak ground acceleration. Let us define success as that event when the peak ground acceleration for a given trial (year) exceeds 0.2g. Thus, the probability of zero exceedence of level 0.2g in 20 years is the same as the probability of 0 successes in 20 trials. Hence, from Eq. 5-2:

$$p_{20}(0) = \binom{20}{0} p^0 (1-p)^{20}$$

$$p_{20}(0) = (1-p)^{20}$$

However,

$$p_{20}(0) = 0.27$$

$$(1-p)^{20} = 0.27$$

or $$p = 0.063.$$

Thus, for Managua, there is a 6.3% chance that in any given year, a peak ground acceleration of 0.20g will be exceeded.

However, the return period is defined as

$$\text{Return Period} = RP = \frac{1}{p} \tag{5-3}$$

Thus, the return period RP in Managua for a peak ground acceleration of 0.20g is $\frac{1}{0.063} \approx 16$ years.

It should be pointed out that this return period of 16 years corresponding to 0.2g, obtained by using the cumulative distribution function (CDF) of PGA A at Managua for a 20 year exposure time does not change if we use the CDF corresponding to 50 year exposure time. For example, for a 50 year exposure time, (see Figure 4-27),

$$P_{50} \ (A > 0.20g) = 0.963 \tag{5-4}$$

Hence, $\quad P_{50} \ (A < 0.20g) = 0.037$

Or $\quad P_{50} \ (0) = (1-p)^{50} = .037$

which gives $\quad p = 0.063 \qquad$ 5-5

and $\quad$ Return period RP $\approx$ 16 years.

Thus, using the CDFs for all the cities in Nicaragua considered in Chapter 4, we can develop a table of peak ground acceleration and return period. Table 5-1 is a general table giving this relationship for the cities considered. The following statements should be understood in using the concept of return period:

(1) A return period is the mean (or average waiting time for an event of interest. Thus, the average (waiting) time between 2 events producing 0.20g in Managua is approximately 16 years.

Table 5-1

Return Period in Years

PGA A in g units	Managua	Leon	Granada	Masaya	Chinandega	Matagalpa	Esteli	San Carlos	Rivas	Juigalpa	Bluefields
.05	1.5	2.0	3.0	2.0	3.0	6.0	7.0	8.0	3.0	6.0	60.0
.10	5.0	7.0	7.0	6.0	7.0	642.0	162.0	262.0	10.0	74.0	1127.0
.15	9.0	13.0	12.0	10.0	12.0	9990.0	2522.0	11755.0	23.0	1153.0	
.20	16.0	32.0	35.0	20.0	106.0				81.0		
.25	35.0	146.0	157.0	72.0	482.0				371.0		
.30	56.0	502.0	540.0	194.0	1657.0				1280.0		
.35	110.0	1430.0	1526.0	794.0							
.40	247.0	3562.0		2370.0							
.45	687.0										
.50	2975.0										

(2) The probability that an event corresponding to a return period RP will occur in any given year is given by $p = \frac{1}{RP}$. Thus, probability of exceeding 0.20g in Managua in any given year is $\frac{1}{16} \approx 0.063$ (same as Eq. 5-5).

(3) The probability that not a single event of the RP type will occur in RP years is given by $\frac{1}{e}$ where e = 2.718, the Naperian base. Thus, probability that in 16 years, there will not be a single event producing a peak ground acceleration of 0.20g in Managua is given by $\frac{1}{e} \approx 0.36$.

Thus, there is 64% chance that in RP years there will be at least one event of RP type. For Managua, there is a 64% chance that in 16 years, 0.20g peak ground acceleration will be exceeded. Consider again Table 5-1. For seismic zoning purposes, the following statements can be made:

The return period corresponding to a peak ground acceleration of 0.20g in Managua is 16 years, in Leon is 32 years, in Granada is 35 years, in Masaya is 20 years, in Chinandega is 106 years and in Rivas is 81 years. Thus, for each city, a graph relating the peak ground acceleration and return period can be plotted. Figures 5-1 through 5-12 show these graphs. We will refer to these graphs as Acceleration Zones. Figure 5-1 shows return period vs. peak ground acceleration for all the cities. It can be seen that for a given return period event (say, 100 years), Bluefields has the lowest value of peak ground acceleration (≈.05g) and Managua has the highest value of peak ground acceleration (.34g). The values for other cities lie between these two limits. Qualitatively, it can be said that for a

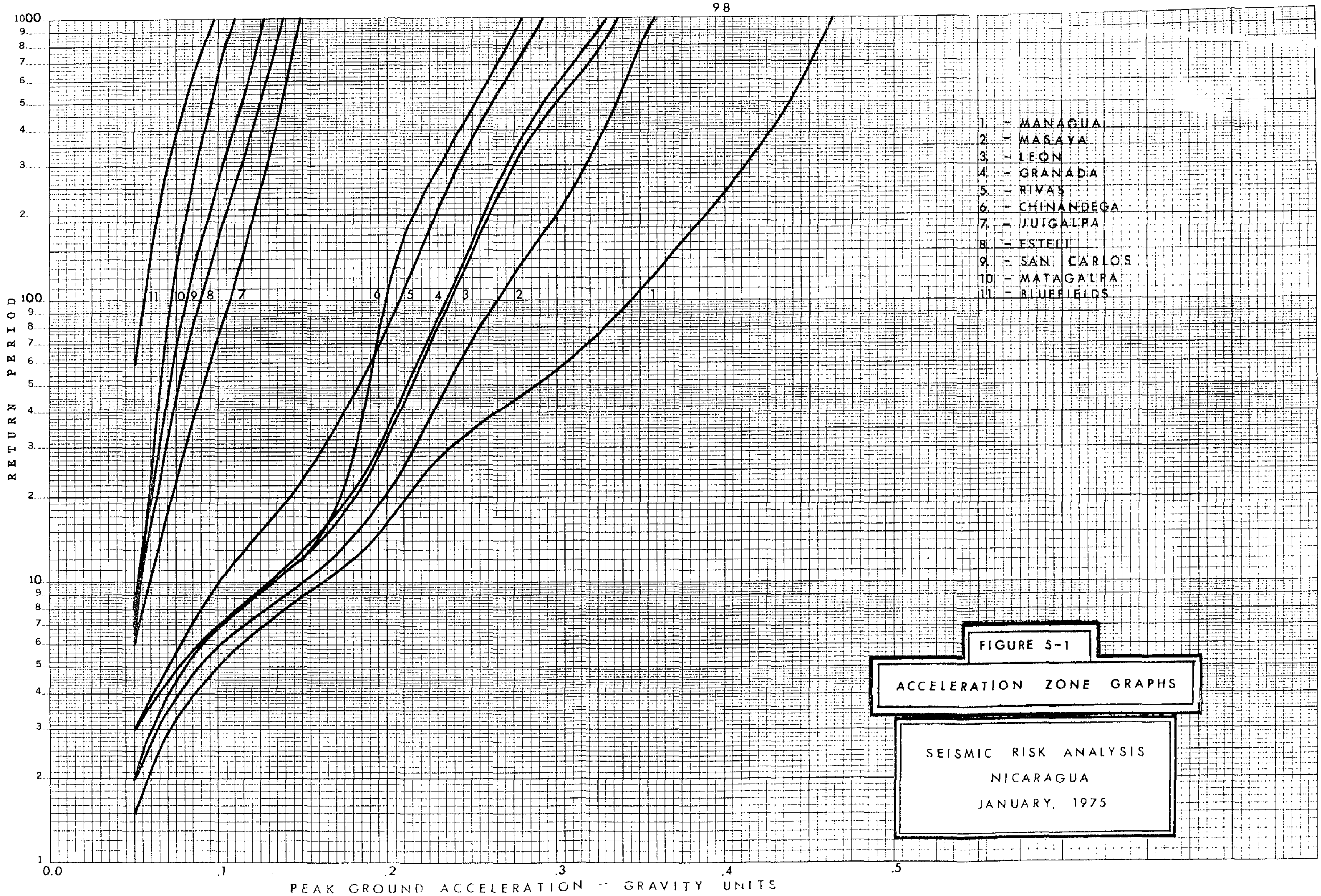


FIGURE 5-1

ACCELERATION ZONE GRAPHS

SEISMIC RISK ANALYSIS

NICARAGUA

JANUARY, 1975

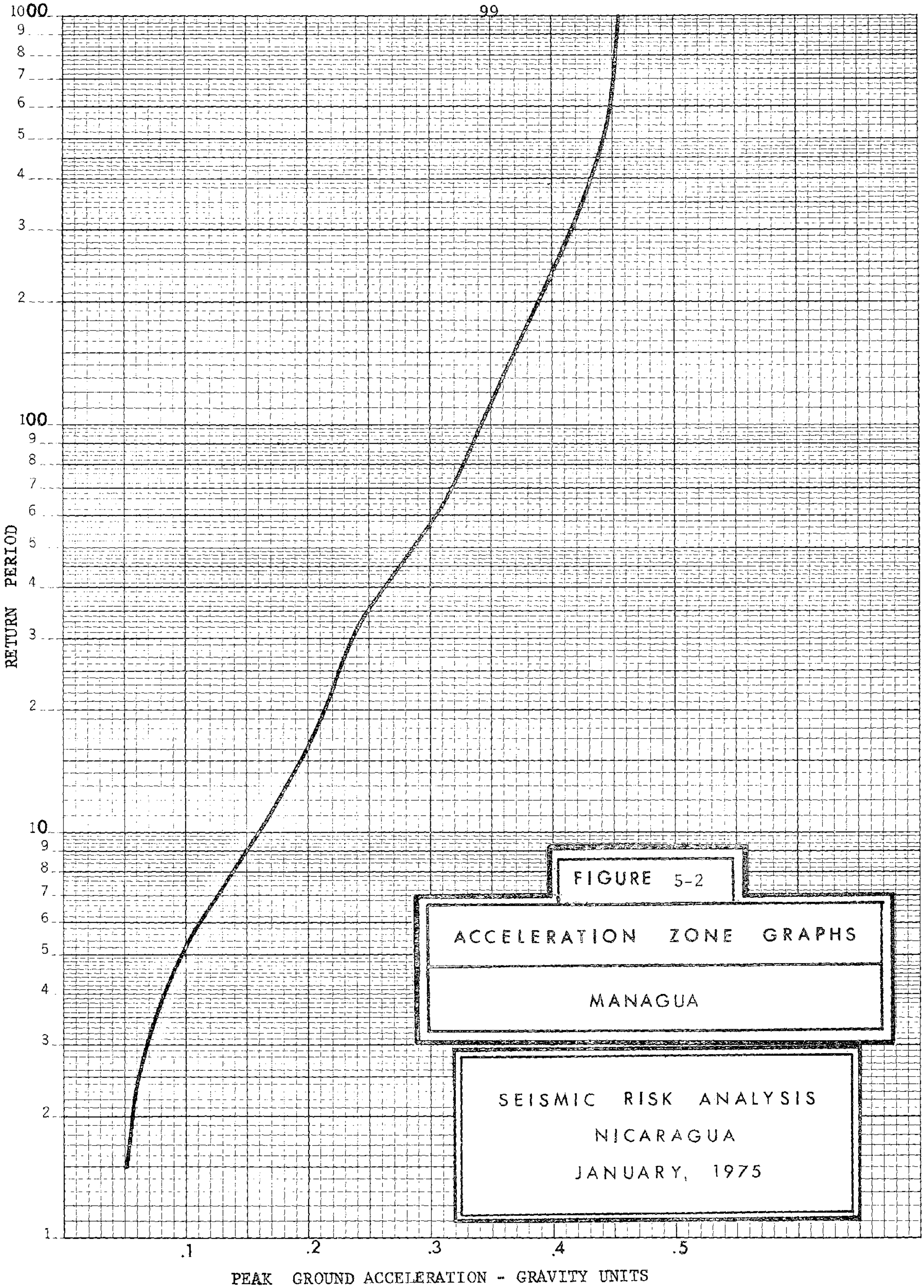


FIGURE 5-2

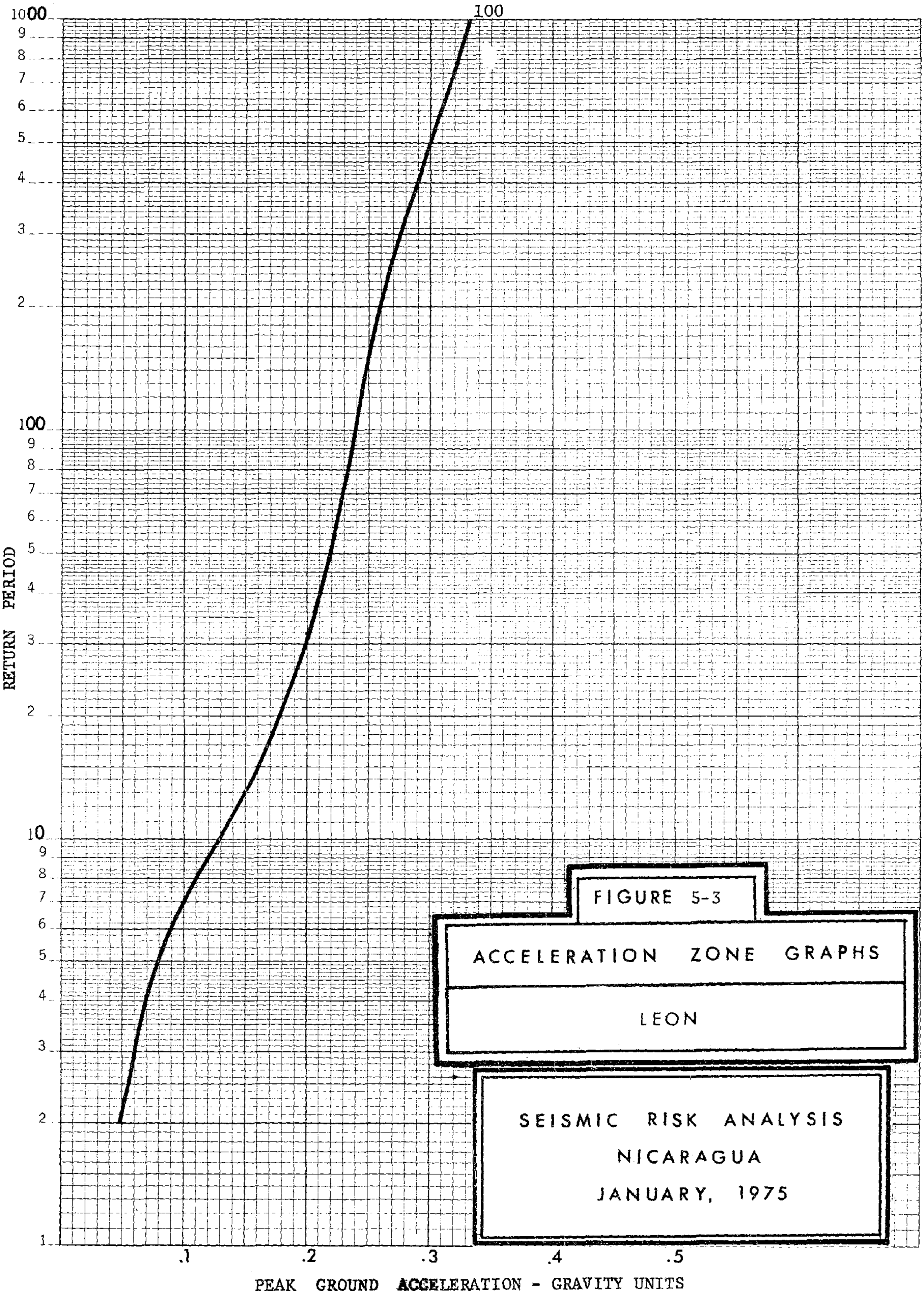
ACCELERATION ZONE GRAPHS

MANAGUA

SEISMIC RISK ANALYSIS

NICARAGUA

JANUARY, 1975


1000
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7
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4
3
2
100
9
8
7
6
5
4
3
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10
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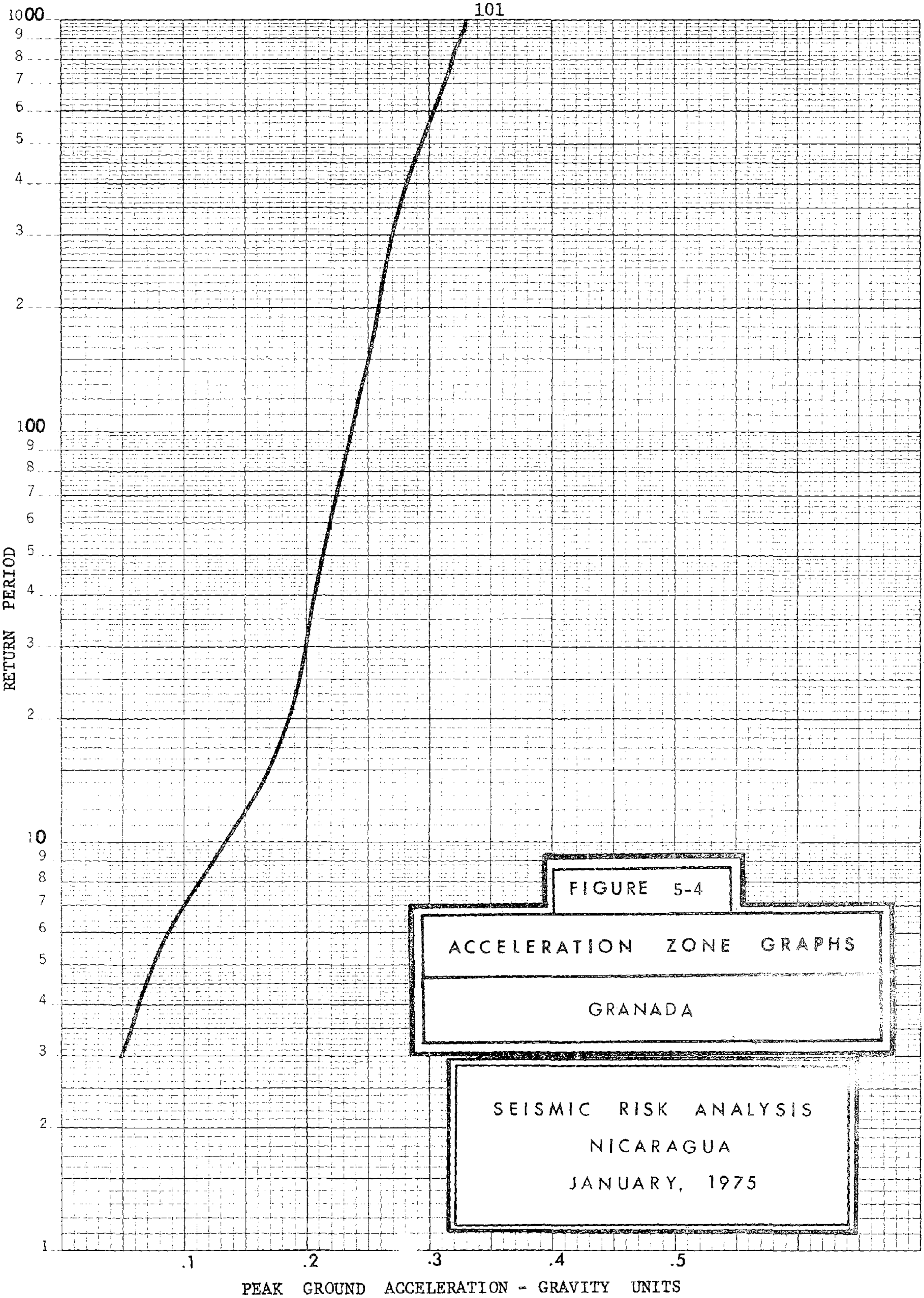
7
6
5
4
3
2
1
RETURN PERIOD
100
.1
.2
.3
.4
.5
PEAK GROUND ACCELERATION - GRAVITY UNITS
FIGURE 5-3
ACCELERATION ZONE GRAPHS
LEON
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975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6
5
4
3
2
1
RETURN PERIOD
.1
.2
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.5
PEAK GROUND ACCELERATION - GRAVITY UNITS
FIGURE 5-4
ACCELERATION ZONE GRAPHS
GRANADA
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

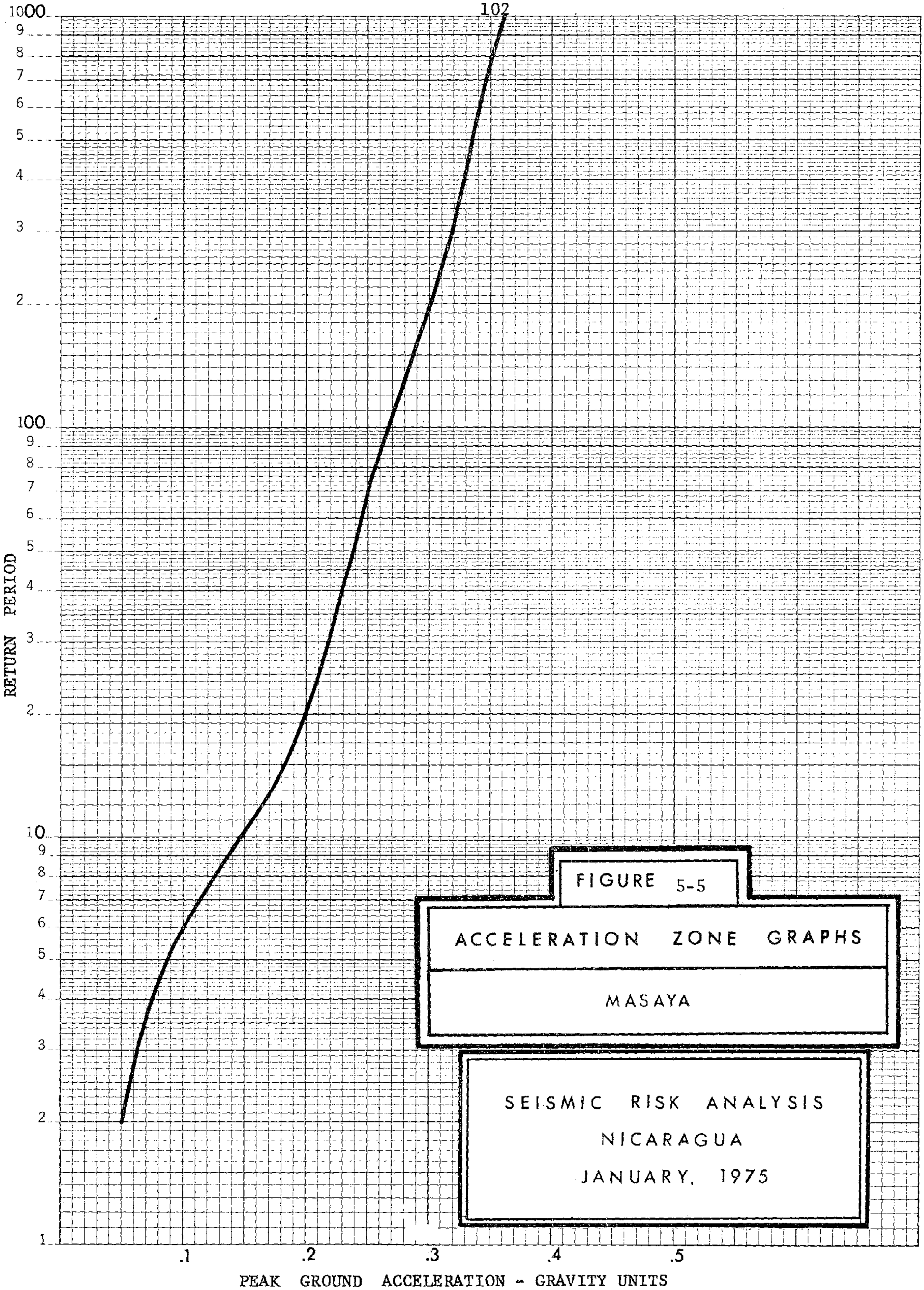
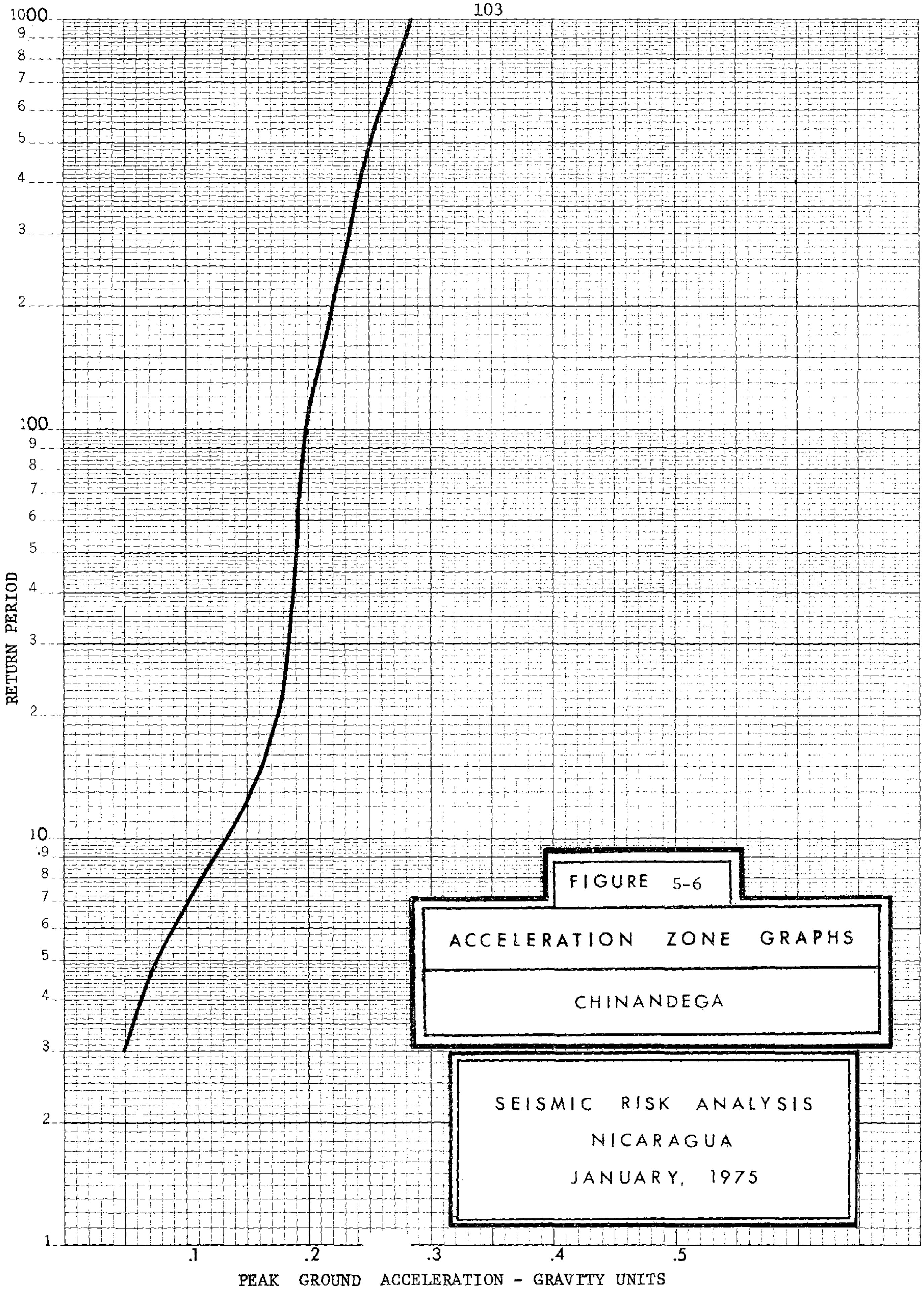


FIGURE 5-5

ACCELERATION ZONE GRAPHS

MASAYA

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975



1000
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1
RETURN PERIOD
.1
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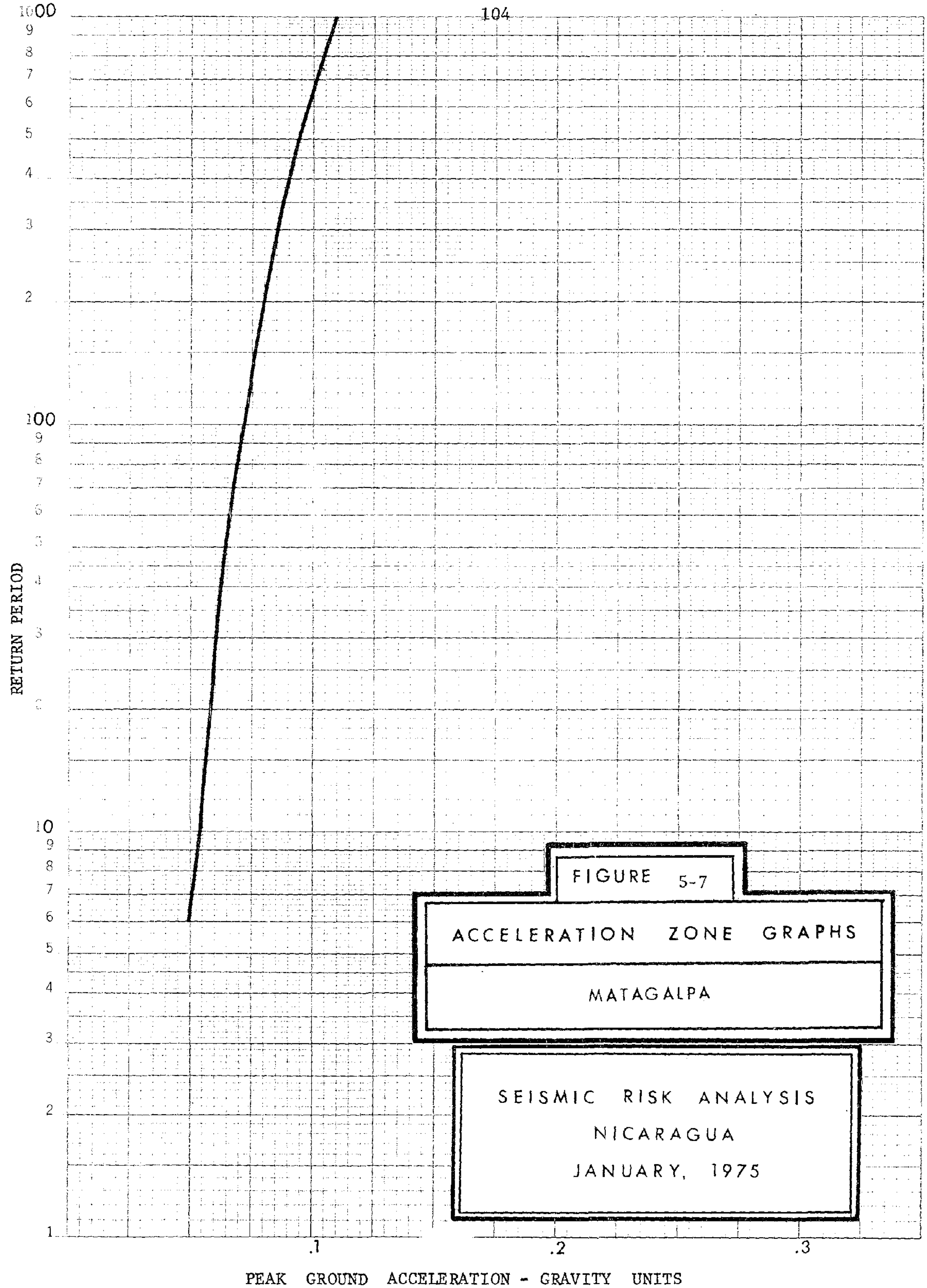
.3
.4
.5
PEAK GROUND ACCELERATION - GRAVITY UNITS
FIGURE 5-6
ACCELERATION ZONE GRAPHS
CHINANDEGA
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975


1000
100
10
1
RETURN PERIOD
.1
.2
.3
PEAK GROUND ACCELERATION - GRAVITY UNITS
FIGURE 5-7
ACCELERATION ZONE GRAPHS
MATAGALPA
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

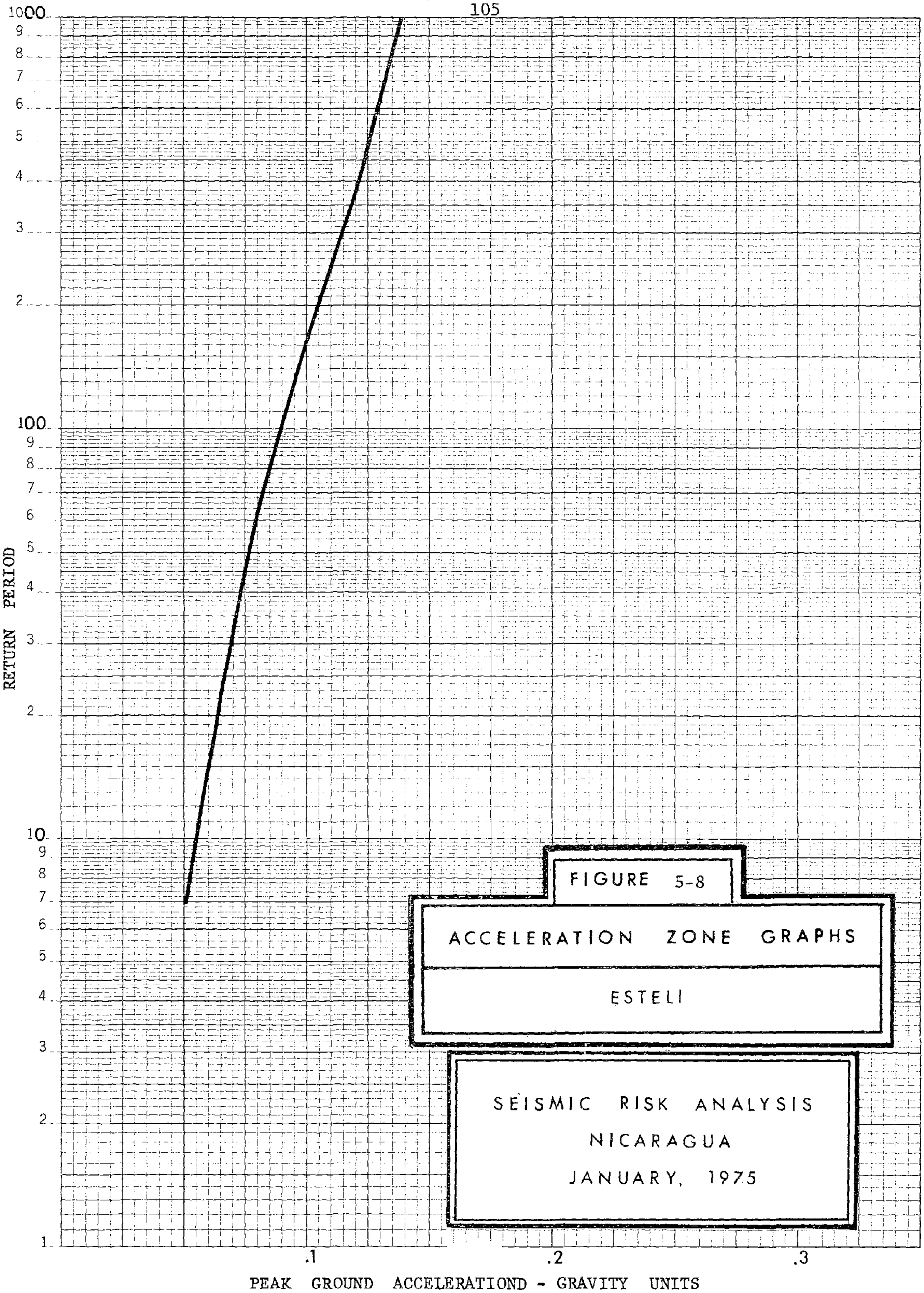


FIGURE 5-8

ACCELERATION ZONE GRAPHS

ESTELI

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

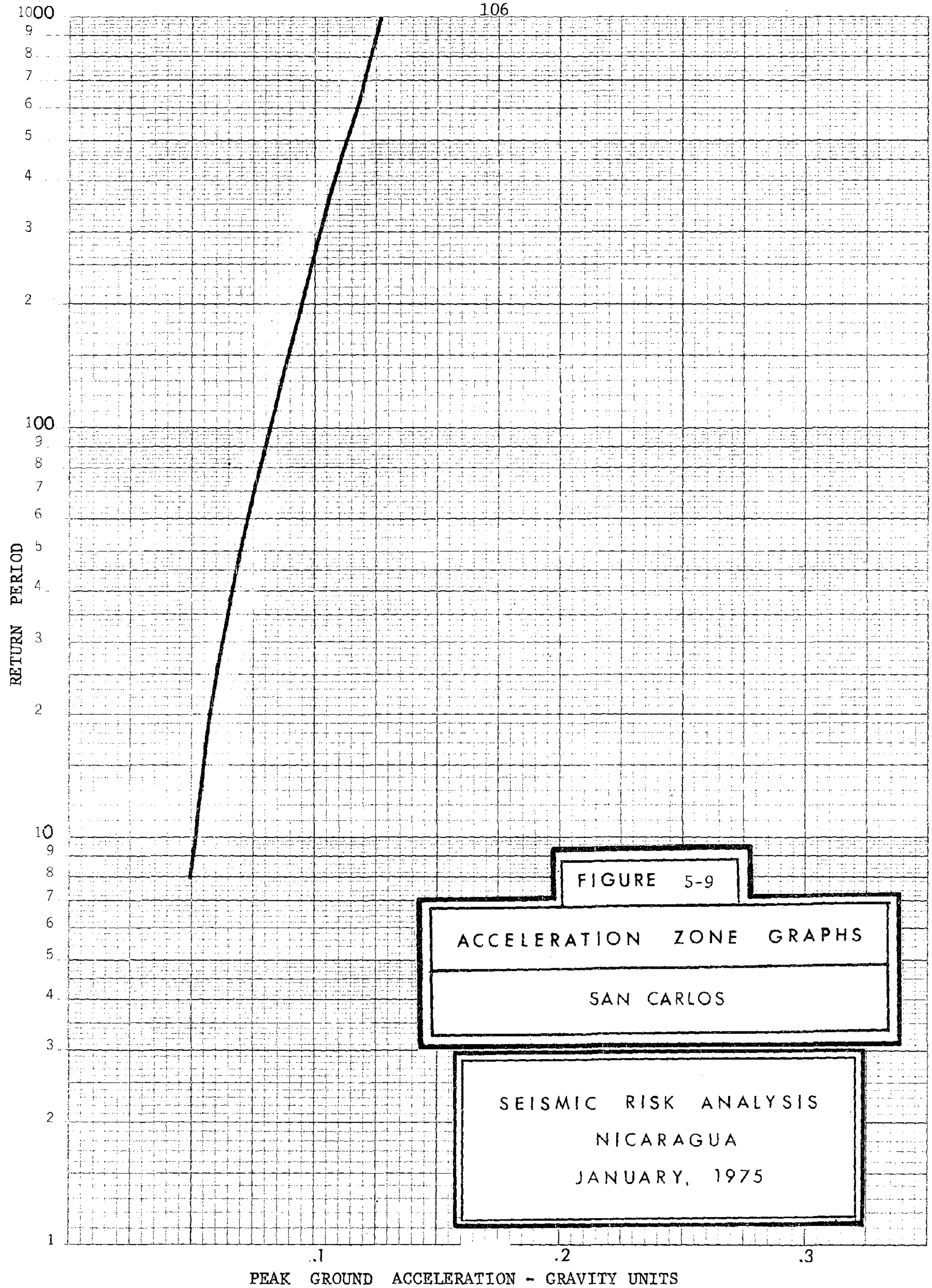


FIGURE 5-9

ACCELERATION ZONE GRAPHS

SAN CARLOS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

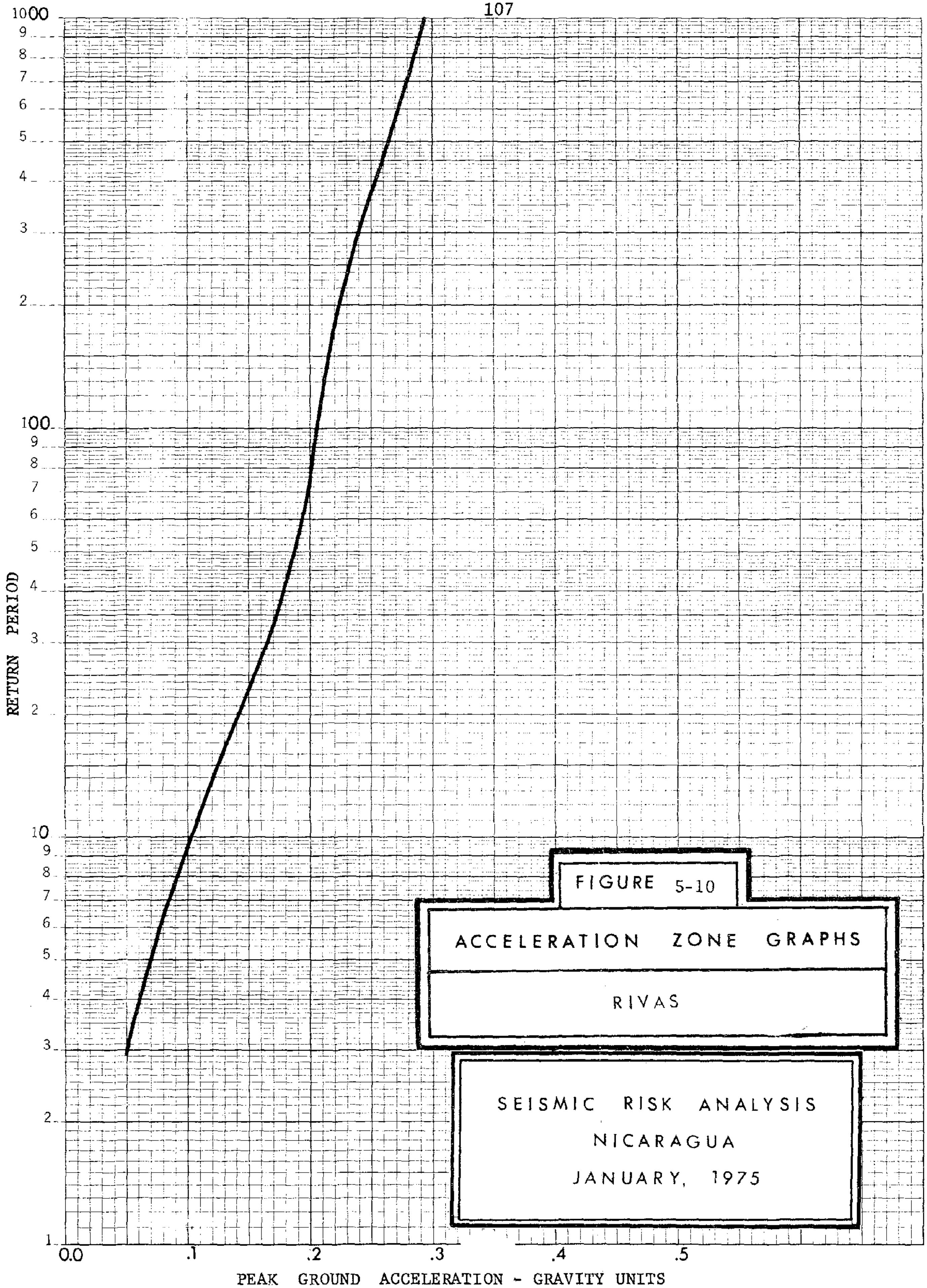


FIGURE 5-10
ACCELERATION ZONE GRAPHS
RIVAS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
RETURN PERIOD
1000
100
10
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0.0
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PEAK GROUND ACCELERATION - GRAVITY UNITS

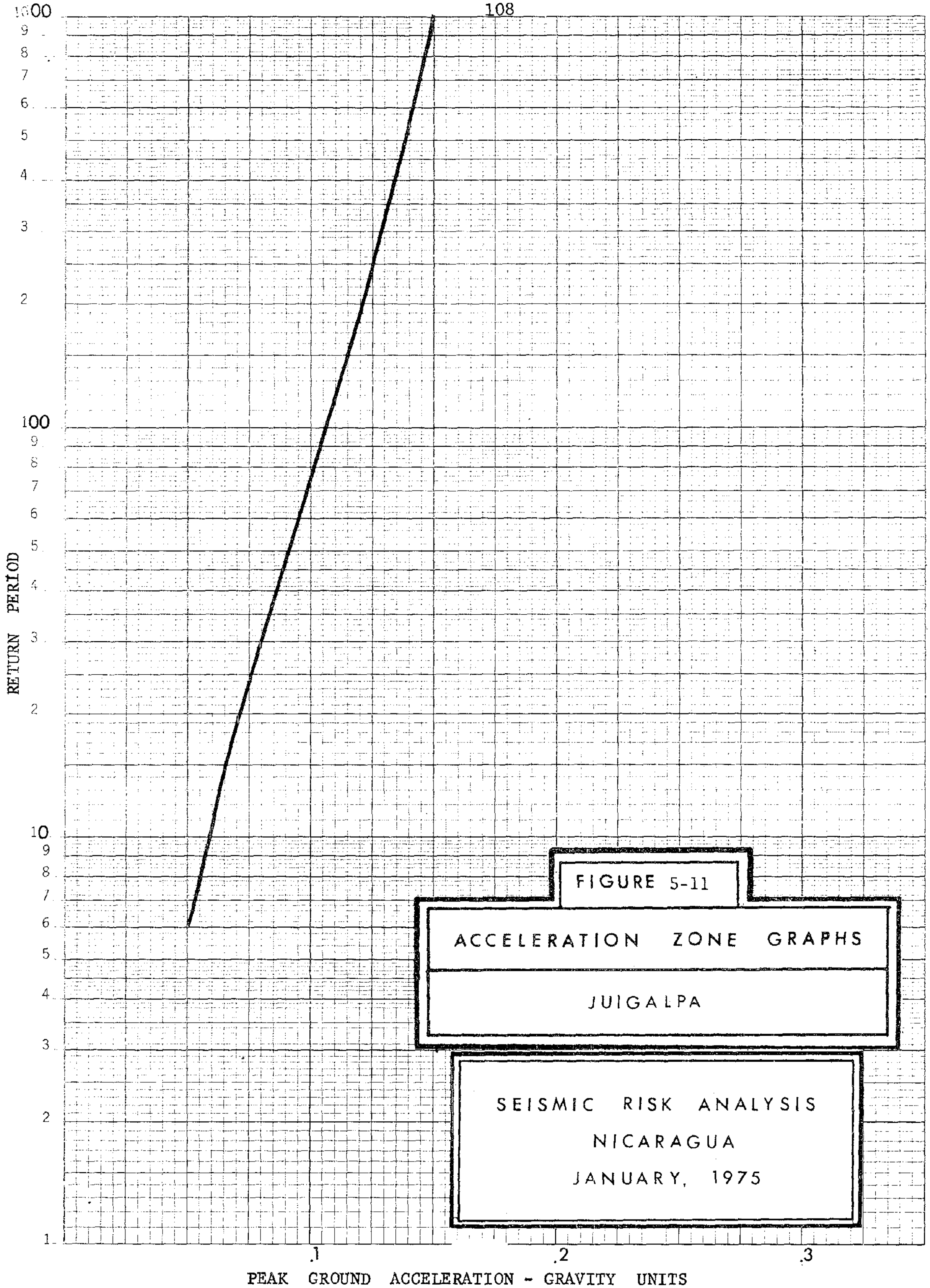

FIGURE 5-11

ACCELERATION ZONE GRAPHS

JUIGALPA

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

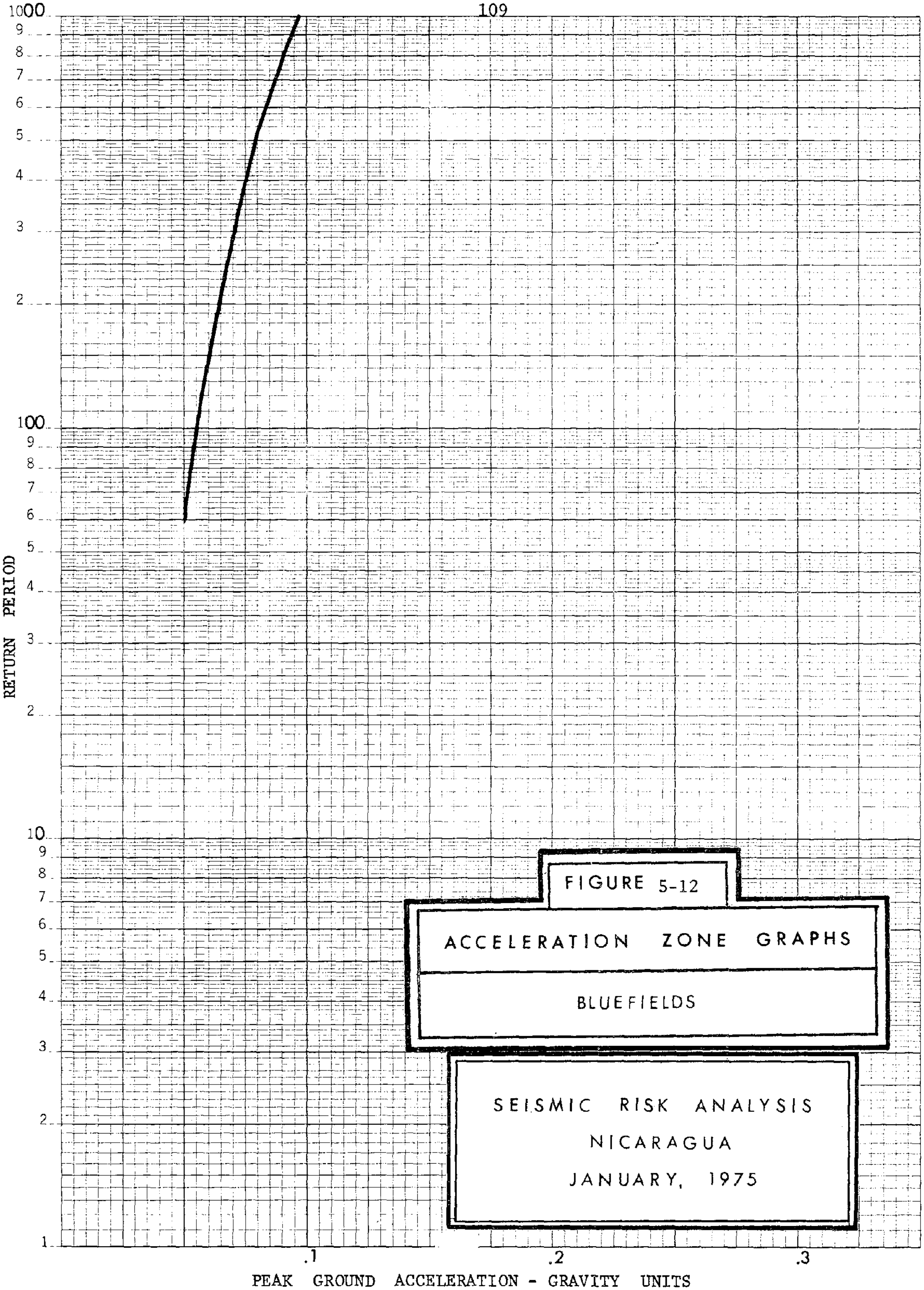

FIGURE 5-12

ACCELERATION ZONE GRAPHS

BLUEFIELDS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

facility requiring a design loading corresponding to a 200 year return period, Bluefields has the lowest seismic zoning requirement; San Carlos, Matagalpa, Esteli and Juigalpa have similar zoning requirements but above the Bluefields level; Rivas, Granada, Chinandega, Masaya and Leon, having similar zoning requirements, come next; and finally, the highest level is for Managua. This type of graph can help in macrozoning a country for a given class and use of a structure or facility. (See Chapter 7.)

Seismic Risk Zoning

In the previous section, we have seen the relationship between the peak ground acceleration and the corresponding return period for different major cities of Nicaragua. However, these relationships by themselves do not help in selecting a return period for a given acceptable level of risk. The next step, in any seismic zoning procedure, is to obtain a relationship between the economic (or exposure) life of a structure, the level of risk one is willing to take, and the return period consistent with the risk and economic life. Consider again the Binomial distribution. The probability of r successes in n independent Bernoulli trials, with probability p of success at each trial, is given by

$$p_n(r) = \binom{n}{r} \; p^r \; (1-p)^{n-r} \qquad \text{Eq. 5-2 repeated}$$

Thus

$$p_{10}\;(0) = \binom{10}{0}\;(p)^0\;(1-p)^{10}$$
$$= (1-p)^{10} = \text{probability of zero success in ten trials (years).}$$

Let $p(0) = (1-p)^{10}$ be equal to 0.90. Then the probability of no occurrence (or success) of a certain level of loading in ten years is given by 0.90.

or
$$(1-p)^{10} = 0.90$$

Hence
$$p = .01048$$

or return period RP = 95 years.

Thus, for a structure whose economic life is ten years, if the acceptable risk level is 90% of not exceeding the specified loading level (i.e., 10% of exceeding), then the structure should be designed for a return period of 95 years. Table 5-2 gives the relationship between acceptable risk level, economic life and return period. If, for example, the acceptable risk level is 80% for a structure whose economic life is 50 years, then the loading level should correspond to a return period of 225 years. If this structure is in Managua, the corresponding peak ground acceleration level is approximately 0.39g. If the same facility for the same risk level is to be built in Matagalpa, the corresponding peak ground acceleration level should be approximately 0.12g. Thus, for a given class and use of structure, having the same economic life (50 years) and same acceptable risk (80%), the two consistent values of peak ground accelerations in Managua and Matagalpa are 0.39g and 0.12g. This is the concept of consistent risk design from one seismic region to another region of different seismicity. Figure 5-13 shows the graph relating the risk level, economic life and the return period. This particular graph is independent of any region and gives return periods only as functions

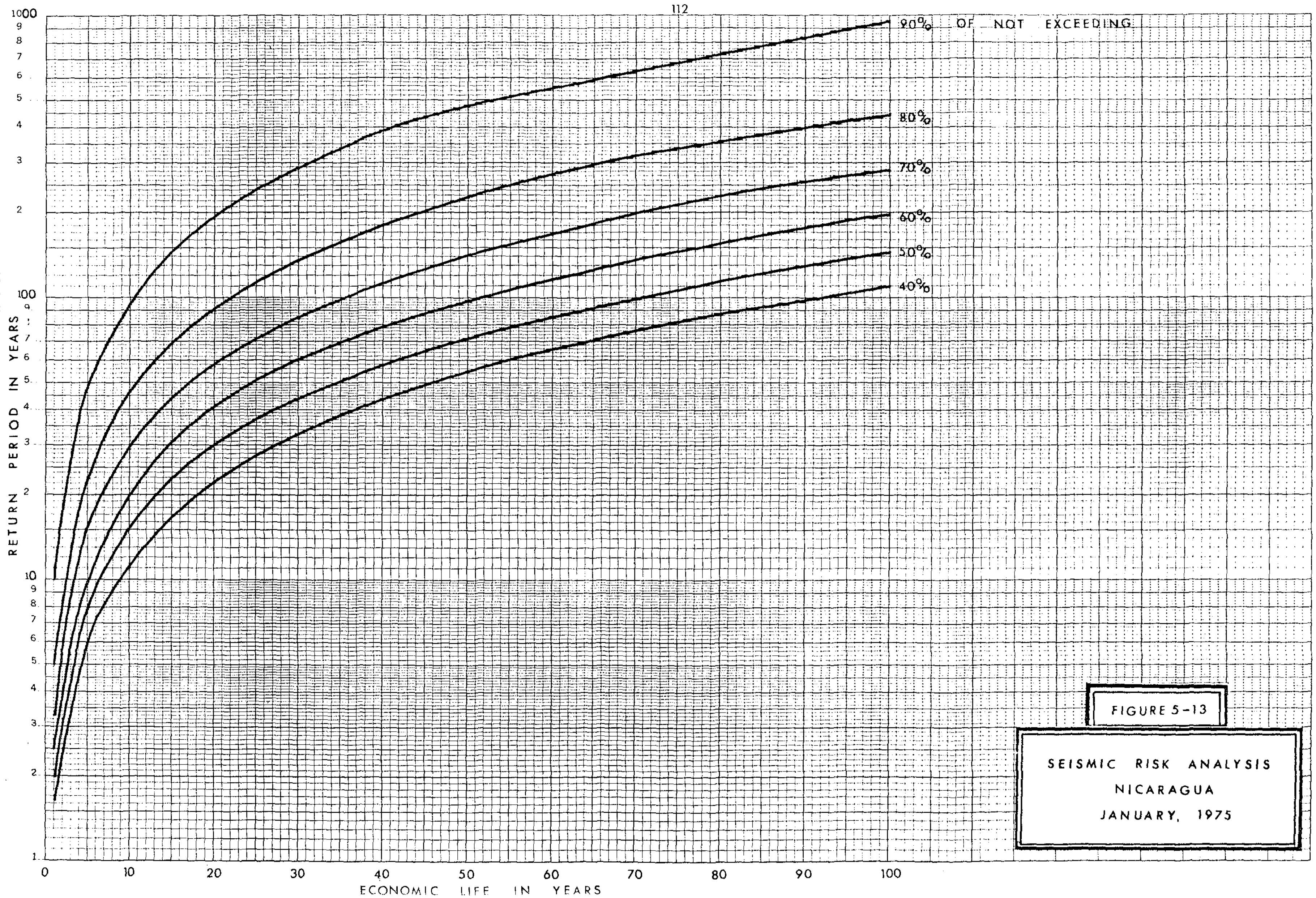

FIGURE 5-13

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

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Table 5-2

Return Period as a Function of Economic Life and Probability of Non-exceedence

Economic Life Years / Probability of not exceeding %	10	20	30	40	50	100
90	95	190	285	390	475	950
80	45	90	135	180	225	449
70	29	57	84	113	140	281
60	20	40	59	79	98	196
50	15	29	44	58	72	145
40	11	22	33	44	55	110
30	9	17	25	34	42	84
20	7	13	19	25	31	63
10	5	9	14	18	22	44
5	4	7	11	14	18	34
1	3	5	7	9	11	22
0.5	2	4	6	8	10	19

of risk and economic life. Such graphs can easily be codified. Once the acceptable risk level for a given economic life is selected for a given class and use of a structure, the corresponding return period is immediately obtained from Figure 5-13. Then, based on the graph of return period vs. peak ground acceleration (similar to Figures 5-1 to 5-12), the loading at a site can be determined. Let us describe this concept of risk, economic life, return period and Acceleration Zone Graphs (AZG).

As an example, consider a design of a hospital facility. Assume that the exposure time or economic life of the system is 50 years. We are to determine the peak ground acceleration level for which this facility should be designed for each of three different cities. The cities are Managua, Leon, and Esteli. Assume that for the hospital, which is a critical facility that must remain functional after a seismic event, the acceptable level of risk corresponding to damage is 20% (see Chapter 7 for details). Thus, whether the planned facility is in Esteli, Managua, or Leon, we will accept a 20% chance of damage during the 50 years economic life of the structure. Then, from Figure 5-13, the return period corresponding to the 50 year economic life and 20% risk is 225 years.

Now let us refer to the AZG corresponding to Managua (see Figure 5-2). The peak ground acceleration for a 225 year return period in Managua is 0.39g. Similarly, referring to the AZG for Esteli and Leon, the peak ground acceleration values corresponding to the 225 year return period are 0.11g and 0.27g. Thus, these three values of peak ground acceleration in the three different cities are consistent with the given acceptable risk.

As an alternate situation, consider two separate classes of structures to be built in Managua. Let a school building with an economic life of 30 years have an acceptable risk level of 20%, and a warehouse with a ten year economic life have a 40% acceptable risk level. Referring to Figure 5-13, the return period for which the school should be designed is 135 years, and the return period for which the warehouse should be designed is 20 years. Again from the Managua AZG (Figure 5-2), the corresponding peak ground acceleration values are 0.36g and 0.21g for the school and warehouse, respectively. If the same two facilities were to be located in Juigalpa, the corresponding peak ground acceleration values would be 0.11g and 0.07g. The major advantage of this method of zoning is that one can keep a consistent risk level from one region to another. Variations in the economic life and acceptable risk levels can be accounted for in arriving at a loading level through the return period transformation. Further application of the AZG to structural design will be presented in Chapter 7 of this report and in Part II of the total study.

CHAPTER VI

PROBABILISTIC INTENSITY FORECASTING -- DAMAGE ESTIMATION

MMI Forecasting

Seismic zoning or seismic risk can be presented, as shown in the previous section, in the form of iso-acceleration maps or cumulative distribution functions (CDF) of peak ground accelerations for different cities or by acceleration zone graphs (AZG). Another very informative and useful parameter for representing future seismic risk is in the form of Modified Mercalli Intensity Scale MMI (see Appendix 2 for definition). This intensity scale describes the behavior of different types and classes of masonary and frame structures at a site due a seismic event. Recently, there has been a considerable amount of work in correlating the damage observed due to a seismic event. (See references 35, 36, and 37.) This damage correlation is usually with the MMI level for a given region. In previous chapters, we have already obtained the probabilistic loading level in the form of peak ground acceleration. Various empirical relationships are available to convert peak ground acceleration to the MMI Scale.
The MMI Scale is discrete, whereas the acceleration scale is continuous. To obtain this discrete probability mass function of the MM intensity at different parts of the country for 20 years and 50 years, a Monte Carlo simulation process was used. The procedure can be described as follows:

(1) Obtain the CDF for peak ground acceleration A for the region under study. (See Figures 4-5 to 4-26.)

(2) Select an empirical equation to convert peak ground acceleration to MMI . The relationship used in this report (see reference 38) is

$$\log_{10}(a) = \frac{I}{3} - 0.5 \qquad 6\ 1$$

where a is the acceleration in cm/sec^2

I is the MM intensity.

Thus, for example, the peak ground acceleration at a site is 0.10g, then

$$a = 0.10g = \frac{10}{100} \times 981.46$$

$$= 98.146.$$

$$\log_{10} 98.146 = \frac{I}{3} - 0.5$$

or

$$I = 7.5.$$

Since I has to be VII or VIII, we pick I = VIII.

(3) Through random number generation, pick a value of the peak ground acceleration by using CDF for the PGA. Substitute this generated value of PGA in equation 6-1 and obtain I.

(4) Repeat step (3) n times and draw a histogram of frequency chart for I. As $n \to \infty$, the frequency of I will approach the probability mass function of I.

This process was repeated for all eleven cities mentioned in

Chapter 5. A time period of 20 years and 50 years was selected for convenience. Figures 6-1 through 6-11 show the probability mass functions for all the cities. The interpretation of these graphs can be explained by means of an example.

For Managua, during the next 20 years, probability that the maximum Modified Mercalli Intensity I will be VIII is given by 0.39 (see Figure 6-1).

Thus,

$$P\ [\text{Maximum MMI in 20 years will be VIII}] = 0.39.$$

Similar statements can be made for other parts of the country. Iso-seismal maps based on such forecasting can be generated for the whole country. It should be pointed out that intensity of shaking is very much a function of local geologic and soil conditions. For proper evaluation of these parameters, a detailed site-specific study and micro-characterization of the site are needed. The values presented here are based on macro study.

"Insurance Risk" or Damage Potential

We have not correlated the damage data for Nicaragua with the observed past intensities. This study will be presented in part II of the current study. Such correlation can be made by observing the percentage damage in a given class of structures due to an observed past seismic event. From the 1972 earthquake in Managua, the information on such Intensity-Damage correlation can be obtained. However, we will present a methodology of using such information to estimate the "insurance risk" for a given region.

FIGURE 6-1

PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]

MANAGUA 20 / 50 YEARS

SEISMIC RISK ANALYSIS
NICÁRAGUA
JANUARY, 1975

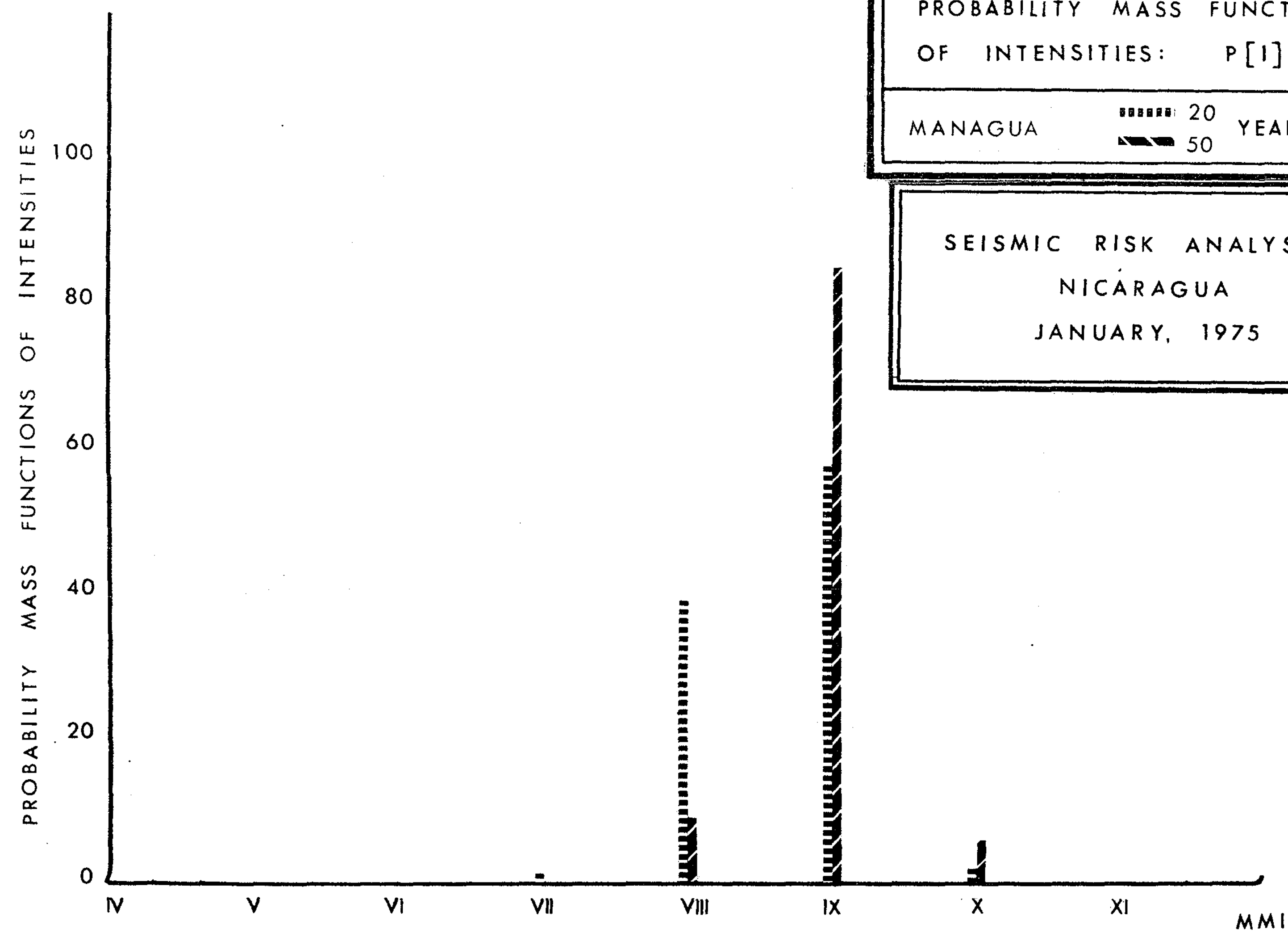
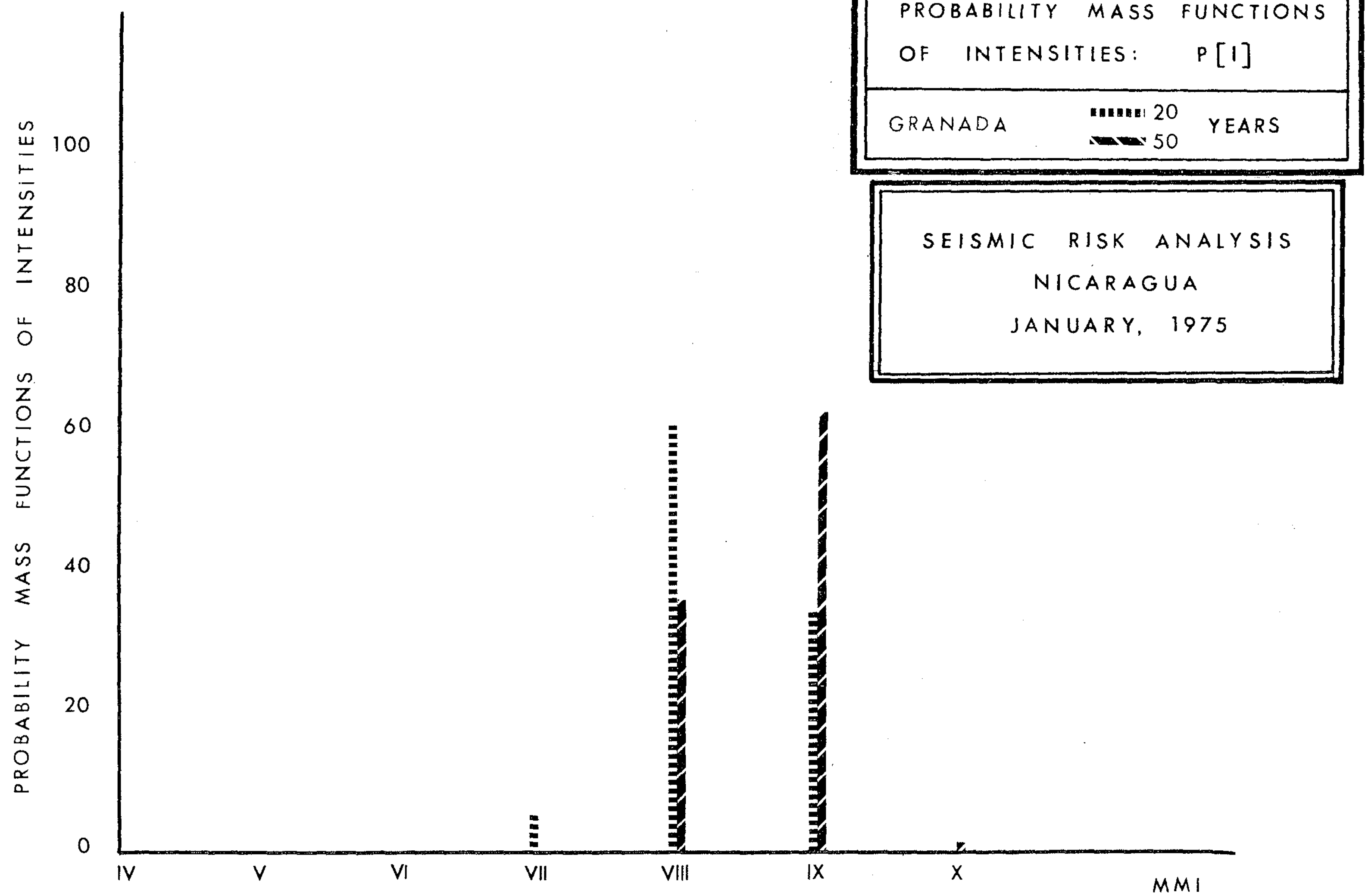


FIGURE 6-2

PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975



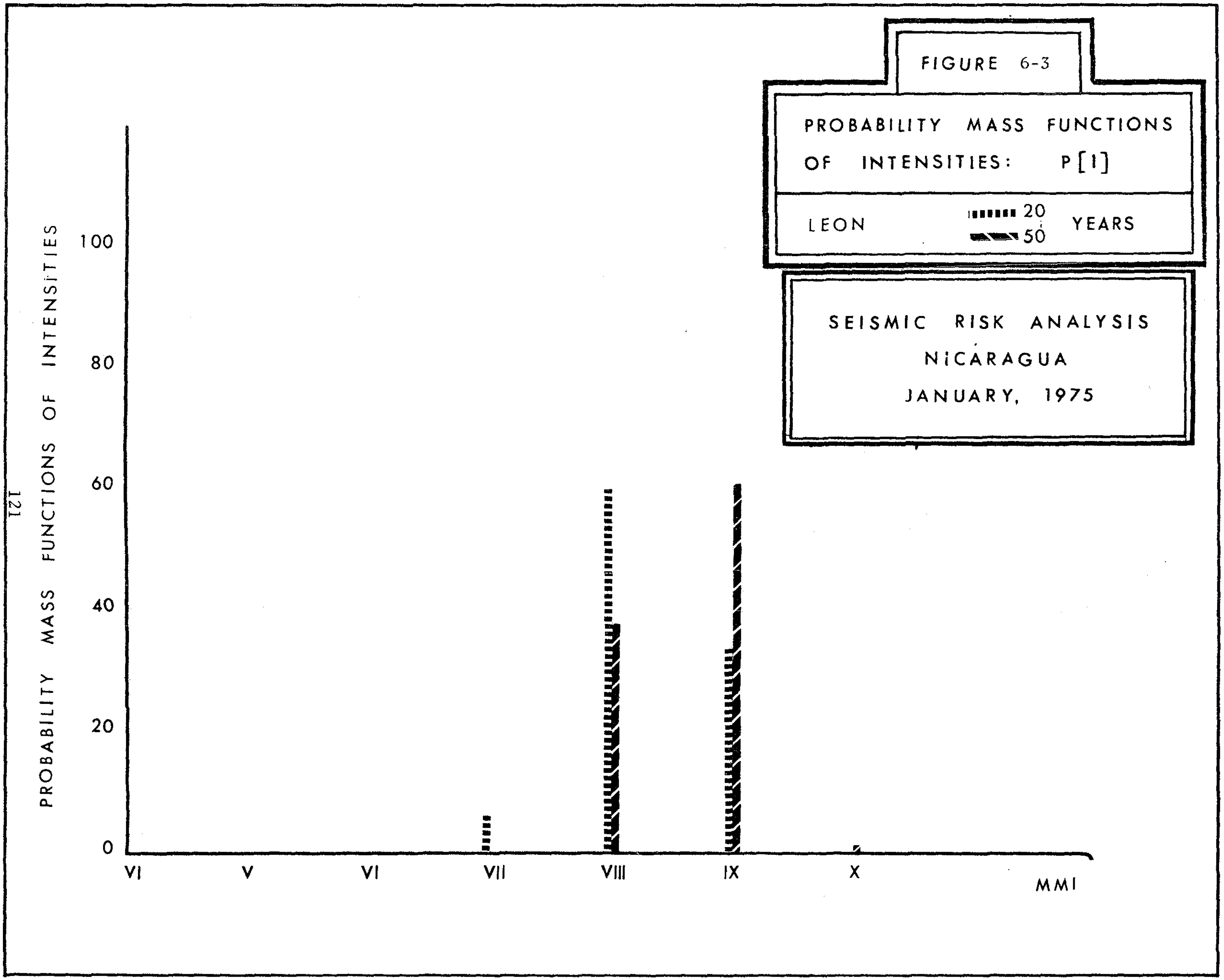


FIGURE 6-3
PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]
LEON
20
50
YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY MASS FUNCTIONS OF INTENSITIES
100
80
60
40
20
0
VI
V
VI
VII
VIII
IX
X
MMI

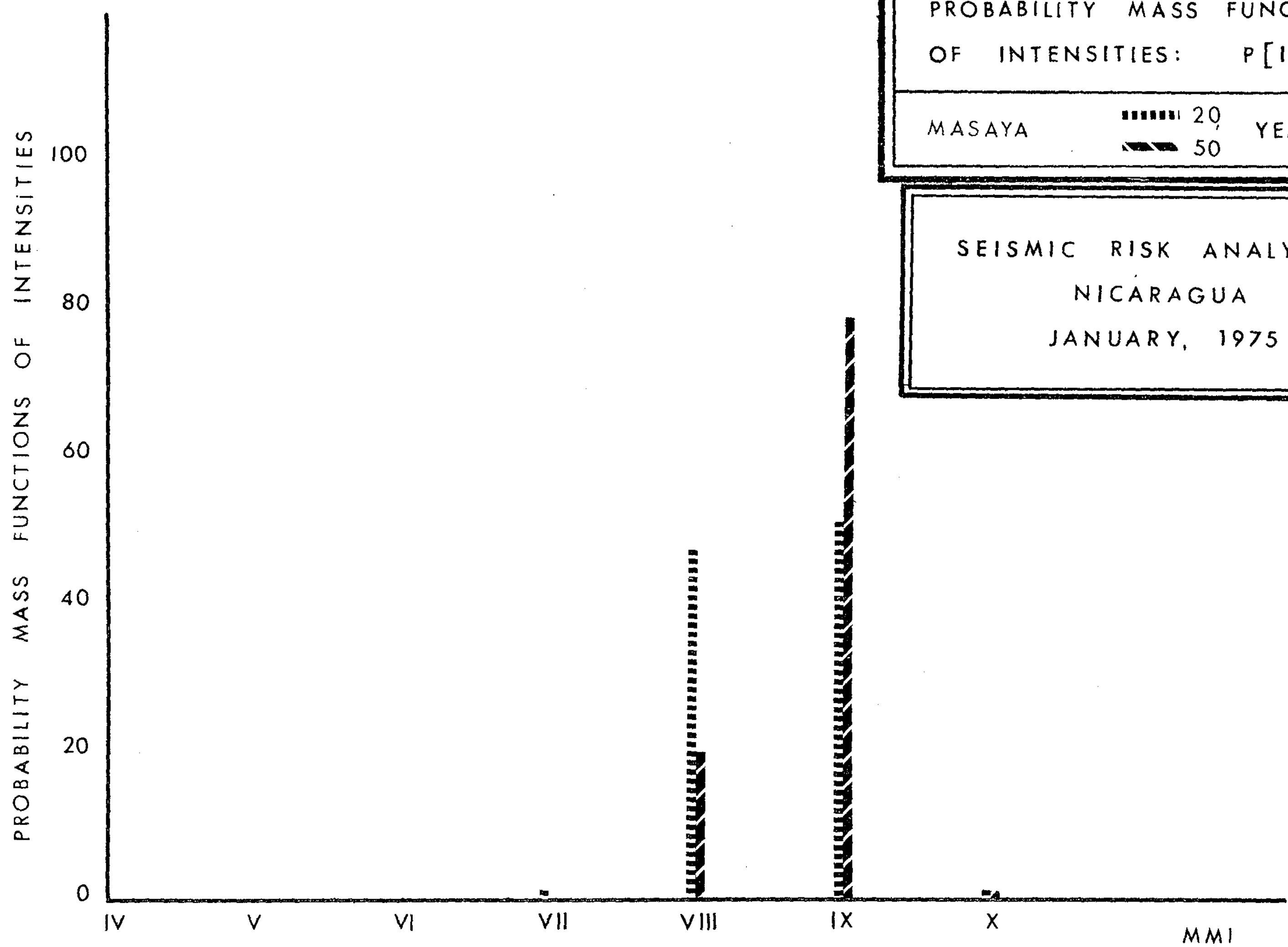


FIGURE 6-4

PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]

MASAYA 20, 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

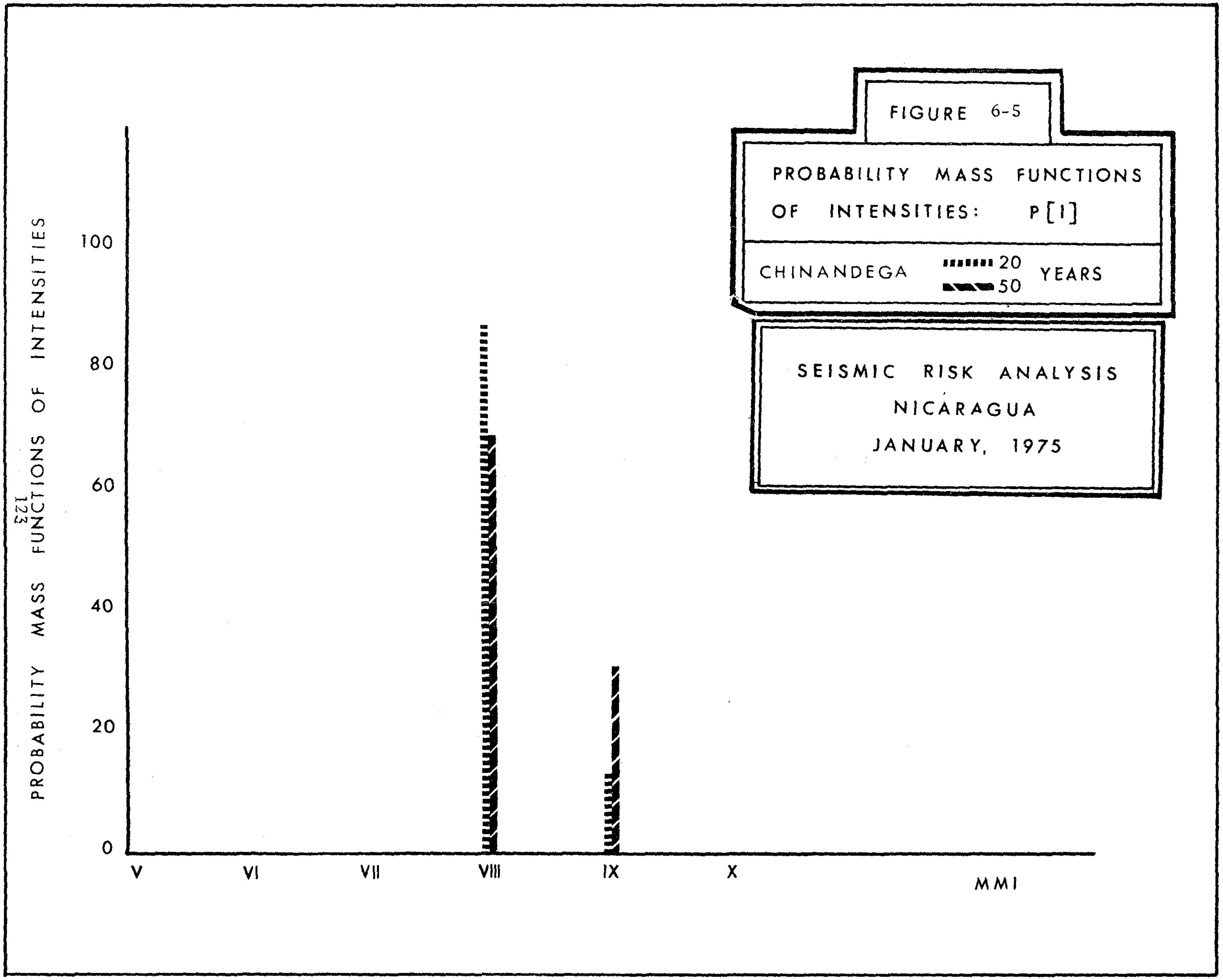


FIGURE 6-5
PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]
CHINANDEGA
20
50
YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY MASS FUNCTIONS OF INTENSITIES
100
80
60
40
20
0
V
VI
VII
VIII
IX
X
MMI

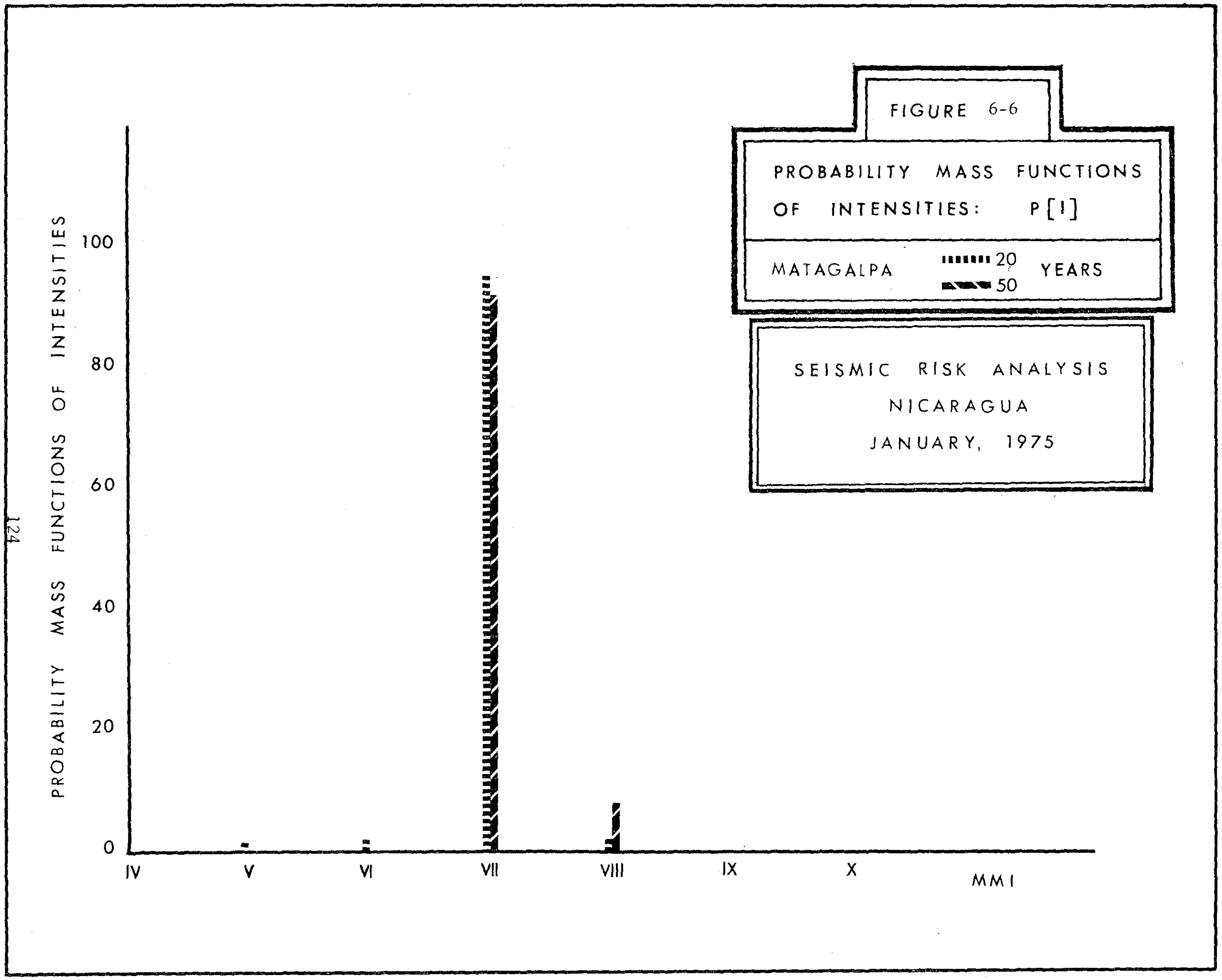


FIGURE 6-6
PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]
MATAGALPA 20 50 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY MASS FUNCTIONS OF INTENSITIES
100
80
60
40
20
0
IV
V
VI
VII
VIII
IX
X
MMI

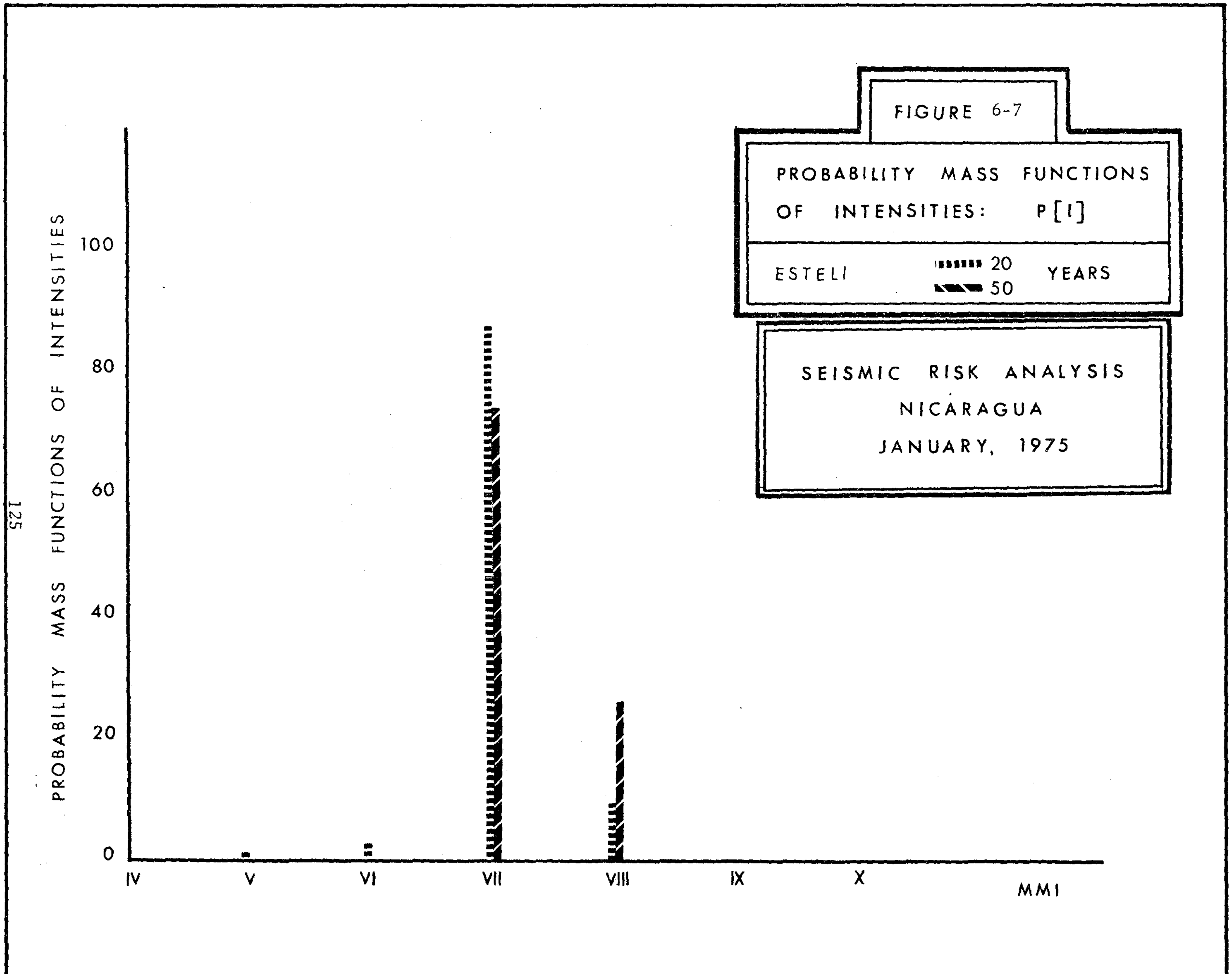


FIGURE 6-7
PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]
ESTELI
20
50
YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY MASS FUNCTIONS OF INTENSITIES
100
80
60
40
20
0
IV
V
VI
VII
VIII
IX
X
MMI

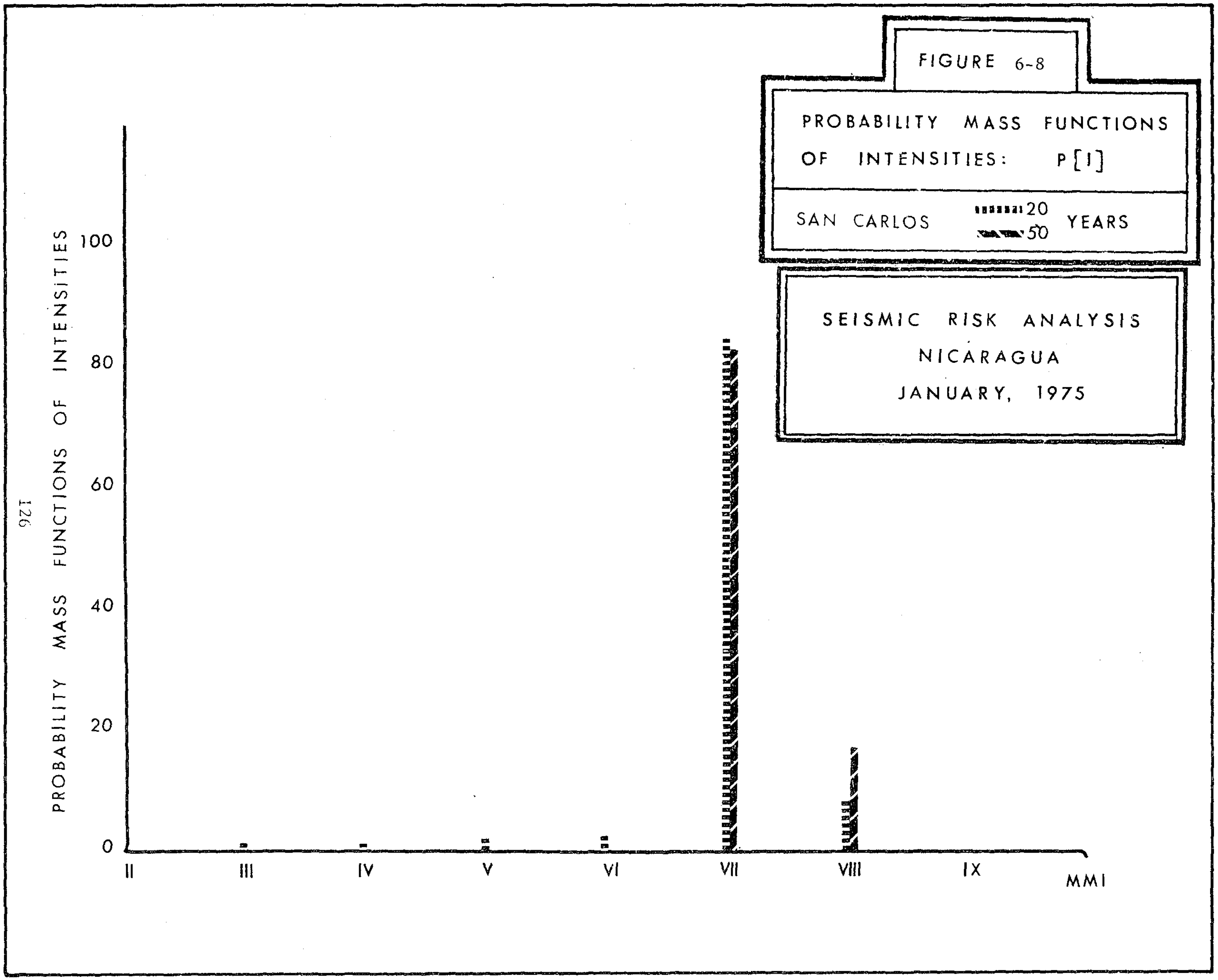


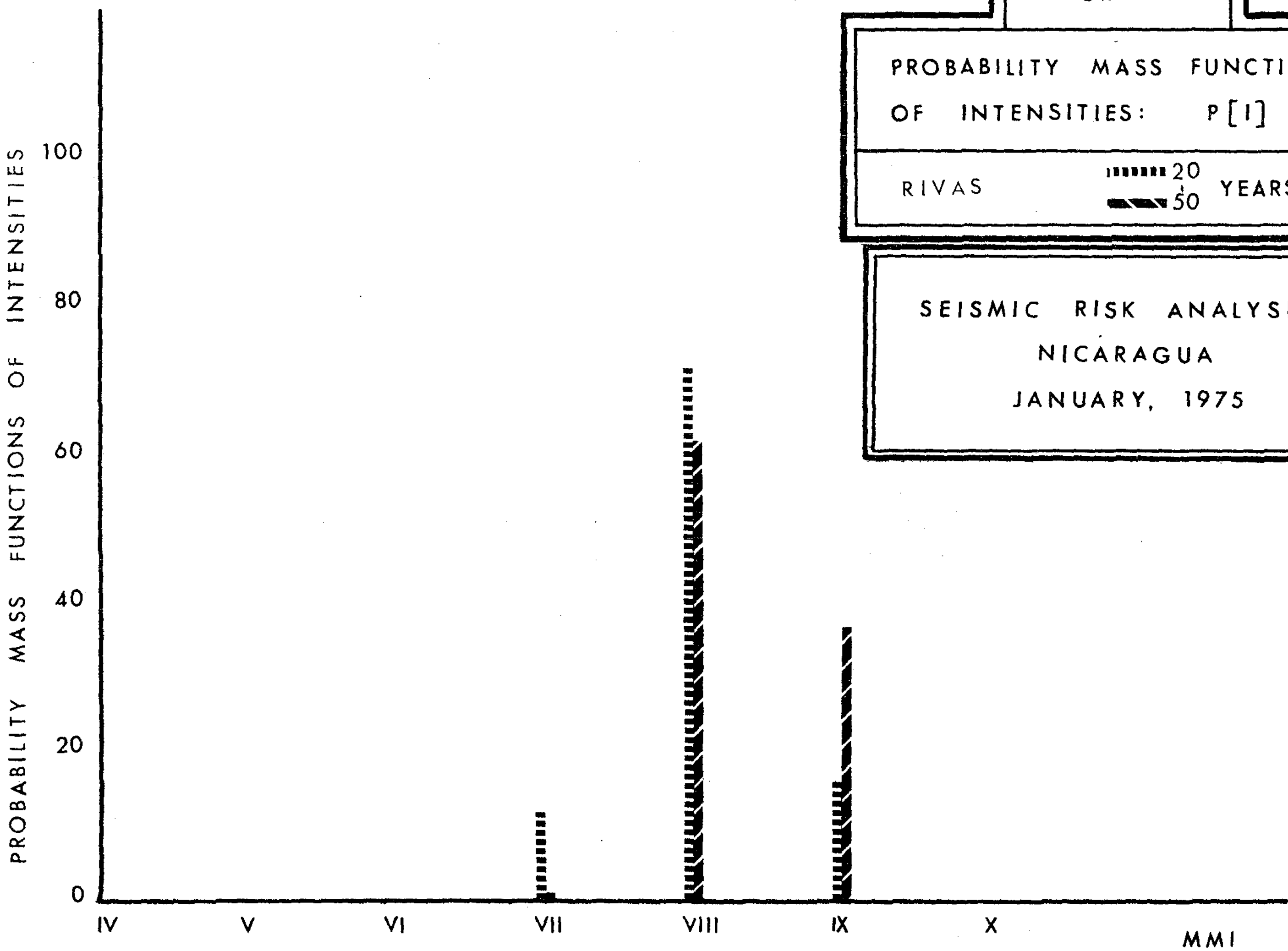
FIGURE 6-8
PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]
SAN CARLOS 20 50 YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY MASS FUNCTIONS OF INTENSITIES
100
80
60
40
20
0
II
III
IV
V
VI
VII
VIII
IX
MMI

FIGURE 6-9

PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]

RIVAS 20 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975



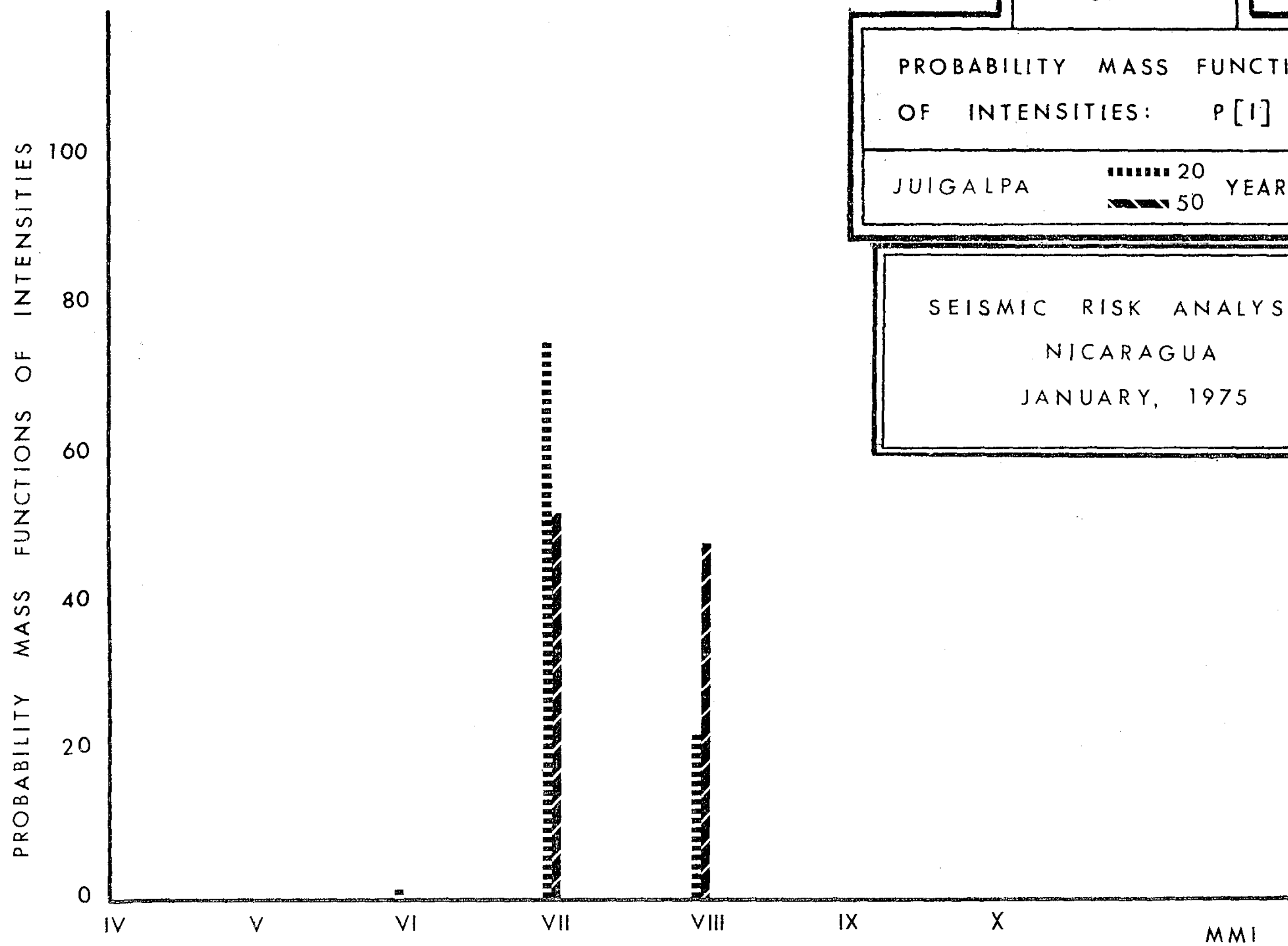


FIGURE 6-10

PROBABILITY MASS FUNCTIONS OF INTENSITIES: P[I]

JUIGALPA 20 / 50 YEARS

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

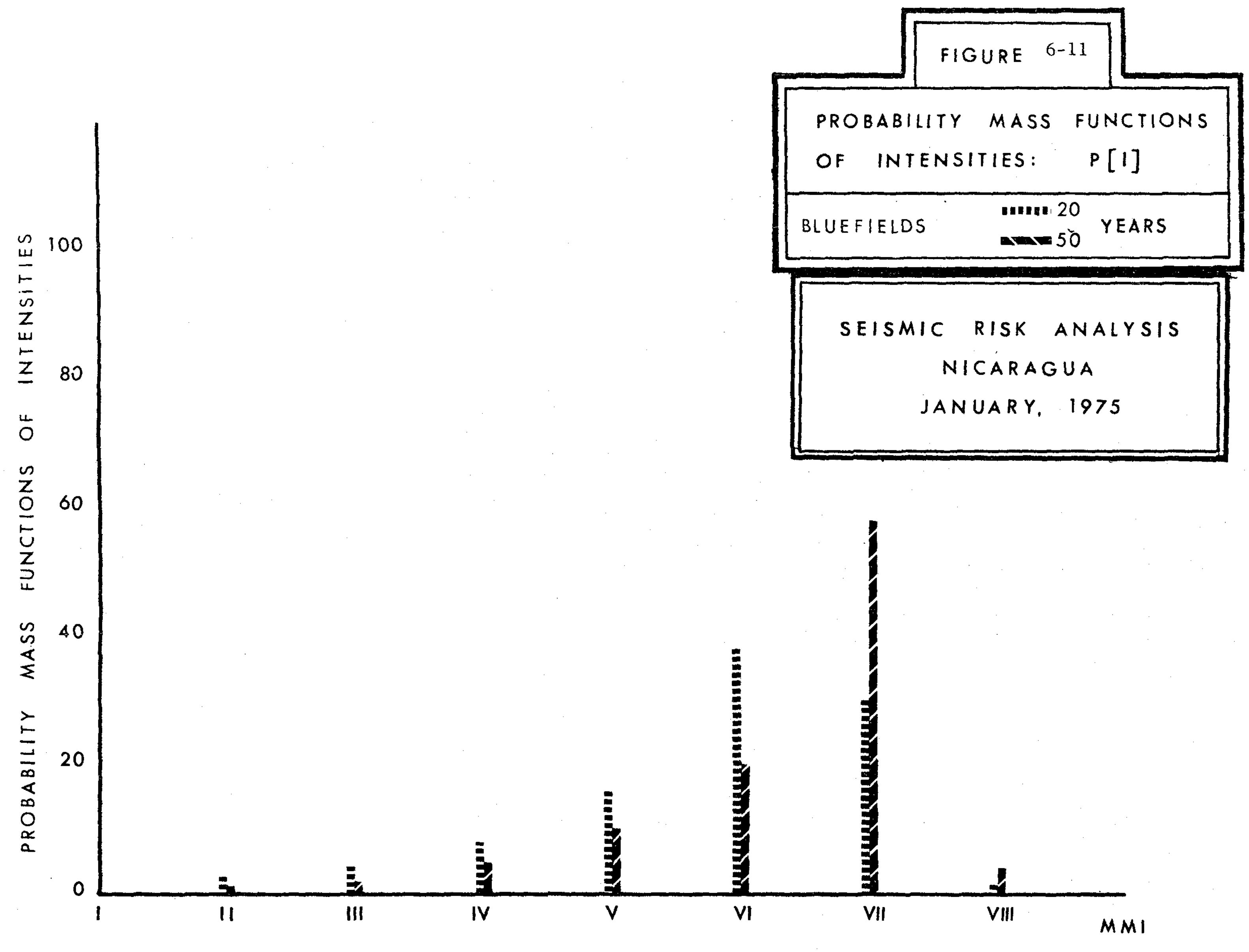


FIGURE 6-11
PROBABILITY MASS FUNCTIONS
OF INTENSITIES: P[I]
BLUEFIELDS
20
50
YEARS
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975
PROBABILITY MASS FUNCTIONS OF INTENSITIES
100
80
60
40
20
0
I
II
III
IV
V
VI
VII
VIII
MMI

A study of MM intensity and dollar damage was conducted after the Long Beach, the Kern County, and the San Fernando earthquakes. Figure 6-12 shows a graph of Median Loss in percent as a function of MMI for different types of structures. Even though we realize that these values are not applicable to Nicaragua, behavior of structures in Nicaragua built similar to those in the Southern California region will exhibit relationships similar to Figure 6-12. The purpose of presenting numerical examples with damage data from California and intensity forecast for Nicaragua is to show the methodology. No conclusions regarding insurance risk or damage potential should be made by using these numerical values. The purpose, to repeat once again, is strictly to demonstrate methodology. However, appropriate and applicable numerical values will be used in Part II of this study. If, on the other hand, residential houses or light industrial buildings are constructed similar to those in Southern California, such as wood frame dwellings and tilt-up structures, then the numerical values presented here can be used with some caution.

The three classes of structures considered in the example are:

(1) All one- and two-story wooden frame residential houses;

(2) Pre-1940 residential homes; and

(3) Light industrial buildings.

Using Figure 6-12, Table 6-1 can be constructed. The losses corresponding to any MM intensity level are in percentages. Thus, for example, due to MMI of V, damage to a wood frame dwelling would be 0.1%. The corresponding loss to pre-1940 design dwellings would be

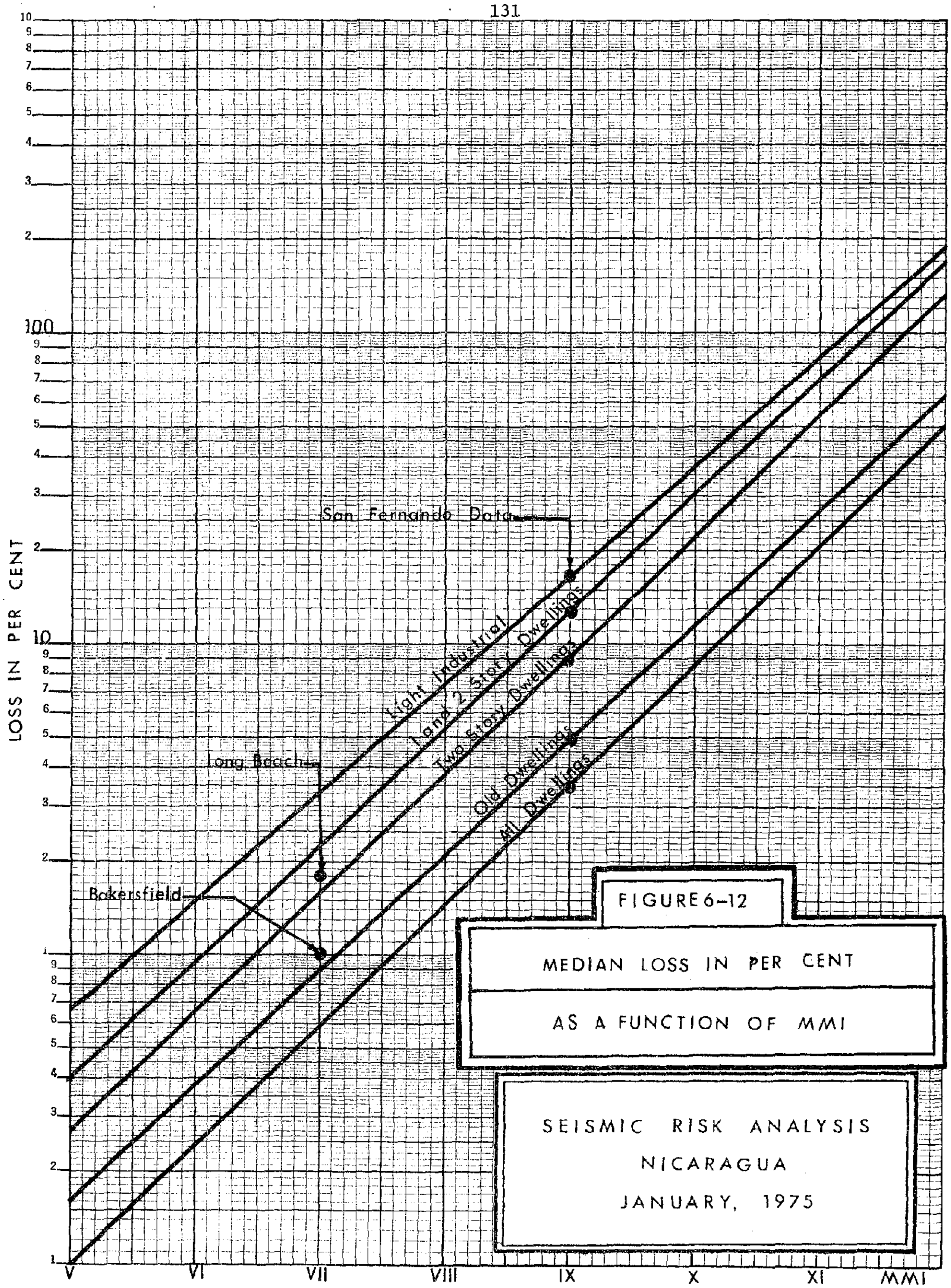


FIGURE 6-12

MEDIAN LOSS IN PER CENT

AS A FUNCTION OF MMI

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY, 1975

Table 6-1

Median Losses Due to Different MMI Levels

Intensity	All Dwellings	Pre-1940 Construction	Light Industrial Buildings
V	0.1	0.2	0.75
VI	0.2	0.4	1.5
VII	0.6	0.9	3.3
VIII	1.4	2.1	7.2
IX	3.3	5.0	16.0
X	7.7	12.0	36.0
XI	18.0	29.0	80.0

0.2%, and to light industrial buildings, it would be 0.75%. To determine the expected loss to any class of structure, the probability of any MMI level must be multiplied by the corresponding loss due to that level. The summation over all intensities will give the expected loss in percentage for that class of structure.

Consider, for example, the probability mass function of MMI for a twenty year period at Masaya (see Figure 6-4):

Intensity	VII	VIII	IX	X
Probability	.01	.47	.51	.01
Damage % All Dwellings	0.6	1.4	3.3	7.7

The expected median damage in 20 years for "all dwellings" is given by

$$
\begin{aligned}
E\ [\text{Damage}] &= (.01)\ (.6) + (.47)\ (1.4) + (.51)\ (3.3) + (.01)\ (7.7) \\
&= 2.424\%
\end{aligned}
$$

Thus, for a \$1,000 valuation, the expected damage in 20 years is given by

$$
\begin{aligned}
\text{Expected Damage} &= \frac{2.424}{100} \times 1{,}000 \\
&= \$24.24 \text{ per } \$1{,}000 \text{ valuation in 20 years.}
\end{aligned}
$$

Similar calculations can be carried out for all eleven cities considered for 20 and 50 years. Table 6-2 shows these loss calculation results. Table 6-3 shows similar expected loss calculations for a 20% chance of exceedance. Thus, in Managua, there is a 20% chance that in twenty years a light industrial building will have an expected loss of

Table 6-2

Expected Median Losses

	20 Yrs.			50 Yrs.		
	All Dwell.	P-1940 Dwell.	L. I. Struct.	All Dwell.	P-1940 Dwell.	L. I. Struct.
Managua	26.2	39.7	128.4	33.9	51.6	164.4
Leon	20.0	30.0	100.0	26.2	39.7	128.6
Granada	20.0	30.3	100.0	26.6	40.3	130.3
Masaya	24.2	36.7	119.4	29.6	44.9	144.4
Chinandega	16.5	24.8	83.4	19.9	30.0	99.3
Matagalpa	6.0	9.0	33.2	6.6	10.0	36.10
Esteli	6.6	9.9	35.7	8.1	12.1	43.1
San Carlos	6.3	9.5	34.4	7.4	11.0	39.6
Rivas	16.1	24.2	81.4	21.0	31.6	104.2
Juigalpa	7.6	11.5	40.9	9.8	14.8	51.7
Bluefields	2.9	4.8	17.5	4.5	7.1	25.8

Table 6-3

Expected Losses with 20% Chance of Exceedence

	20 Yrs.			50 Yrs.		
	All Dwell.	P-1940 Dwell.	L. I. Struct.	All Dwell.	P-1940 Dwell.	L. I. Struct.
Managua	65.5	99.2	321.0	84.8	129.0	411.0
Leon	50.0	75.0	250.0	65.5	99.2	321.2
Granada	50.0	75.8	250.0	66.5	100.8	325.8
Masaya	60.5	91.8	298.5	74.0	112.2	361.0
Chinandega	41.2	62.0	208.5	49.8	75.0	248.2
Matagalpa	15.0	22.5	83.0	16.5	25.0	90.2
Esteli	16.5	24.8	89.2	20.25	30.2	107.8
San Carlos	15.8	23.8	86.0	18.5	27.5	99.0
Rivas	40.2	60.5	203.5	52.5	79.0	260.0
Juigalpa	19.0	28.8	102.3	24.5	37.0	129.2
Bluefields	7.3	12.0	43.8	11.2	17.8	64.5

$321.00. However, the median (expected) loss for the same time period in Managua for the same class of structure will be $128.40. Figure 6-13 shows the behavior of expected losses as a function of time and the class of structure. It can be seen that the expected loss or economic (or insurance) risk for the Managua region in one year is $5.84 per thousand-dollar valuation. However, over a 20 year life of the structure, the expected median loss is $26.20 per $1,000 valuation. It can be seen from this example that expected loss averaged over a twenty year time period gives less mean rate of loss $26.2/20 = $1.31 per $1,000 valuation per year compared to $5.84 per $1,000 valuation when only one year was considered. This is the concept of risk averaging over time. Thus, if it were possible to insure a given facility for a 20 year economic life (say, a light industrial building), then it would be cheaper to buy that insurance for all twenty years at the same time as opposed to buying it year-by-year. In the long range, buying of insurance, for our numerical problem, the cost would be $128.4 per $1,000 valuation for a twenty year economic life. For a year-to-year buying, it would cost $31.84 per $1,000 valuation per year, or $638.8 per $1,000 valuation over a twenty-year span. Of course, in these simple calculations the value of money and the interest rate are not taken into account. From Tables 6-2 and 6-3 it can be seen that the "insurance risk" in decreasing order of magnitude in different cities is:

1. Managua
2. Masaya
3. Granada

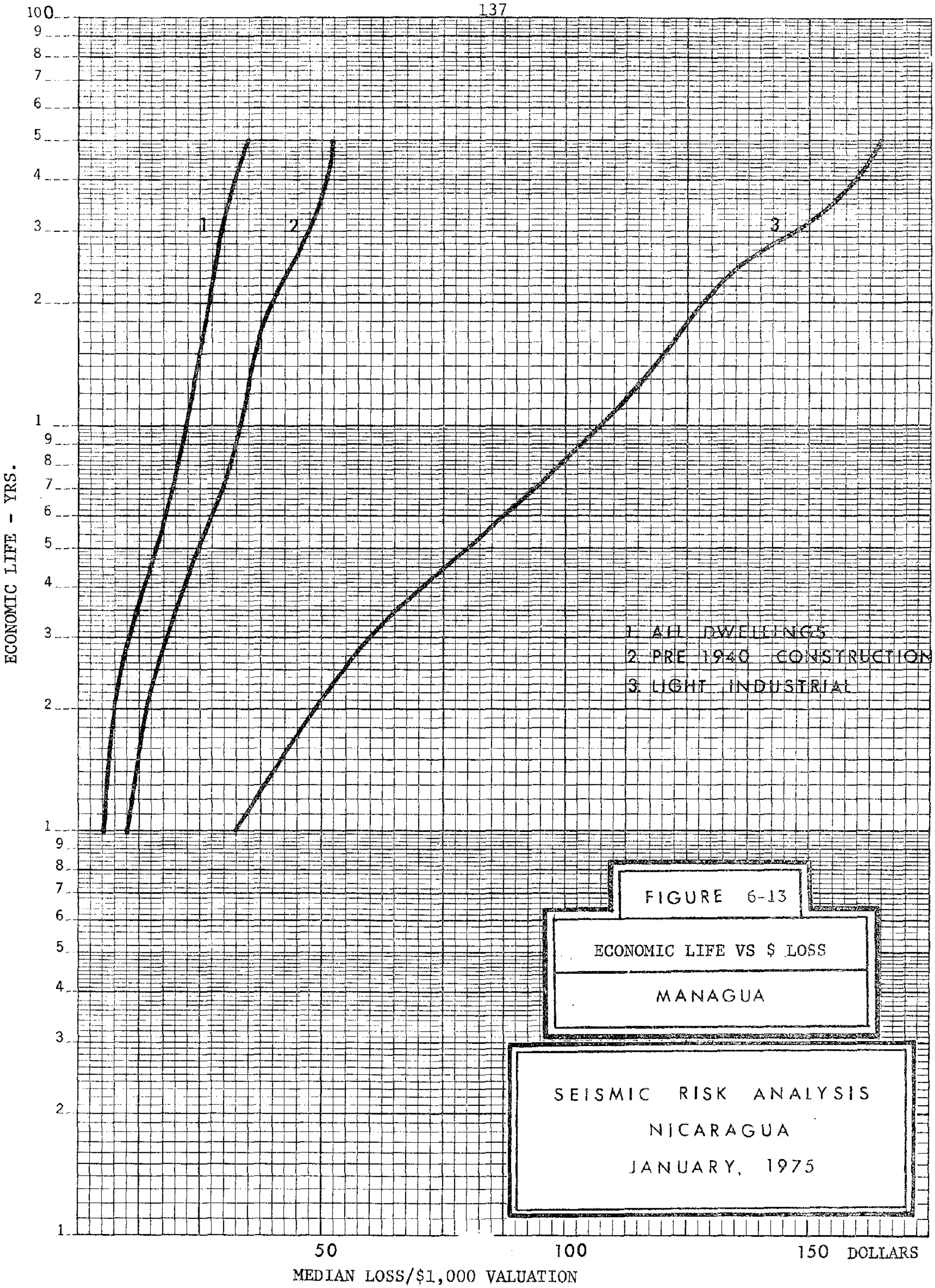


FIGURE 6-13

ECONOMIC LIFE VS $ LOSS

MANAGUA

SEISMIC RISK ANALYSIS

NICARAGUA

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4. Leon
5. Chinandega
6. Rivas
7. Juigalpa
8. Esteli
9. San Carlos
10. Matagalpa
11. Bluefields.

Granada and Leon have almost equal expected economic or insurance risks. Similarly, Juigalpa, Esteli, San Carlos and Matagalpa have very similar insurance risks. The seismic insurance risk at Bluefields is very small. It should be pointed out that even though the damage data used in the numerical problem were from Southern California earthquakes, the order of these cities in their seismic and economic risks is valid. If proper data from Nicaragua are considered with appropriate economic conditions in Nicaragua, the ordering of the cities in their risks will not change substantially.

In conclusion, it can be said that the methodology presented here for determining the probable intensity levels and their use in determining expected damages needs a closer look and further investigation. Part II of the current study will go deeper into that question.

CHAPTER VII

THE RELATIONSHIP OF ISO-ACCELERATION AND ACCELERATION ZONE GRAPHS (AZG) TO SEISMIC DESIGN PROVISIONS

Introduction

From the information as developed in the preceding chapter, ground acceleration values A_g may be established for a given structure location. These values have selected probabilities P of not being exceeded during a given economic structure life L. The purpose of this chapter is to show how these acceleration values are to be incorporated into load criteria for seismic design provisions. Basically, acceleration values must be converted to seismic load information, such that structures, as designed for these load levels, will have a desired reliability R_D of damage protection and a much higher reliability R_C against total building condemnation or incipient collapse during the economic structure life.

While at first thought a building owner may desire full protection against both the hazards of damage and condemnation, a consideration of the complete set of his objectives will show the necessity for acceptance of some level of risk. For a given site location, structure life, and Use Group or Function, these objectives of the building owner are:

. Low construction cost

. Low operating cost

. Functional configuration

. Attractive configuration

. Damage protection

. Condemnation protection.

Perfect and certain fulfillment of all of these objectives is not possible due to the uncertainties in earthquake demands and in structural capacities and behavior. Practical fulfillment of the first four objectives requires the acceptance of a moderate probability of damage P_D (equal to $1-R_D$) and a small probability of structural condemnation P_C during the building's economic life, L. Owners, therefore, must agree to a definite set of values for P_D, P_C, L for the given value, and Use Group of the building. Graphs presented in Chapter 6 can help the owner decide on the level of risk and hence can result in the determination of the appropriate probability values.

For these given values of P_D, P_C, and L, the Acceleration Zone Graphs (AZG) provide the Peak Ground Acceleration values A_D and A_C which have the moderate P_D and small P_C probabilities of exceedence during the structure life L at a given site location.

For example, the use or function of structures may be organized into the following groups which depend on the desired reliabilities of operation and damage protection in the event of a large earthquake.

Group A: Critical facilities necessary for life care and safety; hospitals; penal and mental institutions; gas, water, electric, and waste water treatment facilities; communications facilities; police and fire departments; and disaster control centers.

Group B: Multi-family residences; hotels; recreational and entertainment structures; churches and schools; commercial and industrial structures necessary for normal commerce.

Group C: Facilities which are relatively non-essential for normal commerce and where damage will not create a life safety hazard. An example of such facilities would be warehouses.

Example values of the peak ground accelerations A_D and A_C, at sites in Managua and Leon, are given in the following Tables: 7-1, 7-2, 7-3, and 7-4. These are based on structure lives of 20 and 50 years, and on reasonable values for P_D and P_C corresponding to the structure Use Group. The values given in these tables are strictly for demonstrating the concepts, and are not meant to be used by engineers at this time. As can be seen from these four tables, the same facility and risk in Leon and Managua requires different A_D and A_C values. Obviously, Leon has a lower seismic demand than Managua.

With these known values of A_D and A_C at the structure site, the primary objectives of the structural designer are to:

- Provide a structure with sufficient rigidity such that no significant non-structural damage will occur due to earthquake ground motions of a level represented by A_D.
- Provide a structure with sufficient strength capacity such that no significant structural damage will occur due to deformation demands caused by earthquake ground motions of a level represented by A_D.

Table 7-1

20 Year Economic Life, Managua Region

Group	P_D	RP_D	A_D	P_C	RP_C	A_C
A	20%	90	.33g	10%	190	.38g
B	50%	30	.24g	20%	90	.33g
C	70%	17	.20g	50%	30	.24g

Table 7-2

20 Year Economic Life, Leon Region

Group	P_D	RP_D	A_D	P_C	RP_C	A_C
A	20%	90	0.24g	10%	190	0.26g
B	50%	30	0.20g	20%	90	0.24g
C	70%	17	0.17g	50%	30	0.17g

Table 7-3

50 Year Economic Life, Managua Region

Group	P_D	RP_D	A_D	P_C	RP_C	A_C
A	20%	225	0.40g	10%	475	0.44g
B	50%	72	0.32g	20%	225	0.40g
C	70%	42	0.27g	50%	72	0.32g

Table 7-4

50 Year Economic Life, Leon Region

Group	P_D	RP_D	A_D	P_C	RP_C	A_C
A	20%	225	0.26g	10%	475	0.30g
B	50%	72	0.23g	20%	225	0.26g
C	70%	42	0.21g	50%	72	0.23g

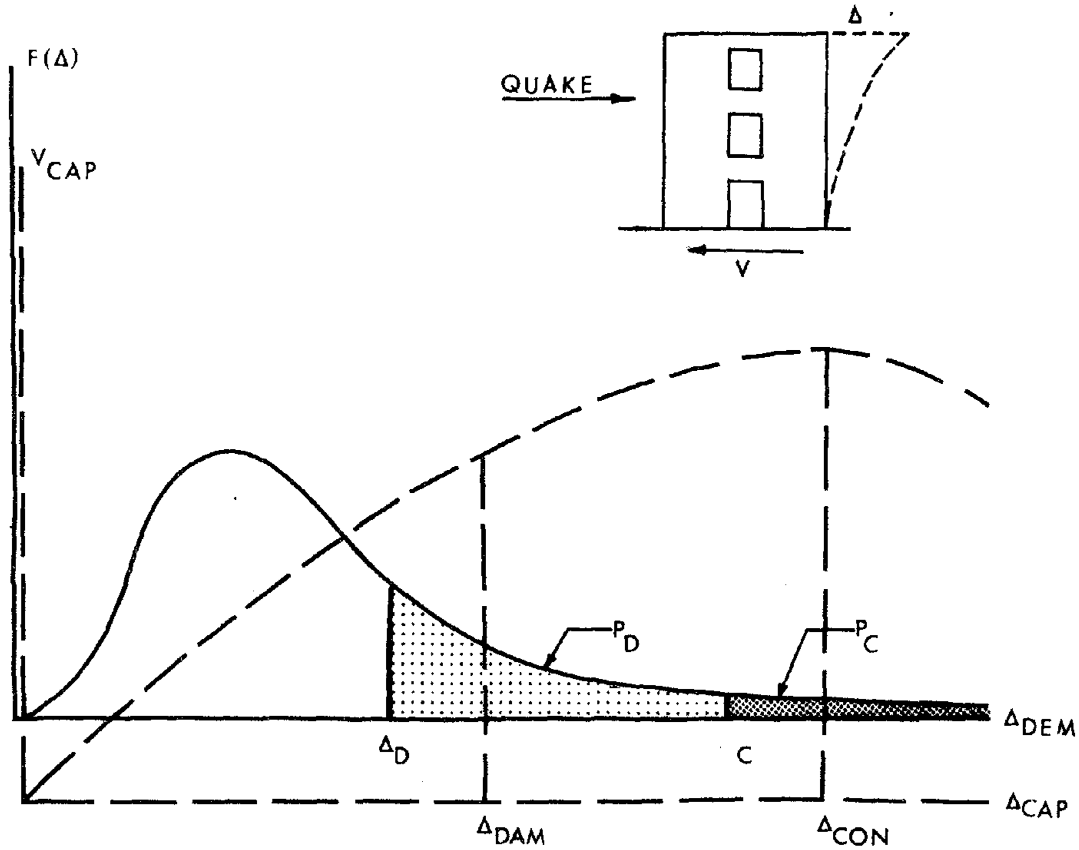
- Provide a structure with sufficient strength, stability, and deformation capacity such that condemnation of the structure will not result from the effects of earthquake ground motions of a level represented by A_C.
- While the possibility of significant damage is admissible with the moderate probability P_D, and the possibility of building condemnation is admissible with the small probability P_C, every prudent effort is to be made to prevent serious injury or death of the building occupants. This life safety objective requires that the details of both the structural and non-structural elements, and the complete structural system are such that neither injurious system failures, injurious falling debris, nor structural collapse will result from ground motions of a level represented by A_C.

The practical consequence of this last objective is that only those types of structural systems which are capable of retaining their integrity and stability at deformations at and beyond the A_C level are to be used.

Within these systems, the details of the connections between structural elements must tie the structure together, and the elements themselves must not have brittle or sudden buckling modes of failure. Multiple systems of frames, or back-up systems in the form of shear wall or vertical bracing must provide a series of lateral force resisting systems such that vertical load capacity is maintained for earthquake deformation demands at and reasonably beyond the A_C level.

The complete set of structural design objectives is shown in Figure 7-1. Since the demands of earthquake ground motions create non-linear structural behavior, this figure indicates the critical design thresholds of damage Δ_D and condemnation Δ_C in terms of structure deformation Δ rather than forces. The solid line coordinate system represents the probability density function f (Δ) of Earthquake Deformation Demands Δ_{DEM} which may occur on a given structure during a life L. The dotted line system indicates the load V versus deformation capacity Δ_{CAP} curve of a given structure which satisfies the stated design objectives. Specifically, the structure has been designed such that its deformation capacities are equal to or greater than the earthquake demands at the damage and condemnation threshold levels. The earthquake of level A_D with probability of exceedence P_D does not exceed the damage capacity level Δ_{DAM}, and the earthquake having the condemnation level A_C with probability P_C, does not exceed the condemnation capacity level Δ_{CON}. Further, the structure load-deformation curve maintains a reasonably constant level for even those highly improbable deformations which might reasonably exceed the condemnation level. This latter characteristic insures the stability of the structure against collapse.

The purpose of this chapter is two-fold. First, the response spectrum method of analysis will be described as the means of relating the AZG values A_D and A_C to their corresponding earthquake demands Δ_D and Δ_C on a given structure. Second, the complete design procedure will be developed such that the resulting structure will have the necessary design requirements of $\Delta_{DAM} \geq \Delta_D$ and $\Delta_{CON} \geq \Delta_C$. In addition,



Δ_{CAP} = Deformation Capacity

V_{CAP} = Shear Capacity

Δ_D = Demand of Earthquake of level A_D

Δ_C = Demand of Earthquake of level A_C

Δ_{DAM} = Damage Threshold Deformation

Δ_{CON} = Condemnation Threshold Deformation

P_D = Probability of $\Delta_{DEM} > \Delta_D$

P_C = Probability of '$\Delta_{DEM} > \Delta_C$

FIGURE 7.1

the types of structural systems and details necessary for the life safety requirement of collapse prevention will be defined. The analysis and design procedures will follow the general concepts as set forth in reference 28. The order of the subjects to be treated are as follows:

. Basic Response Spectra:

Definition of an earthquake response spectrum for an ideal elastic single-degree-of-freedom system; Effective ground acceleration as a working measure of spectral size or level; Spectral shape in terms of the dynamic amplification factor (DAF), its mean and standard deviation (σ) value; the effect of the damping ratio (β); the effect of inelastic behavior as represented by the ductility ratio (μ); site or soil-column response effects on the average ground motion values.

. Response Spectrum Analysis:

Response of multi-degree of freedom systems as the square root of the sum of the squared response of each mode to a given spectrum (SRSS response); use of the SRSS response to an inelastic response spectrum as an approximation of inelastic system response.

. Types and Characteristics of Lateral Force-Resisting Systems in Buildings:

Ductile frames; shear walls; walls and ductile frames; walls and ordinary frames; the effect of the choice of system type on the accomplishment of the design objectives.

. Design Spectra:

Definition and purpose of design spectra for the damage and collapse threshold earthquakes; spectral level established by the effective ground acceleration A for a given structure use group and life L; spectral confidence limits $\kappa\sigma$ for structural system types;

structure-foundation interaction effects; subjective assignment of μ, β, and $\kappa\sigma$ for given structural system types; formulation of a set of example design spectra.

. Proposed Design Method:

Earthquake loading as provided by the SRSS response to the Design Spectra; structure modeling for dynamic modal analysis; Dead, Live, and earthquake Load combination; design on an ultimate strength basis; calculation of inelastic deformation-demands and comparison with allowable ductility limits, and stability limits.

It is important to emphasize that the response spectrum analysis and corresponding design procedures are to be presented in a general descriptive form in this report. The current practice of seismic design is such that these methods are still in a state of development within the design profession. They represent, however, the most effective practical means of achieving the design objectives and are to be developed in detail in the proposed Part II of this study.

Basic Response Spectra

The earthquake response spectrum is to provide the analytical model by which the AZG values of A_D and A_C are to be related to structural load values. These load values, in turn, are to be employed within an appropriate design procedure to provide the necessary sizes and proportions of structural elements required to satisfy the design objectives. Before going into the method of formulating what may be termed as structural design spectra, it is necessary to describe the basic spectral characteristics and parameters. These include: size or level, shape, confidence limit, damping, and ductility. (A typical basic response spectrum is shown in Figure 7-2.)

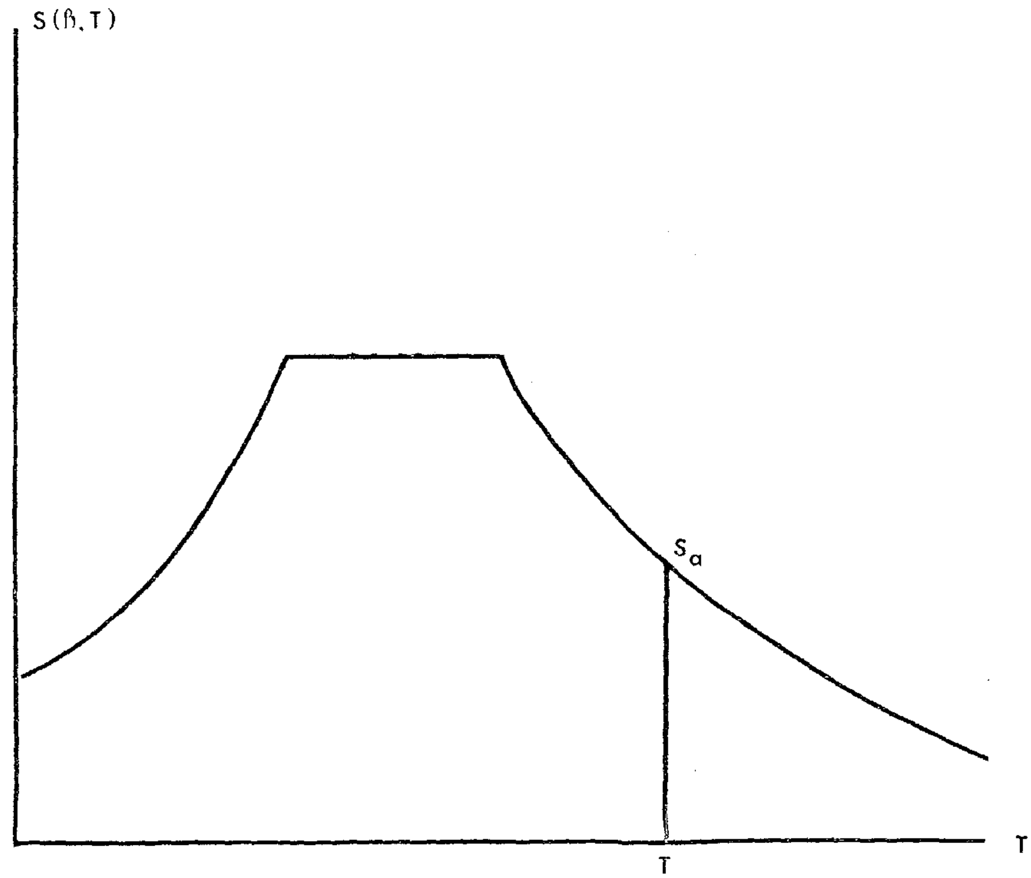


FIGURE 7.2

Definition of an Earthquake Response Spectrum:

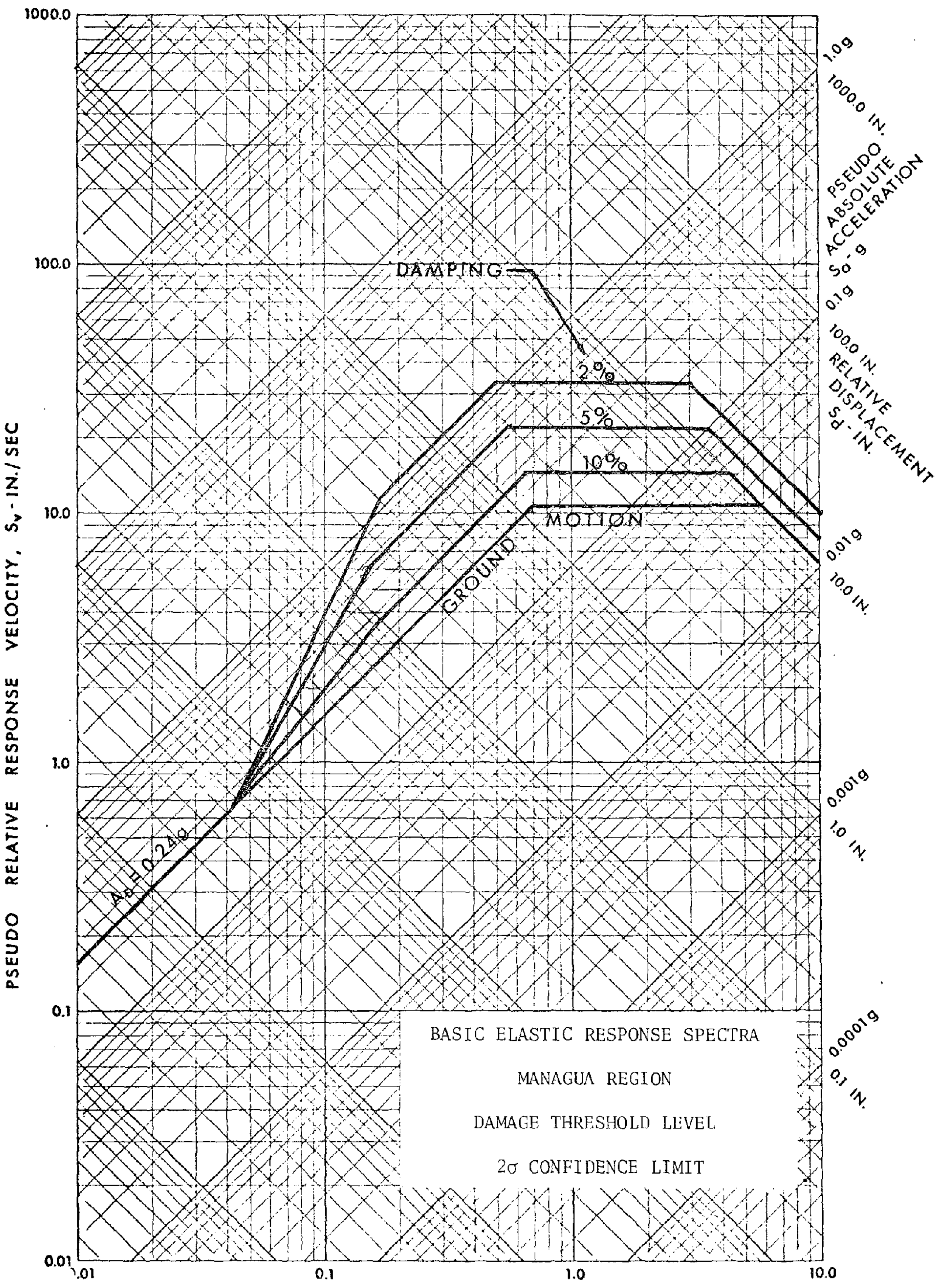
For a given accelerogram or time history of earthquake ground motion, the ordinate $S_a(\beta, T)$ of the acceleration response spectrum (shown in Figure 7-2) is the maximum effective acceleration response felt by an elastic single-degree-of-freedom system, having a damping ratio β and natural period of vibration T.

Basic Response Spectra:

Figures 7-3 and 7-4 show response spectra plotted on special three-way logrithmic paper. They represent the type of basic spectral shape as proposed by Newmark in reference 29, and as extracted in Appendix 5 of this report. The Newmark method of spectrum construction is representative of current practice and is employed for the purpose of this report. Basically, it consists of the following steps:

. Three straight lines representing constant acceleration, velocity, and displacement are used for the ground motion base line. The acceleration leg A_g is the peak effective ground acceleration from the AZG, either A_D or A_C. These A_g values set the level of the spectrum; and the ground velocity V_g and displacement D_g are proportional to this given A_g value.

. The basic response spectrum for a given damping value β is formed by multiplying the A_g, V_g, D_g curve values by DAF values. In the Newmark method these DAF values are at about two standard deviations ($2\ \sigma$) from the mean DAF shape.



1000.0
100.0
10.0
1.0
0.1
0.01
.01
0.1
1.0
10.0
PERIOD - SEC
PSEUDO RELATIVE RESPONSE VELOCITY, S_v - IN./SEC
PSEUDO ABSOLUTE ACCELERATION S_a - g
RELATIVE DISPLACEMENT S_d - IN.
1.0 g
1000.0 IN.
0.1 g
100.0 IN.
0.01 g
10.0 IN.
0.001 g
1.0 IN.
0.0001 g
0.1 IN.
DAMPING
2%
5%
10%
GROUND
MOTION
A_g = 0.24g
BASIC ELASTIC RESPONSE SPECTRA
MANAGUA REGION
DAMAGE THRESHOLD LEVEL
2σ CONFIDENCE LIMIT

FIGURE 7.3

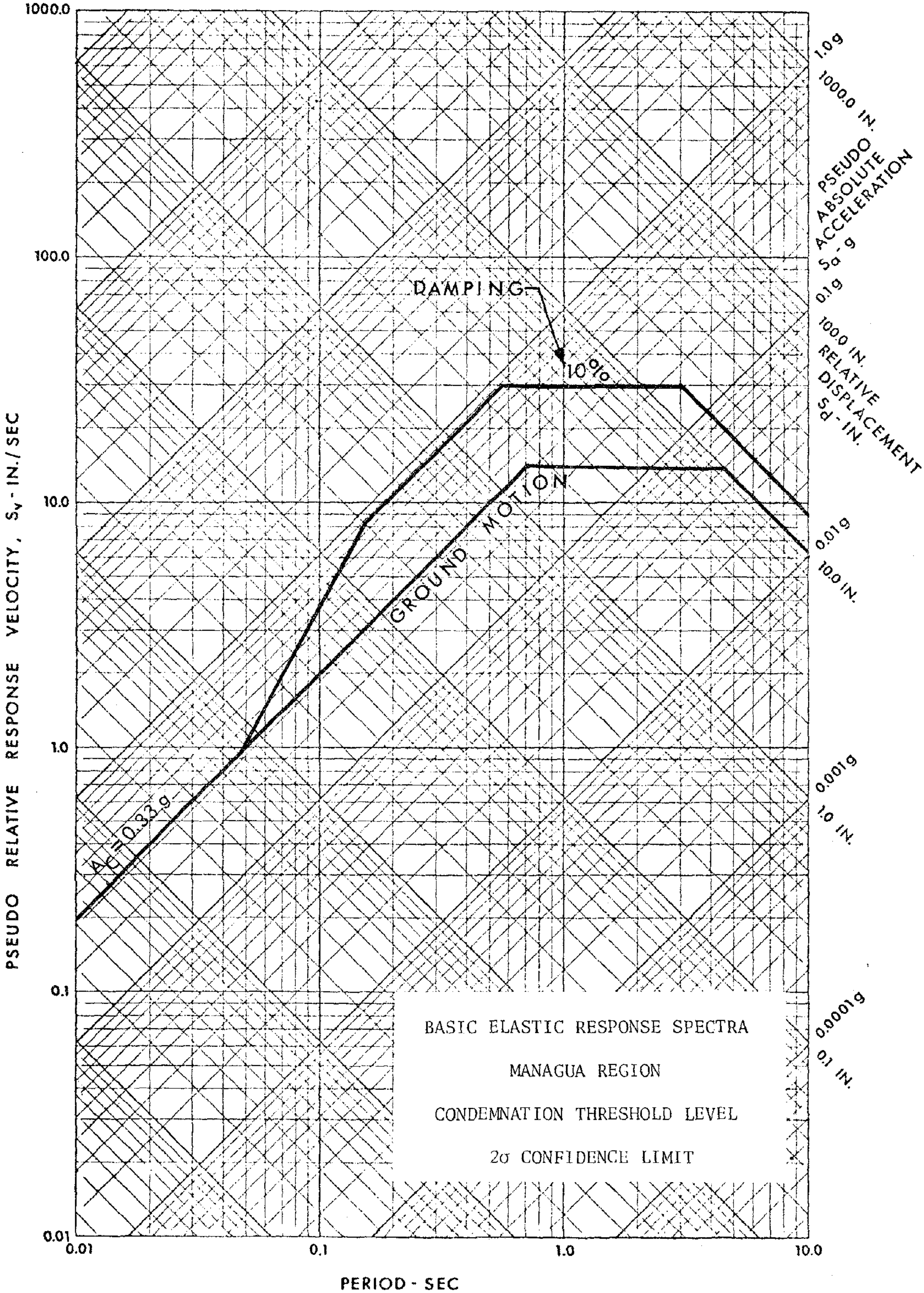

DAMPING
10%
GROUND MOTION
AC=0.33g
BASIC ELASTIC RESPONSE SPECTRA
MANAGUA REGION
CONDEMNATION THRESHOLD LEVEL
2σ CONFIDENCE LIMIT
PSEUDO RELATIVE RESPONSE VELOCITY, S_v - IN./SEC
PERIOD - SEC
PSEUDO ABSOLUTE ACCELERATION Sa - g
RELATIVE DISPLACEMENT Sd - IN.
1000.0
100.0
10.0
1.0
0.1
0.01
0.01
0.1
1.0
10.0
1.0g
1000.0 IN.
0.1g
100.0 IN.
0.01g
10.0 IN.
0.001g
1.0 IN.
0.0001g
0.1 IN.

FIGURE 7.4

Inelastic Response Spectra:

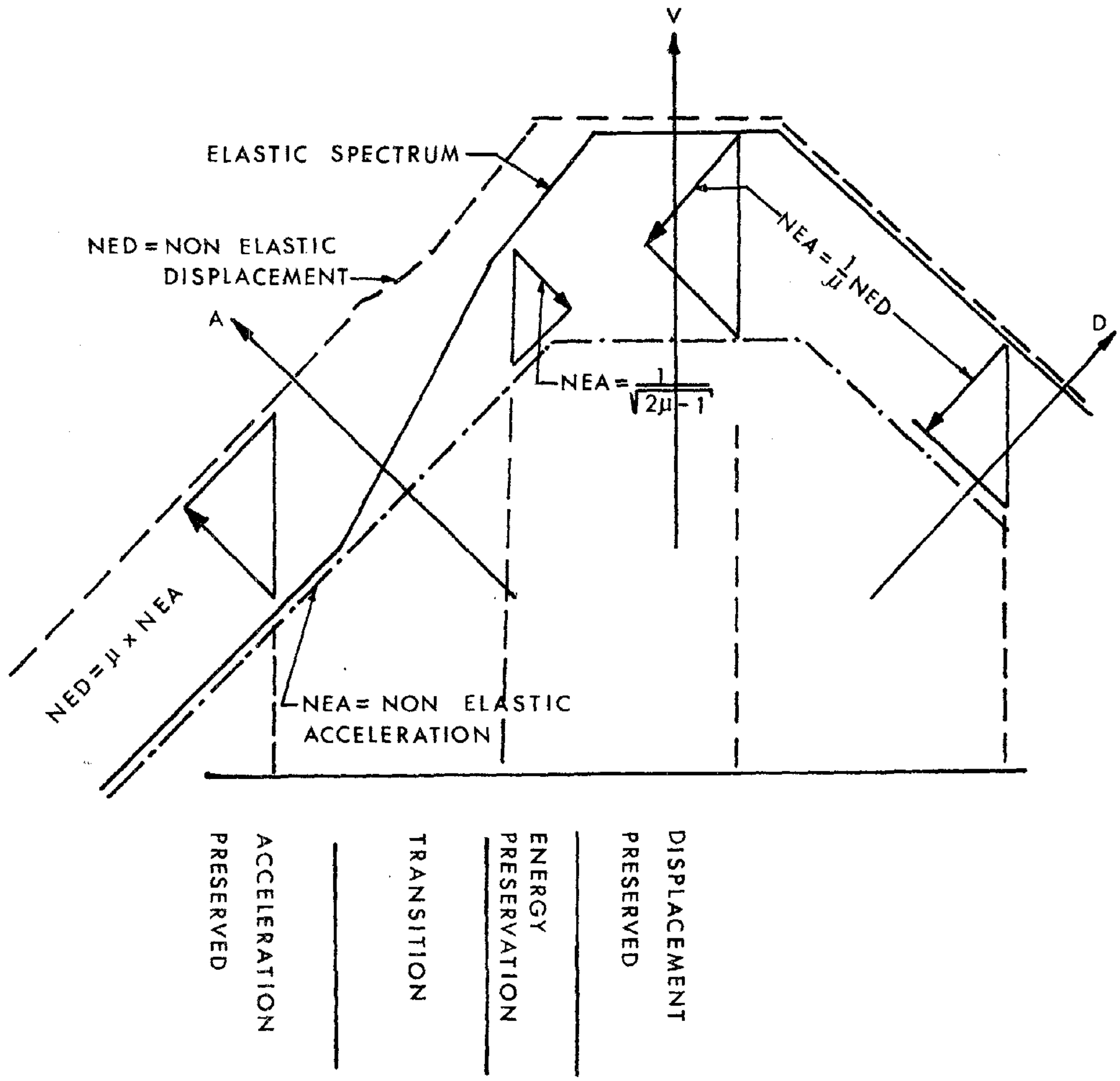
When the inelastic deformation response of ideal elasto-plastic system is known in terms of the ductility factor μ, then the inelastic force and inelastic deformation spectra may be obtained by the rules given in the method and these are represented in Figure 7-5. If the ideal system (with period T) were to have its yield strength level equal to the Force at the Inelastic-Acceleration line, then the total inelastic deformation of the system is given by the Non-Elastic Displacement line.

Some improvements and modifications to this Newmark method which are to be introduced in this Part I report, and subsequently developed in the Part II report, are as follows:

- The ground motion base line values and the resulting shape may need to be modified to represent the envelope of possible effects from the three principal sources of major earthquakes: Local Fault Systems, Volcanic Activity, and the Benioff Zone.

- The DAF in terms of its statistical mean value and standard deviation σ must be evaluated for a macro-region which is representative of Nicaragua; see, for example, reference 30 for this type of study. A similar study as reference 30 will be prescribed for the Nicaragua region in part II of this report.

- The rules for forming the inelastic acceleration and



THE NEWMARK - HALL METHOD

FIGURE 7.5

inelastic displacement spectra must be modified to represent the actual inelastic behavior of structures rather than the given ideal elastic-plastic system behavior. In particular, the low period region of the inelastic displacement spectrum might be better represented by a curve equal to or close to the elastic response spectrum; this is because a real structure must be designed for the inelastic acceleration forces, but in this low period region these forces are equal to the elastic acceleration forces and hence the structure remains elastic with the corresponding elastic (rather than the μ magnified inelastic) displacement value.

. Depending on the local site conditions, the response of the underlying soil column may be significantly different from average base line ground motion values. Therefore, soil-column adjustment factors must be evaluated to modify the basic A_g, V_g, D_g lines of the spectrum. These factors would be applied for either shallow-stiff sites or for deep-soft sites.

Response Spectrum Analysis

Referring back to Figure 7-1, it is necessary for the designer to have some analytical method of computing the earthquake demands of Δ_D and Δ_C. The method to be employed is modal analysis as described in reference 31. Briefly, this consists of the following steps:

. A linear elastic dynamic model of the structure is formulated, and the characteristic mode shapes and frequencies are evaluated.

. For any given Response Spectrum, the force and displacement response of the linear model are assumed to be given by the square root of the sum of the squared response of each mode. This is termed as SRSS response.

. Design spectra are to be formulated (in a following section) such that: the SRSS response to the Damage Threshold Spectrum provides the demand Δ_D, and the SRSS response to the condemnation Threshold Spectrum provides the demand Δ_C. Since both Δ_D and Δ_C may be inelastic deformations, it is necessary to employ the assumption that inelastic structure deformations may be predicted by the elastic dynamic model response to the specially formulated inelastic design spectra. A detailed study of the validity of this assumption will be presented in part II of this report.

Types of Structures in Terms of Their Lateral Force-Resisting Systems

Before proceeding to formulate the Design Spectra, it is necessary to define and consider the inelastic behavior of the following structural systems:

. Ductile Moment Resisting Frames: (the K=0.67 system of the Uniform Building Code).

Symbol	Description
0.67M	Complete Ductile Frames for each Bay Width of the building plan.
0.67P	Ductile Frames around the plan perimeter, with non-ductile columns in the plan interior areas.

. Shear and Bearing Walls: (the K=1.33 System of the UBC).

1.33B	A box system of walls with few openings.
1.33P	A box system of walls with many openings that form an equivalent frame system of piers and spandrels.
1.33C	Cantilever walls or towers with few openings.
1.33S	Cantilever walls or towers with vertical alignment of openings which form sets of coupling spandrels.

. Ductile Frames and Shear Walls: (the K=0.80 System of the UBC).

.080M	Same as 0.67M, but with several shear walls.
.080P	Same as 0.67P, but with few shear walls or towers.

. Ordinary Frames with Semi-Ductile Details and Shear Walls: (the K=1.00 System of the UBC).

1.00M	Same as 0.80M, but with ordinary frames.

1.00P	Same as 0.80P, but with ordinary frames.
1.00MX	Same as 1.00M, but with vertical bracing in place of walls.
1.00MP	Same as 1.00P, but with vertical bracing.

All systems are to have the necessary details required to insure the ductility and integrity of the system; these include: steel and reinforced concrete details for ductile and semi-ductile frames; chord, grid, and collector reinforcing for shear walls; details for vertical bracing systems; horizontal diaphram chords, drag, and shear connection to walls or bracing; and load transfer through construction joints. These details are best exemplified in references 32 and 33.

For a given constant strength level the general inelastic behavior of the various general classes of systems is shown in Figure 7-6, and for a given stiffness or rigidity the behavior is as in Figure 7-7.

Clearly, from Figures 7-6 and 7-7 the K=0.67 systems have the advantage of large ductility, but for some cases they may not have the required rigidity for damage control; and, alternatively, the K=1.33 systems have the desired rigidity, but suffer from a lack of ductility. In addition to these properties, each system has its particular structural damping ratio β_S, and subjective reputation of dependable performance. All of these characteristics--rigidity, ductility, damping,

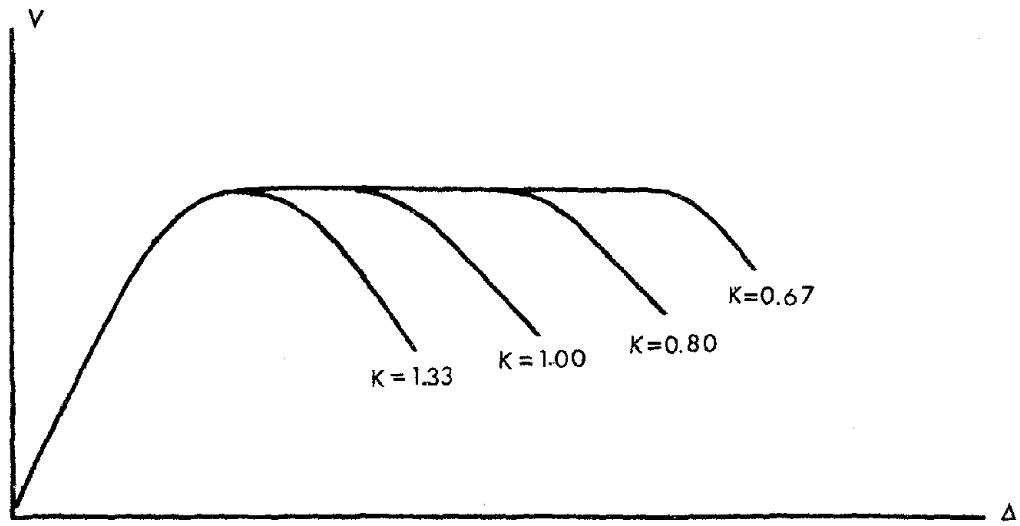


FIGURE 7.6

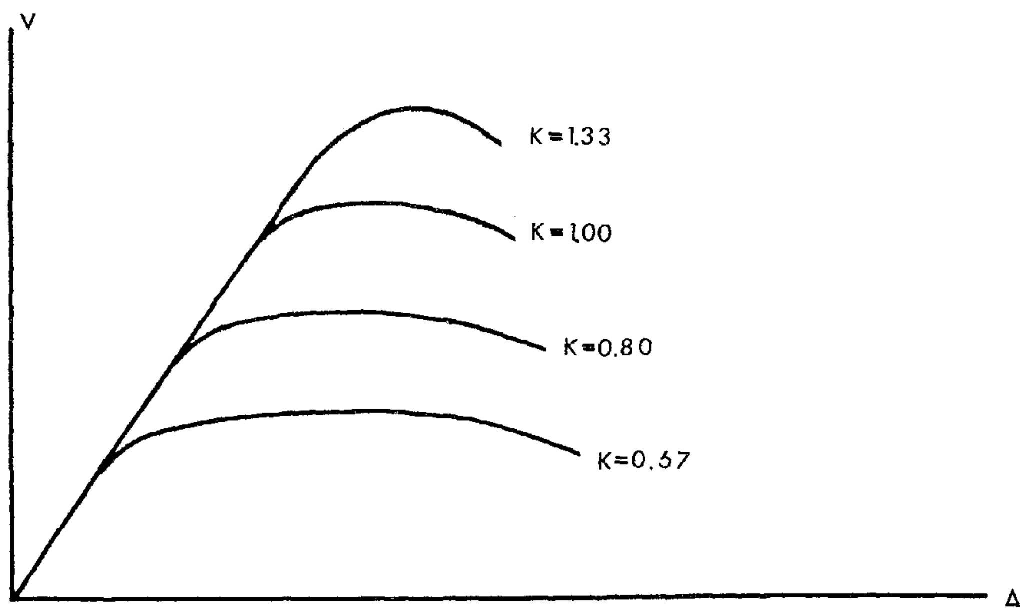


FIGURE 7.7

and dependability--must enter into the formulation of design spectra as given in the next section. However, before going into this formulation, it is well to treat a very important aspect of seismic design. This is the adoption of the appropriate structural system for the given structure configuration and earthquake demand conditions. Consider the following cases:

Insufficient Rigidity of a K=0.67M Frame with $\Delta_{DAM} < \Delta_D$ (see Figure 7-8).

The adoption of the K=0.80M system is preferable to the increase of section sizes in the original frame.

Insufficient Ductility of a K=1.33 Box System with $\Delta_{CON} < \Delta_C$ (see Figure 7-9).

The formation of a K_1=1.00M system is preferable to the general increase of section sizes in the original design. Even more dramatic in this case would be the consideration of a type of brittle pre-cast system (1.33 PC) of wall structure, where a strong back-up frame would be most essential for collapse safety.

Design Spectra

In the section on Basic Response Spectra, two groups of spectral characteristics were discussed: those dealing mainly with ground motion such as the A_g, V_g, D_g base lines and the site soil-column response factor; and those dealing with the elastic system such as the mean and standard deviation of the DAF, damping, and ductility. The Use Group of a structure has already been related to A_g, and the purpose of this section is to relate the actual structural character-

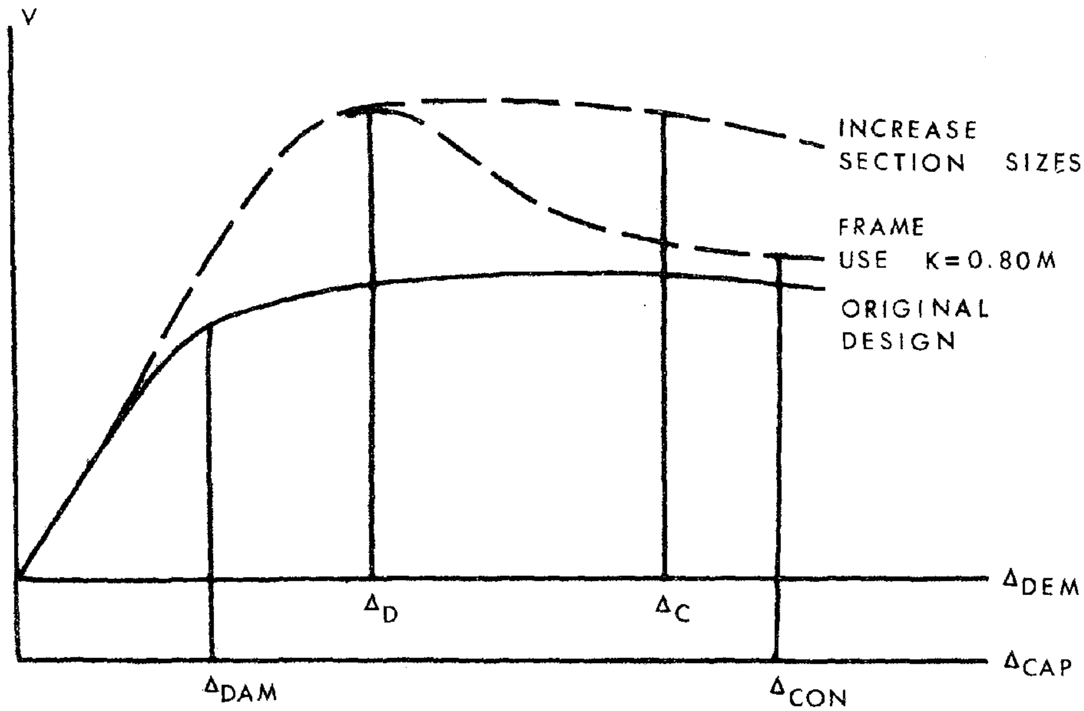


FIGURE 7.8

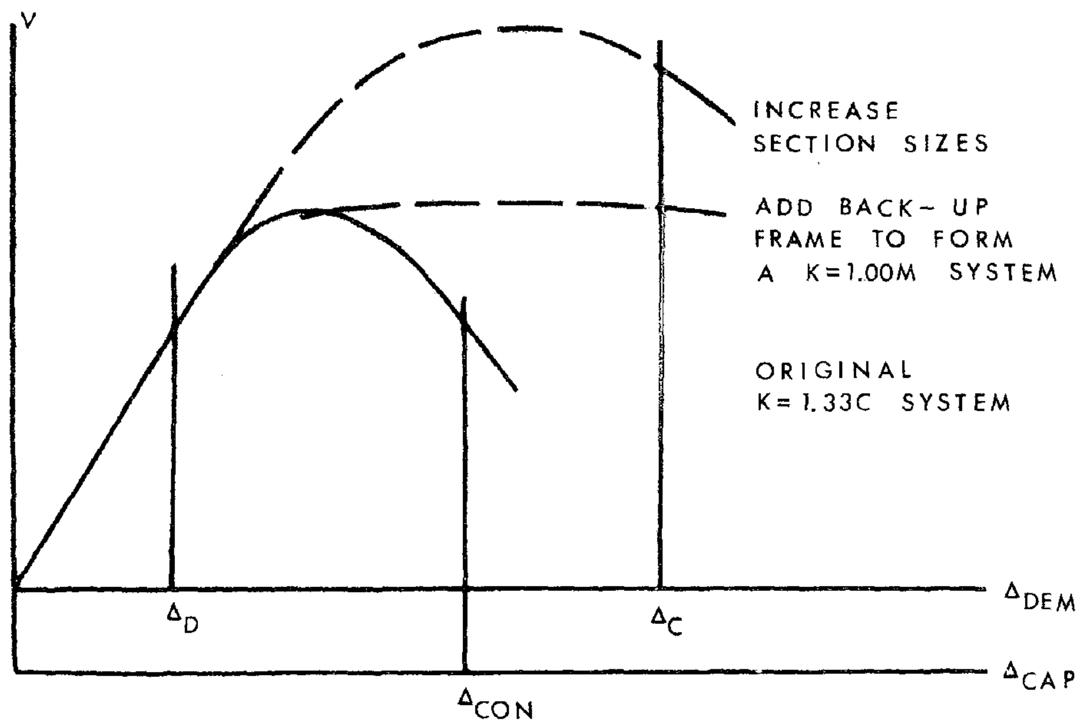


FIGURE 7.9

istics to spectral characteristics which deal with the ideal elastic system. Specifically,

- The confidence level or number of standard deviations $K\sigma$ above the mean DAF is to be related to the structure Use Group and the reputation or dependability of the type of structural system.

- Damping is to be related to structural damping and structure-foundation interaction.

- Ductility is to be related to the member ductility, connection details, and number of back-up frames or indetermineacies contained in the type of structural system.

Confidence Limit for the DAF:

The choice of spectral DAF levels based on the reputation or performance record of a structural system may be best explained in terms of the reliability levels or confidence limits of the amplified response spectrum (or DAF). It should be realized that the DAF values are random variables which can be described in terms of their average value and standard deviation (σ) as shown in Figure 7-10. In the Newmark-Hall paper (Appendix 5), it is stated that the DAF values in Table 2 are such that there is only a 10 percent chance of being exceeded. This level would coincide roughly with the upper (two-σ) confidence level. It is proposed here, that the appropriate DAF confidence level may be different for different types of structural systems. If a system has proven to be reliable from past experience,

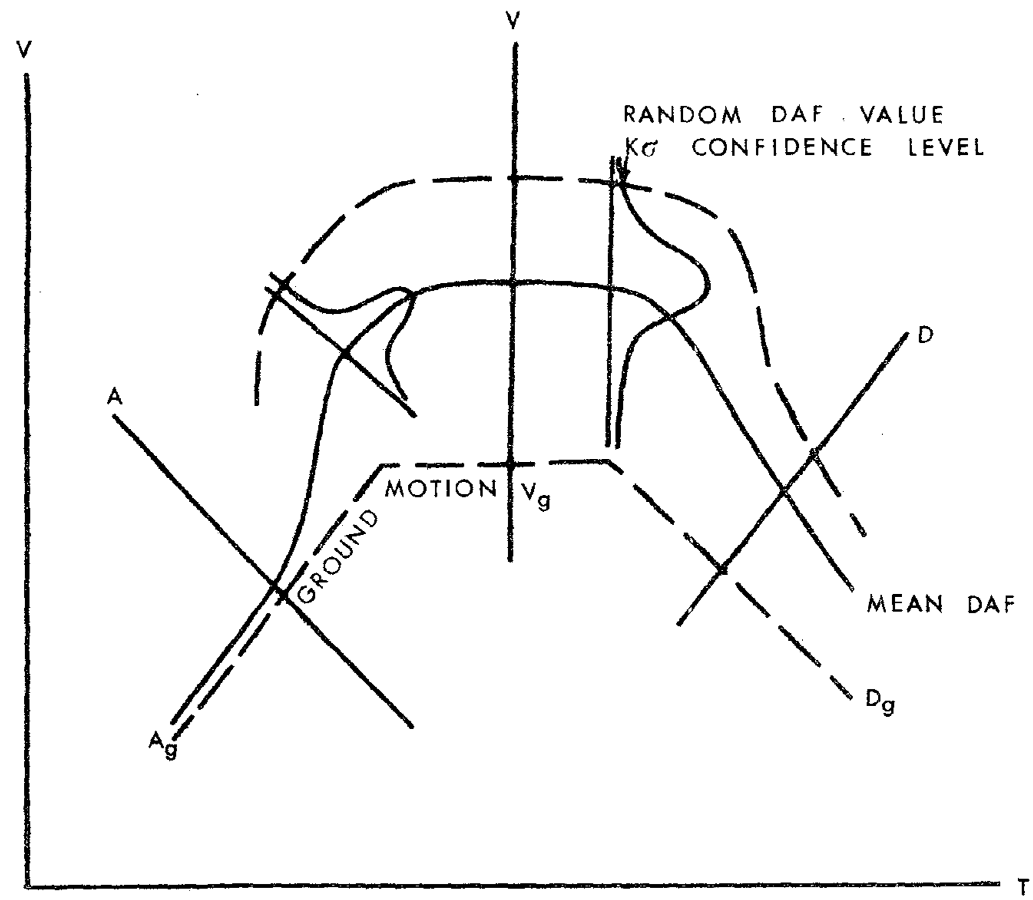

V
V
RANDOM DAF VALUE
Kσ CONFIDENCE LEVEL
D
A
MOTION
V_g
GROUND
MEAN DAF
D_g
A_g
T

FIGURE 7.10

and can tolerate a fairly wide range of displacements before showing significant damage, and has back-up systems to prevent collapse--then it should merit a low confidence level (say the (one-σ) level) for its design spectrum. This is because the system can be depended upon to resist chance exceedences of the design level without failure.

On the other hand, if a system is new and untried, if it is brittle or not well connected, or has no reliable back-up system, then there is a need for that protection against chance excess demands as would be provided by the 2σ exceedence level.

Therefore, for each type of structural system, an appropriate component K_T of the confidence level will be assigned according to the system performance record.

Also, for the purpose of providing a desired level of protection for critical facilities, an additional component K_G of the confidence level will be assigned according to the structure Use Group. The total confidence level for a given group and type is $(K_G+K_T)\sigma$ above the mean DAF. It should be recalled that in addition to this confidence limit, the ground acceleration base line A_g for the spectrum has also been assigned from the AZG according to the accepted probability of exceedance as governed by the building use group.

Damping:

Damping due to the type of structural system is termed as β_S, and will be assigned according to the general material behavior. A new and additional component of damping occurs due to the effects of structure foundation interaction. This will be termed as β_F and will be evaluated in accordance with the methods in reference 34. Total damping for a design spectrum is therefore $\beta' = \beta_S + \beta_F$.

Ductility:

Ductility varies with the type of structural system according to the material, member and connection details, and the number of statistical indeterminacies or back-up systems within the structure. Each structural type will be assigned a ductility value μ' according to its particular description.

Properties

The complete set of spectral properties for the different types of structural systems may be organized as shown in Table 7-5. The values are given for some example systems with a Use Group B. In general, the values are assigned by professional judgment to provide agreement with past experience, reported behavior, and a reasonable level for the final design load values.

Definition and Formulation of Design Spectra in Terms of Modified Inelastic Spectra

While it is generally recognized that the condemnation or collapse threshold level of earthquake motions must be resisted by structural behavior in the inelastic range, this report advances the concept that some inelastic behavior may be tolerated while resisting motions representative of the damage threshold earthquake. This concept is based on the fact that the structural damage threshold occurs at story deformations somewhat greater than the deformation at the attainment of first yield or design strength at the member section

Table 7-5

Given Use Group B, $K_G = 1.0$

Type	Damage Threshold			Collapse Threshold		
	K_T	β_S	μ'	K_T	β_S	μ'
0.67M	1.00	0.05	1.5	1.00	0.05	4.00
1.00M	1.20	0.05	1.5	1.10	0.07	3.00
1.33C	1.50	0.05	1.5	1.20	0.07	2.00

having the highest stress ratio. Figure 7-11 shows the states of strength design, damage threshold, and condemnation threshold.

The following discussion presents the definitions, methods, and reasoning employed for the formation of modified inelastic design spectra:

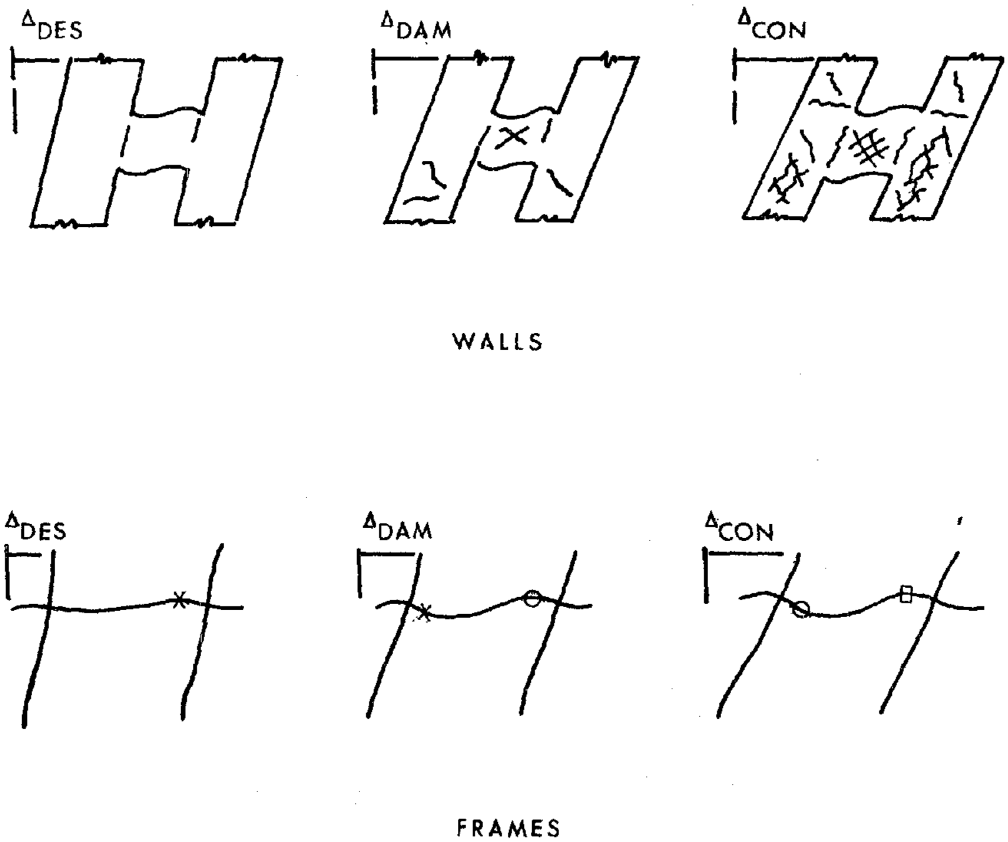
- the Damage Threshold Spectrum for strength determination (DTSS), for setting the design strength of members, and

- the Damage Threshold Spectrum for Deformation determination (DTSD), for the evaluation of P-Delta effects on design strength.

- the Condemnation Threshold Spectrum for Deformation determination (CTSD), for the evaluation of member ductility demand, and detection of instability problems due to P-Delta effects.

The modified, inelastic response, design spectra are formed by the use of what may be termed as spectral modification factors μ' and β'. These factors are used as ductility and damping factor values to obtain the inelastic spectra. They are selected by judgment, consistent with the respective ductility and damping capabilities of the particular type of lateral force-resisting system of a given structure; where these capabilities are as evidenced by test results and past performance of similar systems having undergone strong earthquake ground motions. Referring to Figure 7-12, the objectives are:

1) For the Damage Threshold Earthquake (DTEQ) the factors

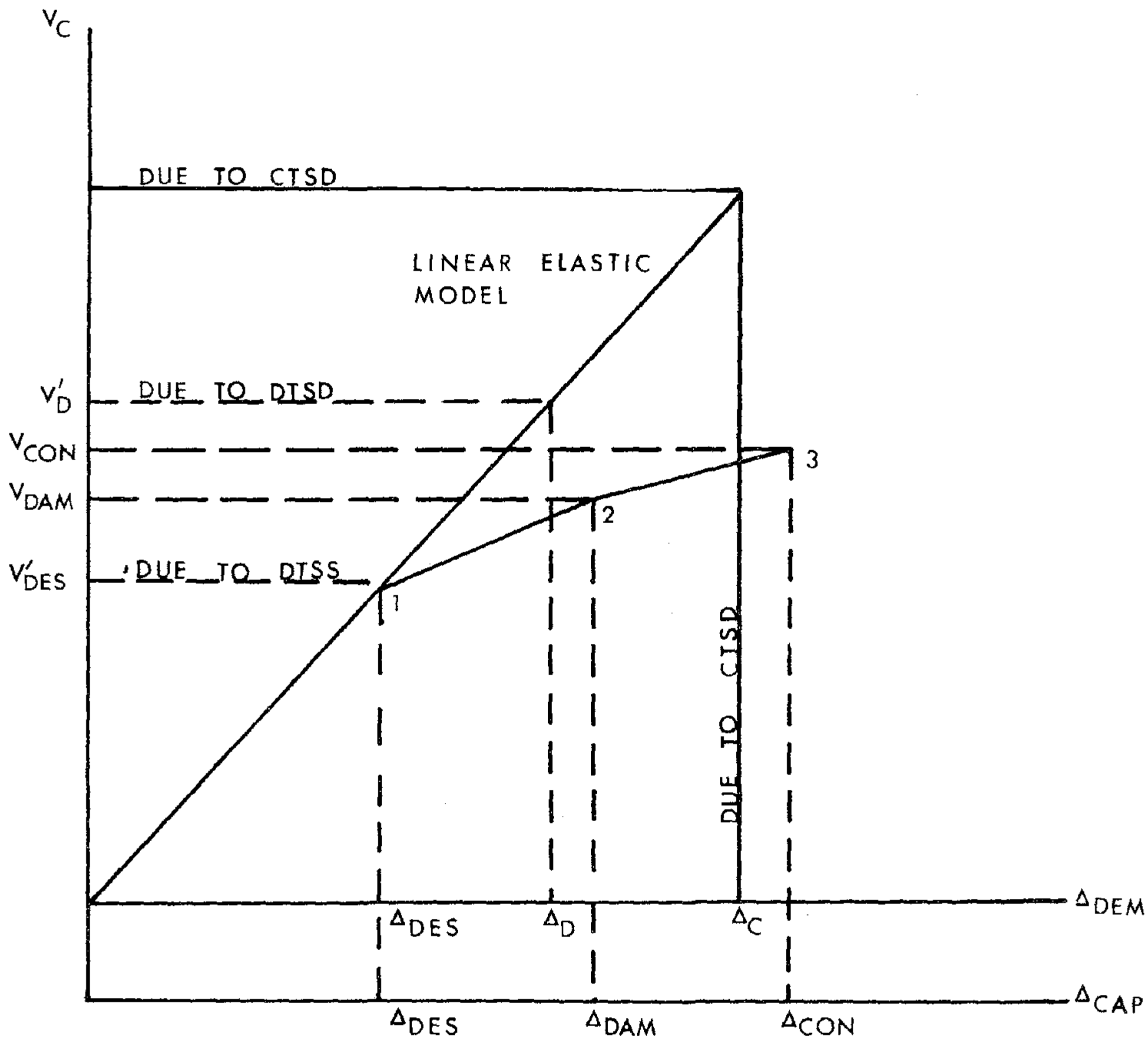


DESIGN LEVEL X
AT ULTIMATE
STRENGTH BASIS

DAMAGE O
TRESHOLD

CONDEMNATION □
TRESHOLD

FIGURE 7-11



V′ = STORY SHEAR ON ELASTIC MODEL
V = STORY SHEAR ON INELASTIC MODEL
DES = MEMBER DESIGN LEVEL

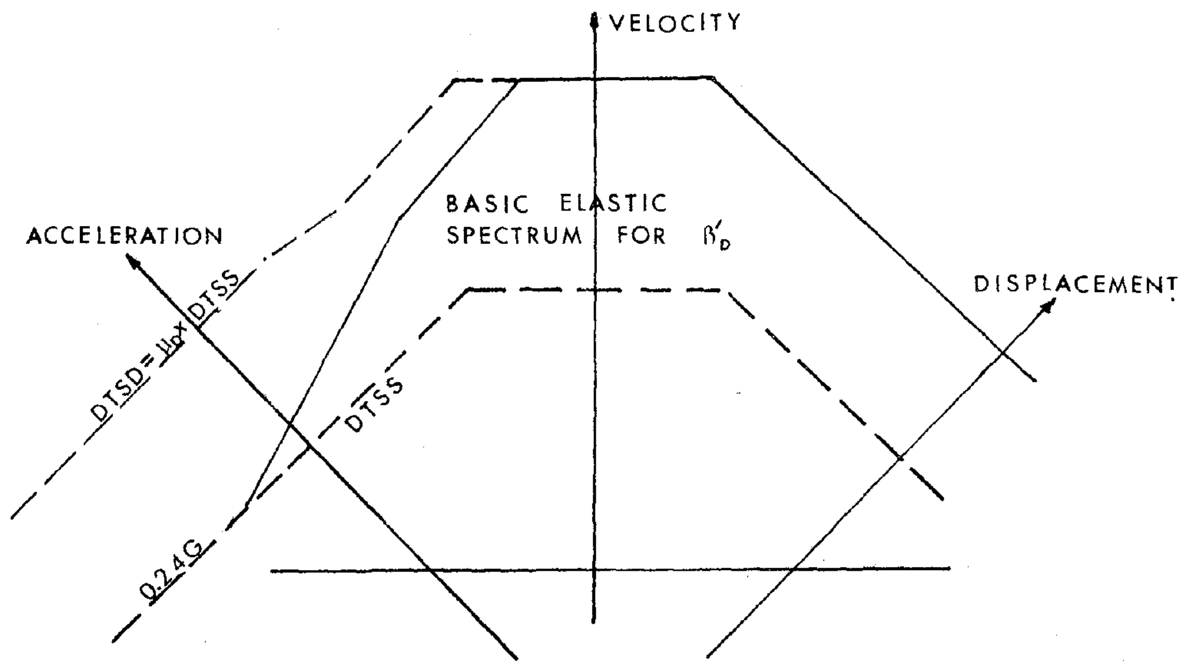
FIGURE 7.12

μ'_D, β'_D are to produce an inelastic acceleration spectrum (see Figure 7-13) (DTSS) such that when members are designed on the ultimate strength basis for forces due to: (this is in accordance with the design procedure given in the next section)

- dead load
- code specified (non-load factored) live load
- DTSS Acceleration
- Extra story shear due to P-Delta effect at DTSD deformation

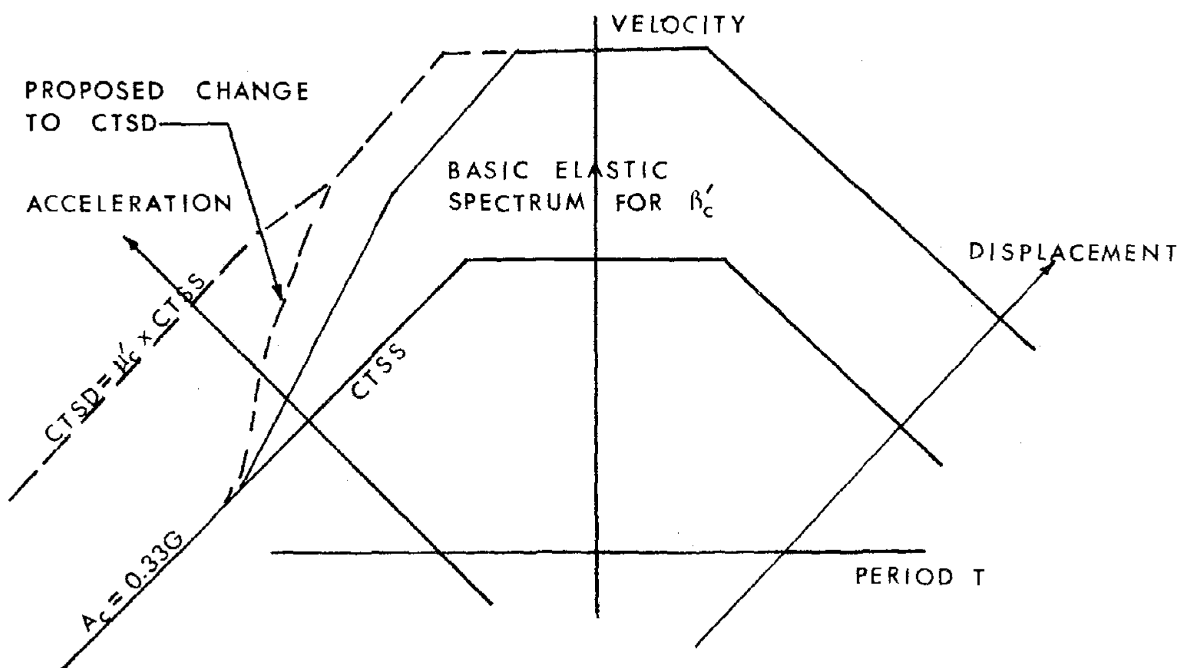
then it is assumed that the structural damage threshold deformation capacity will be equal or larger, with an acceptable level of reliability, than the deformation demands Δ_D of the damage threshold earthquake (DTEQ). If it is further assumed that the damage threshold demand is provided by the DTSD deformation, then it is important that the magnitude or size measure, the shape, and the μ'_D and β'_D values be selected to provide DTSD values having the acceptable level of reliability (corresponding to an upper confidence limit). This damage threshold deformation demand assumption would allow study and formulation of drift limitations for control of non-structural damage.

2) For the Condemnation Threshold Earthquake (CTEQ) the factors μ'_C, β'_C are to produce an inelastic deformation spectrum (see Figure 7-14) (CTSD) that will provide deformation values reliably greater or equal to the deformation demand Δ_C of the CTEQ. Local member ductility demands and story stability



Modified Damage Threshold Spectra with μ'_D and β'_D (Group B, Managua)

FIGURE 7.13



Modified Collapse Threshold Spectra with μ'_C and β'_C (Group,B Managua).

FIGURE 7.14

checks (involving P-Delta effects) evaluated for CTSD deformations will therefore be upper confidence limits for the CTEQ demands.

For this report, the purpose of the μ_C' value is to produce a reasonable, but reliably large estimate of CTEQ deformations. For the formation of an inelastic deformation spectrum, a large value of μ_C' provides conservative (large) values of deformation demand. If it were desired to employ an inelastic acceleration spectrum (CTSS) for the strength design of members, then a large μ_C' value would of course provide non-conservative (low) force demand values. A proposed change to the Newmark method for construction of the CTSD is indicated in Figure 7-14. This may provide a more realistic estimate of actual structure deformation.

It is assumed in this report that the DTSS is larger than the CTSS and therefore controls the strength design of members. Examples of the design spectra DTSS and CTSD will be constructed later in this chapter, after the presentation of the complete proposed Design Procedure in the next section.

Proposed Design Procedure

The DTSS and CTSD design spectra are to be employed in the following design procedure:

. Given Use Group, Life, and Site:

Obtain A_D, A_C, K_G, and site soil-column response factor. The mean DAF and σ values are known. (These will be available from Part II of this study.)

. Given Structure Type and Foundation:

Obtain β_S, β_F, μ, K_T at both Damage D and condemnation C levels.

. Construct Design Spectra:

DTSS for Member Section Design, with μ'_D, $\beta'_D = \beta_S + \beta_F$, $(K_G + K_T)\sigma$.

CTSD for Ductibility Evaluation and Stability Analysis, with μ'_C, $\beta'_C = \beta_S + \beta_F$, $(K_G + K_T)\sigma$.

. Formulate a Linear Elastic Model of the Structure and Find SRSS Value of:

1) Member Force Response to the DTSS;

2) Member Deformation Response to the CTSD.

. Design Members for Load Combinations on an Ultimate Strength Basis for:

1) Load Factored Vertical Dead and Live Load;

2) DTSS Force plus Vertical Dead and Live Load, and Seismic P-Delta Effects;

3) DTSS Force plus two-thirds Dead Load (for vertical acceleration effects). (See Appendix 6 for vertical acceleration effects.)

. Perform Deformation Analysis due to CTSD Response:

1) Evaluate local member ductility demands and compare with established allowable values (to be determined in Part II of this report).

2) Investigate the stability of structural system.

Construction of Example Design Spectra

Given the structure use group, life L, and structural system type, such that the spectral values (A, $K\sigma$, β', and μ') can be determined for both the Damage and Condemnation Level Earthquakes. The basic elastic spectra may be constructed with these known A, $K\sigma$, and β' values. (Examples are shown in Figures 7-3 and 7-4.) Then, with the given μ_D and μ_C ductility values, the inelastic design spectra DTSS and CTSD are constructed according to the Newmark method in Appendix 5. The complete procedure is given in the following example for a structure in the Managua region.

. Structure Use Group: B.

. Region: Managua.

. Type of Structural System: 0.67M.

. Economic Life L = 20 years.

. Site soil-column conditions are average such that the site factor S = 1.00.

. Structure-Foundation Interaction Damping $\beta_F = 0.05$.

. The ground acceleration base lines are found from Table 7-1.

$$A_D = 0.24g, \quad A_C = 0.33g$$

. The structural system properties are found from Table 7-5.

$$K_G = 1.0,$$

For DTSS

$$K_T = 1.00, \ \beta_S = 0.05, \ \mu_D = 1.5$$

For CTSD

$$K_T = 1.00, \ \beta_S = 0.05, \ \mu_C = 4.00$$

. The Spectral Properties are:

<u>For DTSS</u>

$$\beta'_D = \beta_F + \beta_S = 0.10, \quad \mu'_D = 1.5$$

$$K = K_G + K_T = 2.0, \quad K\sigma = 2\sigma$$

<u>For CTSD</u>

$$\beta'_C = \beta_F + \beta_S = 0.10, \quad \mu'_C = 4.00$$

. The Basic Elastic Response Spectra for these properties are constructed and are as shown in Figures 7-3 and 7-4.

. The modified inelastic Design Spectra DTSS and CTSD are constructed and shown in Figure 7-15.

It is interesting to see how design load values from this spectrum compare with loads from the Uniform Building Code. Let us consider a 10 story, Type 0.67M steel frame building with first mode period of 1 second. From Figure 7-15, the acceleration $S_a = 0.14g$, and the resulting base shear value would be about

$$V_1 = (0.14)\ (0.8)\ W = \underline{0.112\ W}$$

where the 0.8 factor allows for the multi-mode response participation factors (or equivalent weights), and W is the structure weight. Member forces from V_1 would be combined with Dead and un-factored Live Load forces for ultimate strength design. The corresponding UBC base shear is

$$V_2 = UKCW = 1.4\ (0.67)\ \left(\frac{0.05}{3\sqrt{1}}\right)\ W$$

$$V_2 = \underline{0.047\ W}$$

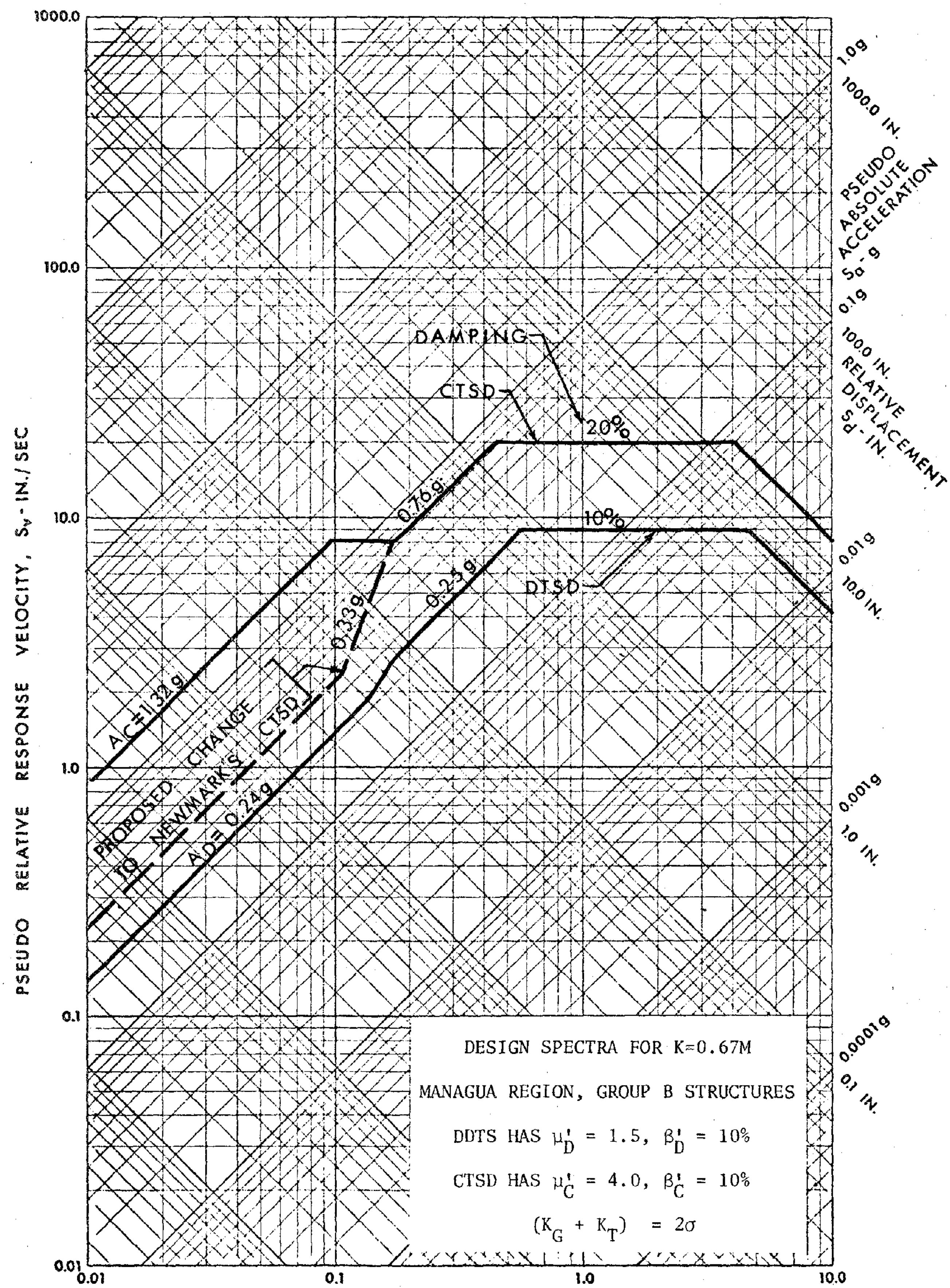

1000.0
100.0
10.0
1.0
0.1
0.01
0.01
0.1
1.0
10.0
PSEUDO RELATIVE RESPONSE VELOCITY, S_v - IN./SEC
PERIOD - SEC
1.0g
1000.0 IN.
PSEUDO ABSOLUTE ACCELERATION S_a - g
0.1g
100.0 IN.
RELATIVE DISPLACEMENT S_d - IN.
0.01g
10.0 IN.
0.001g
1.0 IN.
0.0001g
0.1 IN.
DAMPING
CTSD
20%
10%
DTSD
0.76g
0.25g
0.33g
AC=1.32g
AD=0.24g
PROPOSED CHANGE TO NEWMARK'S CTSD
DESIGN SPECTRA FOR K=0.67M
MANAGUA REGION, GROUP B STRUCTURES
DDTS HAS $\mu_D' = 1.5$, $\beta_D' = 10\%$
CTSD HAS $\mu_C' = 4.0$, $\beta_C' = 10\%$
$(K_G + K_T) = 2\sigma$

FIGURE 7.15

Member forces from V_2 would be combined with a load-factored value of 1.4 times the Dead and Live Load forces, for ultimate strength design. Therefore, although the V_1 is 2.4 times the V_2 value, the resulting V_1 member designs will not differ from the V_1 designs by as much as this amount because of the different method of factoring the Dead and Live Load effects.

It should be realized that these resulting spectra are for example only. The assigned spectral property values are very approximate. These values are to be refined in Part II of this study in order to provide consistent Design Spectra for all Use Groups and Structural Types.

CHAPTER VIII

SUMMARY, CONCLUSIONS AND FURTHER RESEARCH

Summary

In Part I of the seismic risk study for Nicaragua, the following topics are presented:

1. Geological setting for the country in general and the Managua area in particular.
2. Data base for past seismic events was extensively studied. Limitations of the data and approximations were discussed. Seismic recurrence for ten line sources and three area sources was developed.
3. Based on the assumption of the Poisson occurrence of seismic events, probabilities of exceeding different magnitude levels as functions of time for different regions was derived. Using Esteva's attenuation relationship, iso-acceleration maps for the country were constructed. Eleven cities in Nicaragua were considered in this mapping process. Cumulative distribution functions of peak ground accelerations for 20 and 50 years were established. This was shown to be one way of presenting seismic risk for Nicaragua.
4. Based on iso-acceleration maps, the Acceleration Zone

Graphs (AZG) were developed for the eleven cities. A method of determining load levels for consistent risk for the whole country was discussed and suggested. It was proposed that charts such as AZG be used for seismic zoning of Nicaragua.

5. Another parameter, the MMI scale, in understanding seismic risk was presented in probabilistic sense. Based on the damage data in the U.S.A., a method of determining insurance risk was presented.
6. Ground acceleration values from AZG were employed to set the level of the design spectra for structural damage prevention and condemnation control. A design methodology was proposed based on ultimate strength and loads resulting from the above inelastic design spectra.

Conclusions

It can be concluded that there are sufficient data and analytical methods to provide adequate seismic zoning information on an acceptable risk criterion. The methods presented here are simple, easy to use and transferable to structural design procedures. The zoning of the country can be interpreted from iso-acceleration maps or from cumulative distribution plots of peak ground accelerations or from AZG. The method of zoning presented here is general and is completely amenable to availability of additional future data. From the current study, it can be clearly seen that the seismicity of Nicaragua varies significantly from region to region. The Managua region and the region surrounding the line of volcanoes is much more seismic than the central or the eastern region. For example, Bluefields has the lowest probable loading level and the Managua region has the highest probable loading level. Looking at the iso-acceleration maps and AZGs, this information becomes obvious.

It is also obvious that methodologies as presented in this report can be used to convert the loading information based on acceptable risk and economic life of the facility to the structural design process. Further insight can be obtained regarding insurance and economic risk in different parts of the country based on the method presented in this report. Again, it can be seen that for a given region, it is cheaper to buy long-range seismic insurance than to purchase short-term coverage. Based on this insurance risk concept, various parts of the country can be compared for future probable economic impact due to a seismic event.

Further Research

In order to implement and use the procedures presented in their general form in this report, the following tasks are to be accomplished in Part II:

- Refined seismic zoning of the country based on acceptable risk levels for different classes and uses of structures. A detailed look at the acceptable probability levels and their effect on cost and general economy.

- The concept of microzoning a given region in the country.

- Mapping information in the form of effective ground velocity values V_g as predicted in terms of historical data and geological characteristics.
- Inclusion of the site (or soil-column) response factor in the evaluation of A_g, V_g, D_g at a given structure location.
- Representation of foundation-structure interaction in the form of an additive component of structural damping β_F and change in the structure period of vibration.
- Evaluation of the standard deviation values σ and the mean values of the spectral DAF for a given region, with recognition of either the averaging or the predominance of the possible earthquake sources (Benioff Zone, Volcanic, and Local Faulting) as they affect the A_g, V_g, D_g values.
- Formulation of a more precise listing of the Structure Use Groups ranging from critical to non-essential facilities; and establishment of the corresponding acceptable exceedance probability values P for typical structure life times L.
- Assignment of the use Group contribution K_G to the spectrum DAF confidence level.
- Elastic structure modeling techniques to allow reliable prediction by response spectrum analysis of structural element forces and inelastic deformations.
- Improvements in the method of forming the inelastic force and displacement response spectra.
- Categorization of the types of structural lateral force-

resisting systems and the assignment of the corresponding ductility μ, damping β_S, and structure type contribution K_T to the spectrum DAF confidence level, for both the damage and condemnation threshold spectra.

- Formulation of appropriate ultimate strength member design equations for all acceptable materials; specification of essential details necessary for the ductility and stability against collapse; and establishment of allowable ductility demand limits for all acceptable materials and systems.
- Simplification of the design spectrum, the dynamic analysis, and the deformation and stability analysis, to a procedure similar to that employed by the Uniform Building Code. This simplified design procedure would be applicable to the majority of structures. The detailed response spectrum analysis would be required only for those structures which are extremely critical, costly, and/or unique in their configuration and structural systems.
- With damage and economic data from Nicaragua, an insurance and economic risk analysis is to be accomplished. Risk to life and limb will be treated in Part II of this study.

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APPENDIX 1

THE DECEMBER 23, 1972 EARTHQUAKE

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Introduction

The Managua Earthquake actually consisted of three tremors. The major shock, which registered a surface wave magnitude (Ms, NOAA) of 6.2 and a body wave magnitude (Mb, NOAA) of 5.6, was followed by two major aftershocks within one hour, with Mb=5.0 and 5.2, respectively. The quakes were relatively moderate in size, compared to other major earthquakes (e.g. San Francisco, 1906, M=8.3), but caused extensive damage because (1) the epicenters were shallow, (2) surface rupture occurred, and (3) many buildings were constructed with an adobe or taquezal type of construction.

Intensity

The maximum intensity of shaking (Figures A1-1, A1-2), employing the Modified Mercalli Scale, was X along the lakeshore, with VII-IX common throughout most of the city center. (An intensity IX is defined as follows: "Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. . . . Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken." (Reference 7, p. 59.)

The intensity decreased radially outward from the city center (reference 15, p. 18). Near the epicenter, however, intensity contours

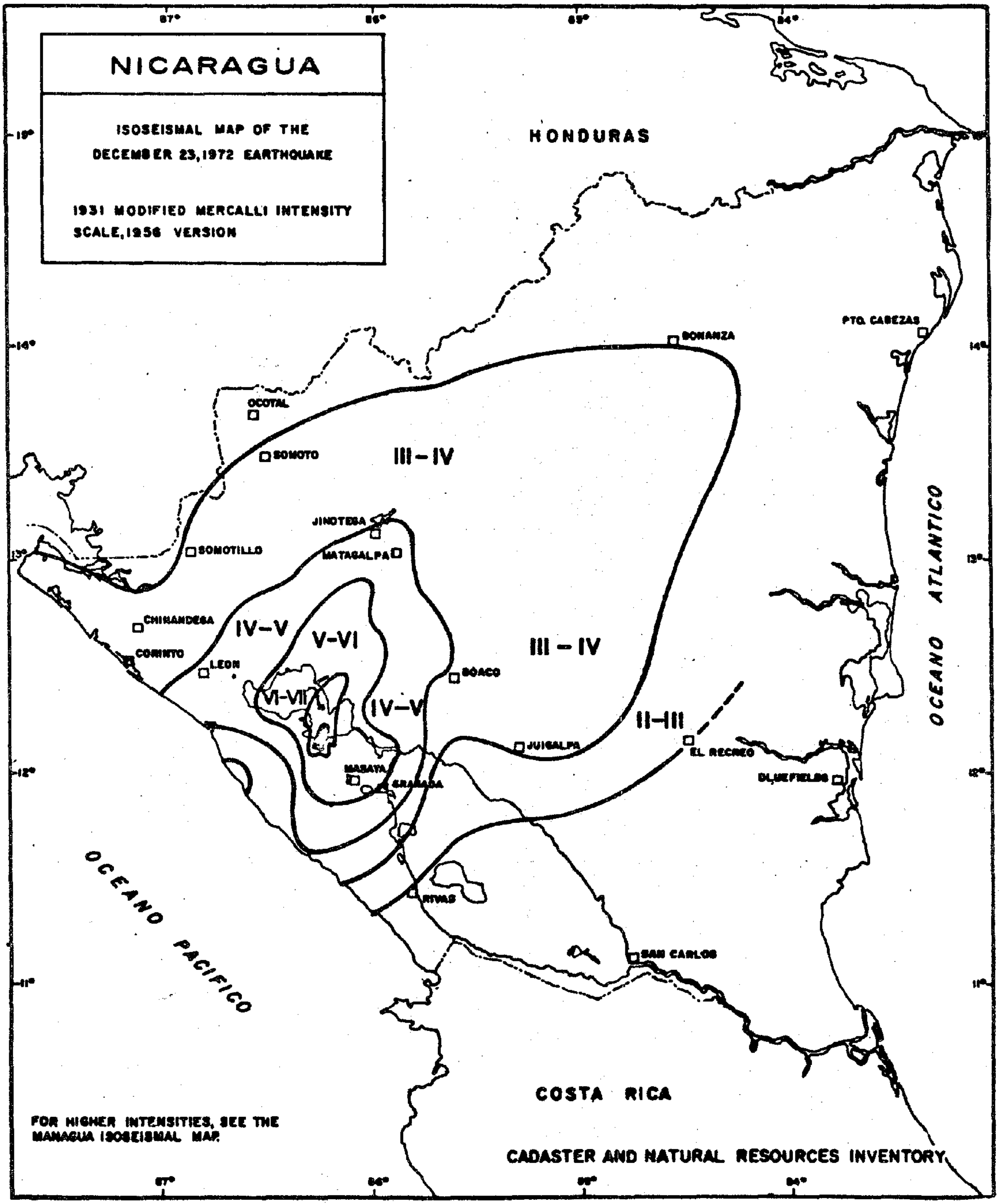

Figure A1-1 Isoseismal Map of Nicaragua
After Hansen and Chavez, 1973

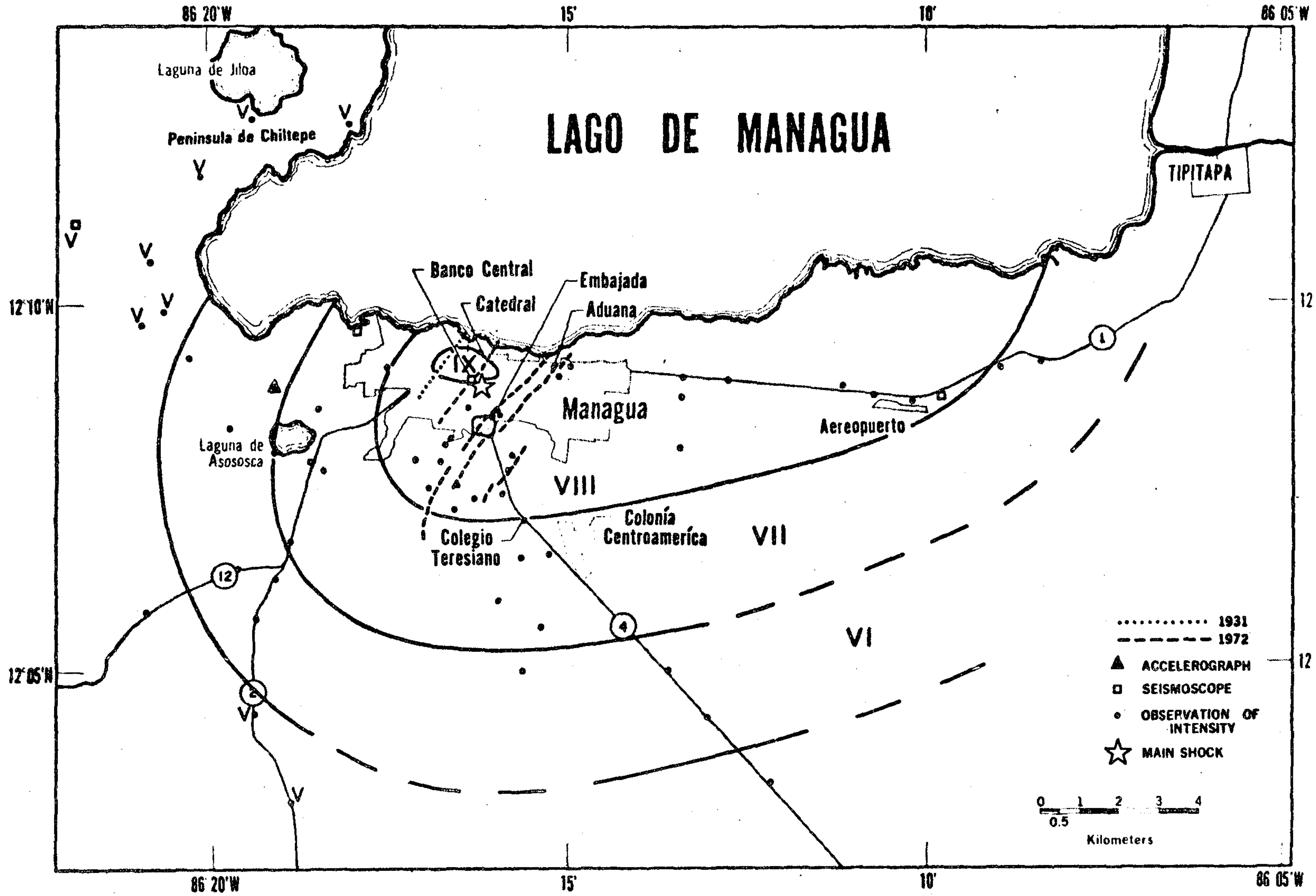

Figure A1-2 Isoseismal map, Managua earthquakes of 23 December 1972. Dashed lines are fault traces from Brown and others (1973).

After Dewey, it al, 1973

parallel the active faults, whereas southwest of the city they cross the faults. This suggests release of energy near the city center, rather than to the southwest (reference 4, p. 70).

Shaking

"Duration of the earthquake in its destructive phase was about 7 seconds" (reference 3, p. 143). The shaking was described as "a series of vertical shakes, followed by horizontal motion, then a vertical 'drop.'" (Reference 15, p. 18.) Peak ground acceleration, recorded 7 km west of the city center, was 0.39 g (reference 15, p. 18).

Damage

Damage occurred as ". . . two broad lanes of heavily damaged buildings of high density, separated by a stretch wherein heavy damage was expressed in a loose, random pattern." (Reference 2, p. 267.) The western lane was bordered by the 1931 fault trace; the eastern was situated between the Tiscapa Fault and the "secondary trace 400 m to the east" (Chico Pelon Fault) (reference 2, p. 267).

Hypocenter Location

The original hypocenter calculation, based on P-wave arrivals, was by NOAA. The location, 27 km northeast of the city center, was erroneous for two reasons: (1) a poor seismic net exists for Central America, and (2) the standard P-wave trend-time tables are incorrect for this area. The "correct" determination, based on an accelerogram recorded at a nearby refinery, placed the hypocenter beneath the center of the city, at a depth no greater than 8 km. This placement has led to a better location of the Benioff Zone in Central America by relocating previous earthquake hypocenters, based on the erroneous velocity data, to the south (reference 4, pp. 66, 70, 75). Aftershock data substantiate the accelerogram hypocentral location, although the depth indicated is 8-10 km, and show three zones of activity. The zones are shown in Figures A1-3 to A1-5.

P-wave first motions, recorded at teleseismic distances, are consistent with fault plane movement parallel to the mapped surface fault traces within the city. It is probable that the major seismic zone, striking N 30-40° E and dipping from 74° W to 82° E, with a 1 km width, bifurcates near the surface into 2 fault traces, the Tiscapa and Chico Pelon Faults. This is supported by the observations that the two faults are at the most only a few hundred meters apart, they trend towards an intersection, and they display roughly equal amounts of displacement from the 1972 tremors. The zone passes through Laguna Tiscapa and the Customs House, and is within 1500 m of nearly all of the severely damaged part of Managua. Aftershock data also suggest left-lateral movement, the same as that observed on the ground. The fault zone responsible for the major portion of the seismic energy was about 15 km long (reference 4, pp. 66, 71; reference 11, p. 97; reference 18, p. 95; reference 14, pp. 115, 127).

Nature and Amount of Fault Movement

Movement was predominantly sinistral, or left-lateral, although local dextral (right-lateral) and normal, or vertical slip, did occur. The faults are manifested in unconsolidated sediments and

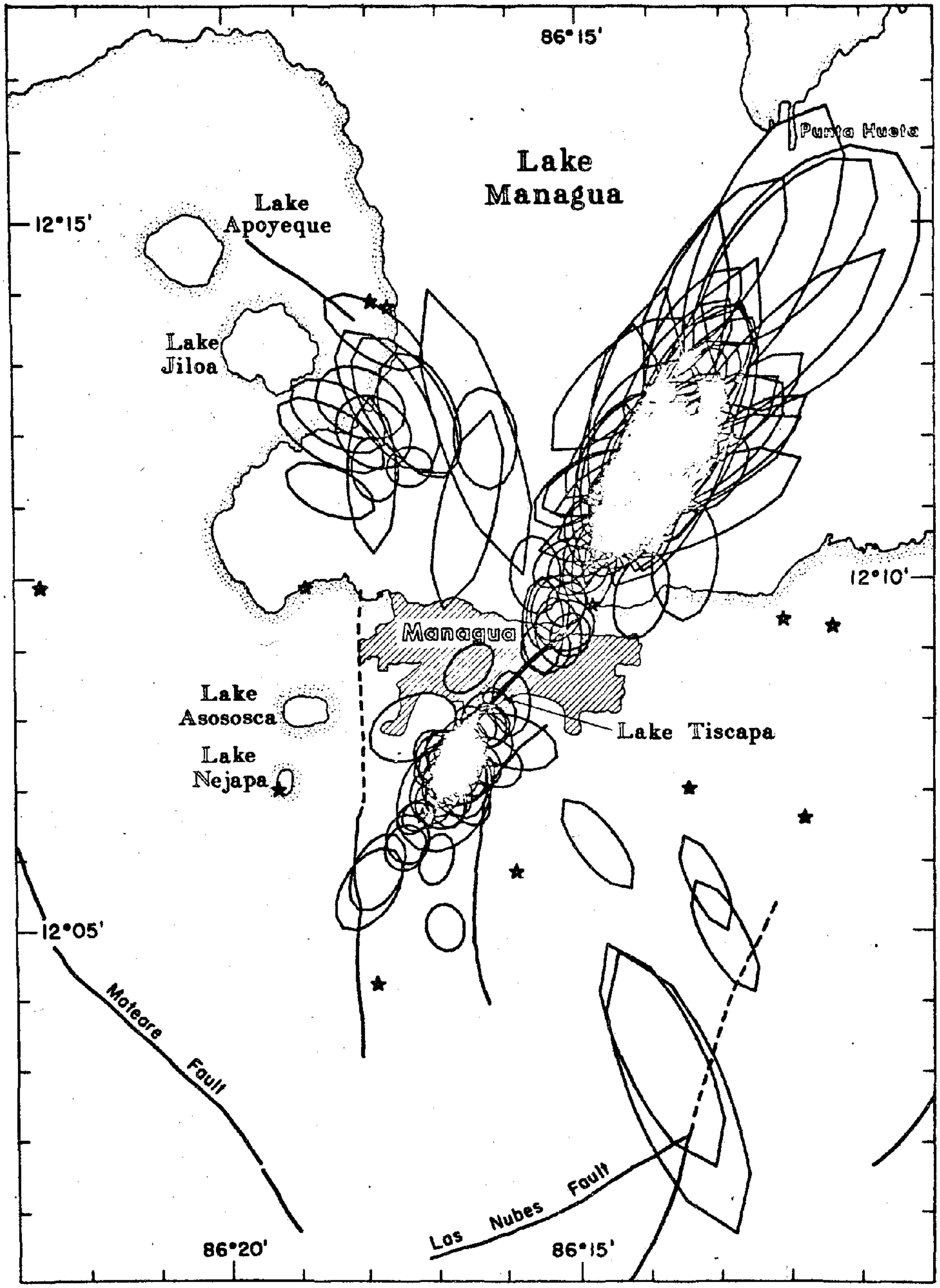

Figure A1-3 Locations of 171 aftershocks. The polygons represent the error in location assuming a possible error in reading arrival times at each station of 0.1 second. Station locations are designated by stars. The wide solid line in Managua represents faults B and C mapped at the surface (Plate 1, Brown and others, 1973).

After Ward, et al, 1973

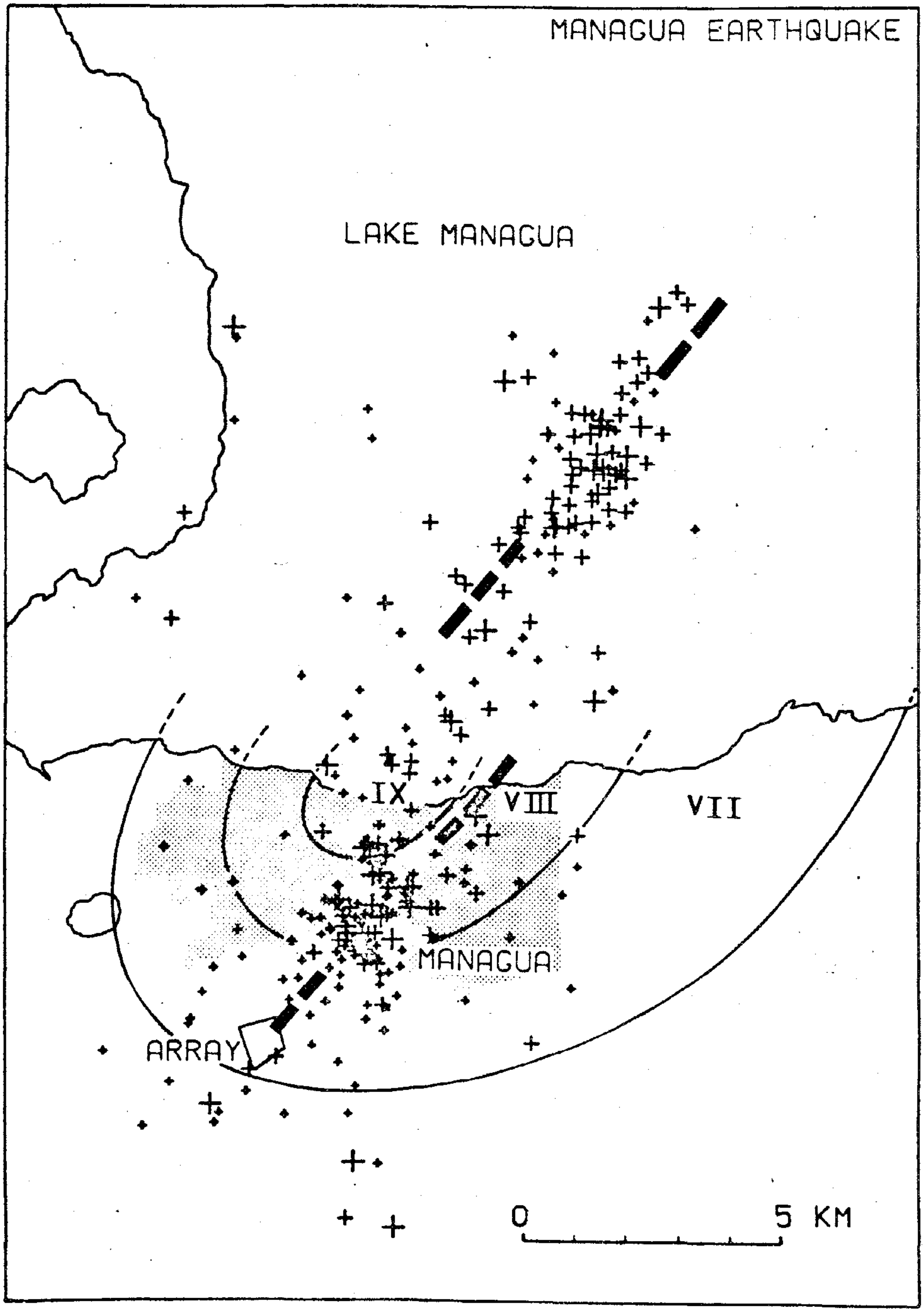

Fig.A1-4 Intensity map for the main shock and epicenters of 300 aftershocks located by data from a 5-station seismic array. At least two linear trends in the aftershock activity are suggested, as indicated by the dashed lines.

After Matumoto and Latham, 1973

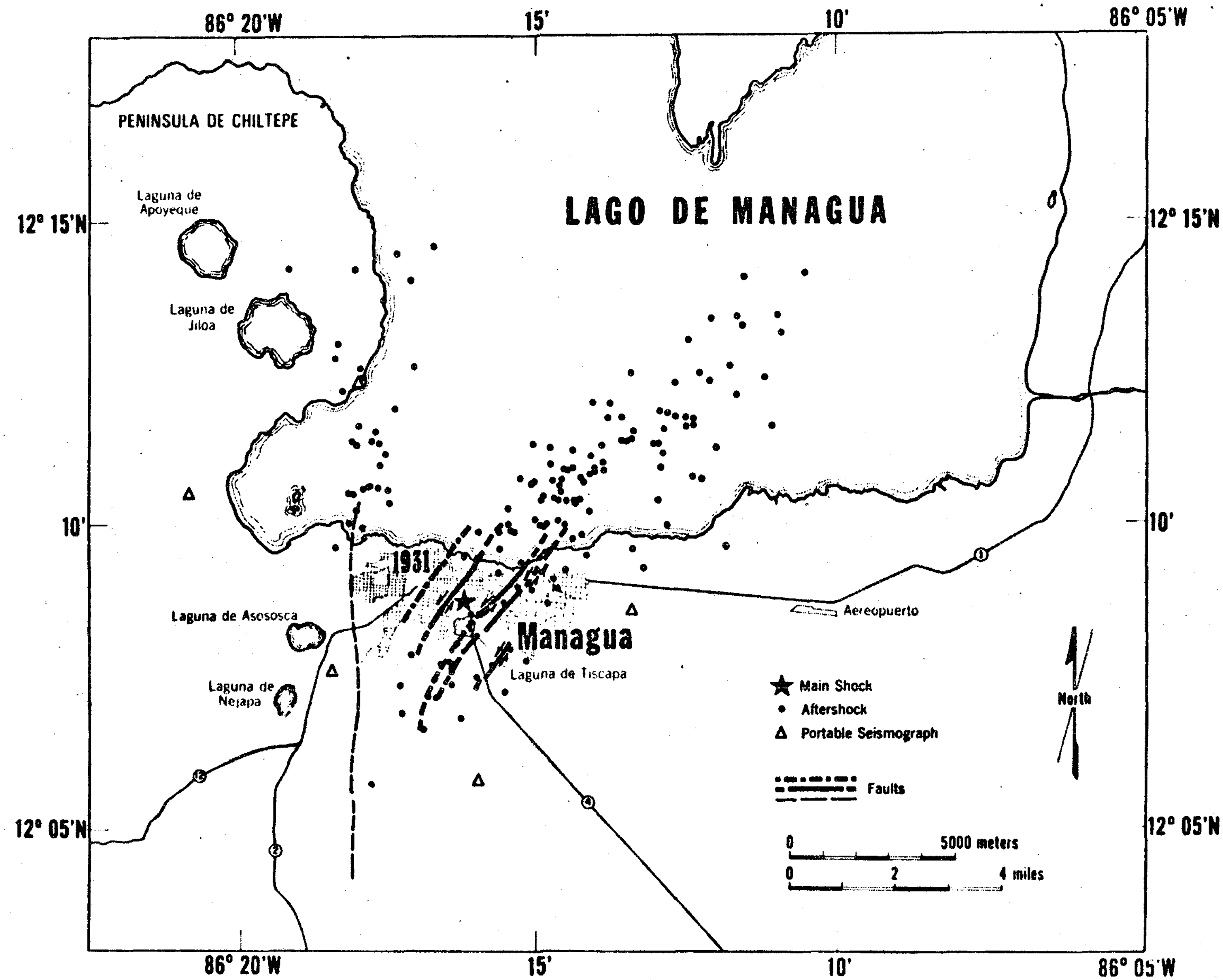

Figure A1-5 Aftershocks to the Managua earthquake, 28 December 1972 - 04 January 1973. Fault traces from Brown and others (1973) (heavy dashed lines) and Kuang and Williams (1971) (light dashed line).
After Dewey, et al, 1973

volcanics by en echelon, rather than continuous fractures. The strike of these fractures is generally more northerly than that of the fault zones themselves. The faults die out within 1.6 - 5.9 km of Lago de Managua.

The maximum horizontal displacement, determined by triangulation, was 40 cm. Maximum vertical movement measured was 10.2 cm.

According to Plafker and Brown, a geometric reconstruction of Tiscapa Crater and the lakefront suggest 10 m sinistral and 30 m vertical displacement. Noting that 1972 movement was predominantly sinistral, they suggest that there has been a late Holocene (past 1,000-2,000 years) change in the tectonic deformation style (reference 14, p. 134). Their geometric reconstruction is open to interpretation, however. In addition, it is possible for an individual seismic trend to be in apparent conflict with the overall, long-term trend.

Cause of the Earthquake

A number of hypotheses have been advanced to explain the earthquake mechanism. They relate to various interpretations of the tectonic setting of Nicaragua.

The 1972 earthquake hypocenter was much too shallow to be related to the underlying Benioff Zone. It was probably caused by tectonic forces associated with the Nicaragua Depression and/or with the chain of volcanoes. This earthquake was typical of numerous Central American tremors, which, although responsible for only a small fraction of the total seismic energy released in the region, and which are of small or moderate magnitude, nevertheless produce intense ground shaking in small, often densely populated areas (reference 4, p. 67).

One explanation of the earthquake relates the tremblors to isostatic and gravity conditions. There is a correlation of earthquakes in the Managua area with years of drought, which is reflected in surface levels of Lago de Managua falling as much as 3.77 m. It is suggested that the low lake level leads to a decrease in weight from the lake onto the earth's crust below. Resisting forces, in years of low lake levels, are thus in excess of equilibrium, strains build up, and earthquakes result. In addition, there is a correlation of earth tide maximums with the 1931, 1968, and 1972 quakes (reference 16, pp. 56-60). One problem with this theory is that it postulates upward movement of the block, whereas slight subsidence, in addition to horizontal movements, in fact occurred. However, earthquakes have been recorded during the loading and unloading of reservoirs, and the decreases in water levels could have been the triggering force needed to initiate rupture along faults already at maximum stress.

A second theory "explains" both the left-lateral offset within Managua and the right-lateral offset of the Cordillera de Marrabios. It suggests that the chain of volcanoes is a secondary spreading center, analogous to the East Pacific Rise or Mid Atlantic Ridge. The rift is thus a zone of crustal extension, a fact suggested by the general structure of the Nicaragua Depression. The entire rift is being pulled apart in the fashion shown in Figure A1-6, but with irregular breaks, known as transform faults. Movement on such a transform fault would show left-lateral displacement within Managua, located within the zone of extension between offsets of the ridge, or

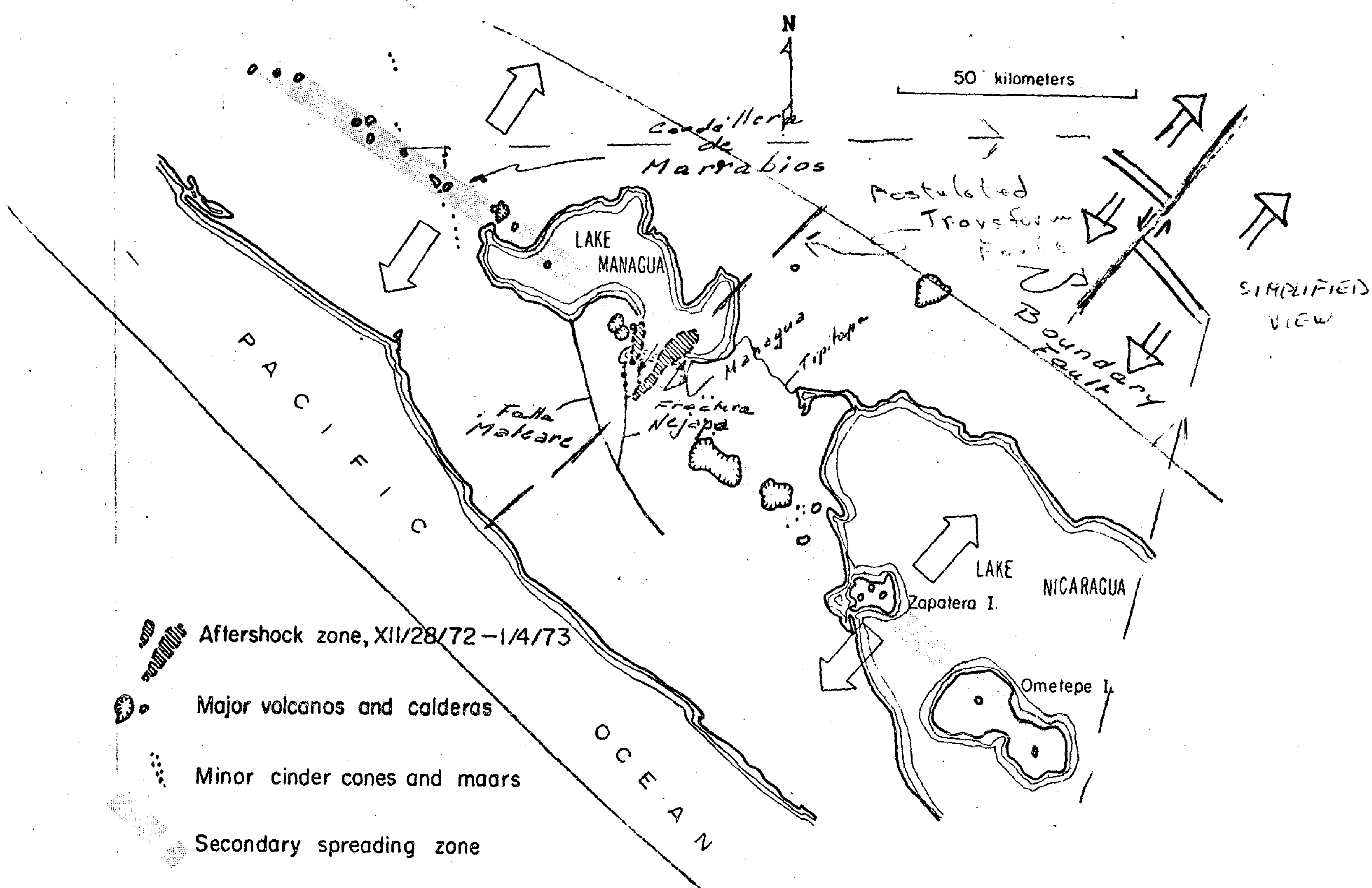

Figure A1-6. The Managua earthquake fault system viewed as a transform fault between offset segments of the principal volcanic chain. Broad arrows indicate relative motion of western Nicaragua and eastern Nicaragua away from secondary spreading zone beneath volcanic chain. Small arrows indicate slip on causative fault of Managua earthquake. Volcanic features from McBirney and Williams (1965) and Ui (1973).

After Dewey, et al, 1973

volcanics, yet beyond the immediate area of the city both sides of the fault would show movement in the same direction (reference 4, pp. 82-84). This theory is attractive because it suggests an explanation for several geologic phenomena. It is especially useful because it predicts earthquakes for two other parts of the Nicaragua Depression, where the Cordillera de Marrabios is offset.

A third theory ascribes the concentration of seismic activity near the Nicaragua volcanics to an unusually thin, rigid lithosphere, resulting in the crust being subjected to unusually high tectonic stresses. Although both north-south compression or east-west tension could produce the observed pattern, geologic evidence favors the latter (reference 4, pp. 84-85). This explanation fits with the apparent tensional nature of the Nicaragua Depression, as well as regional plate movements, and also the observed left-lateral movement of the faults in Managua, but does not explain the line of volcanoe offset. Such a stress system could be caused by movement of the Caribbean Plate (Eastern Nicaragua) eastward, while the Cocos Plate (Pacific Ocean) travels north and plunges under Central America (reference 11, p. 102).

The final theory is that right-lateral offset is occurring along the Cordillera de Marrabios (Figure A1-7). Extension occurs at a kink, and left-lateral oblique faults, members of the conjugate shear set, form (reference 15, p. 12). This explanation is similar to the preceding one, but fails to explain the cause of the kinking, extension, or the origin of the volcanics. Although volcanoes do occur along fault zones, we are not aware of any cases where the line of

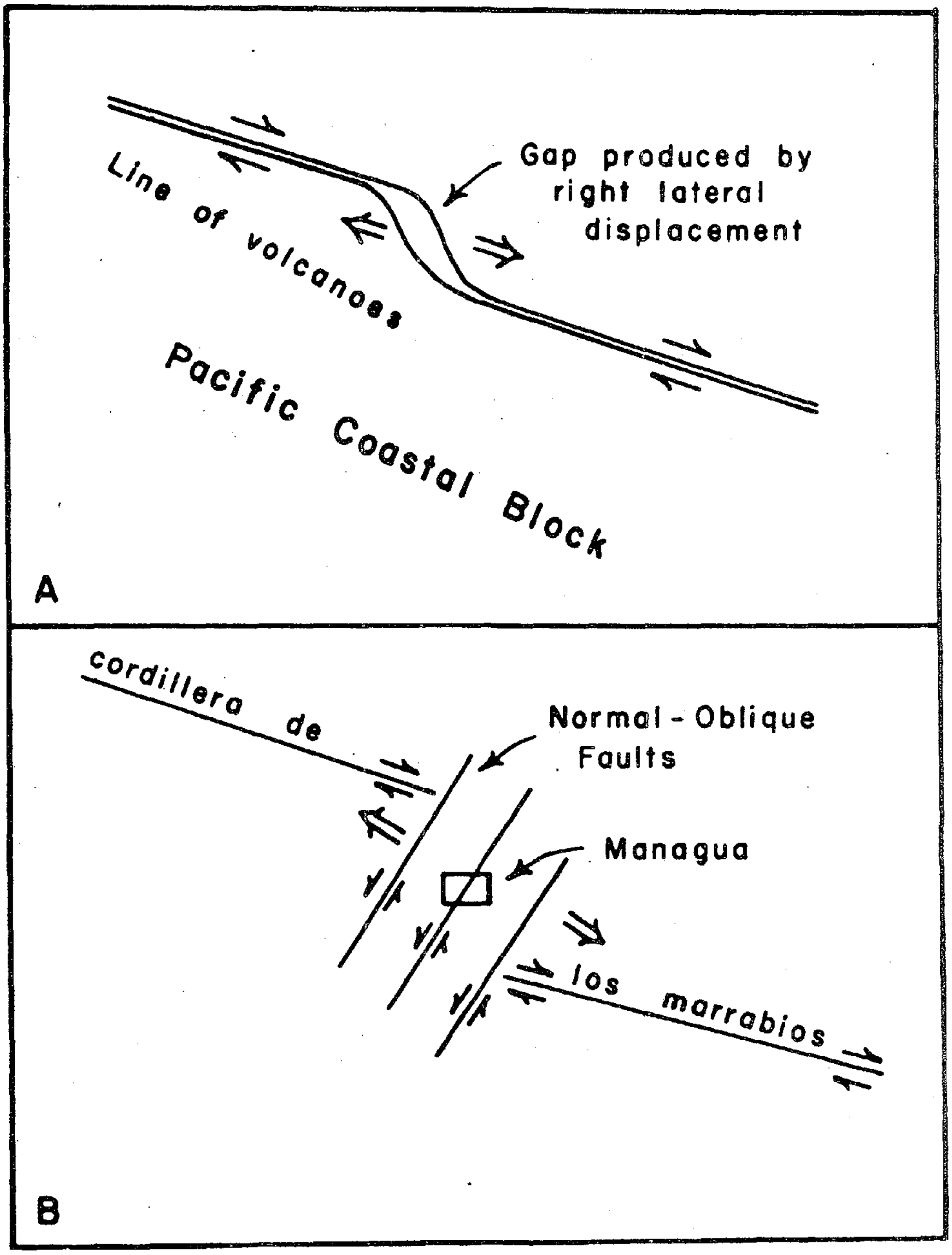

Figure A1-7 Mechanism for Diastrophism in the Managua Area. (A) Right lateral displacement of the Pacific coastal block tends to produce a gap if a fracture, kinked to the right is slipped in a right handed sense. After Brouwer.
(B) In the Managua area, a series of normal left-lateral oblique faults are members of the conjugate shear set produced by extension and by a shear resulting from a northwesterly aligned dextral couple.

After Saint-Amand, 1973

volcanics is so straight or so long.

It is not possible to conclude the cause of the tectonic pattern around Managua. The transform fault explanation seems to be the most reasonable, because it explains all of the observed phenomena. However, it presupposes an interpretation of Plate Tectonics which has not, for this region, at least, been proven. Further detailed work is needed. The explanation will aid in interpretations of the entire Nicaragua Depression.

APPENDIX 2

MODIFIED MERCALLI INTENSITY SCALE

THE MODIFIED MERCALLI INTENSITY SCALE

Mercalli's (1902) improved intensity scale served as a basis for the scale advanced by Wood and Neumann (1931), known as the modified Mercalli scale and commonly abbreviated MM. The modified version is described below with some improvements by Richter (1958). The following remarks are taken almost verbatim from *Elementary Seismology*, Charles F. Richter (W. H. Freeman and Company, San Francisco, copyright © 1958).

To eliminate many verbal repetitions in the original scale, the following convention has been adopted. Each effect is named at that level of intensity at which it first appears frequently and characteristically. Each effect may be found less strongly, or in fewer instances, at the next lower grade of intensity; more strongly or more often at the next higher grade. A few effects are named at two successive levels to indicate a more gradual increase.

Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class *A, B, C* construction).

Masonry A. Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masonry B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Masonry C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Modified Mercalli Intensity Scale of 1931 (Abridged and Rewritten by C. F. Richter.)

I. Not felt. Marginal and long-period of large earthquakes.

Taken From "Fundamentals Of Earthquake Engineering"
N.M.Newmark and E.Rosenblueth. Prentice Hall.

2. Felt by persons at rest, on upper floors, or favorably placed.

3. Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.

4. Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of 4, wooden walls and frames crack.

5. Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.

6. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, and so on, off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry *D* cracked. Small bells ring (church, school). Trees, bushes shaken visibly, or heard to rustle.

7. Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry *D* including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, unbraced parapets, and architectural ornaments. Some cracks in masonry *C*. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.

8. Steering of motor cars affected. Damage to masonry *C*; partial collapse. Some damage to masonry *B*; none to masonry *A*. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and walls. Cracks in wet ground and on steep slopes.

9. General panic. Masonry *D* destroyed; masonry *C* heavily damaged, sometimes with complete collapse; masonry *B* seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames racked. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.

10. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

11. Rails bent greatly. Underground pipelines completely out of service.

12. Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

Other commonly used intensity scales include those of Rossi–Forel (Rossi, 1883; Forel 1884), Cancani (1904), Sieberg, (1923), and Medvedev (1953). The latter is known as the Soviet scale and roughly coincides with the MM intensity scale. Also roughly equivalent is the MSK scale (Medvedev and Sponheuer, 1969). The other scales are falling slowly into disuse. The same is true of the Japanese, Chilean, and other systems of intensity grading that have enjoyed some degree of popularity at national or regional levels.

A now classical piece of work on earthquake intensity and its relation with magnitude is found in a paper by Gutenberg and Richter (1942 and 1956).

APPENDIX 3

LISTING OF EARTHQUAKES

EARTHQUAKES SORTED BY SOURCES
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(THE FIRST DIGIT OF MAGNITUDES MB,MS OR RICHTER CORRESPOND TO UNIT)

DATA SOURCE	DAY	MONTH	YEAR	HOUR	MINUTE	SECOND	LATITUDE	LONGITUDE		MB MAG			MS MAG		RICHTER MAG		RICHTER MAG
* BENIOFF 5 TO 39 KM (159)																	
NOS 45	18	06	71	13	36	01	14.678N	87.567W	7	500MB		072	0		0	022	478
CGS-B	30	01	67	09	30	27	12.196N	87.596W	8	410MB		074	0		0	010*	329
CGS 54	15	08	69	01	43	11	09.473N	83.863W	9	440MB		078	0		0	022	379
CGSPDE	14	07	68	03	55	24	15.237N	88.843W	14	450MB		072	0	F	0	033	395
CGS 15	25	02	69	02	01	44	15.274N	87.544W	13	480MB		072	0		0	G036	445
CGS 15	25	02	69	07	39	00	15.231N	87.466W	15	540MB		072	50MS	F	0	G072	500
CGS 77	16	10	68	06	08	45	13.893N	88.250W	16	430MB	4	073	0	F	0	009	362
CGS-B	15	06	64	11	13	47	12.600N	88.000W	14	410MB		076	0		0	005	329
CGSPDE	23	06	67	13	30	46	12.900N	87.800W	11	420MB		074	0		0	012	345
CGSPDE	13	11	67	16	05	46	10.400N	85.700W	18	460MB		078	0		0	019	412
CGSPDE	14	03	68	10	09	19	12.000N	86.800W	25	450MB		075	0		0	013	395
CGS 23	19	03	69	19	49	22	11.786N	88.087W	23	470MB		076	0		0	021	428
ERL 65	02	11	72	18	24	45	11.711N	87.939W	24	460MB		074	0		0	020*	412
CGS-B	20	08	64	08	26	52	11.700N	87.200W	25	430MB		074	0		0	008	362
ERL 71	11	12	72	12	51	04	11.403N	87.151W	27	440MB		074	0		0	D017*	379
CGS-B	23	06	65	07	37	46	11.400N	87.800W	24	450MB		074	0		0	028	395
CGS-B	31	05	65	20	46	54	11.100N	86.000W	28	470MB		074	0		0	030	428
CGSPDE	11	08	67	12	26	18	11.800N	85.900W	21	470MB		075	0		0	017	428
CGSPDE	03	10	67	18	16	03	10.900N	85.900W	21	580MB		078	0		0	104	611
CGS-B	19	11	66	16	01	46	09.200N	85.800W	25	430MB		077	0		0	012	362
ERL 74	25	12	72	00	26	28	15.215N	88.943W	34	460MB		072	0		0	020	412
CGSPDE	04	10	67	00	12	12	15.700N	88.600W	33	440MB		072	0		0	010	379
CGS-B	13	03	66	21	46	23	14.200N	88.400W	36	420MB		072	0		0	017	345
CGS-B	09	01	64	18	38	10	14.900N	87.900W	33	470MB		072	0		0	005	428
CGS 16	08	02	69	04	56	59	13.624N	88.271W	33	430MB	4	073	0	F	0	N006	362
CGS-B	29	10	64	12	21	52	13.200N	88.500W	33	400MB		073	0		0	008	312
CGS-B	24	04	66	15	41	03	13.700N	88.300W	33	390MB		073	0		0	009	300
CGSPDE	06	11	67	18	49	26	13.500N	88.000W	33	430MB		073	0		0	010	362
CGS-B	27	11	64	10	55	11	13.400N	88.700W	33	410MB		073	0		0	006	329
CGSPDE	03	04	68	00	06	00	12.200N	88.300W	33	430MB		076	0		0	011	362
CGS-B	23	08	66	06	07	36	12.400N	88.200W	39	400MB		076	0		0	011	312
CGS-B	15	08	64	14	43	03	12.000N	88.000W	39	400MB		076	0		0	006	312
CGS-B	16	04	66	13	21	40	12.300N	88.400W	33	440MB		076	0		0	017	379
CGS 65	01	10	69	08	41	31	12.084N	88.358W	33	440MB	3	076	0	F	0	N011	379
CGS-B	12	01	65	03	19	10	12.300N	88.900W	39	430MB		076	0		0	007	362
CGS-B	29	04	64	08	08	41	12.100N	88.400W	33	390MB		076	0		0	007	300
CGS-B	08	05	67	14	40	08	12.974N	88.073W	33	460MB		073	0	F	0	N030	412
CGSPDE	19	09	68	00	37	17	12.143N	88.714W	33	440MB		076	0		0	N014*	379
ERL 47	04	08	73	11	54	16	12.140N	88.983W	33	440MB		076	0		0	N008*	379
CGS-B	30	07	65	06	33	30	12.000N	88.500W	33	420MB		076	0		0	009	345
CGS-B	07	11	64	01	36	56	12.000N	88.000W	33	440MB		076	0		0	012	379
CGS-B	16	12	63	06	23	20	12.200N	88.400W	34	430MB		076	0		0	010	362
CGS-B	21	11	65	02	19	12	12.100N	88.900W	33	430MB		076	0		0	008	362
CGS 41	23	06	69	20	46	40	12.410N	88.052W	33	470MB		076	0		0	N033	428
CGS-B	12	04	66	17	30	49	12.600N	88.000W	33	440MB		076	0		0	019	379
CGS-B	12	04	67	04	56	25	12.217N	88.102W	33	480MB		076	0		0	N025	445
CGS-B	03	12	64	14	51	10	12.300N	88.500W	33	410MB		076	0		0	010	329
CGS-B	23	03	65	02	58	39	12.300N	88.200W	33	410MB		076	0		0	008	329

CGSPDE	01	04	68	07	28	28	12.400N	88.500W	33	440MB	076	0		0	013	379
CGS-B	06	11	63	21	49	57	12.200N	88.000W	37	410MB	076	0		0	010	329
CGS-B	24	07	66	18	50	54	12.200N	88.700W	33	410MB	076	0		0	010	329
CGS-B	15	04	67	22	10	08	12.743N	88.163W	33	430MB	076	0		0	N006*	362
CGS-B	16	08	64	12	34	34	12.000N	88.600W	33	430MB	076	0		0	011	362
CGS-B	27	09	65	17	37	53	12.900N	88.400W	33	420MB	076	0		0	007	345
CGS-B	10	12	65	17	59	11	12.400N	87.300W	33	420MB	074	0		0	011	345
CGS-B	08	09	63	05	28	37	12.300N	87.900W	31	410MB	074	0		0	008	329
ERL 30	07	05	73	11	54	36	12.344N	87.949W	33	430MB	074	40MS		0	N013	400
ERL 58	26	09	72	03	20	16	12.231N	87.412W	37	460MB	074	38MS		0	016*	380
CGS 38	13	05	69	16	39	27	12.447N	87.630W	33	420MB	074	0		0	N007*	345
CGS-B	27	06	65	13	08	28	12.600N	87.900W	33	410MB	074	0		0	007	329
CGS-B	02	04	64	03	48	59	12.500N	87.800W	32	420MB	074	0		0	009	345
ISS	20	11	52	15	37	17	12.100N	87.900W	33	0	074	0		625PAS		625
CGS-B	17	06	64	09	11	42	12.000N	87.200W	33	410MB	074	0		0	008	329
CGS-B	23	12	66	22	10	25	12.600N	87.700W	32	450MB	074	0		0	019	395
CGS 48	05	07	70	05	42	05	12.561N	87.446W	33	450MB	074	0		0	N031	395
GS 51	27	07	73	08	51	11	12.415N	87.407W	33	450MB	074	0		0	N017*	395
CGS-B	24	04	65	13	25	41	12.700N	82.000W	33	440MB	094	0		0	009	379
CGS-B	16	07	65	10	34	12	11.600N	88.000W	30	490MB	076	0		0	022	462
CGS-B	23	03	65	17	18	29	11.800N	88.200W	33	420MB	076	0		0	006	345
CGS-B	18	04	67	08	24	17	11.419N	88.415W	33	400MB	076	0		0	N007*	312
NOS 45	16	06	71	04	18	56	11.722N	88.222W	33	450MB	076	0		0	N012*	395
CGS-B	18	06	65	09	28	21	11.600N	88.200W	33	430MB	076	0		0	007	362
ERL 66	27	08	71	09	21	57	11.839N	88.884W	33	500MB	076	38MS		0	N041	380
CGSPDE	28	04	68	10	03	31	11.800N	88.800W	39	490MB	076	0		0	039	462
CGS-B	29	01	67	20	11	22	11.953N	88.928W	33	450MB	076	0		0	N018	395
CGS-B	28	05	66	14	34	39	11.900N	88.100W	33	430MB	076	0		0	013	362
CGS-B	18	07	64	20	37	35	11.000N	87.000W	33	460MB	074	0		0	010	412
CGS-B	29	04	64	21	53	15	11.800N	87.600W	33	450MB	074	0		0	007	395
CGS-B	0[illegible]	04	65	15	09	15	11.400N	87.400W	33	420MB	074	0		0	009	345
CGS-B	14	04	66	06	57	45	11.800N	87.500W	33	410MB	074	0		0	007	329
CGSPDE	04	07	67	13	29	04	11.600N	87.200W	33	420MB	074	0		0	010	345
CGS-B	22	03	65	00	17	27	11.900N	87.900W	33	430MB	074	0		0	007	362
CGS-B	16	07	65	11	21	10	11.600N	87.800W	33	480MB	074	0		0	016	445
CGS-B	04	10	64	06	30	14	11.300N	87.400W	33	420MB	074	0		0	007	345
CGSPDE	04	07	67	07	39	20	11.800N	87.300W	38	400MB	074	0		0	016	312
NOS 80	25	11	70	05	38	12	11.745N	87.920W	36	480MB	074	0		0	014*	445
NOS 9	11	01	71	17	13	48	11.714N	87.794W	33	480MB	074	0	F	0	N014*	445
CGS 3	06	01	69	09	44	41	11.410N	87.232W	33	430MB	074	0		0	011	362
CGS 3	06	01	69	09	24	23	11.440N	87.245W	33	480MB	074	43MS		0	N026	430
CGS-B	06	06	65	08	34	17	11.100N	87.200W	33	460MB	074	0		0	005	412
CGS-B	30	07	63	12	25	39	11.400N	87.300W	33	420MB	074	0		0	005	345
CGS-B	29	08	64	20	51	54	11.400N	87.200W	33	420MB	074	0		0	005	345
CGS-B	20	08	64	09	12	53	11.000N	87.500W	33	420MB	074	0		0	008	345
CGS-B	13	04	63	18	53	18	11.700N	87.800W	33	430MB	074	0		0	016	362
CGS-B	20	05	67	23	42	19	11.653N	86.714W	33	420MB	074	0		0	N012*	345
CGS-B	08	06	65	01	25	59	11.000N	86.800W	33	470MB	074	0		0	007	428
CGS 92	16	11	68	15	14	37	11.082N	86.911W	33	460MB	074	39MS		0	N018	390
CGS 8	15	01	70	16	57	47	11.614N	86.929W	33	480MB	074	0		0	N008*	445
CGSPDE	02	10	67	15	59	43	11.700N	86.800W	39	470MB	074	0		0	025	428
CGS-B	12	09	64	19	05	47	11.200N	86.900W	33	470MB	074	0		0	014	428
CGS 89	21	11	68	07	19	34	11.203N	86.891W	33	450MB	074	0		0	N016	395
CGS-B	23	03	65	21	21	52	11.000N	86.600W	33	480MB	074	0		0	014	445
CGS-B	30	04	64	15	05	57	11.200N	86.700W	33	430MB	074	0		0	005	362
CGS 94	19	11	68	00	09	27	11.336N	86.714W	33	460MB	074	41MS		0	N022	410
CGS 26	20	03	70	21	00	08	10.428N	87.364W	34	440MB	077	43MS		0	013*	430
CGS 54	20	07	70	04	58	05	10.366N	87.400W	33	460MB	077	0		0	N015*	412
CGS 55	12	08	70	11	07	59	10.867N	87.229W	33	450MB	077	0		0	N009*	395
CGS 37	25	05	70	23	05	18	10.738N	86.939W	33	480MB	077	0		0	N014	445

CGS 62	02	09	70	17	51	35	10.432N	86.015W	33	430MB		077	0		0	N009*	362
CGS 39	13	05	69	14	36	16	10.817N	86.338W	33	450MB		077	0		0	N008*	395
ERL 10	06	02	72	10	05	31	10.592N	86.400W	36	440MB		077	0		0	008*	379
CGSPDE	25	06	67	22	52	12	10.700N	86.000W	33	400MB		077	0		0	007	312
CGSPDE	04	07	67	06	12	57	10.800N	86.400W	33	410MB		077	0		0	016	329
CGS-B	28	07	64	01	54	18	10.800N	86.200W	33	420MB		077	0		0	006	345
CGS-B	07	07	64	09	59	19	10.800N	86.600W	33	490MB		077	0		0	010	462
CGS-B	17	07	65	04	05	16	10.100N	86.500W	33	430MB		077	0		0	006	362
CGSPDE	04	10	67	06	02	16	10.700N	86.000W	33	530MB		077	0		0	066	528
ERL 81	14	10	71	06	59	10	10.824N	86.268W	33	470MB		077	40MS		0	N020*	400
CGS-B	13	01	65	02	19	49	10.100N	86.300W	33	480MB		077	0		0	009	445
CGS-B	30	07	64	15	13	39	10.600N	86.500W	33	470MB		077	0		0	007	428
CGS-B	28	09	63	05	25	03	10.200N	86.000W	33	410MB		077	0		0	008	329
CGSPDE	03	10	67	19	22	39	10.400N	86.200W	33	450MB		077	0		0	019	395
CGS-B	14	12	66	01	33	13	10.600N	86.800W	33	430MB		077	0		0	008	362
CGS-B	02	08	64	04	00	45	10.900N	86.300W	32	470MB		077	0		0	012	428
ERL 19	19	02	72	19	41	33	10.674N	86.085W	33	470MB		077	0		0	N014*	428
CGS-B	07	06	65	07	41	47	10.400N	86.000W	33	470MB		077	0		0	007	428
CGS 45	11	07	69	07	51	04	10.342N	85.369W	33	500MB		078	0	F	450BRK	N049	450
CGS-B	16	01	67	15	30	47	10.641N	85.920W	33	470MB		078	0		0	N019*	428
CGS 46	11	07	69	05	57	12	10.189N	85.484W	30	470MB		078	0	F	0	025	428
ERL 36	14	04	73	09	19	09	10.260N	85.015W	33	430MB		078	0		0	N006*	362
CGS-B	15	05	64	12	10	25	10.500N	85.700W	33	450MB		078	0		0	024	395
ERL 71	13	09	71	15	33	27	09.973N	87.361W	33	440MB		077	0		0	N007	379
ERL 77	28	09	71	05	47	16	09.821N	86.267W	33	500MB		077	0		0	N036*	478
CGS-B	15	02	65	02	29	48	09.900N	86.500W	33	550MB		077	0		0	011	561
CGS-B	31	08	64	17	07	39	09.600N	85.400W	33	420MB		077	0		0	006	345
CGS-B	23	02	66	01	02	01	09.700N	85.800W	35	400MB		077	0		0	008	312
CGS-B	15	05	64	05	46	32	09.800N	85.500W	33	420MB		077	0		0	010	345
CGS-B	09	09	65	12	43	37	09.600N	85.300W	33	450MB		077	0		0	008	395
CGS	20	02	54	02	00	43	11.500N	87.500W	N	0		074	0	F	587PAS	*	587
CGS	19	02	54	00	40	25	11.530N	87.500W	N	0		074	0	F	663PAS	*	663
GUTE	22	12	34	14	29	31	11.500N	87.000W	N	0		074	0		650PAS	*	650
GUTE	12	01	33	01	17	42	11.000N	87.000W	N	0		074	0		560PAS	*	560
D&M 238	13	8	58				11.70	87.20	N	D						***	538
D&M 239	3	9	58				11.70	87.20	N	E						***	351
D&M 302	28	9	62				11.60	87.20	N	E						***	457
D&M 335	16	9	63				11.80	87.40	N	E						***	518
GUTE	08	10	41	04	22	13	10.500N	86.250W	0	0		077	0		575PAS	*	575
ISS	11	11	50	13	51	10	10.400N	85.700W	0	0		078	0		575PAS	*	575
D&M 245	21	2	59				11.70	87.20		D						*	582
D&M 248	2	4	59				11.70	87.20		D						*	571
D&M 184	28	12	54				11.50	87.50		E						*	515
D&M 186	11	1	55				11.00	86.50		E						*	498
D&M 203	16	8	55				11.00	87.00		E						*	464
D&M 259	11	9	59				12.00	87.50		E						*	513
D&M 307	15	1	63				12.00	87.50	20	D						***	582
D&M 317	30	6	63				11.40	87.30	33	4.2						***	4.2
D&M 218	19	5	57				12.14	87.24	24	E						*	413
D&M 306	15	1	63				12.00	87.50	33	D						*	589
D&M 298	17	7	62				11.60	87.10	25	E						*	470

* BENIOFF AND COSTA RICA 40 TO 79 KM (186)

CGS 11	31	01	70	22	23	37	15.727N	88.750W	46	500MB		072	45MS		0	060	450
CGS-B	10	11	63	07	18	07	12.900N	88.500W	45	420MB		076	0		0	007	345
CGS-B	24	05	63	06	10	09	12.100N	88.500W	42	440MB		076	0		0	012	379
CGS-B	08	07	66	13	22	13	12.400N	88.900W	43	430MB		076	0		0	013	362
CGS-B	24	01	66	03	46	57	12.300N	88.400W	46	450MB		076	0		0	018	395
CGS 40	06	06	69	16	16	01	12.467N	88.000W	48	500MB	3	076	45MS	F	0	051	450
CGS 90	28	11	68	13	44	33	12.451N	88.006W	49	490MB		076	0		0	021	462

ERL 85	26	11	71	04	06	38	12.148N	87.626W	47	440MB		074	0		0	013	379
CGS-B	30	01	62	08	34	23	12.700N	87.500W	49	0		074	0	F	488PAL	049	488
CGS-B	04	04	64	06	43	20	12.500N	87.700W	41	430MB		074	0		0	012	362
CGS-B	06	12	65	13	19	13	12.400N	87.400W	44	390MB		074	0		0	007	300
CGS 12	12	02	70	17	22	11	12.139N	86.616W	44	440MB		075	0		0	013	379
ERL 16	05	03	72	21	47	26	11.971N	87.914W	46	460MB	3	074	0	F	0	020*	412
CGS-B	31	08	63	13	08	46	11.900N	87.000W	48	490MB		074	0		0	018	462
CGS-B	08	12	64	04	11	53	11.500N	87.000W	48	500MB		074	0		0	029	478
GS 52	14	07	73	07	55	08	11.637N	87.291W	46	490MB		074	44MS		0	024*	440
ERL 35	28	05	73	06	19	00	11.474N	87.266W	42	460MB		074	40MS		0	014*	400
ERL 42	05	07	73	08	08	07	11.949N	87.541W	48	430MB		074	0		0	015	362
CGS 7	20	01	70	08	27	48	11.482N	86.353W	46	540MB		074	0		0	063	545
CGS-B	12	10	66	20	20	06	11.200N	86.200W	45	510MB		074	0		0	054	495
ERL 9	30	01	72	22	25	03	11.082N	86.750W	43	450MB		074	38MS		0	017	380
CGSPDE	30	06	67	11	42	33	11.000N	86.600W	44	450MB		074	0		0	022	395
CGS-B	30	07	64	05	16	03	11.100N	86.200W	42	570MB		074	0		0	022	594
ERL 7	30	01	72	19	10	07	11.261N	86.773W	48	460MB		074	44MS		0	027	440
CGS 36	13	05	69	17	38	28	11.784N	86.179W	46	490MB		074	52MS		400BRK	042	520
ERL 61	01	10	72	06	00	51	11.015N	86.330W	49	420MB		074	0		0	006*	345
SYKES	27	10	57	18	29	10	15.970N	88.110W	56	0		093	0		400PAL	008	400
CGS 55	03	08	69	21	17	16	13.342N	88.342W	52	430MB		073	0		0	010*	362
CGS 59	04	09	69	06	25	28	13.216N	88.429W	53	460MB	4	073	0	F	0	020	412
ERL 51	31	08	72	02	24	09	13.272N	88.513W	50	540MB		073	48MS		0	068	480
CGSPDE	06	05	68	21	23	11	13.021N	87.734W	56	460MB		072	0		0	D024*	412
CGS-B	25	02	63	08	58	43	12.300N	88.200W	58	410MB		076	0		0	029	329
CGS-B	05	03	64	14	30	18	12.100N	88.100W	53	430MB		076	0		0	009	362
CGS-B	15	06	64	09	41	24	12.600N	88.300W	56	410MB		076	0		0	007	329
CGS-B	19	06	64	17	31	56	12.400N	88.100W	53	400MB		076	0		0	011	312
CGSPDE	18	04	68	04	40	38	12.500N	88.600W	54	490MB		076	0		0	019	462
CGS-B	30	08	64	15	20	50	12.600N	88.500W	54	460MB		076	0		0	015	412
CGS-B	04	07	63	01	43	20	12.800N	88.700W	55	410MB		076	0		0	007	329
CGS-B	13	07	66	08	20	58	12.600N	87.800W	56	530MB		074	0		0	073	528
CGS-B	09	05	63	15	03	43	12.400N	87.000W	50	550MB		074	0		0	047	561
CGS-B	17	07	65	13	59	46	12.200N	87.900W	56	440MB		074	0		0	013	379
CGS-B	27	03	67	14	40	13	12.221N	87.712W	54	420MB		074	0		0	019	345
CGS-B	06	12	65	04	39	10	12.500N	87.300W	50	450MB		074	0		0	024	395
CGS 54	07	08	70	13	03	14	12.325N	87.791W	59	450MB		074	0		0	016	395
NCS 21	25	02	71	04	15	41	12.159N	87.489W	52	530MB		074	0		0	075	528
CGS-B	10	06	64	16	25	09	12.000N	87.900W	55	420MB		074	0		0	006	345
ERL 41	07	07	73	07	07	55	11.842N	87.761W	55	430MB		074	0		0	015	362
CGS 32	24	04	69	22	29	46	11.539N	87.022W	50	440MB		074	0		0	022	379
ERL 40	06	07	73	05	21	49	11.962N	87.588W	51	480MB		074	0		0	023	445
CGS-B	07	10	63	03	59	54	11.600N	86.900W	50	450MB		074	0		0	015	395
ERL 9	04	02	72	01	07	43	11.213N	86.891W	52	480MB		074	0		0	019	445
CGS 23	18	03	70	23	17	57	10.984N	86.015W	56	490MB		074	0		0	037	462
CGS-B	01	03	65	15	17	45	10.800N	86.700W	55	490MB		077	0		0	009	462
CGS-B	18	04	67	07	05	08	10.749N	86.777W	53	480MB		077	0		0	041	445
CGS-B	03	02	64	01	46	27	13.000N	88.000W	61	410MB		073	0		0	006	329
CGS	27	06	58	05	44	28	13.000N	88.500W	60	0		073	0		DF600PAS		600
CGS 80	22	10	68	06	42	01	13.208N	88.247W	64	470MB	5	073	0		0	017	428
CGS-B	20	12	63	22	28	31	13.200N	88.000W	64	430MB		073	0		0	005	362
CGS-B	21	07	64	07	01	59	13.100N	88.400W	68	470MB		073	0		0	030	428
CGS-B	16	08	64	07	30	12	13.300N	87.600W	60	390MB		072	0		0	005	300
CGS-B	02	03	64	16	09	46	12.500N	88.000W	63	440MB		076	0		0	018	379
CGS-B	19	05	61	09	25	41	12.800N	88.200W	68	0	5	076	0	F	463PAL		463
ERL 49	19	08	73	22	17	49	12.651N	88.507W	68	510MB	4	076	0	F	0	D033	495
ERL 14	04	03	72	22	04	23	12.933N	88.743W	64	470MB	4	076	0	F	0	024	428
ERL 15	07	03	73	13	00	45	12.705N	88.027W	64	460MB		076	0		0	013*	412
CGS-B	24	07	66	06	11	55	12.200N	88.300W	61	410MB		076	0		0	013	329
ERL 81	08	11	71	04	26	16	12.472N	88.165W	68	490MB		076	0		0	031	462

CGSPDE	26	01	68	10	37	56	12.730N	88.800W	60	390MB		076	0		0	007	300
CGS-B	31	07	66	19	10	46	12.400N	88.100W	63	420MB		076	0		0	032	345
CGS-B	04	05	66	18	13	55	12.500N	87.700W	68	510MB		074	0		0	036	495
CGS-B	08	08	64	15	45	10	12.500N	87.800W	63	580MB		074	0		0	032	611
CGS 20	10	03	69	08	15	08	12.349N	87.455W	62	530MB		074	0		600BRK	059	600
CGS-B	21	02	64	07	24	08	12.800N	87.900W	62	390MB		074	0		0	007	300
ERL 85	25	11	71	22	06	26	12.759N	87.458W	62	490MB		074	0		0	024	462
CGS-B	18	04	67	08	29	12	11.751N	87.307W	65	460MB		074	0		0	028	412
ERL 8	16	01	72	23	55	10	11.505N	87.582W	62	460MB		074	0		0	014	412
GUTE	06	01	41	09	48	34	11.750N	86.500W	60	0		074	0		600PAS		600
CGS-B	20	03	61	06	16	21	11.300N	86.500W	60	0		074	0	F	613PAS		613
CGS 38	14	05	69	13	49	03	11.518N	86.154W	62	480MB		074	0		0	046	445
NOS 44	10	06	71	11	20	31	11.101N	86.766W	60	490MB		074	0		0	015*	462
NOS 30	26	04	71	18	32	20	11.422N	86.760W	69	470MB		074	0		0	040	428
CGS-B	02	11	65	22	44	41	10.500N	86.300W	69	450MB		077	0		0	010	395
CGS-B	08	03	67	23	07	02	15.101N	88.782W	76	420MB		072	0		0	006*	345
CGS-B	08	03	67	15	52	15	13.049N	88.127W	75	390MB		073	0		0	005*	300
CGS 19	09	03	69	07	12	39	13.101N	88.528W	75	440MB		073	0		0	021	379
GUTE	26	12	39	11	55	11	13.250N	88.250W	75	0		073	0		600PAS	AAA	600
CGS-B	09	01	67	22	31	22	13.138N	88.981W	78	410MB		073	0		0	008	329
CGS-B	04	01	67	00	28	07	13.100N	88.625W	72	410MB		073	0		0	010	329
CGS-B	27	12	66	21	22	16	13.300N	88.800W	78	540MB		073	0		0	090	545
ERL 66	19	11	72	04	35	24	12.973N	88.524W	73	560MB		073	0		0	065	578
CGS-B	10	08	64	14	00	51	12.400N	88.300W	71	400MB		076	0		0	006	312
CGS 33	03	05	69	03	19	06	12.987N	88.449W	77	450MB	5	073	0	F	0	032	395
CGS 25	01	04	69	21	19	53	12.677N	88.233W	71	490MB	4	076	0	F	0	038	462
CGS 26	08	04	69	19	41	58	12.670N	88.340W	76	480MB		076	0		0	023	445
ERL 32	30	04	72	10	55	19	12.896N	88.668W	77	440MB	3	076	0	F	0	019	379
CGS 62	06	09	70	20	44	49	12.783N	88.595W	75	450MB		076	0	F	0	017	395
CGS 34	21	04	70	05	37	08	12.947N	87.880W	79	520MB		074	0		0	045	511
ERL 47	28	07	73	20	48	24	12.333N	87.084W	79	540MB		074	0	F	0	D087	545
CGS-B	22	03	67	09	15	29	12.653N	87.823W	70	440MB		074	0		0	025	379
CGS 71	23	09	70	23	57	45	12.021N	87.446W	70	450MB		074	0		0	029	395
CGS-B	20	10	65	23	54	30	12.500N	87.300W	72	540MB		074	0		0	063	545
CGS-B	25	03	67	07	26	37	12.406N	87.876W	76	400MB		074	0		0	005*	312
CGS-B	25	08	64	05	03	18	12.100N	87.700W	72	400MB		074	0		0	005	312
CGSPDE	28	01	68	08	47	20	12.630N	87.900W	75	440MB		074	0		0	022	379
CGS-B	31	12	63	14	22	07	12.400N	87.900W	77	430MB		074	0		0	007	362
CGSPDE	24	03	68	17	13	20	12.500N	86.500W	79	510MB		075	0		0	043	495
CGS 38	13	05	69	14	15	52	11.438N	86.369W	71	480MB		074	0		0	037	445
CGS 38	13	05	69	14	16	52	11.469N	86.355W	79	560MB		074	0	F	570BRK	104	570
CGS 38	13	05	69	18	11	39	11.523N	86.509W	72	460MB		074	0		0	037	412
CGS 4	15	01	69	06	57	22	11.308N	86.813W	72	450MB		074	0		0	010	395
CGS 8	15	01	70	16	52	42	11.512N	86.733W	70	510MB	3	074	0	F	575BRK	046	575
CGS	19	02	54	21	34	41	12.500N	87.500W	N	0		074	0	F	663PAS	*	663
GUTE	28	03	21	07	49	22	12.500N	87.500W	N	0		074	0		730PAS	*	730
CGS	25	10	56	05	21	40	12.000N	87.000W	N	0		074	0	F	637PAS	*	637
ISS	24	10	56	14	42	12	11.790N	86.460W	N	0		074	0		DF730PAS	*	730
ISS	24	04	59	09	31	33	11.480N	86.400W	N	0		074	0		638PAS	*	638
D&M 113	1	5	16				11.00	85.50	N	D						***	547
D&M 114	23	9	16				11.00	85.50	N	D						***	560
D&M 255	12	7	59				11.50	86.00	N	D						***	547
D&M 313	26	2	63				12.40	87.40	N	E						***	505
D&M 326	13	10	63				12.50	87.10	N	D						***	535
D&M 327	23	11	63				12.00	87.20	N	C						***	605
D&M 331	11	3	64				12.50	87.10	N	D						***	540
D&M 332	29	4	64				12.30	87.40	N	E						***	515
D&M 457	13	4	73				10.50	85.50	N	6.7						***	6.7
D&M 111	27	02	16				11.00	85.50	N	C						*	650
D&M 156	30	4	55				12.00	87.00	N	6.0						*	6.0

D&M 2C1	7	7	55				12.50	88.00	N	E					*	515
D&M 206	13	10	55				12.00	87.00		E					*	358
D&M 258	15	8	59				12.20	87.30	N	D					*	557
D&M 144	20	2	40				12.50	87.50		C					*	681
D&M 139	25	4	38				12.20	86.90		C					*	676
D&M 141	9	2	39				11.10	86.20		E					*	446
D&M 147	5	8	44				12.50	87.50		E					*	425
D&M 157	2	4	51				12.00	87.00		E					*	419
D&M 158	6	5	51				11.00	85.50		E					*	353
D&M 173	12	3	52				11.00	86.00		E					*	389
D&M 179	21	2	54				11.90	87.00		E					*	480
D&M 182	27	11	54				12.00	87.00		E					*	373
D&M 2C4	29	8	55				12.00	87.00		E					*	458
D&M 208	21	4	56				12.00	87.33		D					*	581
D&M 247	31	3	59				13.00	87.00		E					*	482
D&M 249	22	4	59				11.50	86.50		D					*	591
D&M 252	30	5	59				11.00	86.00		E					*	448
D&M 262	21	1	60				12.00	87.25		E					*	420
D&M 264	15	2	60				12.00	87.00		E					*	390
D&M 322	9	8	63				12.90	87.40	80	E					***	383
D&M 369	24	5	68				12.50	86.50	79	5.1					***	5.1
D&M 154	13	11	49				11.00	85.50	60	E					*	384
D&M 260	19	11	59				12.00	87.00	60	E					*	397
D&M 263	26	1	60				13.00	87.50	60	D					*	580
D&M 267	10	7	60				12.50	86.00	150	6.0					*	6.0
D&M 289	22	12	61				11.90	85.50	49	E					*	393
D&M 311	11	2	63				11.60	87.50	41	E					*	496
D&M 314	25	4	63				12.20	87.30	75	E					*	415
D&M 180	21	2	54				12.50	87.00	60	E					*	420
D&M 241	14	11	58				12.36	86.37	72	C					*	670
CGS-B	01	10	63	10	30	31	10.200N	84.500W	46	460MB	078	0		0	022	412
CGS-B	30	12	66	18	35	15	10.100N	84.700W	43	440MB	078	0		0	015	379
CGS-B	02	05	64	19	53	10	09.500N	84.500W	44	430MB	078	0		0	007	362
CGS-B	09	04	66	02	42	11	09.500N	84.100W	49	570MB	078	0		0	079	594
CGS-B	20	06	64	05	02	37	09.200N	84.400W	45	430MB	078	0		0	007	362
CGS-B	C9	04	66	02	34	23	09.400N	84.200W	47	530MB	078	0		0	071	528
CGS-B	16	10	65	14	22	55	09.000N	83.500W	48	510MB	078	0		0	053	495
CGS-B	28	03	66	00	33	30	09.500N	83.100W	46	400MB	078	0		0	008	312
CGS-B	17	07	66	00	14	03	09.200N	82.300W	44	400MB	080	0		0	013	312
CGS 31	16	04	70	05	04	41	09.602N	84.240W	50	430MB	078	0		0	G014*	362
CGS 53	05	08	70	13	22	05	09.841N	84.317W	56	460MB	078	0		0	023	412
CGS-B	15	10	64	21	09	02	09.600N	84.000W	51	420MB	078	0		0	007	345
ISS	13	05	52	19	31	45	10.300N	85.300W	64	0	078	0		690PAS		690
CGS-B	13	10	63	05	10	12	09.800N	84.100W	63	440MB	078	0		0	012	379
CGS-B	27	03	66	23	12	49	09.800N	83.300W	61	420MB	078	0		0	018	345
CGS-B	02	07	63	18	05	33	09.000N	83.000W	64	420MB	078	0		0	011	345
CGS-B	13	05	67	12	48	00	09.247N	82.395W	68	400MB	080	0		0	008*	312
CGS-B	21	06	63	03	32	41	10.700N	85.800W	73	500MB	078	0		0	022	478
GUTE	18	06	39	16	46	05	10.000N	83.000W	70	0	078	0		650PAS	BCB	650
CGS-B	01	05	66	22	28	45	09.600N	83.900W	71	430MB	078	0		0	018	362
GUTE	26	04	16	02	21	30	10.000N	85.000W	0	0	078	0		730PAS	*	730
GUTE	21	12	39	20	54	48	10.000N	85.000W	0	0	078	0		730PAS	*	730
GUTE	06	12	41	01	25	01	10.500N	85.250W	0	0	078	0		600PAS	*	600
GUTE	22	12	39	04	44	00	10.000N	84.500W	0	0	078	0		675PAS	*	675
ISS	09	09	52	12	54	42	09.200N	84.200W	0	0	078	0	F	688PAS	*	688
GUTE	05	10	40	14	38	43	09.500N	84.250W	0	0	078	0		625PAS	*	625
GUTE	27	10	40	05	35	37	09.750N	84.500W	0	0	078	0		675PAS	*	675
CGS	19	07	56	23	26	25	09.500N	84.500W	0	0	078	0	F	600BRK	*	600
GUTE	04	03	24	10	07	42	09.750N	84.000W	0	0	078	0		700PAS	*	700

* BENIOFF 80 TO 109 KM (72)

GUTE	16	11	41	09	39	46	13.250N	88.500W	80	0		073	0		575PAS	AAA	575
CGS-B	20	09	66	03	53	34	13.100N	88.200W	81	400MB		073	0		0	008	312
CGS-B	13	11	64	08	02	38	13.000N	88.600W	86	490MB		073	0		0	024	462
ERL 11	17	02	72	09	19	06	13.269N	88.691W	80	470MB	5	073	0	F	0	031	428
CGS-B	24	02	64	02	29	12	13.000N	87.200W	83	390MB		072	0		0	006	300
CGS-B	05	07	63	14	40	54	12.800N	88.900W	85	410MB		076	0		0	011	329
CGS-B	23	12	66	02	02	19	12.900N	88.600W	88	450MB		076	0		0	023	395
CGSPDE	06	12	67	02	53	06	12.530N	87.200W	87	530MB		074	0		0	057	528
CGS 48	26	07	69	07	21	00	12.625N	87.798W	84	480MB		074	0		0	039	445
CGS-B	18	04	67	13	05	15	12.765N	87.943W	84	390MB		074	0		0	007*	300
CGS-B	01	08	64	08	55	49	11.800N	87.000W	88	430MB		074	0		0	008	362
CGS-B	23	04	67	14	44	10	11.525N	86.480W	82	440MB		074	0		0	014	379
GUTE	07	03	31	00	41	56	11.500N	85.500W	80	0		075	0		600PAS	ACB	600
CGS-B	10	12	66	01	09	39	10.300N	85.100W	87	430MB		078	0		0	011	362
CGS-B	19	07	65	22	14	21	10.600N	85.300W	80	460MB		078	0		0	019	412
NOS 31	01	05	71	14	32	12	13.259N	88.499W	93	540MB	5	073	0	F	0	053	545
CGS-B	16	03	64	06	06	51	13.500N	88.300W	92	390MB		073	0		0	008	300
ISS	06	05	51	23	03	32	13.000N	87.800W	96	0		072	0		625TAC		625
GUTE	08	07	39	21	31	44	12.500N	88.000W	90	0		076	0		550PAS	CCC	550
CGS-B	09	02	66	06	22	44	12.700N	87.800W	98	440MB		074	0		0	021	379
CGS-B	20	10	65	23	54	51	12.700N	87.000W	91	560MB		074	0		0	032	578
GUTE	21	05	32	10	10	07	12.000N	87.500W	90	0		074	0		690PAS	AAA	690
CGS-B	23	05	64	06	45	21	11.700N	86.600W	93	460MB		074	0		0	007	412
ISS	26	02	52	15	39	28	11.500N	86.300W	96	0		074	0	F	600PAS		600
ISS	27	10	56	15	33	02	11.880N	86.460W	90	0		074	0		587PAS		587
CGS-B	04	07	64	02	36	54	11.300N	86.500W	90	410MB		074	0		0	007	329
TAC	13	01	54	00	26	30	14.000N	86.416W	100	0		072	0		575TAC		575
CGS 31	04	04	69	08	43	06	13.534N	88.981W	105	430MB		073	0		0	008*	362
GUTE	07	02	31	03	30	35	13.000N	87.000W	100	0		072	0		575PAS	CCC	575
CGS-B	06	03	64	02	55	11	12.400N	87.600W	100	390MB		074	0		0	008	300
CGS-B	12	04	66	03	34	22	12.600N	87.600W	104	390MB		074	0		0	009	300
CGS-B	03	09	64	01	55	32	12.200N	87.000W	108	400MB		074	0		0	005	312
CGS	03	05	59	04	41	24	12.500N	87.500W	100	0	5	074	0	F	560TAC		560
ERL 40	27	06	72	14	45	32	12.182N	86.588W	104	460MB		075	0		0	014*	412
CGS-B	01	07	64	00	42	55	12.000N	86.700W	108	430MB		075	0		0	008	362
NOS 75	03	11	70	08	39	12	11.644N	86.112W	100	460MB		074	0		0	D023	412
ERL 73	14	12	72	03	55	41	11.858N	86.574W	104	460MB		074	0		0	D019*	412
CGS 42	24	06	69	00	35	05	11.661N	85.717W	100	530MB		075	0		500PAL	G085	500
CGS	10	11	56	00	08	27	10.500N	86.000W	100	0		077	0		600PAS		600
CGS	04	04	55	19	24	04	13.000N	87.000W	N	0		072	0		625PAS	*	625
D&M 310	16	1	63				13.00	86.70	N	E						***	367
D&M 328	21	12	63				13.40	87.20	N	D						***	570
D&M 207	24	1	56				12.20	86.70	I	7.3						*	7.3
D&M 137	12	1	37				12.58	87.16		D						*	590
D&M 243	27	12	58				12.80	87.30		D						*	593
D&M 135	1	9	35				12.58	87.16		E						*	465
D&M 138	25	9	37				11.00	85.00		E						*	444
D&M 152	25	1	49				11.00	86.00		E						*	429
D&M 190	10	3	55				12.50	87.00		E						*	486
D&M 195	30	4	55				12.50	87.00		D						*	597
D&M 216	28	1	57				12.00	86.50		E						*	486
D&M 235	4	6	58				12.00	86.50		E						*	352
D&M 148	5	10	45				13.00	87.00	100	E						*	365
D&M 153	11	6	49				12.50	87.00	100	E						*	397
D&M 159	11	5	51				13.00	87.50	100	E						*	467
D&M 169	17	12	51				11.50	86.30	96	E						*	375
D&M 171	2	3	52				11.50	86.30	100	D						*	572
D&M 183	2	12	54				12.00	86.50	100	E						*	497
D&M 226	26	11	57				11.50	86.50	100	E						*	512

D&M 242	4	12	58				11.50	86.50	100	6.0					*	6.0
D&M 290	8	1	62				12.20	85.40	103	E					*	380
D&M 292	17	1	62				11.90	86.30	97	E					*	389
D&M 303	27	10	62				11.60	86.40	106	E					*	360
ISS	16	02	59	17	54	15	13.110N	87.340W	0	0		072	0	540TAC	*	540
ISS	06	05	51	23	08	01	13.000N	87.800W	0	0		072	0	CD650TAC	*	650
ISS	07	05	51	20	22	21	13.000N	87.800W	0	0		072	0	600TAC	*	600
GUTE	07	03	34	22	41	47	13.250N	87.750W	0	0		072	0	625PAS	*	625
GUTE	29	06	19	23	14	23	13.500N	86.500W	90	0		075	0	675PAS	BCC	675
D&M 130	21	5	33				12.00	87.50	90	6.9	OFL				***	6.9
D&M 334	11	5	63				11.40	87.50	100	D	OFL				***	542
D&M 413	24	9	69				11.66	87.71	100	5.3	OFL				***	5.3

* BENIOFF 110 TO 159 KM (31)

CGS-B	28	08	63	05	55	51	12.600N	87.200W	115	420MB		074	0	0	006	345
CGS-B	17	08	64	19	05	44	12.400N	86.800W	116	450MB		075	0	0	008	395
CGS-B	30	04	67	03	10	49	12.632N	86.852W	115	420MB		075	0	0	009	345
CGS-B	02	11	64	07	07	57	11.800N	86.900W	117	460MB		074	0	0	009	412
CGS-B	28	07	64	02	53	26	11.500N	86.400W	111	420MB		074	0	0	006	345
CGS-B	12	04	61	22	20	33	13.200N	88.900W	122	0	6	073	0	D 588PAS		588
CGS-B	07	08	66	04	11	30	11.000N	86.200W	125	410MB		074	0	0	009	329
CGS-B	01	09	63	22	57	34	11.300N	85.500W	120	440MB		075	0	0	010	379
CGSPDE	07	07	68	19	49	36	14.678N	88.939W	134	450MB		072	0	0	017*	395
CGS-B	23	05	61	16	44	59	12.700N	87.300W	138	0	6	074	0	DF650PAS		650
CGS-B	21	01	65	20	43	55	12.300N	86.700W	138	440MB		075	0	0	016	379
GUTE	05	10	25	04	09	07	12.250N	85.250W	135	0		075	0	675PAS	BBA	675
GUTE	05	11	26	07	55	38	12.300N	85.800W	135	0		075	0	700PAS	BBA	700
CGS-B	07	08	64	08	21	03	11.800N	86.900W	130	470MB		074	0	0	010	428
CGS-B	12	08	64	12	47	46	11.400N	86.100W	135	410MB		074	0	0	007	329
CGSPDE	14	07	67	18	02	08	13.500N	88.800W	147	460MB		073	0	0	014	412
CGS-B	17	09	63	03	48	09	12.600N	87.100W	144	390MB		074	0	0	007	300
NOS 41	11	06	71	11	51	40	12.647N	86.967W	148	450MB		074	0	0	024*	395
CGS-B	24	04	64	14	40	28	13.300N	88.800W	158	510MB		073	0	0	044	495
CGSPDE	20	08	68	00	34	04	13.105N	87.833W	157	430MB		072	0	0	014	362
CGS-B	16	12	65	05	23	06	12.600N	88.800W	152	440MB		076	0	0	019	379
CGS-B	06	03	67	23	31	25	12.389N	87.127W	159	410MB		074	0	0	008*	329
CGS	03	05	54	17	13	32	12.000N	86.000W	150	0		075	0	600PAS		600
CGSPDE	28	01	68	18	20	49	12.100N	86.900W	152	440MB		075	0	0	015	379
GUTE	22	07	19	22	01	35	12.000N	85.000W	150	0		075	0	650PAS	CCC	650
D&M 209	9	8	56				12.80	86.00	N	D					***	571
D&M 300	16	8	62				12.60	85.60	N	D					***	562
D&M 246	15	3	59				12.00	85.00		E					*	386
D&M 393	28	12	68				12.10	86.90	152	4.4					***	4.4
D&M 231	23	12	57				12.50	86.50	150	E					*	368
D&M 265	3	9	61				12.20	86.90	124	E					*	381

* BENIOFF 160 TO 215 KM (41)

CGS 74	15	09	68	04	32	04	13.456N	87.243W	164	390MB		072	0	0	009*	300
ERL 22	09	04	73	03	17	48	12.267N	86.553W	167	470MB		075	0	0	035	428
CGSPDE	15	10	67	08	00	50	11.900N	86.000W	162	620MB		074	0	0	083	677
CGS-B	02	07	63	22	00	48	13.700N	88.900W	173	410MB		073	0	0		329
NOS 16	19	02	71	16	59	40	13.562N	88.795W	176	520MB	3	073	0	F 0	D042	511
CGS 51	17	07	69	13	24	22	12.590N	67.006W	175	440MB		074	0	0	G012	379
CGS-B	01	07	65	16	57	30	12.100N	87.500W	176	430MB		074	0	0	007	362
GUTE	26	01	47	10	06	46	12.500N	86.250W	170	0		075	0	720PAS	BBA	720
CGS 21	14	03	69	08	47	16	12.916N	86.755W	178	560MB	4	075	0	F 580PAS	D109	580
CGS 22	27	03	70	18	39	18	12.631N	86.823W	182	510MB		075	0	0	031	495
CGSPDE	27	08	67	13	08	55	12.300N	86.200W	183	520MB		075	0	0	060	511
CGS-B	30	06	66	10	39	00	11.900N	85.900W	187	400MB		075	0	0	021	312
CGS 68	06	10	70	15	18	54	13.561N	88.312W	192	460MB		073	0	0	018*	412

CGSPDE	11	06	68	05	52	33	13.941N	88.761W	199	530MB		073	0		0	D056	528
ERL 49	27	07	73	19	42	47	12.813N	86.674W	199	530MB	3	075	0	F	0	D111	528
CGSPDE	01	03	68	16	34	44	11.700N	85.500W	190	480MB		075	0		0	033	445
CGS 67	29	09	70	04	42	46	11.473N	85.479W	192	540MB		075	0		540BRK	D132	540
CGS-B	01	07	66	20	17	49	13.700N	88.400W	201	460MB		073	0		0	049	412
CGS-B	09	12	64	14	22	03	13.200N	87.200W	200	390MB		072	0		0	008	300
ERL 45	28	07	72	13	53	40	13.151N	87.753W	209	440MB		072	0		0	016*	379
CGS-B	25	07	66	00	42	58	13.200N	87.800W	210	400MB		072	0		0	006	312
ERL 60	12	08	71	05	59	00	12.877N	87.341W	211	470MB		074	0		0	026	428
CGS-B	19	01	67	02	19	23	12.561N	86.613W	200	450MB		075	0		0	G024	395
GUTE	24	02	34	05	33	30	12.750N	86.750W	200	0		075	0		600PAS	A3B	600
GUTE	07	04	44	13	32	58	12.000N	85.500W	200	0		075	0		600PAS	8CC	600
CGS 11	29	01	70	23	06	38	11.492N	86.086W	215	480MB		074	0		0	G018*	445
GUTE	24	04	16	08	02	12	11.000N	85.000W	N	0		075	0		730PAS	*	730
GUTE	05	10	50	16	09	31	11.000N	85.000W	N	0		075	0		CD770PAS	*	770
D&M 301	10	9	62				12.30	86.70	N	E						*	370
D&M 121	25	10	28				12.30	85.80		C						*	622
D&M 162	9	7	51				11.00	85.00		E						*	466
D&M 189	7	3	55				13.00	87.00		E						*	394
D&M 194	11	4	55				13.50	87.00		E						*	506
D&M 265	24	3	60				13.50	87.00		E						*	368
D&M 266	26	4	60				11.00	84.50		E						*	493
ISS	28	09	51	12	07	07	11.500N	86.300W	20 0	0		07 4	0		575PAS	*	575
D&M 168	28	9	51				11.50	86.30	160	E						*	466
D&M 274	22	3	61				11.80	86.80	172	E						*	449
D&M 118	10	2	22				13.00	85.40	I	C	OFL					*	671
D&M 143	29	12	39				13.00	86.00		E	OFL					*	521
D&M 299	24	7	62				11.20	87.70	173	E	OFL					*	390

* COSTA RICA 5 TO 39 KM (23)

CGS-B	26	07	64	13	53	25	11.800N	88.200W	9	440MB		076	0		0	010	379
NOS 41	05	06	71	14	20	42	09.339N	84.232W	26	540MB	3	T078	51MS	F	0	082	510
ERL 47	04	08	73	00	44	42	09.838N	84.586W	33	530MB		078	54MS	F	520BRK	N083	540
ERL 60	05	08	71	22	58	08	09.641N	84.069W	33	490MB		078	0		0	N019	462
CGS-B	27	03	64	17	12	36	09.200N	84.000W	33	420MB		078	0		0	011	345
CGS-B	13	04	65	04	46	43	09.700N	84.400W	33	400MB		078	0		0	006	312
CGS-B	15	10	64	21	04	11	09.100N	84.000W	36	430MB		078	0		0	011	362
CGS-B	20	02	64	09	54	12	09.500N	84.600W	33	430MB		078	0		0	009	362
ERL 13	26	02	73	00	43	39	09.606N	84.229W	33	470MB		078	40MS	F	0	N031	400
CGS-B	17	12	65	04	03	28	09.300N	84.200W	35	420MB		078	0		0	007	345
GUTE	09	03	37	15	40	20	09.000N	83.500W	30	0		078	0		640PAS		640
CGS-B	09	08	66	11	12	39	09.300N	83.800W	35	470MB		078	0		0	032	428
CGS-B	27	03	64	17	27	35	09.400N	83.900W	33	420MB		078	0		0	015	345
ERL 13	23	02	73	19	36	48	09.845N	83.493W	33	470MB		078	42MS		0	N038	420
CGS-B	21	07	66	13	53	23	09.100N	83.900W	33	400MB		078	0		0	011	312
CGS-B	10	07	66	19	18	57	09.700N	83.900W	33	410MB		078	0		0	008	329
CGS-B	15	10	64	22	52	35	09.600N	83.700W	37	420MB		078	0		0	011	345
CGS-B	31	07	63	18	33	28	09.200N	82.400W	33	410MB		080	0		0	006	329
CGS-B	02	04	64	15	25	41	09.500N	82.700W	33	390MB		080	0		0	007	300
CGSPDE	02	07	68	16	15	48	09.045N	82.934W	33	420MB		080	0		0	N007*	345
CGS-B	12	11	65	08	59	54	10.600N	84.400W	37	500MB		078	0		0	037	478
ERL 36	14	04	73	08	54	29	10.711N	84.763W	33	480MB		078	0		0	N034*	445
ERL 36	14	04	73	08	34	00	10.679N	84.759W	33	570MB		078	65MS		CD650PAS	N117*	650

* COSTA RICA AREA 80 TO 119 KM (10)

CGS-B	11	12	66	17	21	56	10.200N	84.100W	82	430MB		078	0		0	018	362
ERL 11	25	02	72	03	43	47	10.187N	83.414W	88	460MB		078	0		0	019*	412
GUTE	19	11	48	01	04	24	10.000N	83.500W	80	0		078	0		700PAS	B8C	700
CGS-B	11	08	66	09	50	41	10.500N	84.000W	96	430MB		078	0		0	014	362
CGS-B	23	05	61	03	40	24	09.800N	84.000W	93	0		078	0		538PAL		538

CGS-B	23	05	67	17	14	50	10.197N	84.022W	119	430MB		078	0	0	008*	362
CGSPDE	04	07	68	15	15	59	10.464N	84.056W	113	440MB		078	0	0	012	379
CGS-B	23	07	64	04	57	54	10.500N	83.900W	114	420MB		078	0	0	010	345
CGS-B	12	08	63	17	19	05	09.830N	84.200W	114	420MB		078	0	0	007	345
D&M 160	18	6	51				11.00	84.00	96	E					*	449
* PACIFIC COASTLINE 33 KM (12)																
CGS 55	12	08	70	09	24	11	12.015N	86.542W	33	590MB		075	63MS F	600PAS	N077	630
CGS-B	11	05	66	05	57	02	12.300N	86.800W	33	400MB		075	0	0	007	312
CGS 55	12	08	70	10	24	23	12.051N	86.539W	33	560MB		075	55MS	530PAS	N057	550
ERL 12	15	02	73	00	15	53	11.534N	86.473W	33	510MB		074	50MS	0	D053	500
CGSPDE	08	10	67	14	28	07	11.300N	86.000W	33	420MB		074	0	0	009	345
CGS-B	21	03	65	09	42	41	11.700N	86.400W	36	520MB		074	0	0	036	511
CGS 8	15	01	70	19	08	59	11.157N	86.015W	33	460MB		074	0	0	N012*	412
ERL 73	29	08	71	08	26	52	11.547N	86.180W	33	490MB		074	0	0	N034	462
ERL 77	24	08	71	08	09	07	11.509N	86.323W	33	510MB		074	42MS	0	N049	420
D&M 172	3	3	52				11.50	86.30	33	E					*	500
D&M 174	5	12	52				11.50	86.30	33	E					*	486
D&M 253	5	6	59				12.06	86.52	27	E					*	491
* ATLANTIC ALL DEPTH (11)																
CGS-B	25	06	62	18	58	35	14.500N	82.400W	25	0		094	0	463PAL	015	463
CGS-B	20	05	66	03	58	01	11.600N	82.600W	33	390MB		079	0	0	009	300
CGS-B	11	08	65	18	15	39	10.800N	83.200W	33	390MB		078	0	0	007	300
CGS-B	08	10	66	20	38	07	10.700N	82.400W	33	410MB		079	0	0	008	329
GUTE	17	03	26	11	53	36	12.500N	82.500W	50	0		094	0	690PAS		690
GUTE	06	12	37	21	43	09	14.000N	82.000W	60	0		094	0	600PAS		600
CGSPDE	17	07	68	06	23	11	10.394N	83.389W	19	510MB		078	0	0	D052	495
CGSPDE	03	03	68	08	43	24	10.000N	83.100W	16	410MB		078	0	0	010	329
GUTE	20	03	36	18	46	28	11.000N	84.000W	N	0		075	0	560PAS	*	560
GUTE	25	07	13	12	38	06	13.000N	83.000W	0	0		075	0	630PAS	*	630
SYKES	07	12	57	22	18	51	13.550N	82.410W	0	0		094	0	510PAL	023	510
* MANAGUA VOLCANO LINE N (57)																
CGS-B	09	05	63	15	06	24	12.700N	86.600W	33	410MB		075	0	0	008	329
ERL 2	05	01	72	11	52	26	12.236N	86.236W	33	420MB		075	0	0	N019*	345
ERL 1	03	01	72	04	31	04	12.311N	86.300W	33	410MB		075	0	0	N019*	329
D&M 131	11	7	33				12.20	86.30	N	D					***	583
D&M 132	24	8	33				12.20	86.30	N	D					***	531
D&M 156	5	1	51				12.20	86.30	N	D					***	565
D&M 272	16	2	61				12.70	86.50	40	E					*	393
D&M 309	15	1	63				13.00	86.70	33	E					*	444
D&M 161	8	7	51				12.00	86.00		E		DEEP M AY BE			*	489
D&M 175		1	53				13.50	86.50		D		DEEP M AY BE			*	576
D&M 200	30	4	55				12.50	86.50		D		DEEP M AY BE			*	559
D&M 236	5	6	58				12.50	86.50		E		DEEP M AY BE			*	461
D&M 256	12	8	59				12.00	86.00		E		DEEP M AY BE			*	470
D&M 41			50				12.60	86.36	N	D	V				***	598
D&M 42	11	4	50				12.48	86.68	N	C	V				***	674
D&M 43	27	4	50				12.48	86.67	N	D	V				***	551
D&M 44			52				12.42	86.55	N	D	V				***	561
D&M 45	8	5	52				12.16	86.30	N	C	V				***	692
D&M 46	29	6	52				11.95	86.15	N	D	V				***	566
D&M 47		7	52				11.95	86.15	N	D	V				***	575
D&M 49	8	4	53				11.95	86.15	N	D	V				***	547
D&M 50	29	6	53				11.95	86.15	N	D	V				***	595
D&M 51		5	53				11.95	86.15	N	D	V				***	595
D&M 53	2	1	55				11.95	86.15	N	D	V				***	540
D&M	2	1	55				11.95	86.15	N	D	V				***	595
D&M 54		12	56				11.95	86.15	N	C	V				***	682
D&M 55			56				11.95	86.15	N	C	V				***	653

D&M	56			57				11.95	86.15	N	C	V				***	632
D&M	57	11	4	58				11.95	86.15	N	C	V				***	661
D&M	58	25	4	58				11.95	86.15	N	C	V				***	656
D&M	59			58				12.42	86.55	N	D	V				***	552
D&M	60	10	11	58				11.95	86.15	N	C	V				***	644
D&M	61	27	1	59				11.95	86.15	N	C	V				***	680
D&M	62	27	3	59				11.95	86.15	N	D	V				***	581
D&M	64			60				11.95	86.15	N	D	V				***	585
D&M	67		8	63				12.70	87.02	N	D	V				***	568
D&M	69	14	11	67				12.48	86.68	N	B	V				***	729
D&M	70	24	7	68				12.42	86.55	N	C	V				***	670
D&M	71	16	6	70				11.95	86.15	N	C	V				***	689
D&M	72	18	6	70				12.42	86.55	N	C	V				***	674
D&M	82		4	83				11.53	85.65	N	D	V				***	574
D&M	83		5	83				11.53	85.65	N	D	V				***	546
D&M	84	2	5	83				11.53	85.65	N	D	V				***	557
D&M	85	4	5	83				11.53	85.65	N	D	V				***	539
D&M	86	6	5	83				11.53	85.65	N	D	V				***	530
D&M	87	6	5	83				11.53	85.65	N	D	V				***	564
D&M	88	15	5	83				11.53	85.65	N	D	V				***	542
D&M	101	22	5	86				12.42	86.55	N	C	V				***	628
D&M	109	1	5	11				11.50	85.70	N	D	V				***	592
D&M	110	17	10	13				11.95	86.15	N	D	V				***	557
D&M	149	11	2	46				11.60	85.60		C	V				***	696
D&M	151	15	7	47				12.50	86.80	N	C	V				***	643
D&M	358	24	10	68				12.52	86.73	N	C	V				***	611
D&M	428	3	2	71				12.50	86.70	N	D	V				***	581
D&M	431	3	5	71				12.70	87.00	N	D	V				***	581
D&M	438	1	12	71				12.60	86.90	N	D	V				***	585

* MANAGUA AREA 5 KM (5)

ERL	77	23	12	72	07	19	49	12.000N	86.189W	5	520MB		075	0	0	G049	511
CGSPDE		04	01	68	10	03	56	12.100N	86.300W	5	460MB		075	0	0	028	412
ERL	76	23	12	72	06	29	42	12.357N	86.119W	5	560MB		075	62MS	CD620PAS	G099	620
ERL	77	23	12	72	07	17	36	12.050N	86.378W	5	500MB		075	0	0	G029	478
GUTE		31	03	31	16	02	21	12.250N	86.200W	0	0		075	0	560PAS	*	560

* FONSECA AREA 33 KM (8)

D&M	329	18	2	64				13.00	87.50	N	D					***	546
D&M	304	24	11	62				13.00	87.50	25	C					***	615
D&M	284	29	8	61				13.00	87.50	33	D					*	562
D&M	294	16	6	62				13.30	87.30	33	E					*	369
D&M	166	6	8	51				13.00	87.50	33	5.5					*	550
ISS		03	08	51	00	23	58	13.000N	87.800W	33	0		072	0	DF600PAS		600
ISS		03	08	51	05	25	44	13.000N	87.800W	33	0		072	0	550PAS		550
ISS		02	08	51	20	30	17	13.000N	87.800W	33	0		072	0	575PAS		575

EARTHQUAKES IN CHRONOLCGICAL ORDER
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(THE FIRST DIGIT OF MAGNITUDES MB,MS OR RICHTER CORRESPOND TO UNIT)

DATA SOURCE	DAY	MONTH	YEAR	HOUR	MINUTE	SECOND	LATITUDE	LONGITUDE		MB MAG			MS MAG	RICHTER MAG		RICHTER MAG
D&M 41			50				12.60	86.36	N	D	V				***	598
D&M 42	11	4	50				12.48	86.68	N	C	V				***	674
D&M 43	27	4	50				12.48	86.67	N	D	V				***	551
D&M 44			52				12.42	86.55	N	D	V				***	561
D&M 45	8	5	52				12.16	86.30	N	C	V				***	692
D&M 46	29	6	52				11.95	86.15	N	D	V				***	566
D&M 47		7	52				11.95	86.15	N	D	V				***	575
D&M 49	8	4	53				11.95	86.15	N	D	V				***	547
D&M 50	29	6	53				11.95	86.15	N	D	V				***	595
D&M 51		9	53				11.95	86.15	N	D	V				***	595
D&M 53	2	1	55				11.95	86.15	N	D	V				***	540
D&M	2	1	55				11.95	86.15	N	D	V				***	595
D&M 54		12	56				11.95	86.15	N	C	V				***	682
D&M 55			56				11.95	86.15	N	C	V				***	653
D&M 56			57				11.95	86.15	N	C	V				***	632
D&M 57	11	4	58				11.95	86.15	N	C	V				***	661
D&M 58	25	4	58				11.95	86.15	N	C	V				***	656
D&M 59			58				12.42	86.55	N	D	V				***	552
D&M 60	10	11	58				11.95	86.15	N	C	V				***	644
D&M 61	27	1	59				11.95	86.15	N	C	V				***	680
D&M 62	27	3	59				11.95	86.15	N	D	V				***	581
D&M 64			60				11.95	86.15	N	D	V				***	585
D&M 67		8	63				12.70	87.02	N	D	V				***	568
D&M 69	14	11	67				12.48	86.68	N	B	V				***	729
D&M 70	24	7	68				12.42	86.55	N	C	V				***	670
D&M 71	16	6	70				11.95	86.15	N	C	V				***	689
D&M 72	18	6	70				12.42	86.55	N	C	V				***	674
D&M 82		4	83				11.53	85.65	N	D	V				***	574
D&M 83		5	83				11.53	85.65	N	D	V				***	546
D&M 84	2	5	83				11.53	85.65	N	D	V				***	557
D&M 85	4	5	83				11.53	85.65	N	D	V				***	539
D&M 86	6	5	83				11.53	85.65	N	D	V				***	530
D&M 87	6	5	83				11.53	85.65	N	D	V				***	564
D&M 88	15	5	83				11.53	85.65	N	D	V				***	542
D&M 101	22	5	86				12.42	86.55	N	C	V				***	628
D&M 109	1	5	11				11.50	85.70	N	D	V				***	592
GUTE	25	07	13	12	38	06	13.000N	83.000W	0	0		075	0	630PAS	*	630
D&M 110	17	10	13				11.95	86.15	N	D	V				***	557
D&M 113	1	5	16				11.00	85.50	N	D					***	547
D&M 114	23	9	16				11.00	85.50	N	D					***	560
D&M 111	27	02	16				11.00	85.50	N	C					*	650
GUTE	24	04	16	08	02	12	11.000N	85.000W	N	0		075	0	730PAS	*	730
GUTE	26	04	16	02	21	30	10.000N	85.000W	0	0		078	0	730PAS	*	730
GUTE	29	06	19	23	14	23	13.500N	86.500W	90	0		075	0	675PAS	BCC	675
GUTE	22	07	19	22	01	35	12.000N	85.000W	150	0		075	0	650PAS	CCC	650
GUTE	28	03	21	07	49	22	12.500N	87.500W	N	0		074	0	730PAS	*	730
D&M 118	10	2	22				13.00	85.40	I	C	OFL				*	671
GUTE	04	03	24	10	07	42	09.750N	84.000W	0	0		078	0	700PAS	*	700
GUTE	05	10	25	04	09	07	12.250N	85.250W	135	0		075	0	675PAS	BBA	675

GUTE	17	03	26	11	53	36	12.500N	82.500W	50	0		094	0	690PAS		690
GUTE	05	11	26	07	55	38	12.300N	85.800W	135	0		075	0	700PAS	BBA	700
D&M 121	25	10	28				12.30	85.80		C					*	622
GUTE	07	02	31	03	30	35	13.000N	87.000W	100	0		072	0	575PAS	CCC	575
GUTE	07	03	31	00	41	56	11.500N	85.500W	80	0		075	0	600PAS	ACB	600
GUTE	31	03	31	16	02	21	12.250N	86.200W	0	0		075	0	560PAS	*	560
GUTE	21	05	32	10	10	07	12.000N	87.500W	90	0		074	0	690PAS	AAA	690
D&M 130	21	5	33				12.00	87.50	90	6.9	OFL				***	6.9
D&M 131	11	7	33				12.20	86.30	N	D					***	583
D&M 132	24	8	33				12.20	86.30	N	D					***	531
GUTE	12	01	33	01	17	42	11.000N	87.000W	N	0		074	0	560PAS	*	560
GUTE	24	02	34	05	33	30	12.750N	86.750W	200	0		075	0	600PAS	ABB	600
GUTE	07	03	34	22	41	47	13.250N	87.750W	0	0		072	0	625PAS	*	625
GUTE	22	12	34	14	29	31	11.500N	87.000W	N	0		074	0	650PAS	*	650
D&M 135	1	9	35				12.58	87.16		E					*	465
GUTE	20	03	36	18	46	28	11.000N	84.000W	N	0		075	0	560PAS	*	560
D&M 137	12	1	37				12.58	87.16		D					*	590
D&M 138	25	9	37				11.00	85.00		E					*	444
GUTE	09	03	37	15	40	20	09.000N	83.500W	30	0		078	0	640PAS		640
GUTE	06	12	37	21	43	09	14.000N	82.000W	60	0		094	0	600PAS		600
D&M 139	25	4	38				12.20	86.90		C					*	676
D&M 141	9	2	39				11.10	86.20		E					*	446
GUTE	18	06	39	16	46	05	10.000N	83.000W	70	0		078	0	650PAS	BCB	650
GUTE	08	07	39	21	31	44	12.500N	88.000W	90	0		076	0	550PAS	CCC	550
GUTE	21	12	39	20	54	48	10.000N	85.000W	0	0		078	0	730PAS	*	730
GUTE	22	12	39	04	44	00	10.000N	84.500W	0	0		078	0	675PAS	*	675
GUTE	26	12	39	11	55	11	13.250N	88.250W	75	0		073	0	600PAS	AAA	600
D&M 143	29	12	39				13.00	86.00		E	OFL				*	521
D&M 144	20	2	40				12.50	87.50		C					*	681
GUTE	05	10	40	14	38	43	09.500N	84.250W	0	0		078	0	625PAS	*	625
GUTE	27	10	40	05	35	37	09.750N	84.500W	0	0		078	0	675PAS	*	675
GUTE	06	01	41	09	48	34	11.750N	86.500W	60	0		074	0	600PAS		600
GUTE	08	10	41	04	22	13	10.500N	86.250W	0	0		077	0	575PAS	*	575
GUTE	16	11	41	09	39	46	13.250N	88.500W	80	0		073	0	575PAS	AAA	575
GUTE	06	12	41	01	25	01	10.500N	85.250W	0	0		078	0	600PAS	*	600
D&M 147	5	8	44				12.50	87.50		E					*	425
GUTE	07	04	44	13	32	58	12.000N	85.500W	200	0		075	0	600PAS	BCC	600
D&M 148	5	10	45				13.00	87.00	100	E					*	365
D&M 149	11	2	46				11.60	85.60		C	V				***	696
D&M 151	15	7	47				12.50	86.80	N	C	V				***	643
GUTE	26	01	47	10	06	46	12.500N	86.250W	170	0		075	0	720PAS	BBA	720
GUTE	19	11	48	01	04	24	10.000N	83.500W	80	0		078	0	700PAS	BBC	700
D&M 152	25	1	49				11.00	86.00		E					*	429
D&M 153	11	6	49				12.50	87.00	100	E					*	397
D&M 154	13	11	49				11.00	85.50	60	E					*	384
GUTE	05	10	50	16	09	31	11.000N	85.000W	N	0		075	0	CD770PAS	*	770
ISS	11	11	50	13	51	10	10.400N	85.700W	0	0		078	0	575PAS	*	575
D&M 156	5	1	51				12.20	86.30	N	D					***	565
D&M 157	2	4	51				12.00	87.00		E					*	419
D&M 158	6	5	51				11.00	85.50		E					*	353
D&M 159	11	5	51				13.00	87.50	100	E					*	467
D&M 160	18	6	51				11.00	84.00	96	E					*	449
D&M 161	8	7	51				12.00	86.00		E	DEEP M AY BE				*	489
D&M 162	9	7	51				11.00	85.00		E					*	466
D&M 166	6	8	51				13.00	87.50	33	5.5					*	550
D&M 168	28	9	51				11.50	86.30	160	E					*	466
ISS	06	05	51	23	08	01	13.000N	87.800W	0	0		072	0	CD650TAC	*	650
ISS	06	05	51	23	03	32	13.000N	87.800W	96	0		072	0	625TAC		625
ISS	07	05	51	20	22	21	13.000N	87.800W	0	0		072	0	600TAC	*	600
ISS	02	08	51	20	30	17	13.000N	87.800W	33	0		072	0	575PAS		575

ISS		03	08	51	05	25	44	13.000N	87.800W	33		0	072	0	550PAS		550
ISS		03	08	51	00	23	58	13.000N	87.800W	33		0	072	0	DF600PAS		600
ISS		28	09	51	12	07	07	11.500N	86.300W	20	0	0	074	0	575PAS	*	575
D&M	169	17	12	51				11.50	86.30	96		E				*	375
D&M	171	2	3	52				11.50	86.30	100		D				*	572
D&M	172	3	3	52				11.50	86.30	33		E				*	500
D&M	173	12	3	52				11.00	86.00			E				*	389
ISS		26	02	52	15	39	28	11.500N	86.300W	96		0	074	0	F 600PAS		600
ISS		13	05	52	19	31	45	10.300N	85.300W	64		0	078	0	690PAS		690
ISS		09	09	52	12	54	42	09.200N	84.200W	0		0	078	0	F 688PAS	*	688
ISS		20	11	52	15	37	17	12.100N	87.900W	33		0	074	0	625PAS		625
D&M	174	5	12	52				11.50	86.30	33		E				*	486
D&M	175		1	53				13.50	86.50			D	DEEP M AY BE			*	576
D&M	180	21	2	54				12.50	87.00	60		E				*	420
D&M	179	21	2	54				11.90	87.00			E				*	480
TAC		13	01	54	00	26	30	14.000N	86.416W	100		0	072	0	575TAC		575
CGS		19	02	54	21	34	41	12.500N	87.500W	N		0	074	0	F 663PAS	*	663
CGS		19	02	54	00	40	25	11.500N	87.500W	N		0	074	0	F 663PAS	*	663
CGS		20	02	54	02	00	43	11.500N	87.500W	N		0	074	0	F 587PAS	*	587
CGS		03	05	54	17	13	32	12.000N	86.000W	150		0	075	0	600PAS		600
D&M	182	27	11	54				12.00	87.00			E				*	373
D&M	183	2	12	54				12.00	86.50	100		E				*	497
D&M	184	28	12	54				11.50	87.50			E				*	515
D&M	186	11	1	55				11.00	86.50			E				*	498
D&M	189	7	3	55				13.00	87.00			E				*	394
D&M	190	10	3	55				12.50	87.00			E				*	486
D&M	194	11	4	55				13.50	87.00			E				*	506
D&M	200	30	4	55				12.50	86.50			D	DEEP M AY BE			*	559
D&M	195	30	4	55				12.50	87.00			D				*	597
D&M	196	30	4	55				12.00	87.00	N		6.0				*	6.0
D&M	201	7	7	55				12.50	88.00	N		E				*	515
D&M	203	16	8	55				11.00	87.00			E				*	464
D&M	204	29	8	55				12.00	87.00			E				*	458
CGS		04	04	55	19	24	04	13.000N	87.000W	N		0	072	0	625PAS	*	625
D&M	206	13	10	55				12.00	87.00			E				*	358
D&M	207	24	1	56				12.20	86.70	I		7.3				*	7.3
D&M	208	21	4	56				12.00	87.33			D				*	581
D&M	209	9	8	56				12.80	86.00	N		D				***	571
CGS		19	07	56	23	26	25	09.500N	84.500W	0		0	078	0	F 600BRK	*	600
ISS		24	10	56	14	42	12	11.790N	86.460W	N		0	074	0	DF730PAS	*	730
CGS		25	10	56	05	21	40	12.000N	87.000W	N		0	074	0	F 637PAS	*	637
ISS		27	10	56	15	33	02	11.880N	86.460W	90		0	074	0	587PAS		587
CGS		10	11	56	00	08	27	10.500N	86.000W	100		0	077	0	600PAS		600
D&M	216	28	1	57				12.00	86.50			E				*	486
D&M	218	19	5	57				12.14	87.24	24		E				*	413
SYKES		27	10	57	18	29	10	15.970N	88.110W	56		0	093	0	400PAL	008	400
D&M	226	26	11	57				11.50	86.50	100		E				*	512
SYKES		07	12	57	22	18	51	13.550N	82.410W	0		0	094	0	510PAL	023	510
D&M	231	23	12	57				12.50	86.50	150		E				*	368
D&M	235	4	6	58				12.00	86.50			E				*	352
D&M	236	5	6	58				12.50	86.50			E	DEEP M AY BE			*	461
D&M	238	13	8	58				11.70	87.20	N		D				***	538
D&M	239	3	9	58				11.70	87.20	N		E				***	351
CGS		27	06	58	05	44	28	13.000N	88.500W	60		0	073	0	DF600PAS		600
D&M	241	14	11	58				12.36	86.37	72		C				*	670
D&M	242	4	12	58				11.50	86.50	100		6.0				*	6.0
D&M	243	27	12	58				12.80	87.30			D				*	593
D&M	245	21	2	59				11.70	87.20			D				*	582
D&M	246	15	3	59				12.00	85.00			E				*	386
D&M	247	31	3	59				13.00	87.00			E				*	482

D&M 248	2	4	59				11.70	87.20		D						*	571
D&M 249	22	4	59				11.50	86.50		D						*	591
D&M 252	30	5	59				11.00	86.00		E						*	448
D&M 253	5	6	59				12.06	86.52	27	E						*	491
D&M 255	12	7	59				11.50	86.00	N	D						***	547
D&M 256	12	8	59				12.00	86.00		E	DEEP M AY BE					*	470
D&M 258	15	8	59				12.20	87.30	N	D						*	557
D&M 259	11	9	59				12.00	87.50		E						*	513
ISS	16	02	59	17	54	15	13.110N	87.340W	0	0			072	0	540TAC	*	540
ISS	24	04	59	09	31	33	11.480N	86.400W	N	0			074	0	638PAS	*	638
CGS	03	05	59	04	41	24	12.500N	87.500W	100	0		5	074	0	F 560TAC		560
D&M 260	19	11	59				12.00	87.00	60	E						*	397
D&M 262	21	1	60				12.00	87.25		E						*	420
D&M 263	26	1	60				13.00	87.50	60	D						*	580
D&M 264	15	2	60				12.00	87.00		E						*	390
D&M 265	24	3	60				13.50	87.00		E						*	368
D&M 266	26	4	60				11.00	84.50		E						*	493
D&M 267	10	7	60				12.50	86.00	150	6.0						*	6.0
D&M 272	16	2	61				12.70	86.50	40	E						*	393
D&M 274	22	3	61				11.80	86.80	172	E						*	449
D&M 284	29	8	61				13.00	87.50	33	D						*	562
D&M 285	3	9	61				12.20	86.90	124	E						*	381
CGS-B	20	03	61	06	16	21	11.300N	86.500W	60	0			074	0	F 613PAS		613
CGS-B	12	04	61	22	20	33	13.200N	88.900W	122	0		6	073	0	D 588PAS		588
CGS-B	19	05	61	09	25	41	12.800N	88.200W	68	0		5	076	0	F 463PAL		463
CGS-B	23	05	61	16	44	59	12.700N	87.300W	138	0		6	074	0	DF650PAS		650
CGS-B	23	05	61	03	40	24	09.800N	84.000W	93	0			078	0	538PAL		538
D&M 289	22	12	61				11.90	85.50	49	E						*	393
D&M 290	8	1	62				12.20	85.40	103	E						*	380
D&M 292	17	1	62				11.90	86.30	97	E						*	389
D&M 294	16	6	62				13.30	87.30	33	E						*	369
D&M 298	17	7	62				11.60	87.10	25	E						*	470
D&M 299	24	7	62				11.20	87.70	173	E	DFL					*	390
D&M 300	16	8	62				12.60	85.60	N	D						***	562
D&M 301	10	9	62				12.30	86.70	N	E						*	370
D&M 302	28	9	62				11.60	87.20	N	E						***	457
CGS-B	30	01	62	08	34	23	12.700N	87.500W	49	0			074	0	F 488PAL	049	488
CGS-B	25	06	62	18	58	35	14.500N	82.400W	25	0			094	0	463PAL	015	463
D&M 303	27	10	62				11.60	86.40	106	E						*	360
D&M 304	24	11	62				13.00	87.50	25	C						***	615
D&M 309	15	1	63				13.00	86.70	33	E						*	444
D&M 306	15	1	63				12.00	87.50	33	D						*	589
D&M 307	15	1	63				12.00	87.50	20	D						***	582
D&M 310	16	1	63				13.00	86.70	N	E						***	367
D&M 311	11	2	63				11.60	87.50	41	E						*	496
D&M 313	26	2	63				12.40	87.40	N	E						***	505
D&M 314	25	4	63				12.20	87.30	75	E						*	415
D&M 334	11	5	63				11.40	87.50	100	D	DFL					***	542
D&M 317	30	6	63				11.40	87.30	33	4.2						***	4.2
D&M 322	9	8	63				12.90	87.40	80	E						***	383
D&M 335	16	9	63				11.80	87.40	N	E						***	518
CGS-B	25	02	63	08	58	43	12.300N	88.200W	58	410MB			076	0	0	029	329
CGS-B	13	04	63	18	53	18	11.700N	87.800W	33	430MB			074	0	0	016	362
CGS-B	09	05	63	15	03	43	12.400N	87.000W	50	550MB			074	0	0	047	561
CGS-B	09	05	63	15	06	24	12.700N	86.600W	33	410MB			075	0	0	008	329
CGS-B	24	05	63	06	10	09	12.100N	88.500W	42	440MB			076	0	0	012	379
CGS-B	21	06	63	03	32	41	10.700N	85.800W	73	500MB			078	0	0	022	478
CGS-B	02	07	63	18	05	33	09.000N	83.000W	64	420MB			078	0	0	011	345
CGS-B	02	07	63	22	00	48	13.700N	88.900W	173	410MB			073	0	0		329
CGS-B	04	07	63	01	43	20	12.800N	88.700W	55	410MB			076	0	0	007	329

CGS-B	05	07	63	14	40	54	12.800N	88.900W	85	410MB	076	0	0	011	329
CGS-B	30	07	63	12	25	39	11.400N	87.300W	33	420MB	074	0	0	005	345
CGS-B	31	07	63	18	33	28	09.200N	82.400W	33	410MB	080	0	0	036	329
CGS-B	12	08	63	17	19	05	09.800N	84.200W	114	420MB	078	0	0	007	345
CGS-B	28	08	63	05	55	51	12.600N	87.200W	115	420MB	074	0	0	006	345
CGS-B	31	08	63	13	08	46	11.900N	87.000W	48	490MB	074	0	0	018	462
CGS-B	01	09	63	22	57	34	11.300N	85.500W	120	440MB	075	0	0	010	379
CGS-B	08	09	63	05	28	37	12.300N	87.900W	31	410MB	074	0	0	008	329
CGS-B	17	09	63	03	48	09	12.600N	87.100W	144	390MB	074	0	0	007	300
CGS-B	28	09	63	05	25	03	10.200N	86.000W	33	410MB	077	0	0	008	329
CGS-B	01	10	63	10	30	31	10.200N	84.500W	46	460MB	078	0	0	022	412
CGS-B	07	10	63	03	59	54	11.600N	86.900W	50	450MB	074	0	0	015	395
D&M 326	13	10	63				12.50	87.10	N	D				***	535
CGS-B	13	10	63	05	10	12	09.800N	84.100W	63	440MB	078	0	0	012	379
CGS-B	06	11	63	21	49	57	12.200N	88.000W	37	410MB	076	0	0	010	329
CGS-B	10	11	63	07	18	07	12.900N	88.500W	45	420MB	076	0	0	007	345
D&M 327	23	11	63				12.00	87.20	N	C				***	605
CGS-B	16	12	63	06	23	20	12.200N	88.400W	34	430MB	076	0	0	010	362
CGS-B	20	12	63	22	28	31	13.200N	88.000W	64	430MB	073	0	0	005	362
D&M 328	21	12	63				13.40	87.20	N	D				***	570
CGS-B	31	12	63	14	22	07	12.400N	87.900W	77	430MB	074	0	0	007	362
D&M 329	18	2	64				13.00	87.50	N	D				***	546
D&M 331	11	3	64				12.50	87.10	N	D				***	540
D&M 332	29	4	64				12.30	87.40	N	E				***	515
CGS-B	09	01	64	18	38	10	14.900N	87.900W	33	470MB	072	0	0	005	428
CGS-B	03	02	64	01	46	27	13.000N	88.000W	61	410MB	073	0	0	006	329
CGS-B	20	02	64	09	54	12	09.500N	84.600W	33	430MB	078	0	0	009	362
CGS-B	21	02	64	07	24	08	12.800N	87.900W	62	390MB	074	0	0	007	300
CGS-B	24	02	64	02	29	12	13.000N	87.200W	83	390MB	072	0	0	006	300
CGS-B	02	03	64	16	09	46	12.500N	88.000W	63	440MB	076	0	0	018	379
CGS-B	05	03	64	14	30	18	12.100N	88.100W	53	430MB	076	0	0	009	362
CGS-B	06	03	64	02	55	11	12.400N	87.600W	100	390MB	074	0	0	008	300
CGS-B	16	03	64	06	06	51	13.500N	88.300W	92	390MB	073	0	0	008	300
CGS-B	27	03	64	17	27	35	09.400N	83.900W	33	420MB	078	0	0	015	345
CGS-B	27	03	64	17	12	36	09.200N	84.000W	33	420MB	078	0	0	011	345
CGS-B	02	04	64	15	25	41	09.500N	82.700W	33	390MB	080	0	0	007	300
CGS-B	02	04	64	03	48	59	12.500N	87.800W	32	420MB	074	0	0	009	345
CGS-B	04	04	64	06	43	20	12.500N	87.700W	41	430MB	074	0	0	012	362
CGS-B	24	04	64	14	40	28	13.300N	88.800W	158	510MB	073	0	0	044	495
CGS-B	29	04	64	08	08	41	12.100N	88.400W	33	390MB	076	0	0	007	300
CGS-B	29	04	64	21	53	15	11.800N	87.600W	33	450MB	074	0	0	007	395
CGS-B	30	04	64	15	05	57	11.200N	86.700W	33	430MB	074	0	0	005	362
CGS-B	02	05	64	19	53	10	09.500N	84.500W	44	430MB	078	0	0	007	362
CGS-B	15	05	64	05	46	32	09.800N	85.500W	33	420MB	077	0	0	010	345
CGS-B	15	05	64	12	10	25	10.500N	85.700W	33	450MB	078	0	0	024	395
CGS-B	23	05	64	06	45	21	11.700N	86.600W	93	460MB	074	0	0	007	412
CGS-B	10	06	64	16	25	09	12.000N	87.900W	55	420MB	074	0	0	006	345
CGS-B	15	06	64	09	41	24	12.600N	88.300W	56	410MB	076	0	0	007	329
CGS-B	15	06	64	11	13	47	12.600N	88.000W	14	410MB	076	0	0	005	329
CGS-B	17	06	64	09	11	42	12.000N	87.200W	33	410MB	074	0	0	008	329
CGS-B	19	06	64	17	31	56	12.400N	88.100W	53	400MB	076	0	0	011	312
CGS-B	20	06	64	05	02	37	09.200N	84.400W	45	430MB	078	0	0	007	362
CGS-B	01	07	64	00	42	55	12.000N	86.700W	108	430MB	075	0	0	008	362
CGS-B	04	07	64	02	36	54	11.300N	86.500W	90	410MB	074	0	0	007	329
CGS-B	07	07	64	09	59	19	10.800N	86.600W	33	490MB	077	0	0	010	462
CGS-B	18	07	64	20	37	35	11.000N	87.000W	33	460MB	074	0	0	010	412
CGS-B	21	07	64	07	01	59	13.100N	88.400W	68	470MB	073	0	0	030	428
CGS-B	23	07	64	04	57	54	10.500N	83.900W	114	420MB	078	0	0	010	345
CGS-B	26	07	64	13	53	25	11.800N	88.200W	9	440MB	076	0	0	010	379
CGS-B	28	07	64	02	53	26	11.500N	86.400W	111	420MB	074	0	0	006	345

CGS-B	28	07	64	01	54	18	10.800N	86.200W	33	420MB	077	0	0	006	345
CGS-B	30	07	64	15	13	39	10.600N	86.500W	33	470MB	077	0	0	007	428
CGS-B	30	07	64	05	16	03	11.100N	86.200W	42	570MB	074	0	0	022	594
CGS-B	01	08	64	08	55	49	11.800N	87.000W	88	430MB	074	0	0	008	362
CGS-B	02	08	64	04	00	45	10.900N	86.300W	32	470MB	077	0	0	012	428
CGS-B	07	08	64	08	21	03	11.800N	86.900W	130	470MB	074	0	0	010	428
CGS-B	08	08	64	15	45	10	12.500N	87.800W	63	580MB	074	0	0	032	611
CGS-B	10	08	64	14	00	51	12.430N	88.300W	71	400MB	076	0	0	006	312
CGS-B	12	08	64	12	47	46	11.400N	86.100W	135	410MB	074	0	0	007	329
CGS-B	15	08	64	14	43	03	12.000N	88.000W	39	400MB	076	0	0	006	312
CGS-B	16	08	64	12	34	34	12.000N	88.600W	33	430MB	076	0	0	011	362
CGS-B	16	08	64	07	30	12	13.300N	87.600W	60	390MB	072	0	0	005	300
CGS-B	17	08	64	19	05	44	12.400N	86.800W	116	450MB	075	0	0	008	395
CGS-B	20	08	64	08	26	52	11.700N	87.200W	25	430MB	074	0	0	008	362
CGS-B	20	08	64	09	12	53	11.000N	87.500W	33	420MB	074	0	0	008	345
CGS-B	25	08	64	05	03	18	12.100N	87.700W	72	400MB	074	0	0	005	312
CGS-B	29	08	64	20	51	54	11.430N	87.200W	33	420MB	074	0	0	005	345
CGS-B	30	08	64	15	20	50	12.600N	88.500W	54	460MB	076	0	0	015	412
CGS-B	31	08	64	17	07	39	09.600N	85.400W	33	420MB	077	0	0	006	345
CGS-B	03	09	64	01	55	32	12.230N	87.000W	108	400MB	074	0	0	005	312
CGS-B	12	09	64	19	05	47	11.200N	86.900W	33	470MB	074	0	0	014	428
CGS-B	04	10	64	06	30	14	11.300N	87.400W	33	420MB	074	0	0	007	345
CGS-B	15	10	64	21	09	02	09.600N	84.000W	51	420MB	078	0	0	007	345
CGS-B	15	10	64	21	04	11	09.100N	84.000W	36	430MB	078	0	0	011	362
CGS-B	15	10	64	22	52	35	09.600N	83.700W	37	420MB	078	0	0	011	345
CGS-B	29	10	64	12	21	52	13.230N	88.500W	33	400MB	073	0	0	008	312
CGS-B	02	11	64	07	07	57	11.800N	86.900W	117	460MB	074	0	0	009	412
CGS-B	07	11	64	01	36	56	12.000N	88.000W	33	440MB	076	0	0	012	379
CGS-B	13	11	64	08	02	38	13.000N	88.600W	86	490MB	073	0	0	024	462
CGS-B	27	11	64	10	55	11	13.400N	88.700W	33	410MB	073	0	0	006	329
CGS-B	03	12	64	14	51	10	12.300N	88.500W	33	410MB	076	0	0	010	329
CGS-B	08	12	64	04	11	53	11.500N	87.000W	48	500MB	074	0	0	029	478
CGS-B	09	12	64	14	22	03	13.200N	87.200W	200	390MB	072	0	0	008	300
CGS-B	12	01	65	03	19	10	12.300N	88.900W	39	430MB	076	0	0	007	362
CGS-B	13	01	65	02	19	49	10.100N	86.300W	33	480MB	077	0	0	009	445
CGS-B	21	01	65	20	43	55	12.300N	86.700W	138	440MB	075	0	0	016	379
CGS-B	15	02	65	02	29	48	09.900N	86.500W	33	550MB	077	0	0	011	561
CGS-B	01	03	65	15	17	45	10.830N	86.700W	55	490MB	077	0	0	009	462
CGS-B	21	03	65	09	42	41	11.700N	86.400W	36	520MB	074	0	0	036	511
CGS-B	22	03	65	00	17	27	11.900N	87.900W	33	430MB	074	0	0	007	362
CGS-B	23	03	65	21	21	52	11.000N	86.600W	33	480MB	074	0	0	014	445
CGS-B	23	03	65	02	58	39	12.300N	88.200W	33	410MB	076	0	0	008	329
CGS-B	23	03	65	17	18	29	11.800N	88.200W	33	420MB	076	0	0	006	345
CGS-B	09	04	65	15	09	15	11.400N	87.400W	33	420MB	074	0	0	009	345
CGS-B	13	04	65	04	46	43	09.700N	84.400W	33	400MB	078	0	0	006	312
CGS-B	24	04	65	13	25	41	12.700N	82.000W	33	440MB	094	0	0	009	379
CGS-B	31	05	65	20	46	54	11.130N	86.000W	28	470MB	074	0	0	030	428
CGS-B	06	06	65	08	34	17	11.100N	87.200W	33	460MB	074	0	0	005	412
CGS-B	07	06	65	07	41	47	10.400N	86.000W	33	470MB	077	0	0	007	428
CGS-B	08	06	65	01	25	59	11.000N	86.800W	33	470MB	074	0	0	007	428
CGS-B	18	06	65	09	28	21	11.600N	88.200W	33	430MB	076	0	0	007	362
CGS-B	23	06	65	07	37	46	11.400N	87.800W	24	450MB	074	0	0	028	395
CGS-B	27	06	65	13	08	28	12.600N	87.900W	33	410MB	074	0	0	007	329
CGS-B	01	07	65	16	57	30	12.100N	87.500W	176	430MB	074	0	0	007	362
CGS-B	16	07	65	10	34	12	11.600N	88.000W	30	490MB	076	0	0	022	462
CGS-B	16	07	65	11	21	10	11.600N	87.800W	33	480MB	074	0	0	016	445
CGS-B	17	07	65	04	05	16	10.100N	86.500W	33	430MB	077	0	0	006	362
CGS-B	17	07	65	13	59	46	12.200N	87.900W	56	440MB	074	0	0	013	379
CGS-B	19	07	65	22	14	21	10.600N	85.300W	80	460MB	078	0	0	019	412
CGS-B	30	07	65	06	33	30	12.000N	88.500W	33	420MB	076	0	0	009	345

CGS-B	11	08	65	18	15	39	10.800N	83.200W	33	390MB	078	0	0	007	300
CGS-B	09	09	65	12	43	37	09.600N	85.300W	33	450MB	077	0	0	008	395
CGS-B	27	09	65	17	37	53	12.900N	88.400W	33	420MB	076	0	0	007	345
CGS-B	16	10	65	14	22	55	09.000N	83.500W	48	510MB	078	0	0	053	495
CGS-B	20	10	65	23	54	51	12.700N	87.000W	91	560MB	074	0	0	032	578
CGS-B	20	10	65	23	54	30	12.500N	87.300W	72	540MB	074	0	0	063	545
CGS-B	02	11	65	22	44	41	10.500N	86.300W	69	450MB	077	0	0	010	395
CGS-B	12	11	65	08	59	54	10.600N	84.400W	37	500MB	078	0	0	037	478
CGS-B	21	11	65	02	19	12	12.100N	88.900W	33	430MB	076	0	0	008	362
CGS-B	06	12	65	13	19	13	12.400N	87.400W	44	390MB	074	0	0	007	300
CGS-B	06	12	65	04	39	10	12.500N	87.300W	50	450MB	074	0	0	024	395
CGS-B	10	12	65	17	59	11	12.400N	87.300W	33	420MB	074	0	0	011	345
CGS-B	16	12	65	05	23	06	12.600N	88.800W	152	440MB	076	0	0	019	379
CGS-B	17	12	65	04	03	28	09.300N	84.200W	35	420MB	078	0	0	007	345
CGS-B	24	01	66	03	46	57	12.300N	88.400W	46	450MB	076	0	0	018	395
CGS-B	09	02	66	06	22	44	12.700N	87.800W	98	440MB	074	0	0	021	379
CGS-B	23	02	66	01	02	01	09.700N	85.800W	35	400MB	077	0	0	008	312
CGS-B	13	03	66	21	46	23	14.200N	88.400W	36	420MB	072	0	0	017	345
CGS-B	27	03	66	23	12	49	09.800N	83.300W	61	420MB	078	0	0	018	345
CGS-B	28	03	66	00	33	30	09.500N	83.100W	46	400MB	078	0	0	008	312
CGS-B	09	04	66	02	34	23	09.400N	84.200W	47	530MB	078	0	0	071	528
CGS-B	09	04	66	02	42	11	09.500N	84.100W	49	570MB	078	0	0	079	594
CGS-B	12	04	66	03	34	22	12.600N	87.600W	104	390MB	074	0	0	009	300
CGS-B	12	04	66	17	30	49	12.600N	88.000W	33	440MB	076	0	0	019	379
CGS-B	14	04	66	06	57	45	11.800N	87.500W	33	410MB	074	0	0	007	329
CGS-B	16	04	66	13	21	40	12.300N	88.400W	33	440MB	076	0	0	017	379
CGS-B	24	04	66	15	41	03	13.700N	88.300W	33	390MB	073	0	0	009	300
CGS-B	01	05	66	22	28	45	09.600N	83.900W	71	430MB	078	0	0	018	362
CGS-B	04	05	66	18	13	55	12.500N	87.700W	68	510MB	074	0	0	036	495
CGS-B	11	05	66	05	57	02	12.300N	86.800W	33	400MB	075	0	0	007	312
CGS-B	20	05	66	03	58	01	11.600N	82.600W	33	390MB	079	0	0	009	300
CGS-B	28	05	66	14	34	39	11.900N	88.100W	33	430MB	076	0	0	013	362
CGS-B	30	06	66	10	39	00	11.900N	85.900W	187	400MB	075	0	0	021	312
CGS-B	01	07	66	20	17	49	13.700N	88.400W	201	460MB	073	0	0	049	412
CGS-B	08	07	66	13	22	13	12.400N	88.900W	43	430MB	076	0	0	013	362
CGS-B	10	07	66	19	18	57	09.700N	83.900W	33	410MB	078	0	0	008	329
CGS-B	13	07	66	08	20	58	12.600N	87.800W	56	530MB	074	0	0	073	528
CGS-B	17	07	66	00	14	03	09.200N	82.300W	44	400MB	080	0	0	013	312
CGS-B	21	07	66	13	53	23	09.100N	83.900W	33	400MB	078	0	0	011	312
CGS-B	24	07	66	06	11	55	12.200N	88.300W	61	410MB	076	0	0	013	329
CGS-B	24	07	66	18	50	54	12.200N	88.700W	33	410MB	076	0	0	010	329
CGS-B	25	07	66	00	42	58	13.200N	87.800W	210	400MB	072	0	0	006	312
CGS-B	31	07	66	19	10	46	12.400N	88.100W	63	420MB	076	0	0	032	345
CGS-B	07	08	66	04	11	30	11.000N	86.200W	125	410MB	074	0	0	009	329
CGS-B	09	08	66	11	12	39	09.300N	83.800W	35	470MB	078	0	0	032	428
CGS-B	11	08	66	09	50	41	10.500N	84.000W	96	430MB	078	0	0	014	362
CGS-B	23	08	66	06	07	36	12.400N	88.200W	39	400MB	076	0	0	011	312
CGS-B	20	09	66	03	53	34	13.100N	88.200W	81	400MB	073	0	0	008	312
CGS-B	08	10	66	20	38	07	10.700N	82.400W	33	410MB	079	0	0	008	329
CGS-B	12	10	66	20	20	06	11.200N	86.200W	45	510MB	074	0	0	054	495
CGS-B	19	11	66	16	01	46	09.200N	85.800W	25	430MB	077	0	0	012	362
CGS-B	10	12	66	01	09	39	10.300N	85.100W	87	430MB	078	0	0	011	362
CGS-B	11	12	66	17	21	56	10.200N	84.100W	82	430MB	078	0	0	018	362
CGS-B	14	12	66	01	33	13	10.600N	86.800W	33	430MB	077	0	0	008	362
CGS-B	23	12	66	22	10	25	12.600N	87.700W	32	450MB	074	0	0	019	395
CGS-B	23	12	66	02	02	19	12.900N	88.600W	88	450MB	076	0	0	023	395
CGS-B	27	12	66	21	22	16	13.300N	88.800W	78	540MB	073	0	0	090	545
CGS-B	30	12	66	18	35	15	10.100N	84.700W	43	440MB	078	0	0	015	379
CGS-B	04	01	67	00	28	07	13.100N	88.625W	72	410MB	073	0	0	010	329
CGS-B	09	01	67	22	31	22	13.138N	88.981W	78	410MB	073	0	0	008	329

CGS-B	16	01	67	15	30	47	10.641N	85.920W	33	470MB	078	0		0	N019*	428
CGS-B	19	01	67	02	19	23	12.561N	86.613W	200	450MB	075	0		0	G024	395
CGS-B	29	01	67	20	11	22	11.953N	88.928W	33	450MB	076	0		0	N018	395
CGS-B	30	01	67	09	30	27	12.196N	87.596W	8	410MB	074	0		0	010*	329
CGS-B	06	03	67	23	31	25	12.389N	87.127W	159	410MB	074	0		0	008*	329
CGS-B	08	03	67	15	52	15	13.049N	88.127W	75	390MB	073	0		0	005*	300
CGS-B	C8	03	67	23	07	02	15.101N	88.782W	76	420MB	072	0		0	006*	345
CGS-B	22	03	67	09	15	29	12.653N	87.823W	70	440MB	074	0		0	025	379
CGS-B	25	03	67	07	26	37	12.406N	87.876W	76	400MB	074	0		0	005*	312
CGS-B	27	C3	67	14	40	13	12.221N	87.712W	54	420MB	074	0		0	019	345
CGS-B	12	04	67	04	56	25	12.217N	88.102W	33	480MB	076	0		0	N025	445
CGS-B	15	04	67	22	10	08	12.743N	88.163W	33	430MB	076	0		0	N006*	362
CGS-B	18	04	67	08	24	17	11.419N	88.415W	33	400MB	076	0		0	N007*	312
CGS-B	18	04	67	08	29	12	11.751N	87.307W	65	460MB	074	0		0	028	412
CGS-B	18	04	67	07	05	08	10.749N	86.777W	53	480MB	077	0		0	041	445
CGS-B	18	04	67	13	05	15	12.765N	87.943W	84	390MB	074	0		0	007*	300
CGS-B	23	04	67	14	44	10	11.525N	86.480W	82	440MB	074	0		0	014	379
CGS-B	30	04	67	03	10	49	12.632N	86.852W	115	420MB	075	0		0	009	345
CGS-B	08	05	67	14	40	08	12.974N	88.073W	33	460MB	073	0	F	0	N030	412
CGS-B	13	05	67	12	48	00	09.247N	82.395W	68	400MB	080	0		0	008*	312
CGS-B	20	05	67	23	42	19	11.653N	86.714W	33	420MB	074	0		0	N012*	345
CGS-B	23	05	67	17	14	50	10.197N	84.022W	119	430MB	078	0		0	008*	362
CGSPDE	23	06	67	13	30	46	12.930N	87.800W	11	420MB	074	0		0	012	345
CGSPDE	25	06	67	22	52	12	10.700N	86.000W	33	400MB	077	0		0	007	312
CGSPDE	30	06	67	11	42	33	11.000N	86.600W	44	450MB	074	0		0	022	395
CGSPDE	04	07	67	06	12	57	10.800N	86.400W	33	410MB	077	0		0	016	329
CGSPDE	04	07	67	13	29	04	11.600N	87.200W	33	420MB	074	0		0	010	345
CGSPDE	04	07	67	07	39	20	11.800N	87.300W	38	400MB	074	0		0	016	312
CGSPDE	14	07	67	18	02	08	13.500N	88.800W	147	460MB	073	0		0	014	412
CGSPDE	11	08	67	12	26	18	11.800N	85.900W	21	470MB	075	0		0	017	428
CGSPDE	27	08	67	13	08	55	12.300N	86.200W	183	520MB	075	0		0	060	511
CGSPDE	02	10	67	15	59	43	11.700N	86.800W	39	470MB	074	0		0	025	428
CGSPDE	03	10	67	19	22	39	10.400N	86.200W	33	450MB	077	0		0	019	395
CGSPDE	C3	10	67	18	16	03	10.900N	85.900W	21	580MB	078	0		0	104	611
CGSPDE	04	10	67	00	12	12	15.700N	88.600W	33	440MB	072	0		0	010	379
CGSPDE	04	10	67	06	02	16	10.700N	86.000W	33	530MB	077	0		0	066	528
CGSPDE	08	10	67	14	28	07	11.300N	86.000W	33	420MB	074	0		0	009	345
CGSPDE	15	10	67	08	00	50	11.900N	86.000W	162	620MB	074	0		0	083	677
CGSPDE	06	11	67	18	49	26	13.500N	88.000W	33	430MB	073	0		0	010	362
CGSPDE	13	11	67	16	05	46	10.400N	85.700W	18	460MB	078	0		0	019	412
CGSPDE	06	12	67	02	53	05	12.530N	87.200W	87	530MB	074	0		0	057	528
D&M 369	24	5	68				12.50	86.50	79	5.1					***	5.1
CGSPDE	04	01	68	10	03	56	12.100N	86.300W	5	460MB	075	0		0	028	412
CGSPDE	26	01	68	10	37	56	12.700N	88.800W	60	390MB	076	0		0	007	300
CGSPDE	28	01	68	08	47	20	12.600N	87.900W	75	440MB	074	0		0	022	379
CGSPDE	28	01	68	18	20	49	12.100N	86.900W	152	440MB	075	0		0	015	379
CGSPDE	01	03	68	16	34	44	11.700N	85.500W	190	480MB	075	0		0	033	445
CGSPDE	03	03	68	08	43	24	10.000N	83.100W	16	410MB	078	0		0	010	329
CGSPDE	14	03	68	10	09	19	12.000N	86.800W	25	450MB	075	0		0	013	395
CGSPDE	24	03	68	17	13	20	12.530N	86.500W	79	510MB	075	0		0	043	495
CGSPDE	01	04	68	07	28	28	12.400N	88.500W	33	440MB	076	0		0	013	379
CGSPDE	C3	04	68	00	06	00	12.200N	88.300W	33	430MB	076	0		0	011	362
CCSPDE	18	04	68	04	40	38	12.500N	88.600W	54	490MB	076	0		0	019	462
CGSPDE	28	04	68	10	03	31	11.800N	88.800W	39	490MB	076	0		0	039	462
CGSPDE	06	05	68	21	23	11	13.021N	87.734W	56	460MB	072	0		0	0024*	412
CCSPDE	11	06	68	05	52	33	13.941N	88.761W	199	530MB	073	0		0	0066	528
CGSPDE	02	07	68	16	15	48	09.045N	82.934W	33	420MB	080	0		0	N007*	345
CGSPDE	04	07	68	15	15	59	10.464N	84.056W	113	440MB	078	0		0	012	379
CGSPDE	C7	C7	68	19	49	36	14.678N	88.939W	134	450MB	072	0		0	017*	395
CGSPDE	14	07	68	03	55	24	15.237N	88.843W	14	450MB	072	0	F	0	033	395

CGSPDE	17	07	68	06	23	11	10.394N	83.389W	19	510MB		078	0		0	0052	495
CGSPDE	20	08	68	00	34	04	13.105N	87.833W	157	430MB		072	0		0	014	362
CGS 74	15	09	68	04	32	04	13.466N	87.243W	164	390MB		072	0		0	009*	300
CGSPDE	19	09	68	00	37	17	12.143N	88.714W	33	440MB		076	0		0	N014*	379
CGS 77	16	10	68	06	08	45	13.893N	88.250W	16	430MB	4	073	0	F	0	009	362
CGS 80	22	10	68	06	42	01	13.208N	88.247W	64	470MB	5	073	0		0	017	428
D&M 398	24	10	68				12.52	86.73	N	C V						***	611
CGS 92	16	11	68	15	14	37	11.082N	86.911W	33	460MB		074	39MS		0	N018	390
CGS 94	19	11	68	00	09	27	11.336N	86.714W	33	460MB		074	41MS		0	N022	410
CGS 89	21	11	68	07	19	34	11.203N	86.891W	33	450MB		074	0		0	N016	395
CGS 9C	28	11	68	13	44	33	12.451N	88.006W	49	490MB		076	0		0	021	462
D&M 393	28	12	68				12.10	86.90	152	4.4						***	4.4
D&M 413	24	9	69				11.66	87.71	100	5.3 CFL						***	5.3
CGS 3	06	01	69	09	24	23	11.440N	87.245W	33	480MB		074	43MS		0	N026	430
CGS 3	06	01	69	09	44	41	11.410N	87.232W	33	430MB		074	0		0	011	362
CGS 4	15	01	69	06	57	22	11.308N	86.813W	72	450MB		074	0		0	010	395
CGS 16	08	02	69	04	56	59	13.624N	88.271W	33	430MB	4	073	0	F	0	N006	362
CGS 15	25	02	69	07	39	00	15.231N	87.466W	15	540MB		072	50MS	F	0	G072	500
CGS 15	25	02	69	02	01	44	15.274N	87.544W	13	480MB		072	0		0	G036	445
CGS 19	09	03	69	07	12	39	13.101N	88.528W	75	440MB		073	0		0	021	379
CGS 2C	10	03	69	08	15	08	12.349N	87.455W	62	530MB		074	0		600BRK	059	600
CGS 21	14	03	69	08	47	16	12.916N	86.755W	178	560MB	4	075	0	F	580PAS	D109	580
CGS 23	19	03	69	19	49	22	11.786N	88.087W	23	470MB		076	0		0	021	428
CGS 25	01	04	69	21	19	53	12.677N	88.233W	71	490MB	4	076	0	F	0	038	462
CGS 31	04	04	69	08	43	06	13.534N	88.981W	105	430MB		073	0		0	008*	362
CGS 26	08	04	69	19	41	58	12.670N	88.340W	76	480MB		076	0		0	023	445
CGS 32	24	04	69	22	29	46	11.539N	87.022W	50	440MB		074	0		0	022	379
CGS 33	03	05	69	03	19	06	12.987N	88.449W	77	450MB	5	073	0	F	0	032	395
CGS 38	13	05	69	18	11	39	11.523N	86.509W	72	460MB		074	0		0	037	412
CGS 38	13	05	69	14	16	52	11.469N	86.355W	79	560MB		074	0	F	570BRK	104	570
CGS 38	13	05	69	14	15	52	11.408N	86.369W	71	480MB		074	0		0	037	445
CGS 38	13	05	69	16	39	27	12.447N	87.830W	33	420MB		074	0		0	N007*	345
CGS 39	13	05	69	14	36	16	10.817N	86.338W	33	450MB		077	0		0	N008*	395
CGS 36	13	05	69	17	38	28	11.784N	86.179W	46	490MB		074	52MS		400BRK	042	520
CGS 38	14	05	69	13	49	03	11.518N	86.154W	62	480MB		074	0		0	046	445
CGS 40	06	06	69	16	16	01	12.467N	88.000W	48	500MB	3	076	45MS	F	0	051	450
CGS 41	23	06	69	20	46	40	12.410N	88.052W	33	470MB		076	0		0	N033	428
CGS 42	24	06	69	00	35	05	11.661N	85.717W	100	530MB		075	0		500PAL	G085	500
CGS 46	11	07	69	05	57	12	10.189N	85.484W	30	470MB		078	0	F	0	025	428
CGS 45	11	07	69	07	51	04	10.342N	85.369W	33	500MB		078	0	F	450BRK	N049	450
CGS 51	17	07	69	13	24	22	12.590N	87.006W	175	440MB		074	0		0	G012	379
CGS 48	26	07	69	07	21	00	12.625N	87.798W	84	480MB		074	0		0	039	445
CGS 55	03	08	69	21	17	16	13.342N	88.342W	52	430MB		073	0		0	010*	362
CGS 54	15	08	69	01	43	11	09.473N	83.863W	9	440MB		078	0		0	022	379
CGS 59	04	09	69	06	25	28	13.216N	88.429W	53	460MB	4	073	0	F	0	020	412
CGS 65	01	10	69	08	41	31	12.084N	88.358W	33	440MB	3	076	0	F	0	N011	379
CGS 8	15	01	70	16	57	47	11.614N	86.929W	33	480MB		074	0		0	N008*	445
CGS 8	15	01	70	16	52	42	11.512N	86.733W	70	510MB	3	074	0	F	575BRK	046	575
CGS 8	15	01	70	19	08	59	11.157N	86.015W	33	460MB		074	0		0	N012*	412
CGS 7	20	01	70	08	27	48	11.482N	86.353W	46	540MB		074	0		0	063	545
CGS 11	29	01	70	23	06	38	11.492N	86.086W	215	480MB		074	0		0	G018*	445
CGS 11	31	01	70	22	23	37	15.727N	88.750W	46	500MB		072	45MS		0	060	450
CGS 12	12	02	70	17	22	11	12.139N	86.616W	44	440MB		075	0		0	013	379
CGS 23	18	03	70	23	17	57	10.984N	86.015W	56	490MB		074	0		0	037	462
CGS 26	20	03	70	21	00	08	10.428N	87.364W	34	440MB		077	43MS		0	013*	430
CGS 22	27	03	70	18	39	18	12.631N	86.823W	182	510MB		075	0		0	031	495
CGS 31	16	04	70	05	04	41	09.602N	84.240W	50	430MB		078	0		0	G014*	362
CGS 34	21	04	70	05	37	08	12.947N	87.880W	79	520MB		074	0		0	045	511
CGS 37	25	05	70	23	05	18	10.738N	86.939W	33	480MB		077	0		0	N014	445
CGS 48	05	07	70	05	42	05	12.561N	87.446W	33	450MB		074	0		0	N031	395

CGS	54	20	07	70	04	58	05	10.366N	87.400W	33	460MB		077	0		0	N015*	412
CGS	53	05	08	70	13	22	05	09.841N	84.317W	56	460MB		078	0		0	023	412
CGS	54	07	08	70	13	03	14	12.325N	87.791W	59	450MB		074	0		0	016	395
CGS	55	12	08	70	11	07	59	10.867N	87.229W	33	450MB		077	0		0	N009*	395
CGS	55	12	08	70	10	24	23	12.051N	86.539W	33	560MB		075	55MS		530PAS	N057	550
CGS	55	12	08	70	09	24	11	12.015N	86.542W	33	590MB		075	63MS	F	600PAS	N077	630
CGS	62	02	09	70	17	51	35	10.432N	86.015W	33	430MB		077	0		0	N009*	362
CGS	62	06	09	70	20	44	49	12.783N	88.595W	75	450MB		076	0	F	0	017	395
CGS	71	23	09	70	23	57	45	12.021N	87.446W	70	450MB		074	0		0	029	395
CGS	67	29	09	70	04	42	46	11.473N	85.479W	192	540MB		075	0		540BRK	D132	540
CGS	68	06	10	70	15	18	54	13.561N	88.312W	192	460MB		073	0		0	018*	412
NOS	75	03	11	70	08	39	12	11.644N	86.112W	100	460MB		074	0		0	D023	412
NOS	80	25	11	70	05	38	12	11.765N	87.920W	36	480MB		074	0		0	014*	445
D&M	428	3	2	71				12.50	86.70	N	D	V					***	581
D&M	431	3	5	71				12.70	87.00	N	D	V					***	581
NOS	9	11	01	71	17	13	48	11.714N	87.794W	33	480MB		074	0	F	0	N014*	445
NOS	16	19	02	71	16	59	40	13.562N	88.795W	176	520MB	3	073	0	F	0	D042	511
NOS	21	25	02	71	04	15	41	12.159N	87.489W	52	530MB		074	0		0	075	528
NOS	30	26	04	71	18	32	20	11.422N	86.760W	69	470MB		074	0		0	040	428
NOS	31	01	05	71	14	32	12	13.259N	88.499W	93	540MB	5	073	0	F	0	053	545
NOS	41	05	06	71	14	20	42	09.339N	84.232W	26	540MB	3	T078	51MS	F	0	082	510
NOS	44	10	06	71	11	20	31	11.101N	86.766W	60	490MB		074	0		0	015*	462
NOS	41	11	06	71	11	51	40	12.647N	86.967W	148	450MB		074	0		0	024*	395
NOS	45	16	06	71	04	18	56	11.722N	88.222W	33	450MB		076	0		0	N012*	395
NOS	45	18	06	71	13	36	01	14.678N	87.567W	7	500MB		072	0		0	022	478
ERL	60	05	08	71	22	58	08	09.641N	84.069W	33	490MB		078	0		0	N019	462
ERL	60	12	08	71	05	59	00	12.877N	87.341W	211	470MB		074	0		0	026	428
ERL	77	24	08	71	08	09	07	11.509N	86.323W	33	510MB		074	42MS		0	N049	420
ERL	66	27	08	71	09	21	57	11.839N	88.884W	33	500MB		076	38MS		0	N041	380
ERL	73	29	08	71	08	26	52	11.547N	86.180W	33	490MB		074	0		0	N034	462
ERL	71	13	09	71	15	33	27	09.973N	87.361W	33	440MB		077	0		0	N007	379
ERL	77	28	09	71	05	47	16	09.821N	86.267W	33	500MB		077	0		0	N036*	478
ERL	81	14	10	71	06	59	10	10.824N	86.268W	33	470MB		077	40MS		0	N020*	400
ERL	81	08	11	71	04	26	16	12.472N	88.165W	68	490MB		076	0		0	031	462
ERL	85	25	11	71	22	06	26	12.759N	87.458W	62	490MB		074	0		0	024	462
ERL	85	26	11	71	04	06	38	12.148N	87.626W	47	440MB		074	0		0	013	379
D&M	438	1	12	71				12.60	86.90	N	D	V					***	585
ERL	1	03	01	72	04	31	04	12.311N	86.300W	33	410MB		075	0		0	N019*	329
ERL	2	05	01	72	11	52	26	12.286N	86.236W	33	420MB		075	0		0	N019*	345
ERL	8	16	01	72	23	55	10	11.505N	87.582W	62	460MB		074	0		0	014	412
ERL	9	30	01	72	22	25	03	11.082N	86.750W	43	450MB		074	38MS		0	017	380
ERL	7	30	01	72	19	10	07	11.261N	86.773W	48	460MB		074	44MS		0	027	440
ERL	9	04	02	72	01	07	43	11.213N	86.891W	52	480MB		074	0		0	019	445
ERL	10	06	02	72	10	05	31	10.592N	86.400W	36	440MB		077	0		0	008*	379
ERL	11	17	02	72	09	19	06	13.269N	88.691W	80	470MB	5	073	0	F	0	031	428
ERL	15	19	02	72	19	41	33	10.674N	86.085W	33	470MB		077	0		0	N014*	428
ERL	11	25	02	72	03	43	47	10.137N	83.414W	88	460MB		078	0		0	019*	412
ERL	14	04	03	72	22	04	23	12.933N	88.743W	64	470MB	4	076	0	F	0	024	428
ERL	16	05	03	72	21	47	26	11.971N	87.914W	46	460MB	3	074	0	F	0	020*	412
ERL	32	30	04	72	10	55	19	12.896N	88.668W	77	440MB	3	076	0	F	0	019	379
ERL	40	27	06	72	14	45	32	12.182N	86.588W	104	460MB		075	0		0	014*	412
ERL	45	28	07	72	13	53	40	13.151N	87.753W	209	440MB		072	0		0	016*	379
ERL	51	31	08	72	02	24	09	13.272N	88.513W	50	540MB		073	48MS		0	068	480
ERL	58	26	09	72	03	20	16	12.231N	87.412W	37	460MB		074	38MS		0	016*	380
ERL	61	01	10	72	06	00	51	11.015N	86.330W	49	420MB		074	0		0	006*	345
ERL	65	02	11	72	18	24	45	11.711N	87.939W	24	460MB		074	0		0	020*	412
ERL	66	19	11	72	04	35	24	12.973N	88.524W	73	560MB		073	0		0	065	578
ERL	71	11	12	72	12	51	04	11.403N	87.151W	27	440MB		074	0		0	D017*	379
ERL	73	14	12	72	03	55	41	11.858N	86.574W	104	460MB		074	0		0	D019*	412
ERL	77	23	12	72	07	17	36	12.050N	86.378W	5	500MB		075	0		0	G029	478

ERL	76	23	12	72	06	29	42	12.357N	86.119W	5	560MB		075	62MS		CD620PAS	G099	620
EPL	77	23	12	72	07	19	49	12.000N	86.189W	5	520MB		075	0		0	G049	511
ERL	74	25	12	72	00	26	28	15.215N	88.943W	34	460MB		072	0		0	020	412
DEM	457	13	4	73				10.50	85.50	N	6.7						***	6.7
ERL	12	15	02	73	00	15	53	11.534N	86.473W	33	510MB		074	50MS		0	0053	500
ERL	13	23	02	73	19	36	48	09.845N	83.493W	33	470MB		078	42MS		0	N038	420
ERL	13	26	02	73	00	43	39	09.606N	84.229W	33	470MB		078	40MS	F	0	N031	400
ERL	15	07	03	73	13	00	45	12.705N	88.027W	64	460MB		076	0		0	013*	412
ERL	22	09	04	73	03	17	48	12.257N	86.553W	167	470MB		075	0		0	035	428
ERL	36	14	04	73	08	34	00	10.679N	84.759W	33	570MB		078	65MS		CD650PAS	N117*	650
ERL	36	14	04	73	08	54	29	10.711N	84.763W	33	480MB		078	0		0	N034*	445
ERL	36	14	04	73	09	19	09	10.260N	85.015W	33	430MB		078	0		0	N006*	362
ERL	30	07	05	73	11	54	36	12.344N	87.949W	33	430MB		074	40MS		0	N013	400
ERL	35	28	05	73	06	19	00	11.474N	87.266W	42	460MB		074	40MS		0	014*	400
ERL	42	05	07	73	08	08	07	11.949N	87.541W	48	430MB		074	0		0	015	362
ERL	40	06	07	73	05	21	49	11.962N	87.588W	51	480MB		074	0		0	023	445
ERL	41	07	07	73	07	07	55	11.842N	87.761W	55	430MB		074	0		0	015	362
GS	52	14	07	73	07	55	08	11.637N	87.291W	46	490MB		074	44MS		0	024*	440
GS	51	27	07	73	08	51	11	12.415N	87.407W	33	450MB		074	0		0	N017*	395
ERL	49	27	07	73	19	42	47	12.813N	86.674W	199	530MB	3	075	0	F	0	D111	528
ERL	47	28	07	73	20	48	24	12.333N	87.084W	79	540MB		074	0	F	0	D087	545
ERL	47	04	08	73	00	44	42	09.838N	84.586W	33	530MB		078	54MS	F	520BRK	N083	540
ERL	47	04	08	73	11	54	16	12.140N	88.983W	33	440MB		076	0		0	N008*	379
ERL	49	19	08	73	22	17	49	12.651N	88.507W	68	510MB	4	076	0	F	0	D033	495

APPENDIX 4

COMPUTER PROGRAM LISTING

```
C       ******************************************************************
C       PROGRAM GENER.MAG                                STANFORD UNIVERSITY
C       ******************************************************************
C       THIS PROGRAM REPLACES THE LETTERS EARTHQUAKE RICHTER MAGNITUDES
C       A,B,C,D,E (FROM THE DAMES & MOORE NICARAGUA EARTHQUAKE CATALOG)
C       BY A RICHTER MAGNITUDE OBTAINED THROUGH RANDOM NUMBER GENERATION.
C       ******************************************************************
C       INPUT FORMAT
C       ************
C
C        COL      VARIABLE NAME      VARIABLE DESCRIPTION
C
C       1.- EARTHQUAKE RECORD CARD    1 CARD PER RECORD    (10A4)
C        1-36     XMAG(1-9)          COL 1/36 READ IN A FORMAT
C       37-40     XMAG(10)           LETTER EARTHQUAKE MAGNITUDE
C
C       OUTPUT
C       ******
C        THE OUTPUT IS SAVED ON DISK, IT CONTAINS THE SAME INFORMATION AS
C       THE INPUT, BUT THE LETTER MAGNITUDE HAS BEEN REPLACED BY THE
C       RICHTER MAGNITUDE.
C
C       ******************************************************************
        DATA B,C,D,E/'   B','   C','   D','   E'/
        DIMENSION XMAG(10)
C       ******************************************************************
C
        IY=1
        CALL RANDK(IY,YFL,0)
        DO 100 I=1,1000
        READ(5,1000,END=99)XMAG
  1000  FORMAT(10A4)
        CALL RANDK(IY,YFL,0)
C
        IF(XMAG(9).EQ.E) GO TO 110
        IF(XMAG(9).EQ.D) GO TO 120
        IF(XMAG(9).EQ.C) GO TO 130
        IF(XMAG(9).EQ.B) GO TO 140
C
        WRITE(10,2000)XMAG,XMAG(9)
  2000  FORMAT(10A4,31X,A4)
        GO TO 100
C
   110  MAG=(3.5+YFL*1.8)*100.
        GO TO 109
C
   120  MAG=(5.3+YFL*.7)*100.
        GO TO 109
C
   130  MAG=(6.0+YFL)*100.
        GO TO 109
C
   140  MAG=(7.0+YFL*.7)*100.
   109  WRITE(10,2020)XMAG,MAG
  2020  FORMAT(10A4,31X,I4)
   100  CONTINUE
    99  RETURN
        END
        SUBROUTINE RANDK (IY,YFL,INDEX)
C
C       THIS SUBROUTINE GENERATES RANDOM NUMBERS
C
        IY=IY*314159269+453806245
     4  IF(IY.GE.0) GO TO 6
     5  IY=IY+2147483647+1
     6  CONTINUE
        IF(INDEX.GT.0) GO TO 8
        YFL=FLOAT(IY)*.4656613E-9
     8  RETURN
        END
$DATA
```

```
C     *****************************************************************
C     PROGRAM SORT.DEPTH                                STANFORD UNIVERSITY
C     *****************************************************************
C     THIS PROGRAM SORTS EARTHQUAKES BY DEPTH.  THE BAND THICKNESS IS SET
C     SET TO 10 KM.
C     *****************************************************************
C     INPUT FORMAT
C     ************
C     THE DATA IS READ FROM DISK.  ONE CARD PER EVENT.  (8A4,I3,11A4)
C
C      COL      VARIABLE NAME     VARIABLE DESCRIPTION
C
C      1-32     XIN1              COL 1/32 READ IN A FORMAT
C     33-35     NDP               DEPTH OF HYPOCENTER
C     36-79     XIN3              COL 36/79 READ IN A FORMAT
C
C     OUTPUT
C     ******
C     THE OUTPUT IS SAVED ON DISK AND CONTAINS THE DATA SORTED BY
C     BANDWITH OF 10 KM STARTING AT DEPTH 0
C
C     *****************************************************************
      DIMENSION XINF1(21,180,8),NDEP(21,180)
     *, XINF3(21,180,11),IND(21),XIN1(8),XIN3(11)
C     *****************************************************************
C
      DO 130 I=1,21
  130 IND(I)=0
C
  100 READ(5,1000,END=99)XIN1,NDP,XIN3
 1000 FORMAT(8A4,I3,11A4)
      N=(NDP/10)+1
      IF(N.GT.21)N=21
      IND(N)=IND(N)+1
      NED=IND(N)
      NDEP(N,NED)=NDP
      DO 400 J=1,8
  400 XINF1(N,NED,J)=XIN1(J)
      DO 500 J=1,11
  500 XINF3(N,NED,J)=XIN3(J)
      GO TO 100
C
   99 CONTINUE
      DO 200 N=1,21
      NDP=(N-1)*10
      NED=IND(N)
      IF(NED.EQ.0) GO TO 210
      WRITE(11,2010) NED,NDP
 2010 FORMAT(I5, 'EQ AT DEPTH 'I5,'  PLUS')
      PRINT,NED,'  EQ AT DEPTH',NDP,'  PLUS'
      DO 220 I=1,NED
      WRITE(11,2000)(XINF1(N,I,J),J=1,8),NDEP(N,I),
     1   (XINF3(N,I,J),J=1,11)
 2000 FORMAT(' '8A4,I3,11A4)
  220 CONTINUE
      PRINT,'   '
      PRINT,'    '
      GO TO 200
  210 PRINT,'NO EQ AT DEPTH',NDP,'  PLUS'
C
  200 CONTINUE
      RETURN
      END
$DATA
```

```
C     ******************************************************************
C     PROGRAM LOCATE.LINE                              STANFORD UNIVERSITY
C     ******************************************************************
C     THIS PROGRAM FITS A LINE THROUGH A SET OF POINTS IN A PLANE USING
C     REGRESSION ANALYSIS.  IT IS USED TO LOCATE A SEISMIC LINE SOURCE
C     GIVEN A NUMBER OF EPICENTERS.
C     ******************************************************************
C     INPUT FORMAT
C     ************
C
C      COL      VARIABLE NAME     VARIABLE DESCRIPTION
C
C     1.- NUMBER OF SOURCES   1 CARD   (I5)
C      1- 5     NBST                NB OF SOURCES TO BE LOCATED
C
C     2.- SOURCE IDENTIFICATION   1 CARD   (I5,18A4)
C      1- 5     NBRC                NB OF RECCORDS IN THE SOURCE
C      6-77     HED1                IDENTIFICATION OF THE SOURCE
C
C     3.- RECORD CARDS   NBRC CARDS   (4F10.0)
C      1-10     Y( )                LATITUDE OF THE EPICENTER
C     11-20     X( )                LONGITUDE OF THE EPICENTER
C     21-30     D( )                DEPTH OF THE HYPOCENTER
C     31-40     XM( )               RICHTER MAGNITUDE
C
C     REPEAT CARDS 2/3 NBST TIMES FOR THE NBST SOURCES
C
C     OUTPUT
C     ******
C     THE OUTPUT DISPLAYS THE SOURCE IDENTIFICATION, THE NUMBER OF
C     RECORDS FOR IN THE SOURCE, THE COORDINATES OF THE CENTROID
C     OF THE DATA AS WELL AS THE SLOPE AND THE DEPTH OF THE LINE.
C
C     ******************************************************************
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION HED1(18),X(200),Y(200),D(200),XM(200)
      REAL*4 HED1
C     ******************************************************************
C
      READ(5,1000)NBST
      DO 100 IS=1,NBST
      XB=0.D0
      YB=0.D0
      DB=0.D0
      Y2=0.D0
      X2=0.D0
      XY=0.D0
      XT=0.D0
      YT=0.D0
      XYT=0.D0
      READ(5,1000)NBRC,HED1
 1000 FORMAT(I5,18A4)
      WRITE(6,2000)NBRC,HED1
 2000 FORMAT('ONUMBER OF RECORDS',I6,3X,18A4)
      NBZE=0
      DO 110 IR=1,NBRC
      READ(5,1010)Y(IR),X(IR),D(IR),XM(IR)
 1010 FORMAT(19X,F5.3,3X,F5.3,14X,F3.0,25X,F3.2)
      X(IR)=-X(IR)
      XB=XB+X(IR)
      YB=YB+Y(IR)
      DB=DB+D(IR)
      IF(D(IR).EQ.0.0)NBZE=NBZE+1
      Y2=Y2+Y(IR)*Y(IR)
      X2=X2+X(IR)*X(IR)
      XY=XY+X(IR)*Y(IR)
  110 CONTINUE
      XNBRC1=NBRC-NBZE
      XNBRC=NBRC
      XB=XB/XNBRC
      YB=YB/XNBRC
      DB=DB/XNBRC1
      ALPH=(XY-XNBRC*XB*YB)/(X2-XNBRC*XB*XB)
      ALPH1=(XY-XNBRC*XB*YB)/(Y2-XNBRC*YB*YB)
      PRINT,'LINE SOURCE'
      PRINT,'X,Y,SLOPE',XB,YB,ALPH,ALPH1,DB
      PRINT,'NUMBER OF NO DEPTH',NBZE
      DO 130 IR=1,NBRC
      X(IR)=X(IR)-XB
      Y(IR)=Y(IR)-YB
      XT=XT+X(IR)*X(IR)
      YT=YT+Y(IR)*Y(IR)
      XYT=XYT+X(IR)*Y(IR)
  130 CONTINUE
      ALPH=XYT/XT
      ALPH1=XYT/YT
      PRINT,ALPH,ALPH1
  100 CONTINUE
      RETURN
      END
$DATA
```

```
C      *******************************************************************
C      PROGRAM LINE.INTER                              STANFORD UNIVERSITY
C      *******************************************************************
C      THIS PROGRAM COMPUTES THE INTERSECTION OF TWO LINES IN THE SAME
C      PLANE.  IT IS USED TO OBTAIN THE INTERSECTION OF TWO LINE SOURCES
C      AT THE SAME DEPTH.
C      *******************************************************************
C      INPUT FORMAT
C      ************
C
C       COL      VARIABLE NAME     VARIABLE DESCRIPTION
C
C      1.- IDENTIFICATION   1 CARD   (20A4)
C       1-80     HED1               IDENTIFICATION OF THE FIRST LINE
C
C      2.- LINE PARAMTETERS   1 CARD   (3F10.0)
C       1-10     X1                 X COORDINATE OF A POINT OF THE LINE
C      11-20     Y1                 Y COORDINATE OF THE SAME POINT
C      21-30     AL1                SLOPE OF THE LINE
C
C      3.- IDENTIFICATION    1 CARD    (20A4)
C       1-80     HED2               IDENTIFICATION OF THE SECOND LINE
C
C      2.- LINE PARAMETERS   1 CARD    (3F10.0)
C       1-10     X2                 X COORDINATE OF A POINT OF THE LINE
C      11-20     Y2                 Y COORDINATE OF THE SAME POINT
C      21-30     AL2                SLOPE OF THE LINE
C
C      OUTPUT
C      ******
C      THE OUTPUT DISPLAYS THE IDENTIFICATION OF THE LINES AND
C      THEIR THE COORDINATES OF THEIR INTERSECTION POINT.
C
C      *******************************************************************
       DIMENSION HED1(20),HED2(20)
C      *******************************************************************
C
  100  READ(5,1000,END=99)HED1
       READ(5,1001)X1,Y1,AL1
       DO 200 I=1,2
       READ(5,1000)HED2
       READ(5,1001)X2,Y2,AL2
 1000  FORMAT(20A4)
 1001  FORMAT(8F10.0)
C
       X=(AL1*X1-AL2*X2-Y1+Y2)/(AL1-AL2)
       IF(Y2.EQ.0.) X=X2
       Y=AL1*(X-X1)+Y1
C
       WRITE(6,2000)HED1,HED2
 2000  FORMAT(' '20A4)
       PRINT,'LINE1',X1,Y1,AL1
       PRINT,'LINE2',X2,Y2,AL2
       PRINT,'INTERSECTION',X,Y
C
  200  CONTINUE
       GO TO 100
   99  RETURN
       END
```

$DATA

```
C      ******************************************************************
C      PROGRAM LINE.LENGTH                              STANFORD UNIVERSITY
C      ******************************************************************
C      THIS PROGRAM COMPUTES THE LENGTH OF A SEGMENT OF LINE BETWEEN
C      TWO HORIZONTAL COORDINATES.
C      ******************************************************************
C      INPUT FORMAT
C      ************
C
C       CCL      VARIABLE NAME      VARIABLE DESCRIPTION
C
C      1.- IDENTIFICATION   1 CARD   (20A4)
C       1-80     HED1                 LINE IDENTIFICATION
C
C      2.- LINE PARAMETERS   1 CARD   (5F1G.C)
C       1-10     X1                   X COORDINATE OF A POINT OF THE LINE
C      11-20     Y1                   Y COORDINATE OF THE SAME PCINT
C      21-30     ALPH                 SLOPE OF THE LINE
C      31-40     XS                   X COORD OF ORIGIN CF SEGMENT
C      41-50     YX                   X COORD OF END OF SEGMENT
C
C      OUTPUT
C      ******
C      THE OUTPUT DISPLAYS THE LINE IDENTIFICATION AND PARAMENTERS AS
C      AS WELL AS THE LENGTH OF THE SEGMENT.
C
C      ******************************************************************
       DIMENSION HED1(20)
C      ******************************************************************
C
  100 READ(5,1001,END=99)HED1
 1001 FORMAT(20A4)
      READ(5,1000)X1,Y1,ALPH,XS,XE
 1000 FORMAT(8F10.0)
      YS=ALPH*(XS-X1)+Y1
      YE=ALPH*(XE-X1)+Y1
      XLEN=SCRT((XS-XE)*(XS-XE)+(YS-YE)*(YS-YE))
      WRITE(6,2000)HED1
 2000 FCRMAT('0',20A4)
      PRINT,'DATA',X1,Y1,ALPH
      PRINT,'ORIGIN  END  LENTGH  ',XS,YS,XE,YE,XLEN
      GC TO 100
   99 RETURN
      END
$DATA
```

```
C     ******************************************************************
C     REGRESSION ANALYSIS                              STANFORD UNIVERSITY
C     ******************************************************************
C     THIS PROGRAM IS TO BE USED FOR REGRESSION ANALYSIS BETWEEN
C     ONE DEPENDENT VARIABLE AND ONE INDEPENDENT VARIABLE.
C     THE LINEAR SCALE IS USED FOR THE INDEPENDENT VARIABLE (X) AND THE
C     LN SCALE FOR THE DEPENDENT VARIABLE (Y).
C     BOTH VARIABLES ARE INPUT ON THE LINEAR SCALE
C     IF THE DEPENDENT VARIABLE IS NOT DETERMINED, THE PROGRAM WILL
C     COMPUTE IT AS (1-CDF) USING A INCREMENT RMIC OF THE DEPENDENT VARIABLE
C     IT CAN EITHER FIT ONE OR TWO LINES ON THE DATA WITH A BREAKING
C     POINT INPUT AS RMBK
C
C     INPUT FORMAT
C     ************
C
C      COL     VARIABLE    VARIABLE DESCRIPTION
C
C     1.-IDENTIFICATION CARD   1 CARD   (5F10.0,2I5,5A4)
C
C      1-10     RMMN         MINIMUM RM VALUE
C     11-20     RMIC         RM INCREMENT TO BE USED TO COMPUTE CDF
C     21-30     RMBK         BREAK OFF MAGNITUDE
C                            IF (RMBK.EQ.0.) ONLY ONE LINE WILL BE FIT
C     31-40     A            AREA
C     41-50     T            TIME
C                            A AND T ARE USED TO NORMALIZE ALPHA
C     51-55     SKIPCD       = 1  BOTH X AND Y ARE INPUT
C                                 RMMN,RMIC ARE INPUT AS ZEROS
C                            = 0  ONLY X IS INPUT (AS RM), Y IS COMPUTED FROM CDF
C                                 RMMN,RMIC ARE TO BE INPUT
C     56-60     SKIPDZ       = 0  INTERVALS WITH NO EVENT WILL NOT BE
C                                 CONSIDERED IN REGRESSION ANALYSIS
C                            = 1  ALL INTERVALS WILL BE CONSIDERED
C     61-80     HED          TITLE
C
C     2.- X AND Y INPUT CARDS  NBRC CARDS   (2F10.0)
C                            INPUT IF SKIPCD=1
C
C
C      1-10     X            INDEPENDENT VARIABLE
C     11-20     Y            DEPENDENT VARIABLE
C
C     3.- RM INPUT CARDS         NBRC CARDS   (50X,F3.1)
C                            SET 1002 FORMAT AS A FUNCTION OF DATA
C                            INPUT IF SKIPCD=0
C
C      1-10     RM           INDEPENDENT VARIABLE
C
C     4.-END OF FILE CARD (ONLY IF RM IS INPUT)   ONE CARD  (50X,F3.1)
C
C     51-53     RM           VALUE GREATER THAN 9.
C
C     NOTE
C
C     IF RM IS INPUT, SEVERAL FILES CAN BE RUN AT ONE TIME BY REPEATING
C     THE INPUT SEQUENCE DESCRIBED ABOVE.
C
C     OUTPUT
```

```
C     ******
C     THE OUTPUT DISPLAYS THE INPUT, THE INTERVAL FREQUENCY,THE
C     FREQUENCY OF OCCURRENCE ABOVE A GIVEN MAGNITUDE, THE REGRESSION
C     COEFFICIENTS ALPHA AND BETA AND ALPHA NORMALIZED WITH RESPECT
C     TO TIME AND AREA OR LENGTH.
C
C     ******************************************************************
      DIMENSION RM(2500),X(50),Y(50),YY(50),IHIS(50),HED(5),
     1 ALPHA(2),BETA(2)
      INTEGER SKIPCD,SKIPDZ
C     ******************************************************************
C
  111 READ(5,1000,END=99)RMMN,RMIC,RMBK,A,T,SKIPCD,SKIPDZ,HED
 1000 FORMAT(5F10.0,2I5,5A4)
      WRITE(6,2000)HED
 2000 FORMAT('1REGRESSION ANALYSIS'/'OLINEAR-LN SCALE'/'0'7A4//)
      IF(SKIPCD.EQ.0) GO TO 200
C
      NBIC=1
C
  100 READ(5,1001,END=104)X(NBIC),YY(NBIC)
      NBIC=NBIC+1
      GO TO 100
C
  104 NBIC=NBIC-1
 1001 FORMAT(2F10.0)
      WRITE(6,2002)NBIC
 2002 FORMAT('ONUMBER OF RECORDS'I5/'OINPUT VARIABLES'/
     *'OINDEPENDENT  DEPENDENT')
      DO 150 IX=1,NBIC
  150 WRITE(6,2011)X(IX),YY(IX)
 2011 FORMAT(' '2F11.2)
      GO TO 201
  200 CONTINUE
C
C     COMPUTE HISTOGRAM
C      CAUTION  FORMAT SET TO READ DATA FROM TAPE U599
C
      NBRC=0
      RMLW=10.
C
  112 NBRC=NBRC+1
  109 READ(5,1002)RM(NBRC)
 1002 FORMAT(74X,F3.2)
      RM(NBRC)=FLOAT(IFIX(RM(NBRC)*10.+.501))/10.
      IF(RM(NBRC).GE.9.) GO TO 105
      IF(RM(NBRC).LT.RMMN) GO TO 110
      IF(RM(NBRC).LT.RMLW) RMLW=RM(NBRC)
      GO TO 112
  110 WRITE(6,2009) NBRC,RM(NBRC)
 2009 FORMAT(' RECORD',I5,'  HAS MAGNITUDE',F10.2,'  IT IS DISREGARDED')
      GO TO 109
C
C
  105 NBRC=NBRC-1
      RMCK1=RMMN
      WRITE(6,2001)NBRC,A,T
 2001 FORMAT('ONUMBER OF RECORDS  ',I5/'OAREA',10X,F10.2,/
     *'OTIME (YEARS) ',F11.2)
      IF(NBRC.LT.2) GO TO 111
```

```
      WRITE(6,2112)RMMN,RMIC
 2112 FORMAT('OMINIMUM MAGNITUDE',12X,F10.2 /
     *'OMAGNITUDE INCREMENT FOR CDF  ',F10.2)
      DO 120 I=1,50
      RMCK=RMCK1+RMIC
      IF(RMLW.LT.(RMCK-.05)) GO TO 121
      RMCK1=RMCK
  120 WRITE(6,2220) RMMN,RMCK
 2220 FORMAT(' THERE IS NO RECORD BETWEEN MAGNITUDE',F8.2,' AND',F8.2)
C
  121 CONTINUE
      WRITE(6,2004)
 2004 FORMAT('OEARTHQUAKE MAGNITUDES')
      WRITE(6,2003)(RM(IX),IX=1,NBRC)
 2003 FORMAT(' '13F10.2)
      RMIC2=RMIC*.5
      X(1)=RMCK1+RMIC2
      NBIC=0
      DO 102 I=1,49
      X(I+1)=X(I)+RMIC
  102 IHIS(I)=0
C
      RMMN1=RMCK1-.1
      DO 103 IX=1,NBRC
      NBKC=((RM(IX)-RMMN1)/RMIC)+.999
      IHIS(NBKC)=IHIS(NBKC)+1
      IF(NBKC.GT.NBIC) NBIC=NBKC
  103 CONTINUE
C
      IF(SKIPDZ.EQ.1.OR.SKIPCD.EQ.1) GO TO 133
C
      I=0
C
  131 I=I+1
  134 IF(I.GE.NBIC) GO TO 133
      IF(IHIS(I).NE.0) GO TO 131
      NBIC1=NBIC-1
      DO 132 K=I,NBIC1
      X(K)=X(K+1)
  132 IHIS(K)=IHIS(K+1)
      NBIC=NBIC-1
      GO TO 134
C
  133 CONTINUE
      IF(NBIC.LT.2) GO TO 111
C     COMPUTE CDF
      YY(NBIC)=IHIS(NBIC)
      NBIC1=NBIC+1
      DO 130 IX=2,NBIC
      KX=NBIC1-IX
  130 YY(KX)=YY(KX+1)+FLOAT(IHIS(KX))
      WRITE(6,2005)
 2005 FORMAT('0',7X,'RM',10X,'INTERVAL',10X,'CUMULATIVE FREQUENCY' /
     *'        INTERVAL',5X,'FREQUENCY',10X,'OCCURRENCES ABOVE RM' /)
      DO 160 IX=1,NBIC
      RM1=X(IX)-RMIC2
      RM2=X(IX)+RMIC2-.01
  160 WRITE(6,2006)RM1,RM2,IHIS(IX),YY(IX),RM1
 2006 FORMAT(' 'F7.2,' - ',F4.2,I8,12X,F10.0,8X,F6.2)
C
  201 DO 202 IX=1,NBIC
  202 Y(IX)=ALOG(YY(IX))
C     CHECK WHETHER THERE WILL BE ONE OR TWO REGRESSION LINES
C
      IR=1
      NBPT=NBIC
      IF(RMBK.EQ.0.) GO TO 300
C
      NBPT=0.
      DO 320 IX=1,NBIC
      IF(X(IX).GT.RMBK) GO TO 340
  320 NBPT=NBPT+1
C
  340 NBPT1=NBIC-NBPT+1
      WRITE(6,2110)RMBK,NBPT,NBPT1
 2110 FORMAT('OTWO STRAIGHT LINES WILL BE USED TO FIT THE DATA'/
     1 ' BREAK POINT MAGNITUDE',F8.2,/' ',I5,' POINTS IN THE FIRST LINE'
     2 /' ',I5,' POINTS IN THE SECOND LINE' )
      GO TO 300
C
  330 NBPT1=NBPT-1
      NBPT=NBIC-NBPT1
      RMBK=0.
      IR=IR+1
      DO 350 IX=1,NBPT
      X(IX)=X(IX+NBPT1)
  350 Y(IX)=Y(IX+NBPT1)
C
  300 WRITE(6,2130)IR
 2130 FORMAT('OINTERCEPT AND SLOPE OF LINE',I5)
C
C     COMPUTE ALPHA  INTERSEPT
C     COMPUTE BETA  SLOPE
C
C     COMPUTE MEAN
      SMXX=0.
      SMXY=0.
      SMX=0.
      SMY=0.
      PRINT,'NBPT',NBPT
      WRITE(6,3333)(Y(I),I=1,NBPT)
      WRITE(6,3333)(X(I),I=1,NBPT)
 3333 FORMAT(' '13F10.5)
      DO 140 IX=1,NBPT
      SMXX=SMXX+X(IX)*X(IX)
      SMX=SMX+X(IX)
      SMXY=SMXY+X(IX)*Y(IX)
  140 SMY=SMY+Y(IX)
      XNBPT=NBPT
      YM=SMY/XNBPT
      XM=SMX/XNBPT
      BETA(IR)=(SMXY-XNBPT*YM*XM)/(SMXX-XNBPT*XM*XM)
      ALPHA(IR)=YM-BETA(IR)*XM
      WRITE(6,2007)ALPHA(IR),BETA(IR)
 2007 FORMAT('OALPHA',F10.6/'OBETA ',F10.6)
C
      IF(A.EQ.0..OR.T.EQ.0.) GO TO 360
      ALPHAN=ALPHA(IR)-ALOG(A*T)
      WRITE(6,2008)ALPHAN
 2008 FORMAT('ONORMALIZED ALPHA',F10.6 //)
```

```
  360 CONTINUE
C
      IF(RMBK.NE.0.) GO TO 330
      IF(IR.EQ.1) GO TO 111
C
C     COMPUTE INTERSECTION POINT IF THERE ARE TWO LINES
      XM=ABS((ALPHA(1)-ALPHA(2))/(BETA(1)-BETA(2)))
      XLN=ALPHA(1)+BETA(1)*XM
      WRITE(6,2120)XM,XLN
 2120 FORMAT('0INTERSECTION POINT'/' MAGNITUDE',F8.2,/' LN OF N   ',F8.2)
C
      GO TO 111
   99 RETURN
      END
$DATA
```

```
C      ****************************************************************
C      PROGRAM ACC.LINE.AREA                       STANFORD UNIVERSITY
C      ****************************************************************
C      THIS PROGRAM COMPUTES PROBABILITY OF EXCEEDANCE OF GIVEN PEAK
C      GROUND ACCELERATIONS AT SPECIFIED SITES DUE TO LINE OR AREA
C      SEISMIC SOURCES.
C      ****************************************************************
C      INPUT FORMAT
C      ************
C
C      1.- IDENTIFICATION CARD              1 CARD    20A4
C
C       1-80    HED1        TITLE
C
C      2.- ATTENUATION CONSTANTS            1 CARD    3F10.0
C
C       1-10    B1
C      11-20    B2
C      21-30    B3
C      31-40    B4
C      41-50    DELTAL      STEP SIZE FOR LINE INTEGRATION
C      51-60    DELTAC      STEP SIZE FOR CIRCLE INTEGRATION
C                           UNITS OF LENGTH USED IN THE PROGRAM
C               ATTENUATION FORMULA OF THE TYPE
C               ACC=B1*EXP(B2*MAG)/(R+B4)**B3
C
C      3.- PROBLEM DESCRIPTION              4 CARDS
C
C      CARD 1   4I5
C       1- 5    NL          NUMBER OF LINE SOURCES
C       6-10    NA          NUMBER OF AREA SOURCES
C      11-15    NT          NUMBER OF TIME PERIODS
C      16-20    NY          NUMBER OF ACC
C
C      CARD 2   TIME PERIODS, NT VALUES, 8 VALUES PER CARD     8F10.0
C       1-10    T(1)        PERIOD 1
C      11-20    T(2)        PERIOD 2
C      .....
C               T(NT)       PERIODS NT
C
C      CARD 3   PEAK GROUND ACC., NY VALUES, 8 VALUES PER CARD   8F10.0
C       1-10    Y(1)        PGA 1
C      11-20    Y(2)        PGA 2
C      .....
C               Y(NY)       PGA NY
C
C      CARD 4   SEARCHING GRID DESCRIPTION   8F10.0
C       1-10    XBEGIN      ORIGIN OF GRID X COORD
C      11-20    YBEGIN      ORIGIN OF GRID Y COORD
C      21-30    XEND        END OF GRID X COORD
C      31-40    YEND        END OF GRID Y COORD
C                           ORIGIN AND END SHOULD COINCIDE IF ONLY
C                           ONE LOCATION IS REQUIRED
C      41-50    DX          X INCREMENT
C      51-60    DY          Y INCREMENT
C                           DX AND DY MUST BE DIFFERENT FROM ZERO
C
C      4.- LINE SOURCES PROPERTIES     3 CARDS PER SOURCE
C
C      CARD 1
C       1-80    HED2        IDENTIFICATION OF THE LINE SOURCE
C
C      CARD 2   7F10.0
C               PROPERTIES ON THE LEFT OF BREAKING POINT
C       1-10    ALPHA1( )   NORMALIZED INTERCEPT
C      11-20    BETA1( )    SLOPE
C      21-30    XL1( )      X COORD FOR ORIGIN
C      31-40    XL2( )      X COORD OF END
C      41-50    YL1( )      Y COORD OF ORIGIN
C      51-60    YL2( )      Y COORD OF END
C      61-70    HL( )       DEPTH OF LINE
C
C      CARD 3     3F10.0
C               PROPERTIES ON THE RIGHT OF BREAKING POINT
C               INPUT THE SAME VALUES FOR ALPHA AND BETA
C               AND DUMMY BREAKING POINT IF THERE IS NO BREAKING POINT
C       1-10    ALPHA2( )   NORMALIZED INTERCEPT
C      11-20    BETA2( )    SLOPE
C      21-30    RML( )      BREAKING POINT MAGNITUDE
C
C      5.- AREA SOURCES PROPERTIES     3 CARDS PER SOURCE
C
C      CARD 1   20A4
C       1-80    HED         AREA SOURCE IDENTIFICATION
C
C      CARD 2   6F10.0
C               PROPERTIES ON THE LEFT OF BREAKING POINT
C       1-10    ALPHA1( )   NORMALIZED INTERCEPT
C      11-20    BETA1( )    SLOPE
C      21-30    XL1( )      X COORD OF CIRCLE CENTER
C      31-40    YL1( )      Y COORD OF CIRCLE CENTER
C      41-50    XL2( )      RADIUS OF CIRCLE
C      51-60    HL( )       DEPTH OF AREA SOURCE
C
C      CARD 3   3F10.0
C               PROPERTIES ON THE RIGHT OF BREAKING POINT
C               INPUT SAME VALUES FOR ALPHA AND BETA AND
C               DUMMY BREAKING POINT IF THERE IS NO BREAKING POINT
C       1-10    ALPHA2( )   NORMALIZED INTERCEPT
C      11-20    BETA2( )    SLOPE
C      21-30    RML( )      BREAKING POINT MAGNITUDE
C
C      OUTPUT
C      ******
C      DEPENDING UPON THE INPUT, THE OUTPUT WILL DISPLAY FROM ONE
C      PROBABILITY OF EXCEEDANCE AT ONE POINT TO A WHOLE CDF FOR A
C      WHOLE GRID OF POINTS
C
C      ****************************************************************
       IMPLICIT REAL*8 (A-H,O-Z)
       DIMENSION HED1(20),HED2(20)
       COMMON/SOURCE/ALPHA1(20),ALPHA2(20),BETA1(20),BETA2(20),RML(20),
      1               HL(20)
       COMMON/ATTEN/B1,B2,B3,B4,DELTAL,DELTAC
       DIMENSION XL1(20),YL1(20),XL2(20),YL2(20),T(10),YC(30),PDF(30),
      1  FY1(30),FY2(30)
C      ****************************************************************
C
C                READ AND WRITE  ATTENUATION CONSTANTS
```

```
C
      READ(5,1040)HED1
 1040 FORMAT(20A4)
      WRITE(6,1041)HED1
 1041 FORMAT('0',20A4 )
      READ(5,1000)B1,B2,B3,B4,DELTAL,DELTAC
      WRITE(6,1020)B1,B2,B3,B4,DELTAL,DELTAC
 1020 FORMAT('0','ATTENUATION CONSTANTS',//,' B1=',E15.7,3X,'B2=',E15.7,
     &3X,'B3=',E15.7,3X,'B4=',E15.7 /' DELTAL =',E15.7,3X,'DELTAC ='
     1  E15.7 /)
 1000 FORMAT(8F10.0)
C
C     READ AND WRITE GEOMETRIC CONSTANTS
      READ(5,1001)NL,NA,NT,NY
 1001 FORMAT(10I5)
      READ(5,1000)(T(I),I=1,NT)
      WRITE(6,2210)
      WRITE(6,2220)(T(I),I=1,NT)
      NY=NY+1
      READ(5,1000) (YG(I),I=2,NY)
      WRITE(6,2230)
      WRITE(6,2220)(YG(I),I=2,NY)
 2220 FORMAT(' ',10F10.2)
 2210 FORMAT('OTIME PERIODS')
 2230 FORMAT('OACCELERATIONS')
      YG(1)=0.0D0
 2222 FORMAT('0','SITE LOCATION',/,' ','X= ',F10.3,5X,'Y= ',F10.3)
  350 READ(5,1000) XBEGIN,YBEGIN,XEND,YEND,DX,DY
      NXMAX=DABS((XBEGIN-XEND)/DX)+1.9D0
      NYMAX=DABS((YBEGIN-YEND)/DY)+1.9D0
      WRITE(6,1021)NL,NA,NXMAX,NYMAX,NT
 1021 FORMAT('0','GEOMETRIC CONSTANTS',//,' NL=',I5,5X,'NA = ',I5,5X,
     &'NXMAX= ',I5,5X,'NYMAX=',I5,5X,'NT=',I5 / )
      IF(NL.LE.0) GO TO 610
C
C               READ AND WRITE LINE SOURCE PROPERTIES
C
      WRITE(6,1024)
 1024 FORMAT('OLINE SOURCES'/' *************' / )
      DO 616 I=1,NL
      READ(5,1040)HED2
      READ(5,1000)ALPHA1(I),BETA1(I),XL1(I),XL2(I),YL1(I),
     &YL2(I),HL(I)
      READ(5,1000) ALPHA2(I),BETA2(I),RML(I)
      WRITE(6,1041)HED2
      WRITE(6,1022)
      WRITE(6,1023)ALPHA1(I),BETA1(I),XL1(I),XL2(I),YL1(I),YL2(I),HL(I)
 1022 FORMAT(' ',2X,'ALPHA1',10X,'BETA1',11
     &X,'XL1',11X,'XL2',11X,'YL1',11X,'YL2',12X,'HL')
 1023 FORMAT(' ',E13.5,6E14.5)
      WRITE(6,1031)
      WRITE(6,1023) ALPHA2(I),BETA2(I),RML(I)
 1031 FORMAT(' SECOND REGRESSION CONSTANTS',/' ',2X,'ALPHAL2',9X,
     &'L2',10X,'MR')
  616 CONTINUE
      DO 700 IL=1,NL
      IF(XL1(IL).LT.XL2(IL)) GO TO 700
      XO=XL1(IL)
      YO=YL1(IL)
      XL1(IL)=XL2(IL)
      YL1(IL)=YL2(IL)
      XL2(IL)=XO
      YL2(IL)=YO
  700 CONTINUE
  610 IF(NA.LE.0) GO TO 551
C
C         READ AND WRITE AREA SOURCE PROPERTIES
      I1=NL+1
      NN=NL+NA
      WRITE(6,2124)
 2124 FORMAT(' '//' AREA SOURCES'/' *************' / )
C
      DO 615 I=I1,NN
      READ(5,1040)HED2
      READ(5,1000) ALPHA1(I),BETA1(I),XL1(I),YL1(I),XL2(I),HL(I)
      READ(5,1000) ALPHA2(I),BETA2(I),RML(I)
      WRITE(6,1041)HED2
      WRITE(6,2120)
      WRITE(6,1023) ALPHA1(I),BETA1(I),XL1(I),YL1(I),XL2(I),HL(I)
      WRITE(6,2123)
      WRITE(6,1023) ALPHA2(I),BETA2(I),RML(I)
  615 CONTINUE
C
 2120 FORMAT(' ',2X,'ALPHA1',10X,'BETA1',11X,'XO'
     &,12X,'YO',12X,'R',13X,'HA')
 2123 FORMAT(' SECOND REGRESSION CONSTANTS',/,' ',2X,'ALPHA2',10X,
     &'BETA2',10X,'MR')
  551 CONTINUE
C
C     ITERATION ON TIME PERIODS
  650 DO 370 IT=1,NT
      WRITE(6,225)
      WRITE(6,3000) T(IT)
C     WRITE(10,5020) T(IT)
 5020 FORMAT('TIME PERIOD',3X,F6.2)
 3000 FORMAT('0',' TIME PERIOD=',F10.2,' YRS',//)
C
C     ITERATION ON GRID
C     ITERATION IN THE Y DIRECTION
      Y=YBEGIN
      DO 385 IY=1,NYMAX
C
C     ITERATION IN THE X DIRECTION
      X=XBEGIN
      DO 380 IX=1,NXMAX
      WRITE(6,2222) X,Y
      PDF(1)=0.0D0
C
C     ITERATION ON PGA
      DO 360 II=2,NY
      YGII=YG(II)
      YGIIB1=YGII/B1
      SUM = 0.0D0
      SUM1=0.0D0
C
C     ITERATION ON LINE SOURCES
      IF(NL.EQ.0) GO TO 441
      DO 351 IL=1,NL
C
      CALL CONSTL(X,Y,XL1(IL),YL1(IL),XL2(IL),YL2(IL),
```

```
     1              AL1,AL2,DLIL,AIL,BIL)
      HLIL=HL(IL)
      GAM22=DEXP(ALPHA2(IL))
      DEL22=BETA2(IL)/B2
      RHO22=DEL22*B3
      FFY=(DLIL*DLIL)+(HLIL*HLIL)
      RH=(DSQRT(FFY)+B4)**RHO22
      EXCK=RH*GAM22*(YGIIB1**DEL22)*(AL2+AL1)*T(IT)
      IF(EXCK.LE.1.D-5) GO TO 351
      CALL INTGLN(IL,XL1(IL),YL1(IL),XL2(IL),YL2(IL),AL1,AL2,DLIL,
     1 HL(IL),AIL,BIL,YGII,X,Y,FFY,GAM22,DEL22,RHO22,FF,FF1)
      SUM=SUM + FF
      SUM1=SUM1+FF1
  351 CONTINUE
C
C              AREA SOURCE COMPUTATION
C
  441 IF(NA.EQ.0) GO TO 442
      I1=NL+1
      IEND=NL+NA
      DO 850 IA=I1,IEND
C
C     CHECK WHETHER THE CIRCLE CONTRIBUTES ANY TO THE PROB
      RIA=XL2(IA)
      HLIL=HL(IA)
      GAM22=DEXP(ALPHA2(IA))
      DEL22=BETA2(IA)/B2
      RHO22=DEL22*B3
      DIST=DSQRT((X-XL1(IA))*(X-XL1(IA))+(Y-YL1(IA))*(Y-YL1(IA)))
      RH=(DIST+B4)**RHO22
      EXCK=RH*GAM22*(YGIIB1**DEL22)*(6.2831D0*RIA*RIA)*T(IT)
      IF(EXCK.LE.1.D-5) GO TO 850
      SUMP=0.D0
      SUMP1=0.D0
      NR=RIA/DELTAC+0.9D0
      DR=RIA/NR
      DR1=DP*.5D0
C
C       ITERATION ON THE NB OF SEGMENTS IN THE CIRCLE
      DO 202 IR=1,NR
      DL1=DABS(DIST-DR1)
      DL2=DIST+DR1
      AL=DSQRT(RIA*RIA-DR1*DR1)
      FFY=DL1*DL1+HLIL*HLIL
      CALL INTGLN(IA,0.D0,DL1,AL,DL1,0.D0,AL,DL1,HLIL,0.D0,DL1,
     1 YGII,0.D0,0.D0,FFY,GAM22,DEL22,RHO22,FF,FF1)
      SUMP=SUMP+FF*2.D0
      SUMP1=SUMP1+FF1*2.D0
      FFY=DL2*DL2+HLIL*HLIL
      CALL INTGLN(IA,0.D0,DL2,AL,DL2,0.D0,AL,DL2,HLIL,0.D0,DL2,
     1 YGII,0.D0,0.D0,FFY,GAM22,DEL22,RHO22,FF,FF1)
      SUMP=SUMP+FF*2.D0
      SUMP1=SUMP1+FF1*2.D0
      DR1=DP1+DR
  202 CONTINUE
C
      SUMP=SUMP*DP
      SUMP1=SUMP1*DR
      SUM=SUM+SUMP
      SUM1=SUM1+SUMP1
  850 CONTINUE
C
  442 CONTINUE
      FY1(II)=DEXP(-SUM*T(IT))
      FY2(II)=1.0D0-FY1(II)
      PDF(II)=-SUM1*T(IT)*FY1(II)
  360 CONTINUE
C
C     WRITE(10,5000)(FY2(J),J=2,NY)
      NYY=(NY-2)/10+1
      K=2
      NY1=11
      DO 375 JJ=1,NYY
      IF(JJ.EQ.NYY) NY1=NY
      WRITE(6,226)(YG(J),J=K,NY1)
      WRITE(6,227)(FY2(J),J=K,NY1)
      WRITE(6,228)(FY1(J),J=K,NY1)
C     WRITE(6,229)(PDF(J),J=K,NY1)
      NY1=NY1+10
      K=K+10
  375 CONTINUE
 5010 FORMAT('SITE  ',2F10.2)
 5000 FORMAT(10F8.5)
  226 FORMAT('0','PGA =  ',10F10.4)
  227 FORMAT(' ','P(Y>YO)',10F10.4)
  228 FORMAT(' ','P(Y<YO)',10F10.4)
  229 FORMAT(' ','PDF(Y) ',10F10.4)
C
C     MOVE TO THE NEXT POINT ALONG THE GRID
  380 X=X+DX
  385 Y=Y+DY
  370 CONTINUE
  365 CONTINUE
  225 FORMAT('1','*********** P R O B A B I L I T Y  D I S T R I B U T
     &I O N  O F  P E A K  G R O U N D  A C C E L E R A T I O N *******
     &****',//)
      STOP
      END
      SUBROUTINE CONSTL(X,Y,XL1,YL1,XL2,YL2,
     1              AL1,AL2,DLIL,AM,B)
C
C     *****************************************************************
      IMPLICIT REAL*8 (A-H,O-Z)
C     *****************************************************************
C
C  AM = SLOPE OF LINE DEFINED FY (X1,Y1),(X2,Y2)
C  B =  Y-INTERSEPT
C
      AM=(YL2-YL1)/(XL2-XL1)
      B=YL1-AM*XL1
      XBAR=(X+AM*Y-B*AM)/(AM*AM+1.0D0)
      YBAR=(AM*X+Y*AM*AM+B)/(AM*AM+1.0D0)
      DLIL=DSQRT((X-XBAR)*(X-XBAR)+(Y-YBAR)*(Y-YBAR))
C
C CHECK POSITION OF (XBAR,YBAR) ALONF LINE W.R.T. END
C OF POINTS OF LINE SOURCE
      AL1=DSQRT((XL1-XBAR)*(XL1-XBAR)+(YL1-YBAR)*(YL1-YBAR))
      AL2=DSQRT((XL2-XBAR)*(XL2-XBAR)+(YL2-YBAR)*(YL2-YBAR))
C
      IF(XBAR.LT.XL1) AL1=-AL1
```

```
      IF(XBAR.LE.XL2)  RETURN
      DUM = AL1
      AL1 = -AL2
      AL2 = DUM
      RETURN
      END
      SUBROUTINE INTGLN(IL,XL1,YL1,XL2,YL2,AL1,AL2,DLIL,HLIL,AIL,BIL,
     1 YGII,X,Y,FFY,GAM22,DEL22,RHO22,FF,FF1)
C
C     THIS SUBROUTINE COMPUTES THE INTEGRAL ALONG A LINE
C     IT IS CALLED FOR LINE SOURCES AS WELL AS FOR CIRCLES SOURCES
C     THE PARAMETERS ALPHA AND BETA TO BE USED ARE FIRST CHECKED
C
C     ****************************************************************
      IMPLICIT REAL*8 (A-H,O-Z)
      INTEGER CASE
      DIMENSION XX(4),YY(4),DEL(2),GAM(2),RHO(2)
      COMMON/SOURCE/ALPHA1(20),ALPHA2(20),BETA1(20),BETA2(20),RML(20),
     1              HL(20)
      COMMON/ATTEN/B1,B2,B3,B4,DELTAL,DELTAC
C     ****************************************************************
C
C     DETERMINE LINE PARAMATERS
      GAM11=DEXP(ALPHA1(IL))
      DEL11=BETA1(IL)/B2
      RHO11=DEL11*B3
      NBSG=1
      XX(1)=XL1
      YY(1)=YL1
C
C     COMPUTE EPICENTRAL DISTANCE RECK AT WHICH AN EARTHQUAKE
C     OF BREAKPOINT MAGNITUDE RML(IL) WILL GENERATE AN ACCELERATION
C     YG(II)
C     IF RECK IS IMAGINARY OR SMALLER THAN DLIL, ALPHA2 AND BETA2
C     ARE USED.  OTHERWISE POINTS WHERE ALPHA AND BETA CHANGE VALUE
C     ARE DETERMINED ON THE LINE
      TERM=(B1*DEXP(B2*RML(IL))/YGII)**(1.D0/B3)
      IF((TERM-B4).LE.HLIL) GO TO 830
      TERM=(TERM-B4)*(TERM-B4)-(HLIL*HLIL)
C
C     THE EPICENTRAL DISTANCE IS NOT IMAGINARY
      RECK=DSQRT(TERM)
      IF(RECK.LE.DLIL) GO TO 830
C
C     THE EPICENTRAL DISTANCE IS NOT SMALLER THAN DLIL,  POINTS OF
C     INTERSECTION WITH THE LINE ARE COMPUTED
      TERM1=AIL*(BIL-Y)-X
      TERM2=X*X+BIL*BIL+Y*(Y-2.D0*BIL)-RECK*RECK
      TERM3=AIL*AIL+1.D0
      TERM2=DSQRT(TERM1*TERM1-TERM2*TERM3)
C
C     COORDINATES OF THE POINTS OF INTERSECTION
      XI1=(-TERM1+TERM2)/TERM3
      XI2=(-TERM1-TERM2)/TERM3
      YI1=AIL*XI1+BIL
      YI2=AIL*XI2+BIL
      IF(XI1.LT.XI2) GO TO 150
      STOR=XI1
      XI1=XI2
      XI2=STOR
      STOR=YI1
      YI1=YI2
      YI2=STOR
  150 CONTINUE
C
C     CHECK WHETHER THE POINTS OF INTERSECTION ARE ON THE SEGMENT
C     CONSIDERED AND DETERMINE THE USE OF ALPHA AND BETA
C
      IF(XL1.LT.XI1) GO TO 800
      IF(XL1.GT.XI2) GO TO 830
      IF(XL2.GT.XI2) GO TO 820
C
      CASE=1
      GO TO 850
C
  800 IF(XL2.LT.XI1) GO TO 830
      IF(XL2.LT.XI2) GO TO 840
C
      CASE=5
      NBSG=3
      XX(2)=XI1
      YY(2)=YI1
      XX(3)=XI2
      YY(3)=YI2
      XX(4)=XL2
      YY(4)=YL2
      GO TO 870
C
  830 CASE=3
  850 XX(2)=XL2
      YY(2)=YL2
      GO TO 870
C
  820 CASE=2
      XI1=XI2
      YI1=YI2
      GO TO 860
C
  840 CASE=4
  860 NBSG=2
      XX(2)=XI1
      YY(2)=YI1
      XX(3)=XL2
      YY(3)=YL2
C
  870 CONTINUE
      I=1
      IF(CASE.GT.2) I=2
      DEL(I)=DEL11
      GAM(I)=GAM11
      RHO(I)=RHO11
      I=I+1
      IF(I.EQ.3) I=1
      DEL(I)=DEL22
      GAM(I)=GAM22
      RHO(I)=RHO22
C
      FF=0.D0
      FF1=0.D0
      AL=-AL1
```

```
C
      DO 900 IXSG=1,NBSG
      TERM=0.D0
      IX=IXSG
      IF(IX.EQ.3) IX=1
C
C     COMPUTE LENGTH OF SEGMENT CONSIDERED AND DETERMINE
C     NUMBER OF INCREMENTS
      XLEN=DSQRT((XX(IXSG)-XX(IXSG+1))*(XX(IXSG)-XX(IXSG+1))+
     1           (YY(IXSG)-YY(IXSG+1))*(YY(IXSG)-YY(IXSG+1)))
      NBIC=(XLEN/DELTAL)+.9D0
      IF(NBIC.EQ.0) NBIC=1
      XXNBIC=NBIC
      DELTA=XLEN/XXNBIC
C
C     START DO LOOP AT 1/2 DELTA
      DELDO=DEL(IX)
      RHODO=RHO(IX)
      AL=AL+.5D0*DELTA
C
      DO 100 IXIC=1,NBIC
      RH=AL*AL+FFY
      TERM=TERM+(DSQRT(RH)+B4)**RHODO
      AL=AL+DELTA
  100 CONTINUE
C
      AL=AL-.5D0*DELTA
      TERM=TERM*GAM(IX)*((YGII/B1)**DELDO)*DELTA
      FF=FF+TERM
      FF1=FF1+TERM*DELDO/YGII
  900 CONTINUE
      RETURN
      END
$DATA
```

```
C     ******************************************************************
C     PROGRAM CONST.PROB                              STANFORD UNIVERSITY
C     ******************************************************************
C
C     GIVEN THE CDF ON ACCELERATION P(A.GT.A0) FOR EACH NODE OF A GRID
C     THIS PROGRAM FINDS THE ACCELERATION (A0) CORRESPONDING TO A GIVEN
C     PROBABILITY USING LINEAR INTERPOLATION.  THE GRID IS COVERED
C     STARTING FROM  BOTTOM LEFT NODE, ITERATIONS ARE MADE FIRST IN
C     THE HORIZONTAL DIRECTION
C
C     ******************************************************************
C     INPUT FORMAT
C     ************
C
C      COL      VARIABLE NAME      VARIABLE DESCRIPTION
C
C     1.- GENERAL INFORMATION  1 CARD   (4I5)
C      1- 5     NBPD               NB OF RUNS REQUIRED
C      6-10     NBAC               NB OF ACC GIVEN FOR EACH NODE
C     11-15     NBXX               NB OF COLUMNS
C     16-20     NBYY               NB OF ROWS
C
C     2.- CDF PARAMETERS   1 CARD   (3F10.0)
C      1-10     ACST               SMALLEST ACC IN CDF
C     11-20     ACIC               ACC INCREMENT IN CDF
C     21-30     PBCK               PROBABILITY A0
C
C     3.- IDENTIFICATION   1 CARD   (20A4)
C      1-80     HED1               RUN IDENTIFICATION
C
C     4.- CDF CARDS   (NBXX*NBYY) CARDS OR SET OF CARDS   (10F8.0)
C      1-10     PB(1)              PROBABILITY OF EXCEEDANCE CORRESPONDING
C                                  TO THE SMALLEST ACCELERATION
C      ...      PB( )
C      ...      PB(NBAC)           PROBABILITY OF EXCEEDANCE CORRESPONDING
C                                  TO THE LARGEST ACCELERATION
C
C     OUTPUT
C     ******
C     THE OUTPUT IS SAVED ON DISK AND CONTAINS THE RUN IDENTIFICATION
C     AND THE ACCELERATION AT EACH NODE CORRESPONDING TO THE GIVEN
C     PROBABILITY OF EXCEEDANCE.  FORMAT (8F10.6)
C
C     ******************************************************************
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION AC(30),PB(30),ACPB(40),HED1(20),HED2(20)
C     ******************************************************************
C
      READ(5,1010)NBPD,NBAC,NBXX,NBYY
 1010 FORMAT(8I5)
      WRITE(6,2010)NBPD,NBAC,NBXX,NBYY
 2010 FORMAT('0NBPD',I6,'  NBAC',I6,'  NBXX',I6,'  NBYY',I6)
C
      READ(5,1040)ACST,ACIC,PBCK
 1040 FORMAT(8F10.0)
      WRITE(6,2030)ACST,ACIC,PBCK
 2030 FORMAT('0ACST',F10.2,'  ACIC',F10.2,'  PBCK',F10.2)
C
      NBAC1=NBAC+1
      PB(1)=1.D0
      AC(1)=0.D0
      AC(2)=ACST
      DO 120 I=2,NBAC
  120 AC(I+1)=AC(I)+ACIC
C
      PRINT,'AC',AC
C
      DO 100 IP=1,NBPD
      READ(5,1000)HED1
 1000 FORMAT(20A4)
      WRITE(6,2000)HED1
 2000 FORMAT(' ',20A4)
      WRITE(10,1000)HED1
      DO 100 IY=1,NBYY
      DO 110 IX=1,NBXX
C     READ(5,1020)HED1
C1020 FORMAT(//20A4)
C
      READ(5,1030)(PB(I),I=2,NBAC1)
 1030 FORMAT(10F8.0)
C
      DO 150 KP=2,NBAC1
      IF(PBCK.GE.PB(KP)) GO TO 160
  150 CONTINUE
C
      WRITE(6,2020)HED1
 2020 FORMAT('0*** ATTENTION ***'/' EXCEDANCE IS LARGER THAN THE INPUT'
     1 /' '20A4)
      ACPB(IX)=AC(NBAC1)
      GO TO 110
C
  160 ACPB(IX)=AC(KP-1)+((AC(KP)-AC(KP-1))*(PB(KP-1)-PBCK))/
     1  (PB(KP-1)-PB(KP))
  110 CONTINUE
C
      WRITE(6,2040)(ACPB(I),I=1,NBXX)
 2040 FORMAT(' ',12F10.6)
      WRITE(10,3000)(ACPB(I),I=1,NBXX)
 3000 FORMAT(8F10.6)
  100 CONTINUE
      RETURN
      END
$DATA
```

```
C     ******************************************************************
C     PROGRAM ACC.TO.INT                                STANFORD UNIVERSITY
C     ******************************************************************
C     THIS PROGRAM FINDS THE MODIFIED MERCALLI INTENSITIES CORRESPONDING
C     TO A CDF ON ACCELERATION USING THE GUTHENBERG-RICHTER RELATIONSHIP
C     A MONTE-CARLO PROCESS IS USED.  CDF P(A.GT.AO) IS INPUT.
C     ******************************************************************
C     INPUT FORMAT
C     ************
C      COL    VARIABLE NAME      VARIABLE DESCRIPTION
C
C     1.- GENERAL    1 CARD    (3I5)
C      1- 5     NB                  (NUMBER OF POINTS IN CDF AS READ IN)*2
C      6-10     IY                  RANDOM GENERATOR INIATOR
C     11-15     NBPT                NB OF CDF TO BE READ IN
C
C     2.- ACCELERATIONS   8 VALUES PER CARD    (8F10.0)
C      1-10     X(4)                SMALLEST ACC IN CDF
C      ...      X( )
C      ...      X(NB+2)             LARGEST ACC IN CDF
C      ...      X(NB+4)             AN ACC EQUAL OR LARGER THAN X(NB+2).
C                                   MAXIMUM POSSIBLE ACC TO BE USED IN CASE
C                                   INPUT CDF DOES NOT GO UP TO 100%
C
C     THE REMAINING DATA IS READ FROM DISK
C     3.- NODE IDENTIFICATION    1 CARD IMAGE    (20A4)
C      1 80     HED1                IDENTIFICATION
C
C     4.- CDF P(A.GT.AO)    8 VALUES PER CARD IMAGE    (8F10.0)
C      1-10     X(3)                P(A.GT.X(4))
       ...      X( )
       ...      X(NB+1)             P(A.GT.X(NB+2))
C
C     OUTPUT
C     ******
C     THE OUTPUT DISPLAYS THE INPUT, THE NUMBER OF OCCURRENCES OF
C     THE MODIFIED MERCALLI INTESITY IN THE SIMULATION AND THE
C     CORRESPONDING PERCENTAGE.
C
C     ******************************************************************
      DIMENSION X1(44),INTEN(15),HED1(20)
      COMMON/POI/IY,DELPOI
C     ******************************************************************
C
      READ(5,1000)NB,IY,NBPT
      NB2=NB+4
      NB1=NB+1
      X1(1)=0.
      X1(2)=0.
 1000 FORMAT(3I5)
      READ(5,1100)(X1(I),I=4,NB2,2)
 1100 FORMAT(8F10.0)
      X1(NB+3)=100.
      CALL RANDK(IY,YFL,0)
C     YFL=-.01
C
C     READ(10,1020)HED1
C     WRITE(6,2010) HED1
      DO 400 IP=1,NBPT
      READ(10,1020)HED1
 1020 FORMAT(20A4)
      READ(10,1200)(X1(I),I=3,NB1,2)
 1200 FORMAT(10F8.0)
      DO 60 I=3,NB1,2
   60 X1(I)=1.-X1(I)
      DO 220 I=1,NB1,2
  220 X1(I)=X1(I)*100.
C
      WRITE(6,2010)HED1
 2010 FORMAT(' ',//' ',20A4)
      WRITE(6,2050)
 2050 FORMAT('0',7X,'CDF',7X,'ACC')
C
      WRITE(6,2020)X1
 2020 FORMAT(' ',2F10.3)
      DO 90 IT=1,12
   90 INTEN(IT)=0
C
      DO 100 IT=1,1000
      CALL CDF(X1,NB2,1,ACC)
      INT=3.*(ALOG10(ACC*1000.)+.5)+.5
      IF(INT.LT.1) GO TO 100
C     PRINT,'ACC,INT',ACC,INT
      INTEN(INT)=INTEN(INT)+1
  100 CONTINUE
C
      TOT=0.
      DO 110 INT=1,12
C     PRINT,'INT,NB OF OCC',INT,INTEN(INT)
  110 TOT=TOT+FLOAT(INTEN(INT))
C
      WRITE(6,2060)
 2060 FORMAT('0INTENSITY    PERCENTAGE'/' *********************'/)
      DO 120 INT=1,12
      SUB=FLOAT(INTEN(INT))/TOT
  120 WRITE(6,2030)INT,SUB
  400 CONTINUE
 2030 FORMAT(' ',I9,F13.3)
      RETURN
      END
      SUBROUTINE CDF(X1,KEND,ICK,ANS)
C
C     THIS SUBROUTINE FINDS THE VALUE CORRESPONDING TO THE RANDON
C     VARIABLE ON THE CDF, LINEAR FITTING IS APPLIED IF INDEX.EQ.I
C
C     ******************************************************************
      COMMON/POI/IY,DELPOI
      DIMENSION X1(KEND)
C     ******************************************************************
C
      KEN=KEND/2
      CALL RANDK(IY,YFL,0)
C     YFL=YFL+.01
C
      DO 100 I=1,KEN
      IF(YFL.LT.(X1(2*I-1)/100.)) GO TO 110
  100 CONTINUE
      WRITE(6,1000)
 1000 FORMAT(27H0** ERROR IN CDF INPUT **  //
```

```
     1 36H LAST PERCENTAGE INPUT IS NOT 100.01  //)
       I=I-1
  110 IF(ICK-1) 200,300,120
  200 ANS = X1(2*I)
      RETURN
C
C     LINEAR FITTING HALF DISTANCE BETWEEN INPUT POINTS
  120 IF(I.EQ.KEN)GO TO 140
      IF(I.LT.3) GO TO 130
      XT=(X1(2*I)+X1(2*I-2))*.5
      YT=X1(2*I-1)
      XB=(X1(2*I-4)+X1(2*I-2))*.5
      YB=X1(2*I-3)
      GO TO 150
C
  130 XT=(X1(2*I)+X1(2*I-2))*.5
      YT=X1(2*I-1)
      XB=X1(2*I-2)
      YB=0.
      GO TO 150
  140 XT=X1(2*I)
      YT=100.
  160 XB=(X1(2*I-4)+X1(2*I-2))*.5
      YB=X1(2*I-3)
      GO TO 150
C
C     LINEAR FIT TROUGH INPUT POINTS
  300 XT=X1(2*I)
      YT=X1(2*I-1)
      XB=X1(2*I-2)
      YB=X1(2*I-3)
C
C     CHECK FOR VERTICAL INTERPOLATION
  150 ANS=XT
      IF((XT-XB).LT..00001) RETURN
C
      A=(YT-YB)/(XT-XB)
      ANS=(YFL*100.-(YT-A*XT))/A
      RETURN
      END
      SUBROUTINE RANDK (IY,YFL,INDEX)
C
C     THIS SUBROUTINE GENERATES RANDOM NUMBERS
C
      IY=IY*314159269+453806245
    4 IF(IY.GE.0) GO TO 6
    5 IY=IY+2147483647+1
    6 CONTINUE
      IF(INDEX.GT.0) GO TO 8
      YFL=FLOAT(IY)*.4656613E-9
    8 RETURN
      END
$DATA
```

```
C     ****************************************************************
C     PROGRAM PLOT.EPI                                STANFORD UNIVERSITY
C     ****************************************************************
C     THIS PROGRAM PLOTS THE EARTHQUAKE EPICENTERS ON A GRID (MAP)
C     GIVEN THE LATITUDE AND LONGITUDE.  DIFFERENT SYMBOLS ARE
C     USED FOR THE DIFFERENT MAGNITUDES.
C     THE PLOT WILL BE ROTATED 90 DEGREES COUNTERCLOCWISE IF IT IS TOO
C     HIGH TO FIT ON THE PLOTTER.  IF BOTH DIMENSIONS ARE TOO BIG, AN
C     MESSAGE WILL BE GENERATED AND THE PLOT ABORTED.
C     ****************************************************************
C     INPUT FORMAT
C     ************
C
C      COL      VARIABLE NAME      VARIABLE DESCRIPTION
C
C     1.- IDENTIFICATION CARD    1 CARD   (80A1)
C      1-80      HED               RUN IDENTIFICATION
C
C     2.- PLOT FLAGS    1 CARD    (7I5)
C      1- 5      NBPL              NB OF PLOTS
C      6-10      SKIP3             =0, WILL PLOT      EQ OF MAG 3
C                                  =1, WILL NOT PLOT  EQ OF MAG 3
C     11-15      SKIP4                                EQ OF MAG 4
C     16-20      SKIP5                                EQ OF MAG 5
C     21-25      SKIP6                                EQ OF MAG 6
C     26-30      SKIP7                                EQ OF MAG 7
C     31-35      SKIP8                                EQ OF MAG 8
C
C     3.- GRID PARAMETERS    1 CARD    (8F10.0)
C                                  THE ORIGIN OF THE GRID IS DEFINED AS
C                                  THE BOTTOM LEFT CORNER; THE END AS THE
C                                  TOP RIGHT.  THE VERTICAL HAS TO POINT
C                                  NORTH FOR THE DATA TO BE READ IN
C                                  CONSISTENTLY.
C      1-10      XMIN1             X COORD OF ORIGIN
C     11-20      YMIN1             Y COORD OF ORIGIN
C     21-30      XMAX1             X COORD OF END
C     31-40      YMAX1             Y COORD OF END
C     41-50      INDGX             LENGTH OF ONE DEGREE IN INCHES FOR
C                                  THE PLOT (X DIRECTION).
C     51-60      INDGY             LENGTH OF ONE DEGREEE IN INCHES FOR
C                                  THE PLOT (Y DIRECTION)
C     61-70      CAL               PLOTTER TO BE USED (10. OR 29.)
C                                  IF ZERO, WILL BE SET TO 10.
C     71-80      DIVR              THE FRAME COORDINATES ARE ROUNDED OFF
C                                  TO THE CLOSEST (DEGREE/DIVR)
C                                  IF INPUT AS 0, WILL BE SET TO 1
C
C     3.- EPICENTER COORDINATES    1 EPICENTER PER CARD IMAGE    (3F10.0)
C                                  INFORMATION READ FROM DISK
C               X( )               X COORDINATE (LONGITUDE) OF EPICENTER
C               Y( )               Y COORDINATE (LATITUDE) OF EPICENTER
C               RM( )              MAGNITUDE
C
C     OUTPUT
C     ******
C     A FILE IS CREATED ON DISK, IT HAS TO BE TRANSFERRED ON TAPE AND
C     PLOTTED ON A 10 OR 29 INCH CALCOMP PLOTTER
C
C     ****************************************************************
      DIMENSION AMODES(200),END1(2),END2(2),XX4(500),YY4(500),XX5(500)
     &,YY5(500),XX6(300),YY6(300),XX7(100),YY7(100),XX8(100),YY8(100),
     &X(2050),Y(2050),RM(2050),XX(200),YY(200),XX3(500),YY3(500)
      INTEGER SKIP3,SKIP4,SKIP5,SKIP6,SKIP7,SKIP8
      REAL INRAS,INDG,INDGX,INDGY
      LOGICAL*1 HED(70)
C     ****************************************************************
C
      READ(5,1001)HED
 1001 FORMAT(80A1)
      READ(5,1002)NBPL,SKIP3,SKIP4,SKIP5,SKIP6,SKIP7,SKIP8
 1002 FORMAT(10I5)
      READ(5,1000)XMIN1,YMIN1,XMAX1,YMAX1,INDGX,INDGY,CAL,DIVR
      IF(DIVR.EQ.0.) DIVR=1.
 1000 FORMAT(8F10.0)
      INRAS=100.
      IF(CAL.EQ.29.) INRAS=200.
      XRAS=(ABS(XMAX1-XMIN1)*INDGX+1.5)*INRAS
      YRAS=(ABS(YMAX1-YMIN1)*INDGY+1.5)*INRAS
      XMIN1=XMIN1*DIVR
      XMAX1=XMAX1*DIVR
      YMIN1=YMIN1*DIVR
      YMAX1=YMAX1*DIVR
      INDGX=INDGX/DIVR
      INDGY=INDGY/DIVR
C
C     ORIENTATE FRAME
      DELTAX=.75/INDGX
      DELTAY=.75/INDGY
      IF(YRAS.LE.(CAL*INRAS)) GO TO 112
      IF(XRAS.LE.(CAL*INRAS)) GO TO 110
      WRITE(6,2100)
 2100 FORMAT('0THE SCALE IS TOO LARGE FOR THE PLOTTER, REDUCE INDG PER D
     &EGREE')
      STOP
C
C     INTERCHANGE X AND Y SCALES
  110 WRITE(6,2101)
 2101 FORMAT('0THE PLOT WILL BE ROTATED BY 90 DEGREES')
      STOR=XMIN1
      XMIN1=YMAX1
      YMAX1=XMAX1
      XMAX1=YMIN1
      YMIN1=STOR
      STOR=DELTAX
      DELTAX=DELTAY
      DELTAY=STOR
      STOR=XRAS
      XRAS=YRAS+3.5*INRAS
      YRAS=STOR
      GO TO 111
C
  112 XRAS=XRAS+3.5*INRAS
C
  111 CALL MODESG(AMODES,'MORTGAT,BIN 502',15)
C
      DO 500 IP=1,NBPL
      AMODES(45)=1.
      XMAX=XMAX1
```

```
      YMAX=YMAX1
      XMIN=XMIN1
      YMIN=YMIN1
C
      READ(10,1030)NBRC,HED
 1030 FORMAT(I5,70A1)
C
      IF(YRAS.LE.(CAL*INRAS)) GO TO 100
      DO 120 IX=1,NBRC
  120 READ(10,1010)X(IX),Y(IX),RM(IX)
 1010 FORMAT(19X,F5.3,3X,F5.3,42X,F3.2)
      GO TO 130
C
  100 CONTINUE
      DO 140 IX=1,NBRC
  140 READ(10,1010)Y(IX),X(IX),RM(IX)
C
C     CHECK SIGNS
  130 IF(XMAX1.GT.XMIN1) GO TO 150
      XMAX=-XMAX1
      XMIN=-XMIN1
      DO 160 IX=1,NBRC
  160 X(IX)=-X(IX)
C
  150 XFMAX=XMAX+DELTAX*4.25/.75
      XFMIN=XMIN-DELTAX
      IF(YMAX1.GT.YMIN1) GO TO 170
      YMAX=-YMAX1
      YMIN=-YMIN1
      DO 180 IX=1,NBRC
  180 Y(IX)=-Y(IX)
  170 YFMAX=YMAX+DELTAY
      YFMIN=YMIN-DELTAY
  552 WRITE(6,551)(X(IX),Y(IX),RM(IX),IX=1,NBRC)
  551 FORMAT(' ',12F10.3)
C
      AMODES(97)=XRAS
      AMODES(179)=YRAS
      CALL SUBJEG(AMODES,XFMIN,YFMIN,XFMAX,YFMAX)
      CALL OBJECG(AMODES,0.,0.,XRAS,YRAS)
C
      IF(AMODES(12).NE.AMODES(13))WRITE(6,2102)AMODES(12),AMODES(13)
 2102 FORMAT('OTHE SCALES ARE NOT THE SAME',2F10.6)
      END1(1)=IFIX(YMIN)
      END2(1)=IFIX(XMAX)
      END1(2)=IFIX(YMAX)
      END2(2)=IFIX(XMIN)
C
      DELTA2=1./(8.*INDGX)
      XMUL=4.
      DELTA1=4.*DELTA2
      DELTA3=3.*DELTA2
      SIGN=-1.
C
      I=1
      VAR=END1(2)
      FIX=END2(2)
C
      DO 400 K=1,2
C
  401 XX(I)=FIX
      YY(I)=VAR
      XX(I+1)=FIX+DELTA2
      YY(I+1)=VAR
      XX(I+2)=FIX
      YY(I+2)=VAR
      INTG=ABS(VAR)
      XC=FIX-DELTA2*XMUL
      YC=VAR
C
      CALL INUMBG(AMODES,XC,YC,3,INTG)
      WRITE(6,550)XC,YC,INTG
      I=I+3
      IF(VAR.EQ.END1(K)) GO TO 402
      VAR=VAR+SIGN
      GO TO 401
C
  402 STOR=VAR
      VAR=FIX
      FIX=STOR
  404 XX(I)=VAR
      YY(I)=FIX
      XX(I+1)=VAR
      YY(I+1)=FIX+DELTA2
      XX(I+2)=VAR
      YY(I+2)=FIX
      INTG=ABS(VAR)
      XC=VAR
      YC=FIX-DELTA3
C
      CALL INUMBG(AMODES,XC,YC,3,INTG)
      WRITE(6,550)XC,YC,INTG
      I=I+3
      IF(VAR.EQ.END2(K)) GO TO 403
      VAR=VAR-SIGN
      GO TO 404
C
  403 SIGN=-SIGN
      STOR=VAR
      VAR=FIX
      FIX=STOR
      DELTA2=-DELTA2
      XMUL=2.
      DELTA3=-DELTA3
C
  400 CONTINUE
C     DO 500 K=1,I
C 500 WRITE(6,550)XX(K),YY(K)
  550 FORMAT(' ',2F10.3,I6)
      I=I-3
      CALL LINESG(AMODES,I,XX,YY)
C
      I3=0
      I4=0
      I5=0
      I6=0
      I7=0
      I8=0
      DO 300 IX=1,NBRC
      IRM=RM(IX)-1.99
```

```
      IF(IRM.LT.1) GO TO 300
      GO TO (230,240,250,260,270,280),IRM
  230 I3=I3+1
      XX3(I3)=X(IX)
      YY3(I3)=Y(IX)
      GO TO 300
  240 I4=I4+1
      XX4(I4)=X(IX)
      YY4(I4)=Y(IX)
      GO TO 300
  250 I5=I5+1
      XX5(I5)=X(IX)
      YY5(I5)=Y(IX)
      GO TO 300
  260 I6=I6+1
      XX6(I6)=X(IX)
      YY6(I6)=Y(IX)
      GO TO 300
  270 I7=I7+1
      XX7(I7)=X(IX)
      YY7(I7)=Y(IX)
      GO TO 300
  280 I8=I8+1
      XX8(I8)=X(IX)
      YY8(I8)=Y(IX)
  300 CONTINUE
      AMODES(40)=8.*INRAS/100.
      AMODES(41)=14.*INPAS/100.
C
      AMODES(45)=.5
      AMODES(84)=12.
      IF(SKIP3.EQ.0.AND.I3.GT.0) CALL POINTG(AMODES,I3,XX3,YY3)
      IF(SKIP3.EQ.0.AND.I3.GT.0) PRINT,'I3 HAS BEEN CALLED'
      IF(SKIP4.EQ.0.AND.I4.GT.0)CALL POINTG(AMODES,I4,XX4,YY4)
      IF(SKIP4.EQ.0.AND.I4.GT.0)PRINT,'I4 HAS BEEN CALLED'
C
      AMODES(84)=15.
      IF(SKIP5.EQ.0.AND.I5.GT.0)CALL POINTG(AMODES,I5,XX5,YY5)
      IF(SKIP5.EQ.0.AND.I5.GT.0)PRINT,'I5 HAS BEEN CALLED'
C
      AMODES(45)=.35
      AMODES(84)=40.
      IF(SKIP6.EQ.0.AND.I6.GT.0)CALL POINTG(AMODES,I6,XX6,YY6)
      IF(SKIP6.EQ.0.AND.I6.GT.0) PRINT,'I6 HAS BEEN CALLED'
C
      AMODES(45)=2.0
      AMODES(84)=12.
      IF(SKIP7.EQ.0.AND.I7.GT.0)CALL POINTG(AMODES,I7,XX7,YY7)
      IF(SKIP7.EQ.0.AND.I7.GT.0) PRINT,'I7 HAS BEEN CALLED'
C
      AMODES(45)=.65
      AMODES(84)=40.
      IF(SKIP8.EQ.0.AND.I8.GT.0)CALL POINTG(AMODES,I8,XX8,YY8)
      IF(SKIP8.EQ.0.AND.I8.GT.0)  PRINT,'I8 HAS BEEN CALLED'
C
      XT=XMAX-.5/INDGX
      AMODES(50)=90.
      AMODES(46)=90.
      YT=YMIN+DELTA1
      AMODES(45)=.5
      XT=XFMAX-3./INDGX
      CALL VECSG(AMODES,XT,YT,16,' .  MAGNITUDE 4+')
C
      XT=XT+DELTA1
      CALL VECSG(AMODES,XT,YT,16,' +  MAGNITUDE 5+')
C
      XT=XT+DELTA1
      AMODES(45)=.35
      CALL VECSG(AMODES,XT,YT,16,' X  MAGNITUDE 6+')
C
      XT=XT+DELTA1
      AMODES(45)=1.0
      CALL VECSG(AMODES,XT,YT,16,' .  MAGNITUDE 7+')
C
      XT=XT+DELTA1
      AMODES(45)=.55
      CALL VECSG(AMODES,XT,YT,16,' X  MAGNITUDE 8+')
C
      XT=XT+DELTA1
      AMODES(45)=1.5
      CALL VECSG(AMODES,XT,YT,48,HED)
      CALL PICTRG(AMODES)
      PPINT,'MAGNITUDE 3'
      IF(I3.NE.0)WRITE(6,555)(XX3(I),YY3(I),I=1,I3)
      PRINT,'MAGNITUDE 4 +'
      IF(I4.NE.0)WRITE(6,555)(XX4(I),YY4(I),I=1,I4)
      PRINT,'MAGNITUDE 5 +'
      IF(I5.NE.0)WRITE(6,555)(XX5(I),YY5(I),I=1,I5)
      PRINT,'MAGNITUDE 6 +'
      IF(I6.NE.0)WRITE(6,555)(XX6(I),YY6(I),I=1,I6)
      PRINT,'MAGNITUDE 7 +'
      IF(I7.NE.0)WRITE(6,555)(XX7(I),YY7(I),I=1,I7)
      PRINT,'MAGNITUDE 8 +'
      IF(I8.NE.0)WRITE(6,555)(XX8(I),YY8(I),I=1,I8)
  500 CONTINUE
  555 FORMAT(' ',12F10.3)
      CALL EXITG(AMODES)
      RETURN
      END
$DATA
```

```
C     ******************************************************************
C     PROGRAM PLOT.ISO                                 STANFORD UNIVERSITY
C     ******************************************************************
C     THIS PROGRAM PLOTS THE ISO ACCELERATION OR ISO INTENSITY
C     LINES OVER A GRID (MAP) GIVEN THE ACCELERATIONS AT EACH NODE OF THE
C     GRID.  THE ORIGIN OF THE GRID IS THE BOTTOM LEFT CORNER, THE END
C     THE TOP RIGHT CORNER.  THE VALUES HAVE TO BE INPUT FILING COLUMNS
C     FROM BOTTOM TO TOP AS ONE MOVES FROM LEFT TO RIGHT : FIRST VALUE
C     INPUT IS THUS THE ORIGIN AND LAST THE END.
C     THE PLOT WILL BE ROTATED 90 DEGREES COUNTERCLOCWISE IF IT IS TOO
C     HIGH TO FIT ON THE PLOTTER.  IF BOTH DIMENSIONS ARE TOO BIG, AN
C     MESSAGE WILL BE GENERATED AND THE PLOT ABORTED.
C
C     ******************************************************************
C     INPUT FORMAT
C     ************
C
C      COL      VARIABLE NAME     VARIABLE DESCRIPTION
C
C     1.- IDENTIFICATION CARD    1 CARD    (80A1)
C      1-80     HED               RUN IDENTIFICATION
C
C     2.- PLOT FLAGS   1 CARD    (5I5,6F10.0)
C      1- 5     SKIPAC            =0, WILL PLOT ISO ACCELERATION LINES
C                                 =1, WILL PLOT ISO INTENSITY LINES
C      5-10     NF                NB OF FALUTS TO BE PLOTTED
C     11-15     NBPL              NB OF PLOTS
C     16-20     NR                NB OF ROWS IN INPUT
C     21-25     NC                NB OF COLUMNS IN INPUT
C     26-35     DX                DISTANCE (DEGREE) BETWEEN TWO COLUMNS
C     36-45     DY                DISTANCE (DEGREE) BETWEEN TWO ROWS
C     46-55     DC                INTERVAL BETWEEN TWO ISO LINES
C
C     3.- GRID PARAMETES   1 CARD    ((8F10.0)
C                                 THE ORIGIN OF THE GRID IS DEFINED AS
C                                 THE BOTTOM LEFT CORNER; THE END AS THE
C                                 TOP RIGHT.  THE VERTICAL HAS TO POINT
C                                 NORTH FOR THE DATA TO BE READ IN
C                                 CONSISTENTLY.
C      1-10     XMIN1             X COORD OF ORIGIN
C     11-20     YMIN1             Y COORD OF ORIGIN
C     21-30     XMAX1             X COORD OF END
C     31-40     YMAX1             Y COORD OF END
C     41-50     INDGX             LENGTH OF ONE DEGREE IN INCHES FOR
C                                 THE PLOT (X DIRECTION).
C     51-60     INDGY             LENGTH OF ONE DEGREEE IN INCHES FOR
C                                 THE PLOT (Y DIRECTION)
C     61-70     CAL               PLOTTER TO BE USED (10. OR 29.)
C                                 IF ZERO, WILL BE SET TO 10.
C     71-80     DIVR              THE FRAME COORDINATES ARE ROUNDED OFF
C                                 TO THE CLOSEST (DEGREE/DIVR)
C                                 IF INPUT AS 0, WILL BE SET TO 1
C
C     3.- FAULT COORDINATES    NF CARDS    (4F10.0) NO INPUT IF NF=0
C      1-10     YF1( )            Y COORD OF ORIGIN OF FAULT 1
C     11-20     XF1( )            X COORD OF ORIGIN OF FAULT 1
C     21-30     YF2( )            Y COORD OF END OF FAULT 1
C     31-40     XF2( )            X COORD OF END OF FAULT 2
C                                 REPEAT NF TIMES
C
C     4.- ACCELERATION AT THE NODES   8 VALUES PER CARD   (8F10.0)
C      1-10     ACC(1,IC)          ACC AT ORIGIN (IC=1)
C     ...       ACC(IR,IC)
C     ...       ACC(NR,IC)         ACC AT TOP OF FIRST COLUMN (IC=1)
C                                  REPEAT NC TIMES
C
C     OUTPUT
C     ******
C     A FILE IS CREATED ON DISK, IT HAS TO BE TRANSFERRED ON TAPE AND
C     PLOTTED ON A 10 OR 29 INCH CALCOMP PLOTTER
C
C     ******************************************************************
      DIMENSION AMODES(200),END1(2),END2(2),XX(1000),YY(1000),A(4900)
     1,AA(70,70),XF1(20),XF2(20),YF1(20),YF2(20),HED1(20)
      INTEGER*4 NR,NC,ROTA,SKIPAC
      REAL INRAS,INDG,INDGX,INDGY
      LOGICAL*1 HED2(80)
      EQUIVALENCE(A(1),AA(1,1))
C     ******************************************************************
C
      READ(5,1001)HED1
 1001 FORMAT(20A4)
      WRITE(6,2202)HED1
 2202 FORMAT(' '20A4)
      READ(5,1003)SKIPAC,NF,NBPL,NR,NC,DX,DY,DC
      READ(5,1000)XMIN,YMIN,XMAX,YMAX,INDGX,INDGY,CAL,DIVR
      IF(DIVR.EQ.0.0) DIVR=1.
      PRINT,XMIN,YMIN,XMAX,YMAX,INDGX,INDGY,CAL,DIVR
      PRINT,NBPL,NR,NC,DX,DY,DC
C
 1000 FORMAT(8F10.0)
 1003 FORMAT(5I5,6F10.0)
      INRAS=100.
      IF(CAL.EQ.29.) INRAS=200.
C
C     DIVR IS USED TO TO BE ABLE TO PLOT PARTS OF DEGREES, USALLY DEGREE/10
      XMIN=IFIX(XMIN*DIVR+.5)
      YMIN=IFIX(YMIN*DIVR+.5)
      XMAX=IFIX(XMAX*DIVR+.5)
      YMAX=IFIX(YMAX*DIVR+.5)
      INDGX=INDGX/DIVR
      INDGY=INDGY/DIVR
      DX=DX*DIVR
      DY=DY*DIVR
      XRAS=(ABS(XMAX-XMIN)*INDGX+1.5)*INRAS
      YRAS=(ABS(YMAX-YMIN)*INDGY+1.5)*INRAS
      ROTA=0
C
C     ORIENTATE FRAME
      DELTAX=.75/INDGX
      DELTAY=.75/INDGY
      IF(YRAS.LE.(CAL*INRAS)) GO TO 100
      IF(XRAS.LE.(CAL*INRAS)) GO TO 110
      WRITE(6,2100)
 2100 FORMAT('0THE SCALE IS TOO LARGE FOR THE PLOTTER, REDUCE INDG PER D
     &EGREE')
      STOP
C
C     INTERCHANGE X AND Y SCALES
```

```
  110 WRITE(6,2101)
 2101 FORMAT('0THE PLOT WILL BE ROTATED BY 90 DEGREES COUNTERCLOCKWISE')
      ROTA=1
      STOR=DX
      DX=DY
      DY=STOR
      STOR=DELTAX
      DELTAX=DELTAY
      DELTAY=STOR
      STOR=XMIN
      XMIN=YMAX
      YMAX=XMAX
      XMAX=YMIN
      YMIN=STOR
      STOR=XRAS
      XRAS=YRAS+3.5*INRAS
      YRAS=STOR
      IF(NF.EQ.0) GO TO 144
C
C     READ FAULTS
      READ(5,1007)(YF1(I),XF1(I),YF2(I),XF2(I),I=1,NF)
 1007 FORMAT(4F10.0)
C
      GO TO 130
C
  100 XRAS=XRAS+3.5*INRAS
C
      IF(NF.EQ.0) GO TO 144
      READ(5,1007)(XF1(I),YF1(I),XF2(I),YF2(I),I=1,NF)
C
C
  130 DO 143 I=1,NF
      XF1(I)=XF1(I)*DIVR
      YF1(I)=YF1(I)*DIVR
      XF2(I)=XF2(I)*DIVR
  143 YF2(I)=YF2(I)*DIVR
  144 CONTINUE
C
C     CHECK SIGNS
      IF(XMAX.GT.XMIN) GO TO 150
      XMAX=-XMAX
      XMIN=-XMIN
      IF(NF.EQ.0) GO TO 150
      DO 145 I=1,NF
      XF1(I)=-XF1(I)
  145 XF2(I)=-XF2(I)
C
  150 XFMAX=XMAX+DELTAX*4.25/.75
      XFMIN=XMIN-DELTAX
      IF(YMAX.GT.YMIN) GO TO 170
      YMAX=-YMAX
      YMIN=-YMIN
      IF(NF.EQ.0) GO TO 170
      DO 146 I=1,NF
      YF1(I)=-YF1(I)
  146 YF2(I)=-YF2(I)
C
  170 YFMAX=YMAX+DELTAY
      YFMIN=YMIN-DELTAY
C
C     FIND INTERSECTION BETWEEN FAULTS AND FRAME
      IF(NF.NE.0)CALL FRAME(NF,XMIN,YMIN,XMAX,YMAX,XF1,YF1,XF2,YF2)
C
      CALL MODESG(AMODES,'MORTGAT,BIN 502',15)
      AMODES(97)=XRAS
      AMODES(179)=YRAS
      CALL OBJECG(AMODES,0.,0.,XRAS,YRAS)
      IF(ROTA.EQ.0) GO TO 610
      NSTOR=NC
      NC=NR
      NR=NSTOR
  610 CONTINUE
C
      DO 600 IP=1,NBPL
C
      CALL READ(AA,HED2,NR,NC,ROTA)
C
C     FIND LOWER AND UPPER CONTOUR VALUES
      NBVL=NR*NC
      CALL LIMITS(A,NBVL,SKIPAC,CMIN,DC,CMAX)
C
      AMODES(50)=0.
      AMODES(46)=0.
      CALL SUBJEG(AMODES,XFMIN,YFMIN,XFMAX,YFMAX)
C
      IF(AMODES(12).NE.AMODES(13))WRITE(6,2102)AMODES(12),AMODES(13)
 2102 FORMAT('0THE SCALES ARE NOT THE SAME',2E16.5)
C
C     PLOT GRID
      END1(1)=IFIX(YMIN)
      END2(1)=IFIX(XMAX)
      END1(2)=IFIX(YMAX)
      END2(2)=IFIX(XMIN)
C
      DELTA2=1./(8.*INDGX)
      XMUL=4.
      DELTA1=4.*DELTA2
      DELTA3=3.*DELTA2
      SIGN=-1.
C
      I=1
      VAR=END1(2)
      FIX=END2(2)
C
      DO 400 K=1,2
C
  401 XX(I)=FIX
      YY(I)=VAR
      XX(I+1)=FIX+DELTA2
      YY(I+1)=VAR
      XX(I+2)=FIX
      YY(I+2)=VAR
      INTG=ABS(VAR)
      XC=FIX-DELTA2*XMUL
      YC=VAR
C
      CALL INUMBG(AMODES,XC,YC,4,INTG)
      WRITE(6,550)XC,YC,INTG
      I=I+3
      IF(VAR.EQ.END1(K)) GO TO 402
```

```
      VAR=VAR+SIGN
      GO TO 401
C
  402 STOR=VAR
      VAR=FIX
      FIX=STOR
  404 XX(I)=VAR
      YY(I)=FIX
      XX(I+1)=VAR
      YY(I+1)=FIX+DELTA2
      XX(I+2)=VAR
      YY(I+2)=FIX
      INTG=ABS(VAR)
      XC=VAR
      YC=FIX-DELTA3
C
      CALL  INUMBG(AMODES,XC,YC,4,INTG)
      WRITE(6,550)XC,YC,INTG
      I=I+3
      IF(VAR.EQ.END2(K)) GO TO 403
      VAR=VAR-SIGN
      GO TO 404
C
  403 SIGN=-SIGN
      STOR=VAR
      VAR=FIX
      FIX=STOR
      DELTA2=-DELTA2
      XMUL=2.
      DELTA3=-DELTA3
C
  400 CONTINUE
C     DO 500 K=1,I
C 500 WRITE(6,550)XX(K),YY(K)
  550 FORMAT(' ',2F10.3,I6)
      I=I-3
      CALL LINESG(AMODES,I,XX,YY)
C
C     PLOT TITLE
      XT=XFMAX - 3.*DELTAX
      IF(CAL.EQ.29.) AMODES(45)=3.
      AMODES(50)=90.
      AMODES(46)=90.
      YT=YMIN+DELTA1
      CALL VECSG(AMODES,XT,YT,48,HED2)
      AMODES(45)=1.
C
      IF(NF.EQ.0) GO TO 142
C     PLOT FAULTS
C
      PRINT,'FAULTS AS THEY ARE PLOTTED'
      WRITE(6,2011)(I,XF1(I),YF1(I),XF2(I),YF2(I),I=1,NF)
 2011 FORMAT(' ',I5,4F10.3)
      CALL SEGMTG(AMODES,NF,XF1,YF1,XF2,YF2)
  142 CONTINUE
C
C     CHANGE SUBJECT SPACE TO PLOT CONTOURS, THE BOTTOM LEFT CORNER
C     HAS TO HAVE (0.,0.) COORDINATES
      XFMAX1=ABS(XFMAX-XFMIN)-DELTAX
      YFMAX1=ABS(YFMAX-YFMIN)-DELTAY
      PRINT,'DELTAX,DELTAY,XFMAX,YFMAX',DELTAX,DELTAY,XFMAX,YFMAX
      XFMIN1=-DELTAX
      YFMIN1=-DELTAY
C
      PRINT,'RASOR,XRAS,YRAS',RASOR,XRAS,YRAS
      CALL SURJEG(AMODES,XFMIN1,YFMIN1,XFMAX1,YFMAX1)
      PRINT,'SCALE',AMODES(12),AMODES(13)
C
C     CREATE COUNTOURS AND PLOT THEM
      CALL CONTRG(AMODES,A,NR,NC,DX,DY,CMIN,DC,CMAX)
C
      CALL PICTRG(AMODES)
  600 CONTINUE
C
      CALL EXITG(AMODES)
      RETURN
      END
      SUBROUTINE READ(AA,HED2,NROW,NCOL,NROT)
C
C     ******************************************************************
      DIMENSION AA(NROW,NCOL)
      LOGICAL*1 HED2(80)
C     ******************************************************************
C
C     THIS SUBROUTINE READS THE DATA TO BE INTERPOLATED
      READ(10,1002)HED2
 1002 FORMAT(80A1)
C
      IF(NROT.EQ.1) GO TO 110
C
C     THE DATA IS STORED IN A TWO DIMENSIONAL ARRAY, A(1,1) BEING AT
C     THE ORIGIN OF THE PLOT (BOTTOM LEFT CORNER)
C
      DO 100 IR=1,NROW
  100 READ(10,1000)(AA(IR,IC),IC=1,NCOL)
      GO TO 99
C
C     THE PLOT HAS BEEN ROTATED BY 90 DEGREES COUNTERCLOCKWISE,
C     THE ROWS AND COLUMNS ARE INVERTED IN THE CALL STATEMENT
  110 CONTINUE
      DO 200 I=1,NCOL
      IC=NCOL+1-I
  200 READ(10,1000)(AA(IR,IC),IR=1,NROW)
   99 CONTINUE
C
      DO 300 IR=1,NROW
  300 WRITE(6,1001)(AA(IR,IC),IC=1,NCOL)
 1001 FORMAT(' ',13F10.3)
 1000 FORMAT(8F10.5)
C
      RETURN
      END
      SUBROUTINE LIMITS(A,NBVL,SKIPAC,CMIN,DC,CMAX)
C
C     ********************************************************
      DIMENSION  A(1)
      INTEGER SKIPAC
C     ********************************************************:
C
C     SEARCH FOR LOW BOUND
```

```
      IF(SKIPAC.NE.1) GO TO 110
C
C     FIND INTENSITIES USING RICHTER GUTENBERG RELATIONSHIP
      DO 99 IV  =1,NBVL
      A(IV)=3.*(ALOG10(A(IV)*1000.)+.5)
   99 CONTINUE
C
  110 CONTINUE
      XLOW=A(1)
      DO 100 IV=2,NBVL
      IF(A(IV).LT.XLOW) XLOW=A(IV)
  100 CONTINUE
C
C     SEARCH FOR UPPER BOUND
      XHIGH=A(1)
      DO 200 IV=2,NBVL
      IF(A(IV).GT.XHIGH) XHIGH=A(IV)
  200 CONTINUE
C
C     FIND LOW AND HIGH CONTOUR LINES TO BE PLOTTED
      CMIN=-DC
  300 CMIN=CMIN+DC
      IF(CMIN.LT.XLOW) GO TO 300
C
      CMAX=CMIN-DC
  400 CMAX=CMAX+DC
      IF(CMAX.LT.XHIGH) GO TO 400
      CMAX=CMAX-DC
C
      PRINT,'XLOW,XHIGH,CMIN,CMAX',XLOW,XHIGH,CMIN,CMAX
C
      RETURN
      END
      SUBROUTINE FRAME(NF,XMIN,YMIN,XMAX,YMAX,XF1,YF1,XF2,YF2)
C
C     ************************************************************************
      DIMENSION XF1(1),XF2(1),YF1(1),YF2(1)
C     ************************************************************************
C
C     LIMIT THE FAULTS TO THE AREA UNDER CONSIDERATION
      DO 500 IF=1,NF
C
C     CHECK THE X COORDINATES OF THE FAULT
      SIGN=1.
      CHECK=XMIN
C
C     CHECK FOR LOWER AND UPPER BORN
      DO 100 X=1,2
      XF=XF1(IF)*SIGN
C
C     CHECK FOR ORIGIN AND END OF SEGMENT
      DO 200 I=1,2
      IF(XF.GE.CHECK) GO TO 200
      ASL=(YF1(IF)-YF2(IF))/(XF1(IF)-XF2(IF))
      BET=YF1(IF)-ASL*XF1(IF)
      IF(I.EQ.2) GO TO 300
C
C     CHANGE COORDINATES
      XF1(IF)=SIGN*CHECK
      YF1(IF)=ASL*XF1(IF)+BET
      GO TO 200
  300 XF2(IF)=SIGN*CHECK
      YF2(IF)=ASL*XF2(IF)+BET
  200 XF=XF2(IF)*SIGN
      SIGN=-1.
  100 CHECK=-XMAX
C
C     CHECK THE Y COORDINATES OF THE FAULT
      SIGN=1.
      CHECK=YMIN
C
C     CHECK FOR LOWER AND UPPER BORN
      DO 110 X=1,2
      YF=YF1(IF)*SIGN
C
C     CHECK FOR ORIGIN AND END OF SEGMENT
      DO 210 I=1,2
      IF(YF.GE.CHECK) GO TO 210
      ASL=(YF1(IF)-YF2(IF))/(XF1(IF)-XF2(IF))
      BET=YF1(IF)-ASL*XF1(IF)
      IF(I.EQ.2) GO TO 310
C
C     CHANGE COORDINATES
      YF1(IF)=SIGN*CHECK
      XF1(IF)=(YF1(IF)-BET)/ASL
      GO TO 210
  310 YF2(IF)=SIGN*CHECK
      XF2(IF)=(YF2(IF)-BET)/ASL
  210 YF=YF2(IF)*SIGN
      SIGN=-1.
  110 CHECK=-YMAX
C
  500 CONTINUE
      RETURN
      END
      SUBROUTINE CONTRG(AMODES, A, NR, NC, DX, DY, CMIN, DC, CMAX )
C
C                    CONTOUR PLOTTING SUBROUTINE
C                    BY  ROBERT BEEBE (STANFORD)
C                    MARCH 3, 1970
C
C         USE:
C              CALL CONTRG(AMODES,A,NR,NC,DX,DY,CMIN,DC,CMAX)
C         WHERE:
C              AMODES IS THE 200 ELEMENT MODE-SET-ARRAY,
C
C              A IS A ONE DIMENSIONAL ARRAY CONTAINING FUNCTION VALUE
C                OF A UNIFORMLY SPACED GRID.  AA IS ORDERED AS AN
C                EQUIVALENT TWO DIMENSIONAL ARRAY  WITH THE FIRST COLUMN
C                AT THE BOTTOM OF THE GRID (Z=0.) AND THE FIRST COLUMN
C                AT THE LEFT SIDE ((X=0.),  THE SUBJECT SPACE HAS TO BE
C                SET AS FOLLOW (0.,0.,NC*DX,NR*DY)
C
C              NR IS THE NUMBER OF POINTS IN THE Y-DIMENSION.
C
C              NC IS THE NUMBER OF POINTS IN THE X-DIMENSION.
C
C              DX IS THE X-DISTANCE BETWEEN POINTS. (SUBJECT UNITS)
C
C              DY IS THE Y-DISTANCE BETWEEN POINTS. (SUBJECT UNITS)
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C
C               CMIN IS THE LEAST CONTOUR VALUE.
C
C               DC   IS THE CONTOUR INTERVAL.
C
C               CMAX IS THE MAXIMUM CONTOUR VALUE.
C
C   A(CHPI+1)   ICOL    A(CHPI+NR+1)
C       PB       COL         PC
C       .------------------.
C       I         2        /I
C       I       (TNUM)    / I
C       I                /  I
C       I 1            3/   I
C       I              /    I          THIS SKETCH SHOWS SOME
C       I             /     I          OF THE VARIABLE NAMES THAT ARE
C       I            /6     I          USED
C       I           /       I
C       I          /        I
C       I         /         I
C       I        /          I
C       I       /           I
C       I      /            I
C       I     /             I
C       I    /   (TNUM+1)   I
C       I   /        5      I  IROW
C       I/                  I  ROW
C       .------------------.
C       PA                 PD
C   A(CHPI)             A(CHPI+NR)
C
C     ******************************************************************
      REAL*4 A(1), DX, DY, CMIN, DC, CMAX, CNTR, UPPER, LOWER
      REAL*4 PA, PB, PC, PD  , INTERF, AMODES(200)
C     ******************************************************************
C
C   CALCULATE THE MINIMUM DIMENSION FOR THE ARRAY STATUS:
C  MIN = (NR-1)*(NC-1)*2
      LOGICAL*1 STATUS(9804), FULL/.FALSE./, EMPTY/.TRUE./
C
      INTEGER*4 NR, NC, RECTR, RECTC, NTRNG, ROW, COL, NCNTR, LEVEL
      INTEGER*4 SIDIN, SIDOUT, CHPI, TNUM
C
      INTERF(O,I,B,A)= O*((I-1)+(CNTR-A)/(B-A))
C
C
C  SEE IF MODE-SET ARRAY HAS BEEN INITIALIZED
      IF (AMODES(1) .NE. 1.)     CALL ERRZZ(AMODES,36,0.)
C
C  SEE IF THE ERROR CONDITION IS TOO HIGH
      IF (AMODES(15) .GT. AMODES(22))    RETURN
      PRINT,'VECTOR A AS IT IS PLOTTED'
      NVL=NC*NR
      WRITE(6,1000)(A(I),I=1,NVL)
 1000 FORMAT(' ',12F10.3)
C
C
C
C  HOW MANY CONTOURS
      NCNTR = (CMAX-CMIN)/ DC + 1.
      RECTR = NR-1
      RECTC = NC-1
      CNTR = CMIN
      NTRNG = 2*RECTR*RECTC
      INCR = RECTC*2 + 1
C
C  IF THE VALUE OF ANY POINT IS EXACTLY EQUAL TO THAT OF
C  ONE OF THE CONTOUR LEVELS, WE WILL GET UNDERFLOWS AND
C  DIVIDE CHECKS ALL OVER THE PLACE. WE WILL LOWER ANY
C  OFFENDING POINT BY THIS SMALL AMOUNT:
      OFFSET = DC/1000.
C  ONCE FOR EACH CONTOUR
      DO 18 LEVEL = 1, NCNTR
C
      DO 5  I= 1, NTRNG
    5 STATUS(I) = EMPTY
      IPEN = 0
C
C  ONCE FOR EACH ROW OF ENCLOSED GRIDS
      DO 16 IROW = 1, RECTR
C
C  ONCE FOR EACH COLUMN OF ENCLOSED GRIDS
      DO 12 ICOL = 1, RECTC
      ROW = IROW
      COL = ICOL
C
      CHPI = (COL-1) * NR  + ROW
C  ELEVATIONS OF THE 4 CORNERS OF THE GRID
      PA = A(CHPI)
      PB = A(CHPI+1)
      PC = A(CHPI+NR+1)
      PD = A(CHPI+NR)
C
C  MAKE SURE NO ELEV MATCHES A CONTOUR.
C   (THAT WOULD CAUSE A DIVIDE EXCEPTION)
      IF( PA .EQ. CNTR )  PA = PA+OFFSET
      IF( PB .EQ. CNTR )  PB = PB+OFFSET
      IF( PC .EQ. CNTR )  PC = PC+OFFSET
      IF( PD .EQ. CNTR )  PD = PD+OFFSET
      UPPER=AMAX1( PA,PB,PC,PD )
      LOWER=AMIN1( PA,PB,PC,PD )
      IF( CNTR.GE.LOWER.AND.CNTR.LE.UPPER) GO TO 100
C
C  GO TO THE END OF THE COLUMN LOOP
      GO TO 12
C
C       THIS OUT-OF-LINE CODING IS FOR CLOSING CONTOURS THAT HAVE
C       RUN OFF THE EDGE OF THE GRID
   25 CALL LINEG(AMODES,IPEN,X,Y)
      GO TO 12
C
C          THIS OUT-OF-LINE CODING  SHOULD FOLLOW A CONTOUR
C          FROM ITS FIRST APPEARANCE TO EITHER,
C          CLOSURE, OR EXIT FROM THE GRID.
  100 IF( CNTR .GE. PA ) GO TO 110
      IF( CNTR .GE. PB ) GO TO 150
      IF( CNTR .GE. PD ) GO TO 170
      GO TO 160
  110 IF( CNTR .LT. PB ) GO TO 150
      IF( CNTR .LT. PD ) GO TO 170
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APPENDIX 5

NEWMARK AND HALL PAPER

(Taken from reference 52)

PROCEDURES AND CRITERIA FOR EARTHQUAKE RESISTANT DESIGN

by

N. M. Newmark and W. J. Hall

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I. Introduction

When a building or other structure is subjected to earthquake motion, its base or support moves with the ground. Since this motion is relatively rapid, it causes stresses and deformations throughout the structure. If we neglect temporarily the interaction between the base of the structure and its foundation, when the structure is quite rigid, its motion is nearly the same as that of the ground and the dynamic forces acting on it are very nearly equal to those associated with the ground accelerations applied to the structure as a rigid body. If the structure is quite flexible, large relative motions or strains can be induced in the structure because of the differential movements between the supports and the masses of the structure. In order to survive the earthquake motions the structure must be either strong enough or ductile enough to resist the forces generated by these deformations; the combination of the required strength and ductility is a function of the stiffness or flexibility of the structure.

Seismic effects on a structure, component or element, depend not only on the earthquake motion but also on the properties of the structure, component or element itself. Among these properties, the most important are the energy absorption within it or at interfaces between the item under consideration and its support, either due to damping or inelastic behavior, its period of vibration, and its strength or resistance.

It is the purpose of this paper to describe the general nature of the principles upon which earthquake resistant design is based, and to consider the development of design procedures for the design of structures, facilities or components.

Examples of structures that did not have sufficient strength and ductility to resist the earthquakes to which they were subjected are well known. Failures occurred in the columns and frames of buildings in Caracas, for example, when inadequate strength and energy absorbing capacity were available for the earthquake of 1967. Other failures in earthquakes are clearly due to lack of adequate support details, or lack of adequate continuity between individual elements.

Emphasis is placed herein on design as contrasted with analysis, and essentially on preliminary design or the selection of the general outline, type of framing, and first estimate of requirements. This choice of emphasis is made because methods suitable for such purposes generally can assure adequate performance and serve as a check on designs made by more sophisticated methods.

The general concepts presented in this paper have been adapted from those given in References [1], [2], [3], and [4].

The design of a structure, as either a complete system in itself, or as part of the system of which the structure is only a component, can be a highly complex matter involving a number of input data of various types and a host of special requirements. Once the structure has been dimensioned, that is, laid out in plan and the size and strength of its various elements selected, then the analysis of the structure for given conditions of loading and foundation motion can be made by relatively well understood methods, even though the analysis can be a tedious and lengthy one for a complex system. However, unless the designer uses a so-called "direct design" procedure, he is faced with a problem of the preliminary selection of the structural layout, framing, element strength, and the like, before he has a structure which he can analyze. Even with direct design procedures, for important structures he will want to have some handy approximation that can be used for his preliminary studies.

The steps which the designer must take are generally as follows:

(1) Select the earthquake hazard.

(2) Select the safety factor, or the allowable limits of deformation, or the allowable probability of damage or failure. This may depend on step (3).

(3) Select the type or layout of the structure, and estimate its dynamic (and static) parameters. These include a) dynamic resistance, b) natural frequency or period of vibration, c) damping or energy absorption, d) ductility that can be counted on before failure. These may be assigned in a direct-design procedure, or are subject to successive revision in more traditional procedures.

(4) Verify the adequacy of the structure selected, and make any necessary changes in strength or other parameters, or in the complete layout or plan.

(5) Repeat steps (3) and (4) until a satisfactory design is achieved.

(6) Make a more accurate analysis of the final design, and make further changes that may be necessary. If these are not minor, steps (3) to (6) may need repeating. In some cases revisions in steps (1) and (2) may be desirable. In other cases an upper bound direct procedure may be used involving essentially only steps (1), (2), and (3). Most so-called "static design codes" are intended to be of this type.

II. Earthquake Hazard

Earthquakes are relatively rare occurrences, but in many regions of the world one can count on a high probability of at least a small earthquake occurring once in the lifetime of a building. However, the stronger or more intense the earthquake the smaller is the probability of its occurrence. An earthquake that has a relatively high probability of occurrence is appropriately considered as a loading for which the design must provide in such a way that the cost of the minor repairs required is not excessive. Major strengthening of a structure to resist intense forces is expensive, and the cost of such design provisions must be weighed against the possible cost of repairs in order to design whether the additional design strength or ductility is economically justified.

It is generally agreed that structural collapse of such a nature that it might endanger a great many lives should be prevented by the design, even for the maximum credible earthquake. But it would be unreasonable and uneconomical to provide for resistance to an extreme earthquake with the same factor of safety or margin of safety that one normally uses. The selection of the factor of safety for the maximum credible earthquake is in part dependent on the nature and importance of the structure, and on the consequences should the structure fail.

Unfortunately the earthquake hazard for which designs should be made is subject to a high degree of uncertainty. In only a few areas of the country are there relatively long periods of observation of strong earthquake motions. By correlating the available strong motion records with the more common records available from the sensitive recording instruments used by seismologists, and by use of qualitative reports of the effects of earthquakes where motion records are not available, some measure can be obtained of the maximum intensities which have occurred in various geological regions, and predictions can be made of those which might occur in the future. In other regions of the country, where records are scarcer, estimates of a similar nature can be inferred but are much more uncertain.

However, the maximum historical earthquake determined by such a procedure is not a proper measure of the possible intensity of an earthquake which might occur in the future nor of the earthquake for which the design should provide. At some sites, maximum or extreme earthquakes might never have occurred in the past; it is almost certain that they did not occur within the period of recorded history.

In order to specify adequately the earthquake intensity for either the historical or the extreme earthquake, one must do more than determine the possible or probable acceleration of the ground. The character of the earthquake motions must also be described in a way that is representative of the geologic conditions, taking into account the local soil conditions including overburden depths and characteristics, presence of water, depth to basement rock, and the like. A better measure of the free-field earthquake motions is a description which includes not only the maximum ground acceleration, but also the maximum ground velocity and the maximum ground displacement, with some measure of the number of pulses or the duration of the strong motions that should be considered. All of these quantities are dependent on the geologic and soil characteristics, and are in part dependent on the soil-structure interaction of the structure supported on the soil or rock.

Earthquakes in different parts of the world on different types of foundations and rock or soil strata have greatly different characteristics. The Niigata Earthquake of 1964 was characterized by the phenomenon of liquefaction of the soil and foundation failures causing tilt and overturning of a number of buildings that otherwise would not have been badly damaged. The earthquakes in Mexico City are affected by the natural frequency of the huge bowl of soft jellylike soil which underlies most of the central part of the city, and which emphasizes and amplifies motions in the range of periods of vibration of 2 to 2.5 sec. and diminishes those of very low period, of less than 0.5 sec.

The low buildings, the old churches and the cathedral in Mexico, many of which have been in existence several hundred years during which time Mexico has been subjected to serious shaking from earthquakes, generally have not been severely damaged by earthquakes, but modern tall buildings were seriously damaged, especially in the earthquake of 1957. Nevertheless, one building, construction of which was completed during the early 1950's, the Latino Americana Tower, survived this earthquake and, subsequently, the several slightly less intense earthquakes of the past decade with no damage, not even cracking of window panes. This is due to the fact that the expected nature of the earthquake motions was taken into account in its design. The building is of interest because it was the first building which was designed in accordance with modern analytical methods that was actually subjected to an earthquake approaching the intensity of the earthquake for which it was designed.

With the increased knowledge of the characteristics of earthquakes from the records obtained of strong motions in earthquakes in various parts of the world, we now have the basis for a much more detailed description of the type and intensity of earthquake motions that should be considered for design of structures of various types in various regions, taking into account the geologic conditions as well as the foundation materials under the buildings.

III. General Design Concepts

General Principles

The designer's freedom of choice in selecting methods of resisting earthquake motions is restricted by the necessity that he comply with the architectural form selected for the building. If the form follows the function, the constraints are generally minimal. However, it is not necessarily true that an efficient earthquake resisting capability can be put into any arbitrary form of envelope for the structure. The designer must, therefore, have latitude in his selection of the resisting elements of the structure. He may choose a flexural framework; or a structure having resistance primarily in the outer walls as a monocoque assembly; or a structure strengthened by shear walls or by bracing; or a structure with a resisting core from which the lower parts of the building are hung; or various modifications and combinations of these. The methods of achieving strength and ductility in these various forms are necessarily different and the design criteria have to take this into account.

The permissible level of response of a structure, element or component, must be associated with a loading criteria. The response criteria should properly be dependent on the type of structure, the relative cost of repairs for minor damage, and the hazard in terms of possible loss of life should the item fail or reach extreme deformation limits. The seismic resistance of an element is a function primarily of its yield strength, its natural frequency of vibration, its damping and energy absorption in the elastic range, and its ductility and energy absorption capacity in the range before unacceptable damage occurs.

Dynamic Resistance

Detailed descriptions of the response of simple elastic systems, or more complex structure and elements, subjected to dynamic loading and especially to seismic loading, are given in References [2], [3], and [4]. In general, it can be shown that the response of a simple damped oscillator to a dynamic motion of its base can be represented graphically in a simple fashion by a logarithmic plot as shown in Figure 1. In this figure, there are shown on the one plot, using four logarithmic scales, the following three quantities:

D = Maximum relative displacement between the mass of the oscillator and its base

V = Maximum pseudo relative velocity = ωD

A = Maximum pseudo acceleration of the mass of the oscillator = $^{2}\omega D$

In these relations, ω is the circular natural frequency of the oscillator.

The effective maximum ground motions for the earthquake disturbance for which Fig. 1 is drawn are maximum ground displacement d_m = 10 in., maximum ground velocity v_m = 15 in. per sec., maximum ground acceleration a_m = 0.3g, where g is the acceleration of gravity. The curve shown is a smooth curve rather than the actual jagged curve that one obtains from a precise calculation. The symbols 1, 2 and 3 on the curve represent oscillators, item 1 having a frequency of 20 cps, item 2 of 2.5 cps, and item 3 of 0.25 cps. It can be seen that for item 1 the maximum relative displacement is extremely small, but for item 3 it is quite large. On the other hand, the pseudo acceleration for item 3 is relatively small compared with that for item 2. The pseudo relative velocities for items 2 and 3 are substantially larger

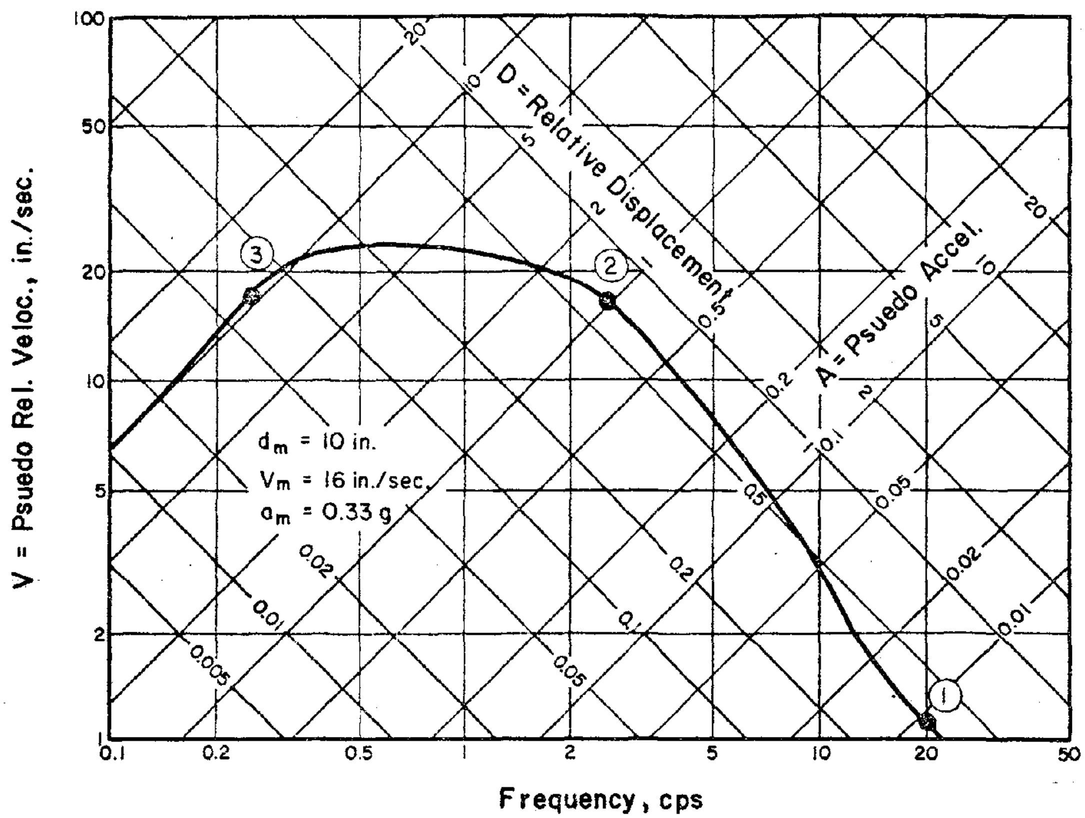

FIG. I RESPONSE SPECTRUM FOR TYPICAL EARTHQUAKE than that for item 1.

The advantage of using the tripartite logarithmic plot, with frequency plotted also logarithmically, is that one curve can be drawn to represent the three quantities D, V and A. The pseudo relative velocity is nearly the same as the maximum relative velocity for higher frequencies, but differs substantially for very low frequencies. It is, however, a measure of the energy absorbed in the spring. The maximum energy in the spring, neglecting that involved in the damper of the oscillator, is $MV^2/2$, where M is the mass of the oscillator.

The pseudo acceleration is practically the same as the maximum acceleration, and the quantity MA is precisely the maximum force in the spring. Therefore, the pseudo acceleration is exactly the same as the maximum acceleration when there is no damping.

In the discussion and figures which follow, the terms "velocity" will be used for V and "acceleration" for A without the explanatory words maximum, pseudo, relative or absolute.

There are many strong motion earthquake records available. One that has been used for a number of years is that for the El Centro earthquake of May 18, 1940. The response spectra computed for the earthquake for several different amounts of damping are shown in Figure 2. The oscillatory nature of the response spectra, especially for low amounts of damping, is typical of the nature of response spectra for earthquake motions in general. A replot of Fig. 2 is given in Fig. 3 in a dimensionless form where the scales are given in terms of the maximum ground motion components. In this figure, the ground displacement is given by the symbol y, and the subscript m designates a maximum value. Dots over the y indicate differentiation with respect to time.

It can be seen from Fig. 3 that for relatively low frequencies, below something of the order of about 0.05 cps, the maximum displacement response D is practically equal to the maximum ground displacement. For intermediate frequencies, however, greater than about 0.1 cps, up to about 0.3 cps, there is an amplified displacement response, with amplification factors running up to about three or more low values of the damping factor β.

For high frequencies, over about 20 to 30 cps or so, the maximum acceleration is practically equal to the maximum ground acceleration. However, for frequencies below about 6 cps, ranging down to about 2 cps, there is nearly a constant amplification of acceleration, with the higher amplification corresponding to the lower values of damping. In the intermediate range between about 0.3 to 2 cps, there is nearly a constant velocity response, with an amplification over the maximum ground velocity. The amplifications also are greater for the smaller values of the damping factor.

The results shown in Fig. 3 are typical for other inputs, either for other earthquake motions or for simple types of dynamic motion in general. The data from which Fig. 3 was drawn, as well as other similar figures, are taken from Reference [2].

Natural Frequency

The dynamic response of a structure is a function, among other things, of its natural frequencies of vibration in its various modes. Natural frequencies can be computed from the mass and stiffness distributions of the structure but such calculations involve an idealization of the structure for the purpose of the analysis. The influence of nonstructural components on natural frequencies can be of particular importance. Also the natural frequencies may be affected to a large degree by the foundation-structure

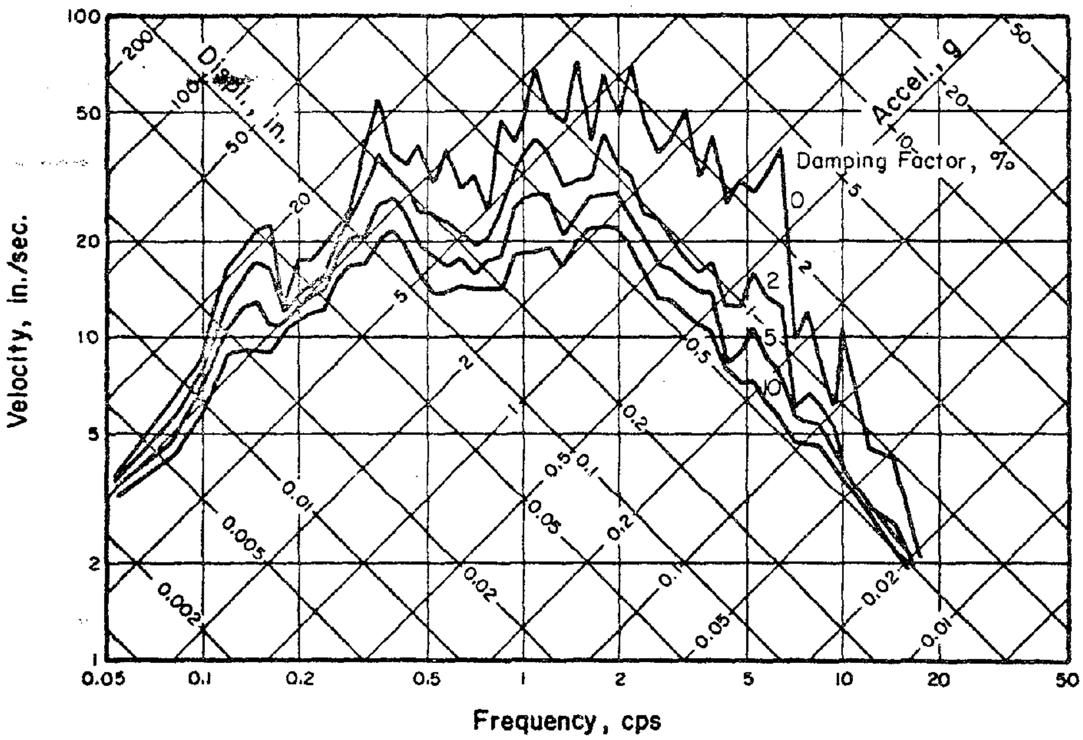

FIG. 2 RESPONSE SPECTRA, EL CENTRO EARTHQUAKE, MAY 18, 1940, NORTH-SOUTH DIRECTION

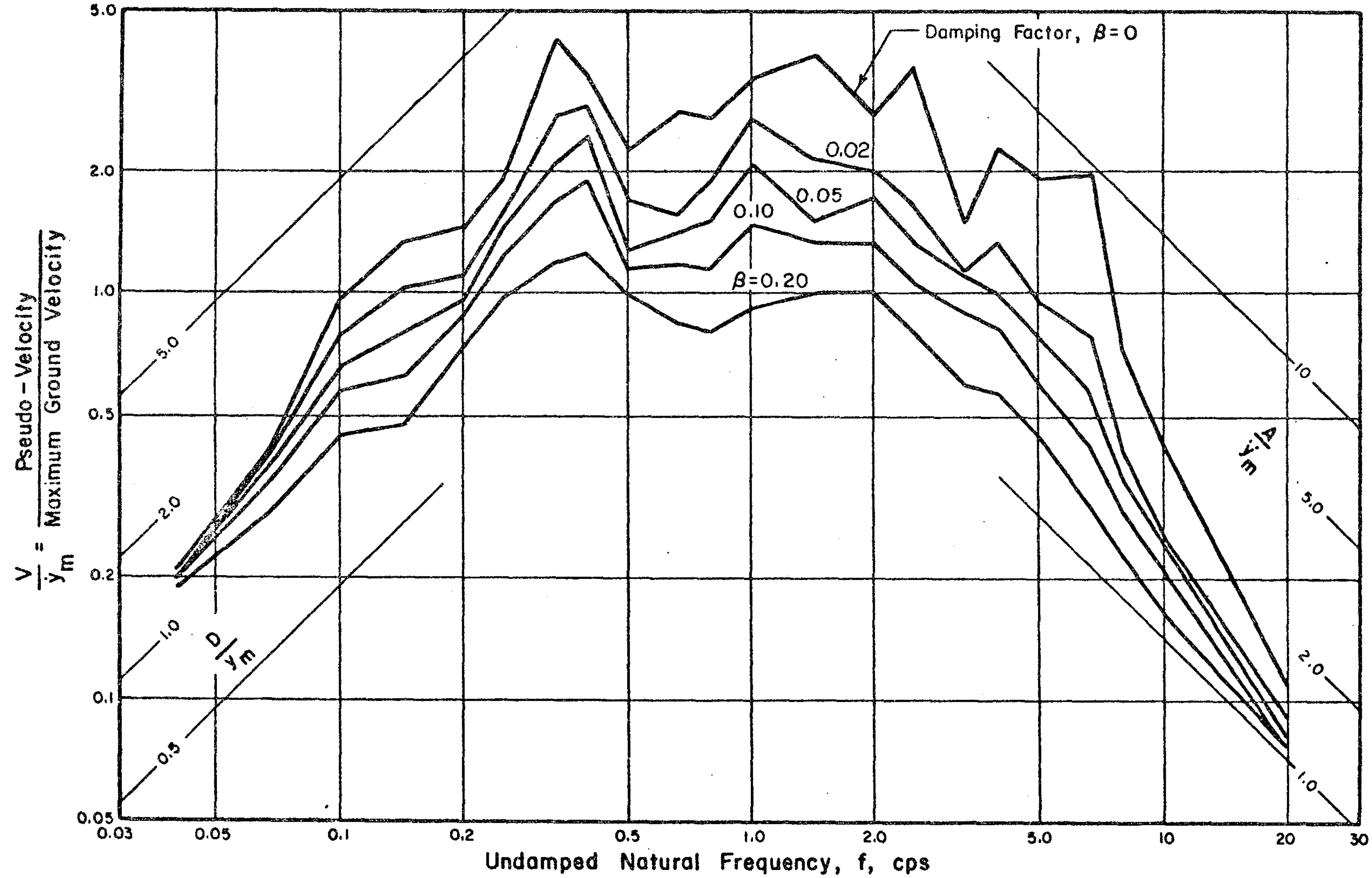

FIG. 3 RESPONSE SPECTRA, ELASTIC SYSTEMS, EL CENTRO 1940 EARTHQUAKE

interaction.

Design specifications which involve natural frequencies have the disadvantage that the structure must be designed, at least in a preliminary way, before the frequencies can be determined, or else the frequencies must be estimated from factors involving judgment and overall dimensions. Hence, such methods may involve relatively large errors in the response or else the method of design must be one of continuing approximations and revisions.

Damping

Energy absorption in a structure arises in various ways including damping or energy absorption of various types within the structure itself, friction or viscous damping, or other types of damping in the structure as well as in the parts of the structure interfering with each other or moving against one another. These can all be generally approximated by use of a damping coefficient. The damping is a function of the intensity of motion and of the stress levels induced within the structural components, and it is highly dependent on the makeup of the structure and the energy absorption mechanisms within it and at its interfaces with the foundation or with other structures. The importance of damping is indicated by the fact that the dynamic response of a structure in an earthquake may be affected to as great a degree by damping as by almost any other parameter. This is especially true in those instances when long sustained nearly harmonic motions are involved. It is because of this reason that the greatest difficulties are found with design specifications other than of the performance type in which the design forces do not properly reflect the differences in damping associated with different materials, different types of framing, and different levels of allowable deformation and stress.

Inelastic Behavior and Ductility

Let us now consider the situation in which the simple oscillator has a spring which can deform inelastically during the response. The simple resistance-displacement relationship for the spring is shown by the light line in Fig. 4, where the yield point is indicated, with a curved relationship showing a rise to a maximum resistance and then a decay to a point of maximum useful limit or failure at a displacement u_m. An equivalent elasto-plastic resistance curve is shown by the heavy line in the figure, rising on a straight line to a point where the yield displacement is u_y and the resistance r_y, and then extending without appreciable increase in resistance to the maximum displacement u_m. The effective resistance curve is drawn so as to have the same area between the origin and u_y as the actual curve, and again the same area to the maximum displacement point. The ductility factor μ is defined as the ratio between the maximum permissible or useful displacement to the yield displacement, for the effective curve.

It is convenient to use an elasto-plastic resistance-displacement relation because one can draw response spectra for such a relation in generally the same way as the spectra were drawn for elastic conditions in Fig. 2 and 3. In Fig. 5 there are shown acceleration spectra for elasto-plastic systems having 2% of critical damping for the El Centro 1940 earthquake. Here, the symbol D_y represents the elastic component of the response displacement, but is not the total displacement. Hence, the curves also give the elastic component of maximum displacement as well as the maximum acceleration, A, but they do not give the proper value of maximum velocity. This is designated by the use of the symbol V' for the pseudo velocity drawn in the figure. The figure is drawn for ductility factors ranging from 1 to 10. It is typical of other acceleration spectra for elasto-plastic systems, as indicated by the acceleration spectra shown in Fig. 6 for the step displacement pulse sketched in the figure.

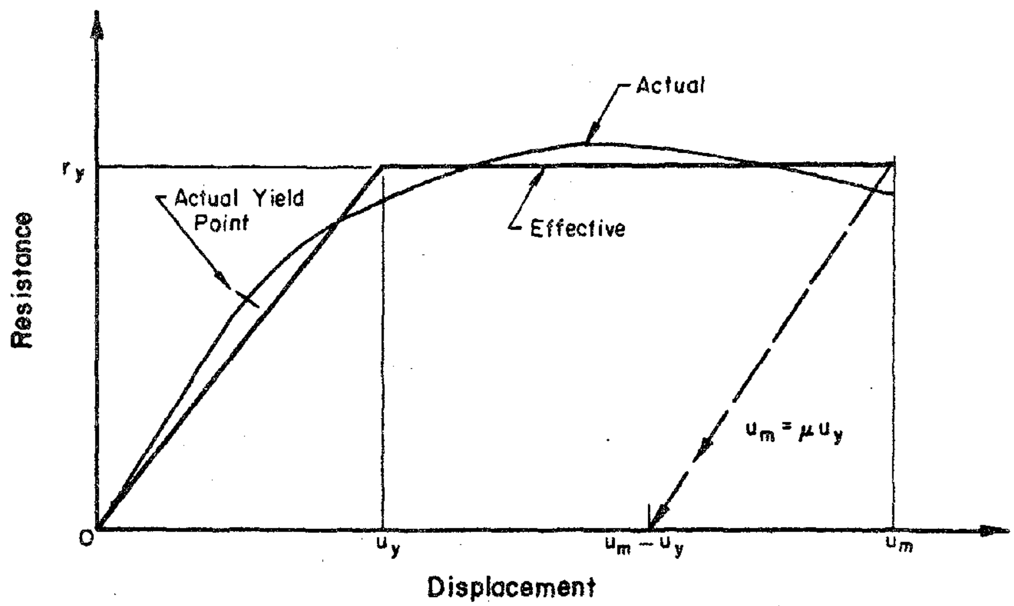

FIG. 4 RESISTANCE — DISPLACEMENT RELATIONSHIP

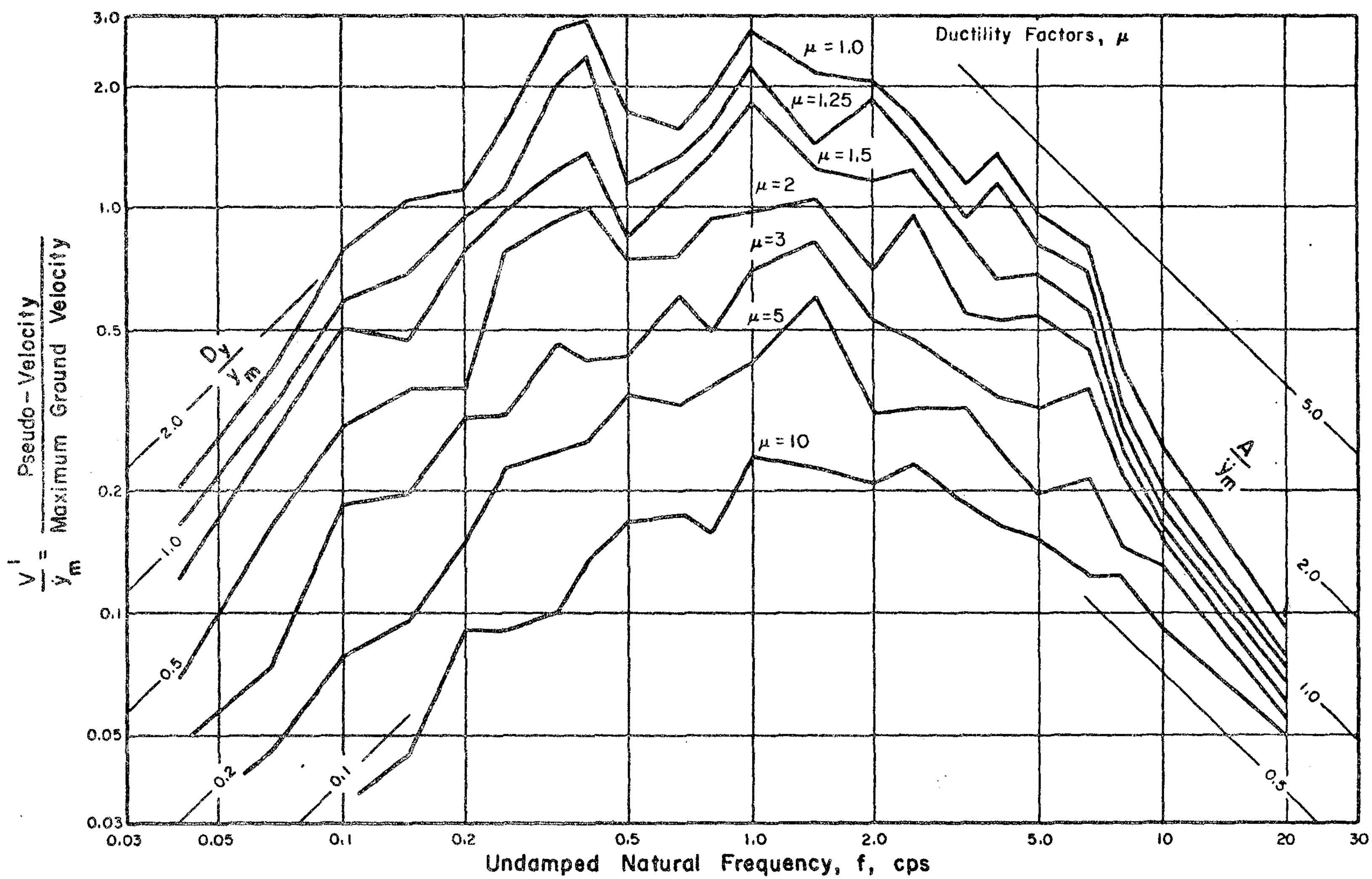

FIG. 5 ACCELERATION SPECTRA FOR ELASTOPLASTIC SYSTEMS, TWO PERCENT CRITICAL DAMPING, EL CENTRO 1940 EARTHQUAKE

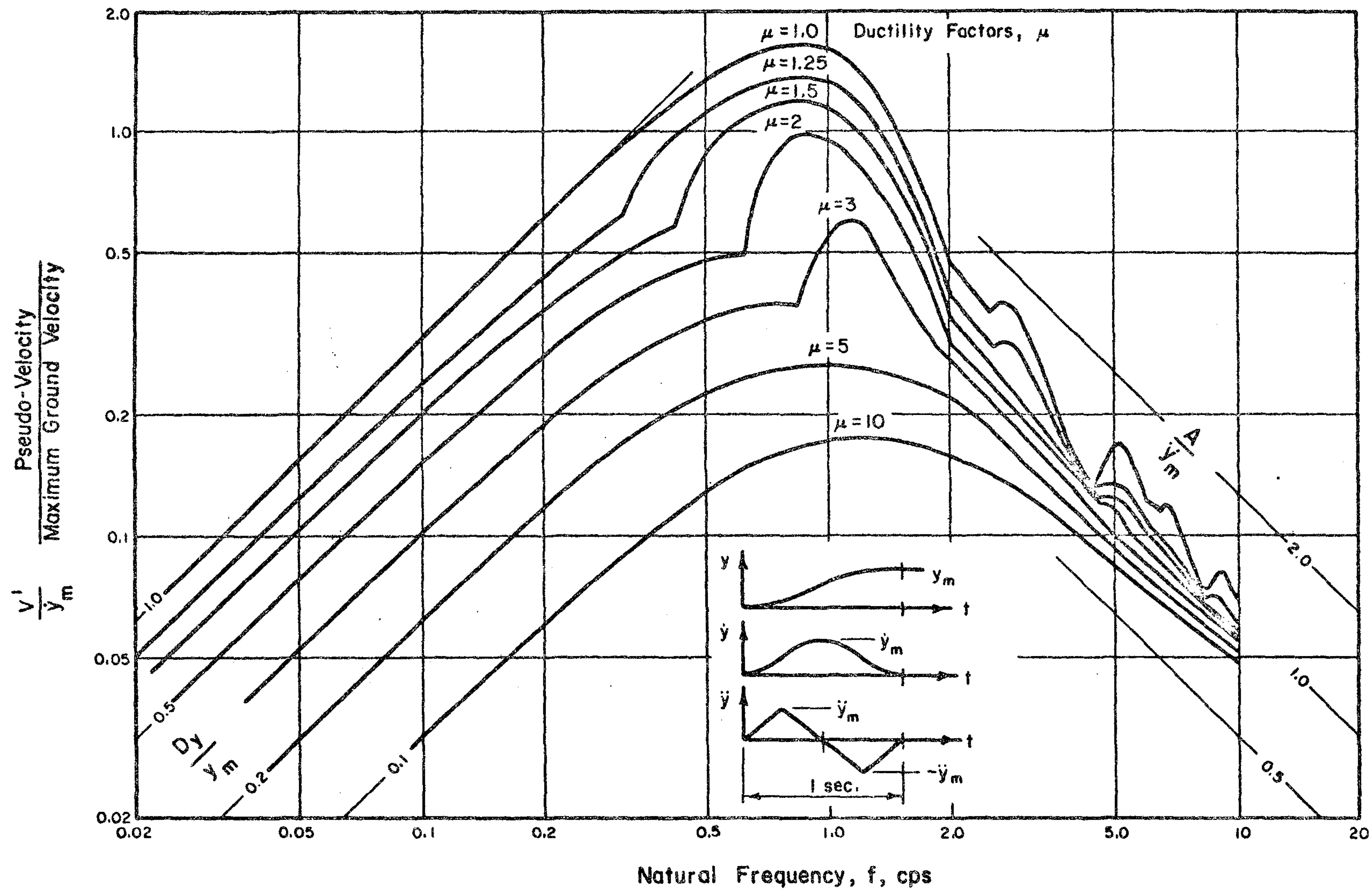

FIG. 6 ACCELERATION SPECTRA FOR UNDAMPED, ELASTOPLASTIC SYSTEMS, STEP DISPLACEMENT

Figure 6 is drawn for a step displacement pulse corresponding to the two triangular pulses of acceleration shown, where the total length of time required to reach the maximum ground or base displacement is 1 second. The frequency scale shown in Fig. 6 will be changed for any other length of time, t, to reach the maximum displacement by dividing the frequencies f by t. In other words, for a step displacement pulse that takes 0.2 sec., the abscissa for a frequency of 1 cps would be changed to 5 cps, and that for 3 cps in the figure would be changed to 15 cps, etc. The general nature of the similarity between Figs. 5 and 6 is important.

One can also draw a response spectrum for total displacement, as shown in Fig. 7. This is drawn for the same conditions as Fig. 5, and is obtained from Fig. 5 by multiplying each curve's ordinates by the value of ductility factor μ shown on that curve. It can be seen that the maximum total displacement is virtually the same for all ductility factors, actually perhaps decreasing even slightly for the larger ductility factors in the low frequency region, for frequencies below about 2 cps. Moreover, it appears from Fig. 5 that the maximum acceleration is very nearly the same for frequencies greater than about 20 to 30 cps for all ductility factors. In between, there is a transition. These remarks are applicable to the spectra for other earthquakes also. One can generalize about them in the following way for general nonlinear relations between resistance and displacement, for single degree of freedom structures.

For low frequencies, corresponding to something of the order of about 0.3 cps as an upper limit, displacements are preserved. As a matter of fact, the inelastic systems have perhaps even a smaller displacement than elastic systems. For frequencies between about 0.3 to about 2 cps, the displacements are very nearly the same for all ductility factors. For frequencies between about 2 up to about 6 cps, the best relationship appears to be to equate the energy in the various curves, or to say that energy is preserved, with a corresponding relationship between deflections and accelerations or forces. There is a transition region between 6 and 20 to 30 cps, depending on the damping ratio. Above 20 to 30 cps, the force or acceleration is nearly the same for all ductility ratios.

Structure-Foundation Interaction

Earthquake motions are transmitted through the ground to the foundation of a structure and then to the structure itself. The interaction between the foundation components of the structure and the earth upon which it rests are of particular importance in defining ~~the nature of~~ the forces and motions transmitted to the structure. Energy absorption can take place at the interfaces between the structure and the foundation, and between the foundation and the supporting medium. Under certain conditions amplifications of motion may even occur. The interaction between the foundation medium and the foundation structure can be particularly complicated when the building is set into the soil or rock rather than resting upon it.

Design specifications either of the cookbook type, the intermediate type, or the type solely concerned with environmental and performance criteria fall short of their requirements if they do not consider the interaction between the structure and its supports, and especially the type of supports, whether it is pile or caisson foundations, isolated footings, or a mat, or some combination of these.

Nonstructural Components

In buildings, particularly, it is necessary to make a distinction between those components which are essential parts of the structure in its resistance to loads and deformations, and nonstructural components which are those parts needed to perform the proper function of the structure but which

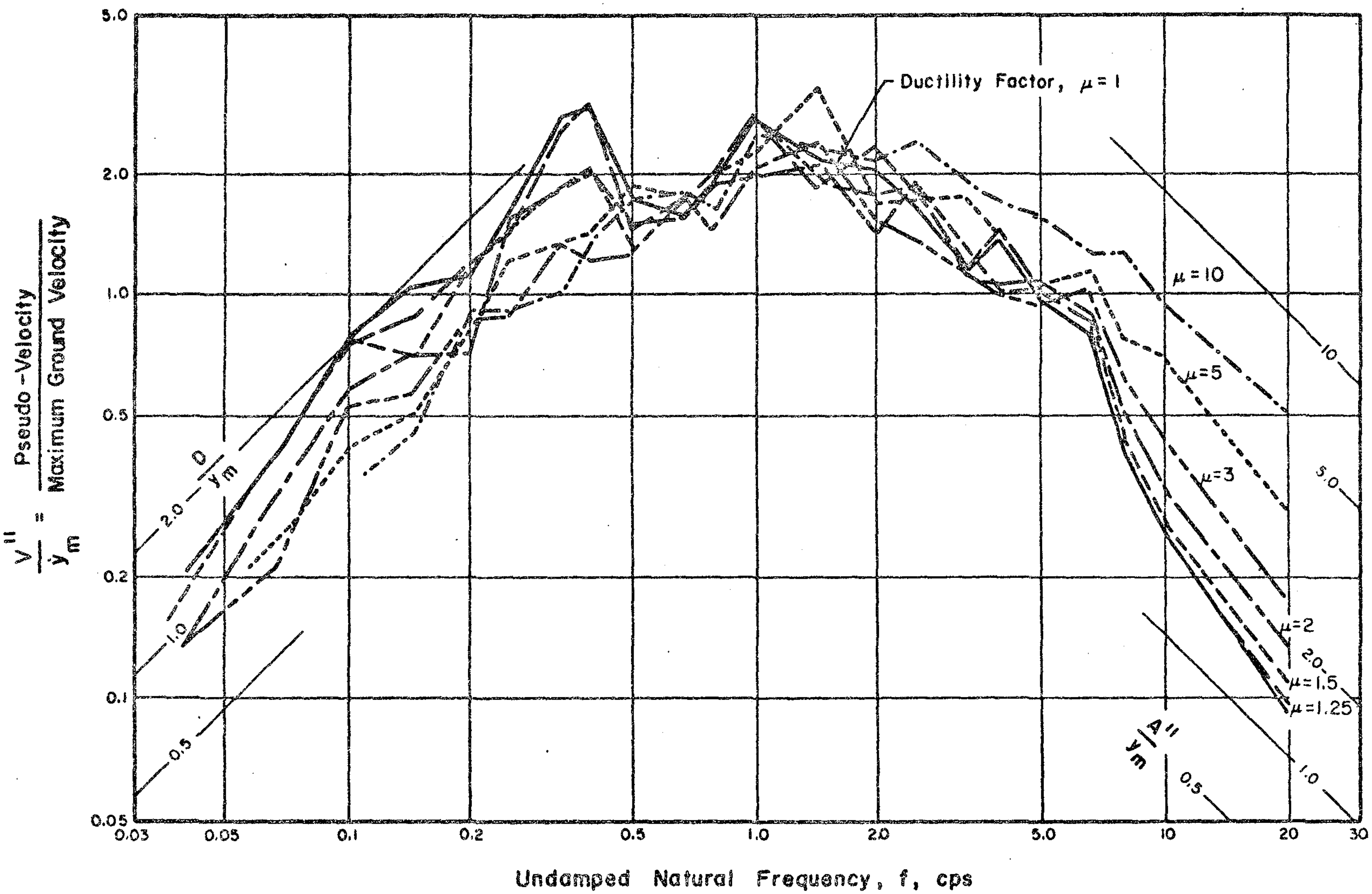

FIG. 7 TOTAL DISPLACEMENT SPECTRA, ELASTOPLASTIC SYSTEMS, TWO PERCENT CRITICAL DAMPING, EL CENTRO 1940 EARTHQUAKE

are not added primarily for resistance to lateral forces. Partitions in a building may be structural or nonstructural depending upon whether they are designed to act as part of the load-carrying framing. However, whatever the designer's intent may be, all the elements of the structure, whether functional or otherwise, have an effect on the behavior of the building under dynamic excitation, and must be considered in terms of dynamic response, strength, and the damage which may be caused by exceeding allowable stress or deformation limits. Even nonfunctional ornamentation on a building that can be dislodged because of lateral motion can cause hazard to life as well as property.

IV. Design Procedures

General Approach

The designer has considerable freedom of choice, in general, as to the type of resisting structure he will use in the design. He may choose a flexible, energy-absorbing structure which can comply with the ground motion readily, or he can use a rigid structure to limit the relative deformation within the structure itself. In one case the strains in the structure are determined primarily by the maximum transient ground displacement or velocity, and in the other they are determined primarily by the maximum transient ground acceleration.

If the structure is in an intermediate range of stiffness, then its energy absorbing capacity is of the greatest importance, which involves both its strength and ductility in some balanced manner. Under these conditions, one may reduce the strength and increase the ductility, or increase the strength and reduce the ductility, in both cases arriving at a satisfactory design. Although the designer must be careful in the determination of these balances, and must look into the strength and ductility of elements or components as well as those of the completed assemblage, he can make up for deficiency in one by an overdesign in the other, in many instances. Constraining the designer to use always highly ductile elements may be unreasonably restrictive since it appears possible to design a structure with as much margin to resist failure by making it less ductile but stronger, in an appropriate fashion. It appears unwise to establish design criteria solely on preconceived notions as to either strength or ductility without considering the combination of both of these that is required for adequate performance.

Theoretically and to a considerable extent practically, it is possible to use any material in almost any fashion one chooses to use it, by providing the proper combination of strength and ductility associated with the particular structural configuration and dimensions, thereby to insure that the completed structure will be able to perform adequately under the appropriate loading or motion environmental conditions.

It has already been noted that in many structures it is desirable, and in fact quite proper and reasonable, for the structure to go well into the inelastic range of behavior, especially for the maximum or extreme environmental seismic conditions. Different types of framing and different materials pose a variety of problems for an adequate specification of performance involving deformations and stresses beyond yield. This has been taken into account in existing codes in various ways, usually by specifying the relative intensity of loading to be considered for different types of framing. Each material must be studied from the point of view of its particular characteristics of strength and ductility, when fabricated into structural members or elements, or when connected together to form a structure. The performance criteria must be prepared in such a way as to avoid unusual handicaps to any one type of framing or material, or to give unusual advantages to any other type.

Selection of Parameters

In the light of the preceding discussion, we can now develop a basis for design of structures, elements or components, where these are subjected directly to the ground or base motion for which we have maximum values of displacement, velocity and acceleration. We first proceed with selection of values of damping. Table 1 is reproduced with some changes from References [5] and [6], and gives the percentage of critical damping for various types and conditions of structures or elements, as a function of stress level. It represents the best information available at the present time, but certainly involves a great deal of judgment and interpretation.

The damping in structural elements and components and in supports and foundations of the equipment is a function of the intensity of motion and of the stress levels introduced within the structural component or structure, as well as being highly dependent on the makeup of the structure and the energy absorption mechanisms within it. For example, a structure with riveted or bolted joints that can undergo relative motion during deformation will absorb a great deal of energy in friction in these joints. A reinforced concrete beam that is cracked, where the elements on the two sides of the crack can move relative to one another with the absorption of energy at the faying surface, will also absorb considerable energy. On the other hand, a homogeneous solid structure or a welded steel structure has relatively small amounts of lost energy because of play in the joints, and a concrete beam before cracking has a relatively small amount of energy losses except those within the material itself. Hence, the degree of damping depends on the framing and makeup of the structure or elements, and on the material used and the stress level within the material for the degree of excitation which it experiences in the shaking motion. For low stress levels and for homogeneous structures, steel or reinforced concrete below cracking levels, the damping may be no greater than in the range of one-half to one percent. For stresses at the level of working stresses or at about half the level of yield point values, the damping may range from about 2 percent for welded steel structures, for well reinforced concrete structures with only small amounts of cracking or for prestressed concrete structures, to 3 percent to 5 percent for ordinary reinforced concrete structures with considerable cracking, and possibly above 5 percent for riveted or bolted connections, or for wood structures with nailed joints and the like. At or near yield point values of stress, the damping may be in the range of about 5 percent for steel structures and prestressed concrete structures that have not completely lost their prestress, ranging to 7 to 10 percent for ordinary reinforced concrete, and as high as 10 to 15 percent for structures with play in the joints, or for masonry structures.

The fundamental frequency of vibration, or its reciprocal, the fundamental period, is best estimated by a simple calculation by use of standard methods of analysis such as are described in Reference [3]. For buildings simple rules, also given in [3], are often used to approximate the fundamental frequency, but are generally not reliable for unusual types of framing or for extremely heavy or extremely light construction.

The ductility factors for various types of construction are more difficult to characterize. They depend on the use of the building, the hazard involved in its failure, the material used, and the framing or layout of the structure, and above all on the method of construction and the details of fabrication of joints and connections. A discussion of these topics is given in Reference [3] also.

Design Spectrum - Elastic

In either analysis or design for earthquake resistance it is convenient to use the concept of the response spectrum. A response spectrum developed to give design coefficients is called a "Design Spectrum".

Table 1

RECOMMENDED DAMPING VALUES

Stress Level	Type and Condition of Structure	Percentage of Critical Damping
Working stress, no more than about 1/2 yield point	a. Vital piping	0.5 to 1.0
	b. Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2
	c. Reinforced concrete with considerable cracking	3 to 5
	d. Bolted and/or riveted steel, wood structures with nailed or bolted joints	5 to 7
At or just below yield point	a. Vital piping	2
	b. Welded steel, prestressed concrete (without complete loss in prestress)	5
	c. Prestressed concrete with no prestress left	7
	d. Reinforced concrete	7 to 10
	e. Bolted and/or riveted steel, wood structures, with bolted joints	10 to 15
	f. Wood structures with nailed joints	15 to 20

In general, for any given area or site, estimates might be made of the maximum ground acceleration, maximum ground velocity, and maximum ground displacement. The lines representing these values can be drawn on the tripartite logarithmic chart of which Fig. 8 is an example. The lines showing the ground motion maxima in Fig. 8 are drawn for a maximum ground acceleration of 1.0g, velocity of 48 in/sec., and displacement of 36 in. These data represent motions more intense than those generally considered for any postulated design earthquake hazard. They are, however, approximately in correct proportion for a number of areas of the world, where earthquakes occur either on firm ground, soft rock, or competent sediments of various kinds. For relatively soft sediments, the velocities and displacements might require increases above the values corresponding to the given acceleration as scaled from Fig. 8. However, it is not likely that maximum ground velocities in excess of 4 to 5 ft per second are obtainable under any circumstances.

Amplification factors for the various ranges in the response spectrum were considered in References [5] and [6]. The values determined therein for a number of earthquakes, with some smoothing and reduction of peaks to present a reasonably consistent probability of failure (of the order of about 10 percent or less), are given in Table 2. The amplification factors given in that table are used in connection with Fig. 8, as explained below.

For each of the amounts of damping shown in Fig. 8 or tabulated in Table 2, the amplified displacements are shown on the left, the amplified velocities at the top, and the amplified accelerations in that part of the right-hand side of the figure for which the lines are parallel to the maximum ground acceleration line, but lie above it. We shall identify these portions of the line as the amplified displacement region, the amplified velocity region, and the amplified acceleration region, respectively.

At a frequency of about 6 cps, the amplified acceleration region line intersects a line sloping down toward the maximum ground acceleration value, and intersecting that line at various frequencies, depending on the damping. The intersection is at a frequency of about 30 cps for 2% damping, and the other lines are parallel to the line for 2% damping. These lines are designated as the acceleration transition region of the spectra. Finally, beyond the intersection with the maximum ground acceleration line, the response spectrum continues with the maximum ground acceleration value for higher frequencies.

The spectra so determined can be used as design spectra for elastic responses. The spectra are completely described when the maximum ground motion values are given for the three components of ground motion, and the damping is known. When only the maximum ground acceleration is given, the values used for maximum ground velocity and displacement are taken as proportional to those in the figure, or as scaled by the same scale factor relative to the maximum ground acceleration compared with 1 g.

The amplification factors given in Table 2 and shown in Fig. 8 are still under study, but it is not expected that major revisions in them will be required.

Design Spectrum - Inelastic

To use the design spectrum to approximate inelastic behavior, the following suggestions are made. In the amplified displacement region of the spectra, the left-hand side, and in the amplified velocity region, at the top, the spectrum remains unchanged for total displacement, and is divided by the ductility factor to obtain yield displacement or acceleration. The upper right-hand portion sloping down at 45°, or the amplified acceleration region of the spectrum, is relocated for an elasto-plastic resistance curve, or for any other resistance curve for actual structural materials, by choosing

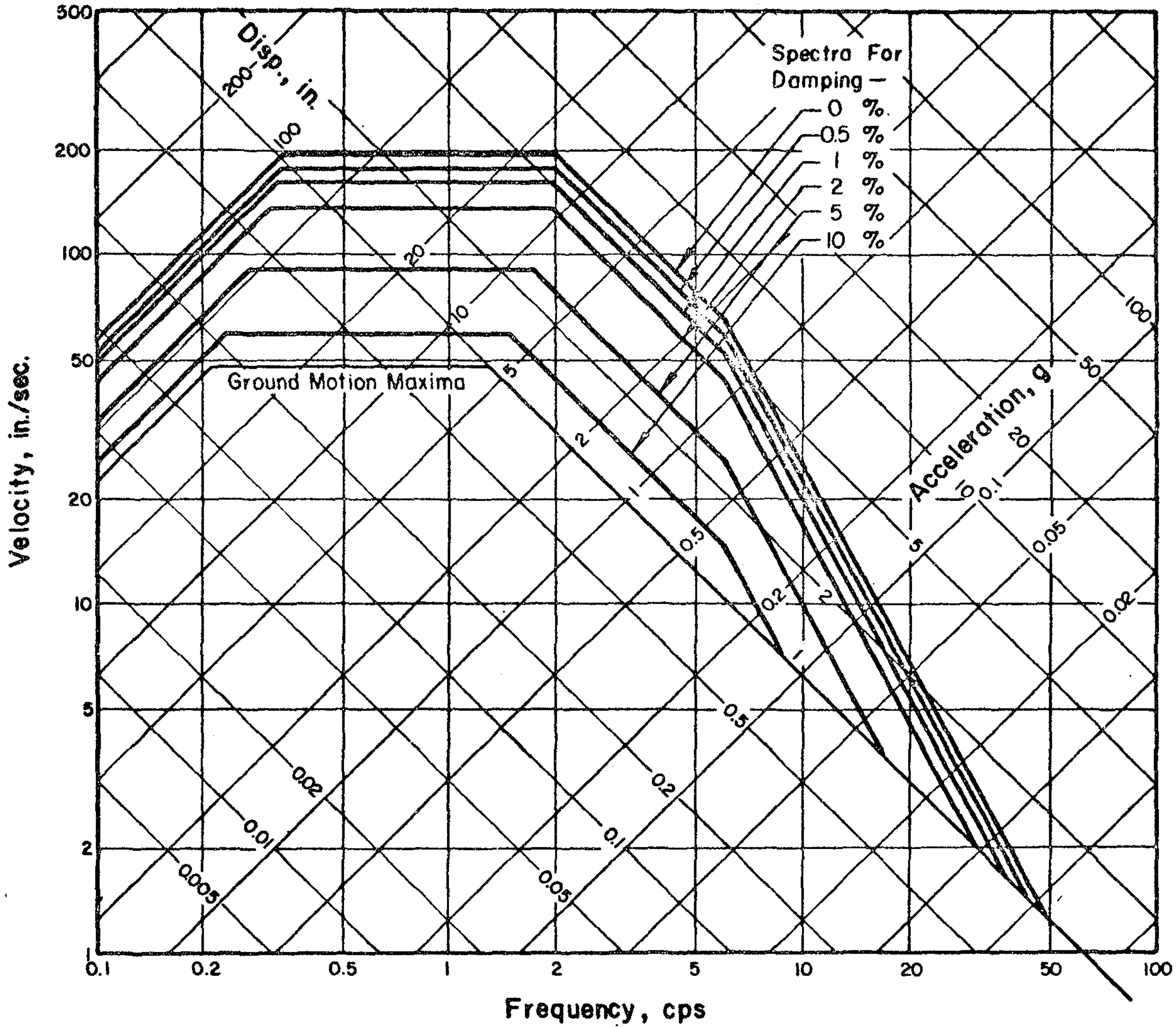

FIG. 8 BASIC DESIGN SPECTRA NORMALIZED TO 1.0 g

Table 2

RELATIVE VALUES OF SPECTRUM AMPLIFICATION FACTORS

Percent of Critical Damping	Amplification Factor For		
	Displacement	Velocity	Acceleration
0	2.5	4.0	6.4
0.5	2.2	3.6	5.8
1	2.0	3.2	5.2
2	1.8	2.8	4.3
5	1.4	1.9	2.6
7	1.2	1.5	1.9
10	1.1	1.3	1.5
20	1.0	1.1	1.2

it at a level which corresponds to the same energy absorption for the elasto-plastic curve as for an elastic curve shown for the same period of vibration. The extreme right-hand portion of the spectrum, where the response is governed by the maximum ground acceleration, remains at the same acceleration level as for the elastic case, and therefore at a corresponding increased total displacement level. The frequencies at the corners are kept at the same values as in the elastic spectrum. The acceleration transition region of the response spectrum is now drawn also as a straight line transition from the newly located amplified acceleration line and the ground acceleration line, using the same frequency points of intersection as in the elastic response spectrum.

In all cases the "inelastic maximum acceleration" spectrum and the "inelastic maximum displacement" spectrum differ by the factor μ at the same frequencies. The design spectrum so obtained is shown in Fig. 9.

An earlier procedure for the definition of inelastic response spectra for design was presented in Reference [2]. In that presentation, the displacement bound, the velocity bound, and the acceleration bound were determined, respectively, by keeping the displacement constant, the energy constant, and the force in the spring constant, and drawing the corresponding maximum response displacement limits.

The revised procedure presented in this report is shown in Fig. 9 for 2% damping, for an elasto-plastic system with a ductility factor of 5. Both the maximum displacement and maximum acceleration bounds are shown, for comparison with the elastic response spectrum.

The solid line $DVAA_o$ shows the elastic response spectrum. The heavy circles at the intersections of the various branches show the frequencies which remain constant in the construction of the inelastic design spectrum.

The dashed line $D'V'A'A_o$ shows the inelastic acceleration, and the lines $DVA''A''_o$ shows the inelastic displacement. These two differ by a constant factor $\mu = 5$ for the construction shown, but A and A' differ by the factor $\sqrt{2\mu - 1} = 3$, since this is the factor that corresponds to constant energy, as indicated in Reference [2].

Of course, the elasto-plastic or other inelastic response spectra can be used only as an approximation for multi-degree-of-freedom systems.

In the development of a design spectrum one may choose to use an "effective" value of maximum ground acceleration rather than an actual value, particularly in cases where the higher spikes of acceleration are associated with very short durations and correspond to velocity changes much smaller than the maximum ground velocity, or where the duration of the earthquake motion is extremely short and the influence on failure or inelastic behavior is thereby lessened.

Vertical and Horizontal Excitation

Since the ground moves in all three directions in an earthquake, and even tilts and rotates, consideration of the combined effects of all these motions must be included in the design of important structures. When the responses in the various directions may be considered to be uncoupled, then consideration can be given separately to the various components of base motion, and individual response spectra can be determined for each component or direction of transient base displacement. Calculations have been made for the elastic response spectra in all directions for a number of earthquakes. The complete results for the three components of motion for these are not yet available, but the trends are summarized below.

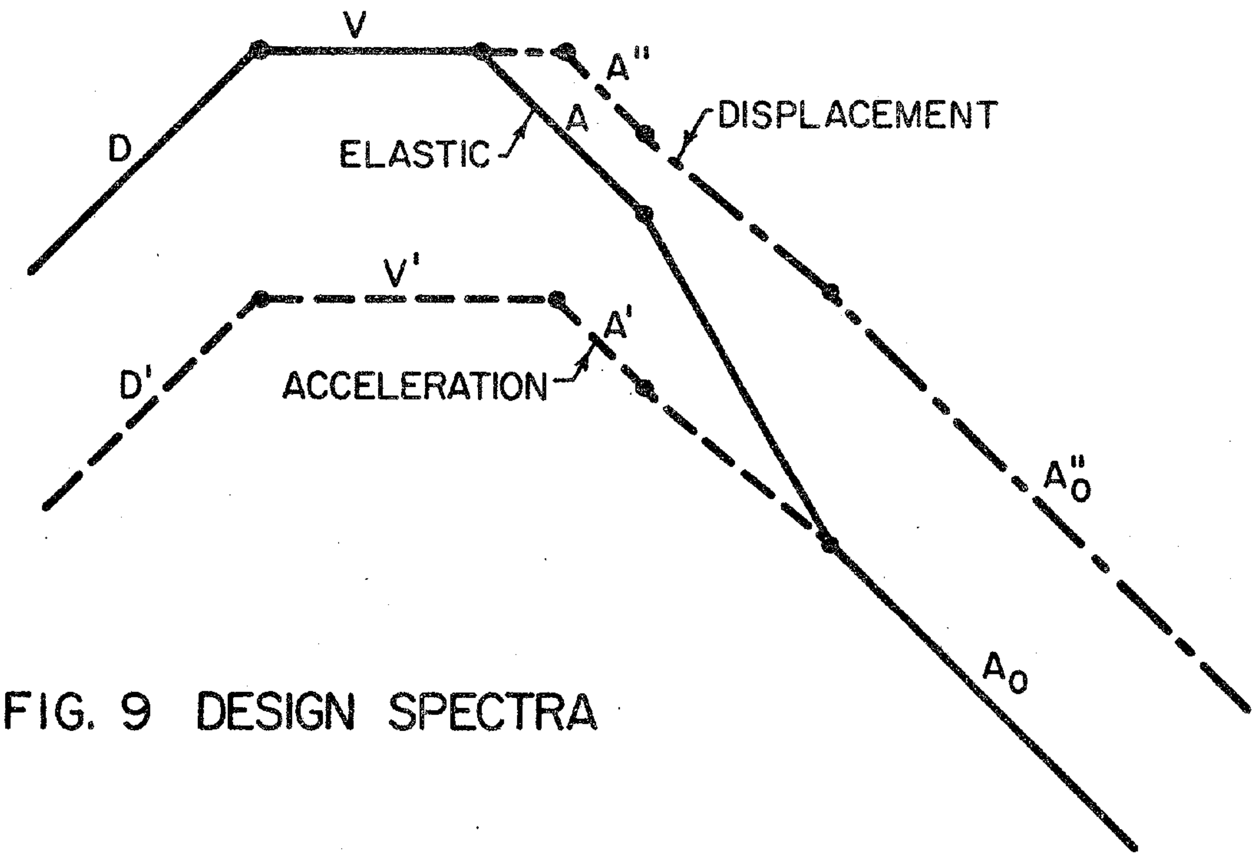

FIG. 9 DESIGN SPECTRA

There are several interesting features of the response spectra. For example, the frequencies at which spikes and valleys occur are generally not the same for the different directions of any earthquake nor for the same directions at the same site for different earthquakes. The responses for the two horizontal directions show cross-overs and significant differences in some ranges of frequency. The vertical response is often equal to or greater than the maximum horizontal response in the high frequency region, but is somewhat to a great deal less in the intermediate and low frequency regions.

It is suggested that until further information becomes available the following design criteria be used:

(a) The design spectrum for vertical response be considered equal to that for horizontal response for frequencies in the amplified acceleration range or higher frequencies. In other words, the acceleration bounds are the same for both horizontal and vertical response.

(b) The design spectrum for vertical response be considered equal to two-thirds that for horizontal response for frequencies in the amplified velocity or displacement ranges.

Combined Effects of Earthquake Motions

Since the responses for motions in the various directions (horizontal and vertical) may not occur at the same time, it is considered reasonable to combine the effects of the several components of motion in a probabilistic manner, by taking the maximum stress, deflection, or other specific response as the square root of the sums of the squares of the corresponding responses to the individual components of motion.

The effects of transient tipping, tilting, and rotation of the ground during an earthquake have not been studied extensively. An elementary treatment of some aspects of these movements has been given in Section 7.7 of Reference [3], and the effects of rotation of the ground about a vertical axis on the accidental torsion in symmetrical buildings, for example, is given in Section 15.6 of the same reference.

When the responses of the structure or component are coupled, the analysis becomes much more complex and a three-dimensional (or at least two-dimensional) response analysis must be considered. However, data regarding the simultaneous input motions must be used in such an analysis, and little guidance is available on this topic.

The motions due to an earthquake occur in both horizontal and vertical directions in a complex manner. It is necessary to consider the interactions between the responses in the various directions, and especially important to consider the interaction between the vertical and the maximum horizontal response. Vertical loads, and eccentricities of the vertical loads caused by horizontal displacements, must often be taken into account with especially heavy structures that carry large masses at or near the points which may deflect a great deal. Some of the resisting capacity for horizontal motions may be used up by the secondary effects of the eccentricities of the gravity loads.

Quite often, the vertical motions may produce vertical stresses in the structure or element that exceed by a large amount those stresses due to the inertial forces corresponding to the vertical acceleration multiplied by the mass of the element. This is true when the frequencies of vibration in the vertical direction of the element or component are in the range where major amplification of response can occur.

Special Considerations and Quality Control

A number of points are often overlooked in the design of structures or components to resist dynamic motions. A summary of some of the more important factors, but by no means a complete listing of all of them, is contained herein.

One of the factors that is commonly overlooked is the matter of relative motions between the parts or elements of a system having supports at different points, because the support motions may not occur simultaneously. Hence, there may be transient relative motions which produce strains in the structure, in addition to the strains produced by the dynamic effects of the overall motion. This is especially important in piping, electric wiring, or other elements connecting parts of a facility.

Finally, there are a group of items which do not lend themselves readily to analytical consideration. These concern the details and material properties of the element or component, and the inspection and control of quality in the construction procedure. The details of connections of the structure to its support or foundations, as well as the various elements or items within the structure or component, are of major importance. Failures often occur at the connections and joints because of inadequacy of these to carry the forces to which they are subjected under dynamic conditions. Inadequacy in properties of the materials can often be encountered, leading to brittle fracture where sufficient energy cannot be absorbed even though such energy absorption may have been counted on in the design.

In order to insure that the intent of the designer is achieved, control of construction procedures and appropriate inspection practices are necessary. It is important that the practical aspects of seismic design be emphasized and that both designers and constructors be fully aware of their importance.

V. Desirable Features of Design Codes

Relation Between Analysis and Design

When the configuration of a structure is fixed by architectural or other requirements, the designer has a restricted choice in the development of the strength and ductility required to insure adequate seismic behavior. It is not always possible to say that some design layouts are better than others for dynamic resistance, although it is fairly clear that different choices of framing can lead to vastly different requirements of strength and ductility. For example, a framed structure is generally less stiff and usually lower in frequency than a shear wall structure with nearly solid walls providing lateral resistance. Hence the design forces may be smaller for the framed structure than for the shear wall structure, although the required ductility may be larger. Methods of design for the dynamic loadings arising from earthquakes are in general simpler and better understood for structures for which there is a great deal of experience. However, unless methods are developed and specifications are devised to take account of new structural types or of new imaginative architectural designs, such designs will be placed at a disadvantage relative to more standard designs because of the necessity for providing greater margins of safety for those designs for which experience is unavailable.

The methods of analysis, and also the details of the design specifications, have implications on the cost and the performance capability of the design. If the specifications are unduly conservative the design will not only be unduly conservative, but may also be forced into a type that is stronger and less ductile than is desirable. It is difficult to avoid differences in the degree of conservatism among different types of structures, and in some cases it is undesirable to do so. Some materials by their nature, including their variability or lack of adequate control of properties, may require a greater factor of safety than other materials the properties of which are more accurately determinable and controllable. The margin between incipient failure and complete collapse may differ for different materials and may therefore involve a difference in the factor of safety required in the design.

Basic Function of Design Codes

The designer, as well as anyone else who has a responsibility for the final structure, has to have some general method of knowing that gross errors have been avoided, and must have some basis of comparison to insure that the design is adequate in an overall sense. It is the purpose of building codes and specifications to perform these functions. However, it is not yet established that building codes can do this kind of job without introducing constraints and controls that may be a severe handicap on the development of new design concepts and procedures. Where building codes are used to insure, by rule-of-thumb methods, that a design is adequate, they embody the result of experience and judgment and must therefore deal implicitly, if not explicitly, with particular structural types and configurations.

The most desirable type of design code or specification is one which puts the least restrictions on the initiative, imagination and innovation of the designer. Such a code might involve only criteria for: (1) the loading or environment; and (2) the level of response, the stresses and deformation, or the performance of the structure under the specified loading or environmental conditions. Such an approach need not, and preferably should not, indicate how the designer is to reach his objective, provided he can show that he has achieved a structural capability to resist the specified environmental conditions. This approach is generally the one that is now used for the design of nuclear reactor power stations. Experience over the past several years in approaching seismic design criteria in this way has indicated a number of problems, but has also been reasonably successful in avoiding constraints due solely to the specifications themselves, although there have been constraints based on the environmental conditions and the stress and deformation levels allowed.

Seismic Response Criteria

The permissible level of response of a structure must, of course, be associated with the loading criteria. One cannot be specified independently of the other. This implies, for example, that different response criteria are to be associated with the probable earthquake or the historical earthquake from those used for the maximum credible or extreme earthquake. Moreover, for either of these, the response criteria should properly be a function of the type of structure, the relative cost of repairs for minor damage, and the hazard in terms of possible loss of life should the structure or any of its elements fail. Hence, the response criteria could be greatly different for individual homes than for multistory buildings housing hundreds or even thousands of people, and certainly different even from these for high dams above large centers of population or for nuclear reactor power stations.

It appears reasonable to establish such criteria in terms of the consequences of failure, and in relative terms associated with yield points or buckling loads of similar dynamic limit loads for the particular material or structural elements used. The aim of the criteria should be consistent with the basic seismic design philosophy stated earlier. Appropriate performance criteria may well be stated most rationally in terms of probabilities of failure or collapse associated with various levels of the probability of the hazard considered.

Performance Criteria

It is essential that the response levels or maximum stresses and deformations be limited, for structures, components, and details such as joints and connections, in order to insure adequate strength and ductility of a structure as well as of its various component parts. However, it is desirable, in the development of the basis for a performance criterion, that the designer's approach not be too greatly constrained. For example, it may be unwise to prescribe limits for both strength and ductility in such a way that the balance between the two cannot be adjusted to take account of new material properties or new structural types as they are developed. A trade-off between ductility and strength should be available in the methods that are permitted, so as to achieve economy without the sacrifice of safety. But whether one is interested in achieving strength or ductility, or both, the materials have to be used in an appropriate fashion, and adequate methods of inspection and control of construction are needed to insure that their use is proper.

Methods of Analysis

A variety of methods of analysis are now available, ranging from simple upper-bound static coefficient methods to modal response combinations, including time history analyses either in the elastic or inelastic range, and extending to probabilistic methods or methods involving consideration of random vibrations. It should be possible to use any of these that are sufficiently justified by either general acceptance or by demonstrated mathematical and physical validity, with requiring that any one particular method be used. Of course, it is desirable always to allow the use of simple methods that are adequately conservative. Properly stated criteria, used with appropriate methods of approximate analysis and design, may make it unnecessary in many cases to perform detailed and costly analyses, particularly in those instances where the simpler approximate methods insure adequate margins of safety.

To push these concepts to the limit involves generally the idea of approximate methods that give reasonably accurate results or, at least, results that are consistent with those obtained from more precise analyses. Depending on the degree and extent of the approximation, it may be necessary, from the point of view of achieving a reasonable degree of precision, to have special modifications of a general procedure for various unusual structural classifications. Methods may be used that do not require knowledge of the period of vibration of the structure, for example, or the methods may involve an approximate determination of the natural period. Approximate damping factor determinations may be involved as well.

For the complete development of this type of approach it may be desirable to explore the possibility of a hierarchy of methods ranging from the crudest approximation, for very simple structures and structural elements, to more accurate approximate methods, for structures of intermediate complexity, and to relatively precise and accurate calculations, for extremely complex structures.

Special Structures

Although many of the problems associated with the design of special structures such as nuclear reactors, high dams, schools and hospitals, are similar to those involved in other more ordinary types of structures, there are some implications of failure of these special structures that require special consideration in selecting margins of safety and the development of design procedures and criteria governing them.

Many structures or parts of structures and many items of equipment can be severely damaged without any implication of loss of life or even of major property damage in an earthquake. For such items and structures it is unnecessary and certainly uneconomical to provide great margins of safety for unlikely earthquakes. The margin of safety of the provision made for an earthquake of a reasonable degree of probability of occurrence within the lifetime of a structure need only be great enough to offset the cost of repairs or reconstruction.

However, structures whose loss of function might cause hazard to life, or structures which are important to prevent damage to major services, have to be designed on an entirely different basis. For such structures, an earthquake even of relatively low probability of occurrence, but one that possibly could occur, should not cause collapse or damage of such a nature that endangers the health or life of large numbers of the population.

Hence, for such structures, much greater accuracy in procedures and assumptions is required, and the type of design specification must be more carefully framed and more clearly stated to give an assuredly adequate margin of safety against failure even for unusual types of structure and framing.

The type of design specification used for major nuclear reactor power plants has emphasized loading or environmental criteria, and performance criteria in terms of stress and deformation levels of response. The experience that has been gained with criteria of these types indicates that benefits are possible for other types of special structures with similar kinds of design specifications. The advantages are in the encouragement of the designer to explore various types of structures, to consider the use of a variety of materials, and to look for economical ways of achieving the desired levels of safety and performance.

Detailed prescriptions of methods and procedures were necessary when the majority of practicing engineers had neither the sophistication nor the computational aids to take account of the more accurate methods of analysis and design that are now available. However, this situation has changed and is continuing to change at a rapid rate. With the increase in numbers of more highly trained engineers, and with a greater store of knowledge available, together with more efficient ways of using that knowledge which have become possible with the general availability of and accessibility to high speed digital computers, it appears that it is now possible to make a major change in our methods of specifying or codifying seismic design.

General Comments and Conclusions

The field of earthquake engineering is relatively new, not much more than three decades old, and advances in knowledge are progressing at a rapid rate, not only because of a greater emphasis on analytical and experimental work in the laboratory, but also because of the availability of more definitive information on earthquake motions, the accumulation of strong-motion records, and recent accelerated expenditures on research. It is important that the discoveries from observations, and studies from the research laboratory and the analyst, find their way into practice as soon as possible. But this is difficult because engineers are traditionally unwilling to take chances on things that they have not proved out. Too little attention has been given to methods of demonstrating adequate performance capability for major structures and structural elements. Although this is not necessarily a part of our consideration in design specifications, it might well be desirable to look for ways of determining the capability of completed structures or for ways of proving the performance capability in the design stage by appropriate methods, so as to encourage the development of more economical designs and methods.

In order to insure that the intent of the designer is achieved, control of construction procedures and appropriate inspection practices are necessary. This point cannot be overemphasized. It is difficult to synthesize and distill the collective experience and judgment of the engineering profession into a set of rules, especially when they cannot be put into a mathematical formulation. This is a difficulty, however, not only with performance and environmental criteria but also with more standard types of design specifications of the current and past eras. Nevertheless, it is important that the practical aspects of seismic design be emphasized and that engineers and constructors be fully aware of their importance.

It is the intent of this discussion to focus attention on the aims and objectives of seismic design in such a way as to encourage the development of methods and practices suitable for structural design of the future; methods that will permit more freedom and latitude to the architectural and engineering innovator.

VI. References

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APPENDIX 6

VERTICAL GROUND MOTION IN EARTHQUAKES--
ITS NATURE AND ITS EFFECTS ON THE
RESPONSE OF BUILDINGS

APPENDIX 6

VERTICAL GROUND MOTION IN EARTHQUAKES--ITS NATURE AND ITS EFFECTS ON THE RESPONSE OF BUILDINGS

by

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Introduction

Vertical ground motions currently are given surprisingly little consideration in the earthquake resistant design of structures. This disregard is mainly based on the assumption that structures are almost "rigid" in the vertical direction, and hence the effects of vertical ground motions can be neglected. However, the few investigations carried out on this subject show that the vertical periods of many standard structures and important structural components are in the range of amplifications of response spectra developed from vertical acceleration components. The amplification of the vertical acceleration at the top of multistory steel structures may be in the order of four times the ground acceleration (references 45, 47). Such accelerations could contribute to the compression failure of columns, increase the ductility requirements in beams, reduce the factor of safety against overturning, or cause a critical condition in a prestressed beam by reducing the effective dead load on the beam (references 42, 46). More research on this subject is therefore urgently needed.

The following summary of information available on vertical ground motions and its effects on the response of structures and structural components is based on the investigations reported in References 42 to 51. The summary is by no means complete, since it only includes work carried out in the United States and disregards research done in Japan and other countries.

Nature of Vertical Ground Motion

A large number of records are available on vertical accelerations in past earthquakes from accelerographs recording horizontal and vertical components. All accelerogram records were digitized by and are available through the Earthquake Engineering Research Laboratory of the California Institute of Technology. Response spectra for single degree of freedom damped oscillators derived from these records are presented in references 46 and 48, and are also available through U.S. Coast and Geodetic Survey. A typical accelerogram for vertical accelerations is shown in Figure A6-1, and a comparison of horizontal vs. vertical spectral responses of SDF oscillators (reference 48) is presented in Figures A6-2 and A6-3.

The following general observations can be made from the data presented in the available references:

1. The peak vertical accelerations may not occur at exactly the same time as the peak horizontal accelerations; however, major vertical accelerations do occur within the same general time as the major horizontal ones. Thus the vertical and horizontal accelerations could be conservatively assumed to act simultaneously. (See Reference 46). Iyengar

and Shinozuka (reference 44) point out that there is no reason to believe that the horizontal and vertical components are uncorrelated, and they propose to represent the two components by a bivariate stationary Gaussian process with a cross spectrum between horizontal and vertical acceleration.

2. An examination of the available response spectra indicates that for most cases the response spectra for vertical ground motion are flatter (broader) than those for horizontal motions (reference 48). The spectra for vertical motion show significant amplification in the high frequency range (4 to 20 cps), and most of them also indicate an attenuation of the low period components (reference 46). It can also be seen that the response spectra for vertical motion approach the ground acceleration at higher frequencies than is the case for horizontal motion (reference 48). This phenomenon is due mainly to the presence of higher frequencies in vertical components of ground motion (reference 47). This will be reflected in the construction of design spectra, where the vertical ground acceleration is amplified over a larger frequency domain than the horizontal acceleration.

3. Vertical accelerations increase rapidly as the distance from the fault decreases, more so than horizontal accelerations. This is confirmed with experience from underground nuclear explosions, where vertical accelerations

close to the point of detonation in some cases exceed the horizontal accelerations (see reference 46).

ad/v^2 Ratio

The nondimensional quantity ad/v^2 can be taken as a basic index for monitoring the shape, and especially the breadth of the response spectra. The discussion of this parameter and others necessary to characterize the vertical ground motion is extracted from reference 48. A summary of the maximum values of the ground motion data studied in reference 48 is presented in Table A6-1.

Recognizing that the maximum ground displacements have to be obtained from the recorded accelerations through double integration and that these values are very sensitive to the base line adjustment procedure, it can be stated that the accelerations, the velocities, and the ad/v^2 ratio are the most important parameters characterizing a ground motion. It also is established that the ad/v^2 ratio is a function of the focal distance of the earthquake and the attenuation of motion in the ground. This ratio is therefore of foremost importance for the construction of design spectra and needs to be carefully studied.

The values of ad/v^2 range from 1.84 to 30.58, but disregarding the El Centro earthquake of 5-18-40, most values fall in the range from about 5 to 15. In general it will be noted that the ad/v^2 values are higher for vertical motion, as would be expected in part because the high frequency components in the vertical direction are more pronounced than in the horizontal direction.

The average values of ad/v^2 for different site conditions are summarized in Table A6-2. The average value for horizontal motions was about 5.6, and differs little from rock sites to alluvial sites. For vertical motions, the average value for alluvial sites was equal to 10.0 and that for rock sites was 13.0. Since only three sets of data were available for rock sites, more confidence can be placed in the values obtained for alluvial sites. Based on the available data, it was concluded that ad/v^2 ratios of 6 and 10 may be representative for horizontal and vertical ground motions, respectively.

Peak Horizontal vs. Vertical Accelerations

Newmark and Hall state in Reference 49 that the vertical components of motion may be somewhat less, or in some circumstances equal to or slightly greater than the horizontal motions, depending upon whether the associated fault motions in the earthquake are primarily horizontal (strike-slip motions) or primarily vertical (thrust fault or dip-slip motions). This general statement was confirmed by the study of particular earthquakes that show a range of ratios of vertical to horizontal peak acceleration from 0.27 for the Ferndale earthquake (12-21-54) to 1.29 for the Holiday Inn record of the San Fernando earthquake. It is clear that this ratio is a function of the fault motion, the distance from the fault, the ground motion intensity and the geological site conditions. Average values of these ratios for different site characteristics are presented in Table A6-3. The values range from 0.40 to 0.72, depending on the acceleration level included and the geological conditions. On the basis of these data, and realizing that only a few samples existed for the rock site, it was decided in reference 48 to take the ground acceleration for a given site for the vertical spectrum to be equal to two-thirds of that for the horizontal motion.

Response Amplifications

Studies of the response amplification in various ranges of frequencies were carried out in reference 7. The input ground motion was normalized to the following values:

Maximum ground displacement	1.0 inch
Maximum ground velocity	10.0 in/sec
Maximum ground acceleration	1.0 g

The results for mean response amplification for 5 percent critical damping is shown in Figure A6-4 for the horizontal and the vertical ground motion.

The following observations were made regarding the vertical response amplification:

1. The amplification factors for accelerations are virtually constant for the range of frequencies from about 3 to 10 hertz, and then decrease fairly uniformly to intersect the ground motion accelerations at frequencies of about 50 hertz for all values of damping.
2. For the intermediate range of frequencies (0.3 to 3.0 hertz), a slight drop in the velocity amplification factor is evident as the frequency increases. However, the drop is only of the order of about 10 percent over the frequency range for which velocity amplification is valid, and it therefore appears reasonable to use a constant

amplification for the velocity range as well as for the acceleration and displacement range.

3. In the low frequency range of the spectrum the displacements are amplified by a constant factor for all values above the 0.05 hertz level.

The statistical values of response amplifications for horizontal and vertical motion obtained from a normal distribution curve for 0.5, 2, 5, and 10 percent of critical damping are presented in Table A6-4. It is recommended that these amplification factors for vertical ground motion be applied to the following frequency ranges:

Displacement amplification	0.1 to 0.3 hertz
Velocity amplification	0.3 to 3.0 hertz
Acceleration amplification	3.0 to 10.0 hertz

Design Spectra for Vertical Motion

It has to be emphasized that it is difficult to generalize as to the shape of the vertical spectra versus the shape of the horizontal spectra. This shape will be a function of the distance to the fault, the site characteristics, the ground motion intensity, etc. In part such a generalization has been attempted in reference 48, where some of the parameters required in constructing the vertical design spectra are expressed as fractions of the parameters used for horizontal design spectra. The procedure utilized in reference 48 for the construction of design spectra for vertical ground motion can be briefly summarized as follows:

1. The peak vertical ground acceleration is taken to be two-thirds of the peak horizontal acceleration, regardless

of site.

2. The ratio of v/a for the vertical motion to that of the horizontal motion is 0.90, regardless of site.

3. The ad/v^2 for vertical motion is taken to be 10.0, regardless of site.

4. These three parameters define the ground motion bounds. The design spectra are obtained by multiplying these ground motion values by the amplification factors given in Table A6-4. The frequency ranges applicable for the amplification of acceleration, velocity, and displacement are listed previously. The faring frequency in the high frequency region was taken to be 50 hertz for all values of damping.

It should be noted that the above parameters were computed from a statistical study including only strong motion data, i.e., disregarding all traces that had accelerations less than 0.1g in the case of horizontal motion and 0.05g in the case of vertical motion.

The values of ground motion bounds of acceleration, velocity and displacement for horizontal and vertical ground motions are presented in Table A6-5. The values are normalized with respect to a horizontal ground acceleration of 1.0g. The displacement values were calculated on the basis of the a, v, and ad/v^2 values, and were then rounded off to a representative number. The actual response spectra bounds, obtained through multiplication with the appropriate amplification factors, are tabulated in Table A6-6 for horizontal and vertical motion. A typical vertical response spectrum is shown in Figure A6-5.

Effects of Vertical Ground Motion on the Response of Buildings

The types of structures and structural components that may experience significant effects from vertical ground motions are high rise buildings, cantilevered structures such as grandstand roofs, aircraft hangars and marquees, cable suspended structures, slabs, and long span elements, particularly of prestressed concrete (see references 46, 47).

Various opinions are expressed in the literature regarding the importance of the effects of vertical accelerations. Larson (reference 47) states that studies of the behavior of structures in the Anchorage, Caracas, and Santa Rose earthquakes disclosed many effects which could be ascribed to vertical components of seismic excitation. Jennings (reference 45), to the contrary, concluded from a damage study of the San Fernando earthquake: "There were no instances where it appeared to the writers that vertical accelerations had been major contributors to the damage sustained by building structures, and it is not recommended at this time that vertical accelerations be included in normal seismic design procedures for typical buildings. There are special structures, however, which could be more sensitive to vertical accelerations and vertical motions should be considered in such cases." Rosenblueth (reference 50) reports the appearance of vertical tensions during nearby earthquakes. These, as well as large compressions, also caused by vertical accelerations, can combine with the effects of overturning moments. He also states that we may have to expect appreciably permanent deformations in floor systems after several nearby strong earthquakes.

Regardless of the wide range of opinions expressed in the literature, it is well established that vertical accelerations are greatly amplified from the ground floor to the top of multistory buildings. This can be seen from the records of the San Fernando earthquake presented in Table A6-7 (see reference 45), which show an amplification of vertical acceleration ranging from 1.4 to 4.5. This is also evident from analytical investigations that show that the vertical period of many typical structures lies in the range of acceleration amplification.

Dynamic Properties of Structures in the Vertical Direction

Building structures can be modeled through a lumped mass-spring system, with each point mass representing the total weight at the floor level, and each spring representing the axial deformation of the columns between floors (see reference 46). The effects of interconnecting girders on the columns could be included, but is not considered to be significant. The vertical period of buildings depends on the height of the structure and on the level of stresses in the column. For multistory steel structures the vertical period can be estimated from Figure A6-6, taken from reference 46.

Results of Analytical Investigations

A series of steel structures, ranging from 5 to 51 stories, was studied in reference 46. The response spectra for the Taft and Golden Gate Park earthquakes were used as input in the analysis.

Elastic behavior was assumed in the analysis. This is a reasonable approximation for studies of structures in the vertical direction because of the current design philosophy to avoid plastic hinges in the columns. The findings of this study are briefly summarized below.

1. The vertical periods of steel structures lie in the range of acceleration amplification of the spectra developed for vertical motion.
2. The amplification of the vertical acceleration at the top of steel structures may be in the order of four times the ground acceleration. Similar results should be expected for buildings of other materials. This amplification could increase or decrease column dead loads by 20 percent or more.
3. Vertical accelerations may contribute to the overstress or compression failure of columns and may reduce the factor of safety against overturning.

A study of a ten story unbraced steel frame subjected to gravity loads and a combination of horizontal and vertical ground motion components is reported in reference 42. An inelastic dynamic analysis of this structure was carried out, using the Pacoima Dam and the Taft earthquake records as an input. Both records were normalized to a maximum vertical ground acceleration of 0.31g. For comparison, a second analysis was carried out with the structure subjected only to gravity loads and horizontal accelerations. The maximum vertical accelerations of the selected nodal points in the structure are shown in Figure A6-7, for the Pacoima Dam record. The following conclusions were drawn from a study of the response of the structure that was designed for seismic loads specified in the Uniform Building Code and following the allowable stress design procedure.

1. The inclusion of vertical components of ground motion increases the ductility requirements in both the columns and girders of the upper stories. Vertical motion is particularly significant in causing inelastic action at the mid-span of the girders. Inelastic action in the columns is increased by the amplification of the vertical motion in these members and by the amplified vertical response of the girders.
2. The relative importance of the vertical motions is dependent on the main characteristics of the complete time histories of both the horizontal and vertical components of the earthquake. The duration and frequency content as well as the maximum acceleration of the ground motion need to be considered in determining the effects of earthquakes on structures.
3. Ductility requirements at the critical regions of individual members cannot be evaluated with accuracy from just analyzing maximum lateral displacements and maximum relative story drifts.

Iyengar and Shinozuka (reference 44) used statistical models for the horizontal and vertical components of ground motion to study the effect of self-weight and vertical acceleration on the tip deflection, base bending moment, and base shear force of cantilever type structures. As expected, the effects were of more significance in taller structures than in shorter ones, but in either case the differences caused by including self-weight and vertical acceleration seem to be considerable.

In conclusion, it has to be stated that no evaluation of the effects of vertical ground motion for structures older than steel structures and for important structural components could be found in the literature. Further research in this area seems to be urgently needed.

Record description	Maximum ground acc. a, g	Maximum ground vel. v, in/sec	Maximum ground displ. d, in	$\frac{ad}{v^2}$	Site description	Remarks
Ferndale, 10-7-51, 2011 PST (Record IA 2)					40-80 ft of alluvium over 100 ft of sandstone over siltstone	Instrument on the ground floor of a 2 story frame structure Ref. (8)
N 46 W	0.120	2.86	0.95	5.39		
S 44 W	0.123	1.73	1.07	16.99		
Vertical	0.032	1.02	1.24	14.74		
Ferndale, 12-21-54, 1156 PST (Record IA 9)					40-80 ft of alluvium over 100 ft of sandstone over siltstone	Instrument on the ground floor of a 2 story frame structure Ref. (8)
N 46 W	0.209	9.79	4.92	4.15		
N 44 E	0.166	14.10	8.09	2.61		
Vertical	0.045	3.13	2.49	4.42		
Eureka, 12-21-54, 1156 PST (Record IA 8)					100' sandstone (poorly consolidated) over 360 ft of siltstone over sandstone	Instrument in the basement of a brick and stone building Ref. (8)
N 11 W	0.189	5.92	8.45	17.61		
N 79 E	0.271	9.23	3.14	3.86		
Vertical	0.110	2.64	2.22	13.54		
Hollister, 4-8-61, 2323 PST (Record IA 18)					500 ft of alluvium over cenozoic rock water table at 50 ft	Instrument on the first floor of the public library, a 2 story structure Ref. (8)
S 01 W	0.076	3.10	3.03	9.26		
N 89 W	0.189	6.45	1.97	3.46		
Vertical	0.056	1.73	1.03	7.45		

Maximum Ground Motions and Site Description

Table A6-1. (reference 48)

Record description	Maximum ground acc. a, g	Maximum ground vel. v, in/sec	Maximum ground displ. d, in	$\frac{ad}{v^2}$	Site description	Remarks
El Centro, 2-9-56, 0633 PST (Record IA 11)					Alluvium to about 5000 ft	Instrument on the first floor of a 2 story massive concrete, heavily reinforced structure
NS	0.036	1.52	1.27	7.65		
EW	0.055	3.11	2.48	5.45		
Vertical	0.016	0.75	0.55	6.05		
El Centro, 4-8-68, 1830 PST (Record IA 19)					Alluvium to about 5000 ft	Instrument on the first floor of a 2 story massive concrete, heavily reinforced structure
NS	0.142	10.49	3.68	1.84		
EW	0.058	4.72	4.68	4.71		
Vertical	0.036	1.16	1.36	14.06		
Hollywood Storage Basement, 7-21-52, 0453 PDT (Record IA 6)					700' ± of alluvium	Instrument in the basement of a 14 story RC building
NS	0.059	2.58	1.41	4.83		
EW	0.046	3.74	2.73	3.47		
Vertical	0.023	1.12	0.85	6.02		
Hollywood Storage PE Lot, 7-21-52, 0453 PDT (Record IA 7)					700' ± of alluvium	
NS	0.063	2.60	1.26	4.54		
EW	0.043	4.11	2.89	2.84		
Vertical	0.023	1.22	0.81	4.84		
San Francisco Golden Gate Park, 3-22-57, 1144 PST (Record IA 15)					Siliceous sandstone 1-2 miles from San Andreas Fault	Instrument in a small shack used to house electrical equipment
N 10 E	0.106	1.09	0.20	6.90		
S 80 E	0.127	1.26	0.18	5.56		
Vertical	0.051	0.41	0.16	18.76		

Table A6-1. (continued)

Record description	Maximum ground acc. a, g	Maximum ground vel. v, in/sec	Maximum ground displ. d, in	$\frac{ad}{v^2}$	Site description	Remarks
Pacoima Dam, 2-9-71, 0600 PST (Record IC 041)					Highly jointed diorite gneiss 4 km from surface faulting	Small building houses the instrument
S 74 W	1.250	22.49	5.11	4.88		
S 16 E	1.241	43.70	23.18	5.82		
Vertical	0.718	23.06	13.75	7.17		
Castaic, 2-9-71, 0600 PST (Record ID 056)					Sandstone	Small building houses the instrument
N 21 E	0.333	6.73	2.05	5.82		
N 69 W	0.281	10.55	3.22	3.14		
Vertical	0.181	2.75	1.42	13.13		
Holiday Inn (First Floor), 2-9-71, 0600 PST (Record IC 048)					Alluvium 8 km from surface faulting	Instrument on the first floor of a 7 story RC building structure
NS	0.258	12.13	8.70	5.90		
EW	0.137	9.68	6.37	3.60		
Vertical	0.177	12.81	6.37	2.66		
15250 Ventura Boulevard (Basement), 2-9-71, 0600 PST (Record IH 115)					Alluvium water table at 55'	Instrument in the basement of a 12 story RC building structure
N 11 E	0.234	10.96	7.07	5.32		
N 79 W	0.154	7.88	4.48	4.29		
Vertical	0.108	4.77	3.09	5.67		
El Centro, 5-18-40, 2037 PST (Record IA 1)					Alluvium to about 5000 ft	Instrument on the first floor of a 2 story massive concrete, heavily reinforced structure
NS	0.352	13.88	4.74	3.35		
EW	0.223	11.72	6.58	4.13		
Vertical	0.280	3.95	4.41	30.58		

Table A6-1. (continued)

Site	Direction	No. of Records	ad/v^2
alluvium & rock	horizontal	28	5.6
alluvium	horizontal	22	5.7
rock	horizontal	6	5.4
alluvium & rock, a>0.1g	horizontal	20	5.7
alluvium, a>0.1g	horizontal	14	5.9
rock, a>0.1g (same as above)	horizontal	6	5.4
alluvium, a<0.1g	horizontal	8	5.3
alluvium & rock	vertical	14	10.7
alluvium & rock*	vertical	13	9.1
alluvium	vertical	11	10.0
alluvium*	vertical	10	7.9
rock	vertical	3	13.0
alluvium & rock, a>0.05g	vertical	8	12.4
alluvium, a>0.05g	vertical	5	12.0
alluvium, a>0.05g*	vertical	4	7.3
rock, a>0.05g (same as above)	vertical	3	13.0
alluvium, a<0.05g	vertical	6	8.4

* Discarding the extreme value, El Centro, 5-18-40, 2037 PST, $ad/v^2 = 30.58$

Summary of Average ad/v^2 Values

Table A6-2. (reference 48)

Site	No. of records	Average $\frac{\text{a - vertical}}{\text{a - horizontal}}$
alluvium & rock	28	0.53
alluvium	22	0.53
rock	6	0.54
alluvium & rock, $a_h>0.1g$, $a_v>0.05g$	15	0.65
alluvium, $a_h>0.1g$, $a_v>0.05g$	9	0.72
alluvium & rock, $a_h<0.1g$, $a_v<0.05g$	13	0.40
alluvium, $a_h<0.1g$, $a_v<0.05g$ (same as above)	13	0.40

Summary of Vertical to Horizontal Acceleration Ratios

Table A6-3. (reference 48)

Percentile	Damping	All Records (28)			Records with a > 0.1g (20)		
		D	V	A	D	V	A
50	0.5	1.98	2.86	4.00	1.97	2.58	3.67
	2.0	1.69	2.23	2.91	1.68	2.06	2.76
	5.0	1.39	1.74	2.20	1.40	1.66	2.11
	10.0	1.13	1.38	1.72	1.15	1.34	1.65
75	0.5	2.66	3.81	5.02	2.66	3.41	4.65
	2.0	2.23	2.89	3.52	2.24	2.68	3.36
	5.0	1.80	2.19	2.59	1.83	2.10	2.48
	10.0	1.43	1.69	1.97	1.47	1.66	1.89
90	0.5	3.27	4.67	5.95	3.28	4.16	5.53
	2.0	2.72	3.48	4.06	2.74	3.23	3.90
	5.0	2.17	2.60	2.93	2.21	2.51	2.82
	10.0	1.71	1.98	2.20	1.75	1.94	2.11
95	0.5	3.64	5.19	6.50	3.65	4.60	6.05
	2.0	3.02	3.84	4.39	3.04	3.57	4.22
	5.0	2.39	2.84	3.14	2.44	2.75	3.03
	10.0	1.87	2.15	2.33	1.91	2.11	2.24

Statistical Value of Response Amplifications
(Normal Distribution - Horizontal Components)

Table A6-4. (reference 48)

Percentile	Damping	All Records (14)			Records with a > 0.05g (8)		
		D	V	A	D	V	A
50	0.5	1.77	2.74	4.22	1.86	2.52	4.02
	2.0	1.57	2.10	2.86	1.65	1.97	2.80
	5.0	1.33	1.56	2.08	1.40	1.51	2.05
	10.0	1.09	1.22	1.62	1.16	1.17	1.59
75	0.5	2.33	3.67	5.47	2.48	3.39	5.46
	2.0	2.04	2.77	3.60	2.17	2.61	3.70
	5.0	1.70	2.06	2.52	1.81	1.97	2.57
	10.0	1.38	1.55	1.91	1.47	1.49	1.92
90	0.5	2.83	4.51	6.59	3.04	4.17	6.76
	2.0	2.46	3.37	4.27	2.63	3.18	4.51
	5.0	2.04	2.47	2.92	2.18	2.37	3.04
	10.0	1.63	1.84	2.17	1.75	1.78	2.22
95	0.5	3.13	5.02	7.26	3.37	4.64	7.53
	2.0	2.71	3.73	4.67	2.91	3.52	4.99
	5.0	2.24	2.73	3.16	2.40	2.62	3.32
	10.0	1.79	2.01	2.32	1.92	1.95	2.40

Statistical Values of Response Amplifications
(Normal Distribution - Vertical Components)

Table A6-4. (continued) (Ref. 48)

Site	Direction	Acceleration a, g	Velocity v, in/sec	Displacement d, in	ad/v^2
alluvium	horizontal	1.0	48	36	6.04
rock	horizontal	1.0	28	12	5.92
alluvium	vertical	2/3	29	33	10.10
rock	vertical	2/3	17	11	9.80

Horizontal and Vertical Design Ground Motions
Table A6-5. (reference 48)

Percentile	Damping percent	Amplification D	Amplification V	Amplification A	Faring frequency hertz	Spectrum bounds (alluvium) D in	Spectrum bounds (alluvium) V in/sec	Spectrum bounds (alluvium) A g	Spectrum bounds (rock) D in	Spectrum bounds (rock) V in/sec	Spectrum bounds (rock) A g
50	0.5	1.97	2.58	3.67	40	71	124	3.67	24	72	3.67
	2.0	1.68	2.06	2.76	30	60	99	2.76	20	58	2.76
	5.0	1.40	1.66	2.11	20	50	80	2.11	17	46	2.11
	10.0	1.15	1.34	1.65	20	41	64	1.65	14	38	1.65
75	0.5	2.66	3.41	4.65	40	96	164	4.65	32	95	4.65
	2.0	2.24	2.68	3.36	30	81	129	3.36	27	75	3.36
	5.0	1.83	2.10	2.48	20	66	101	2.48	22	59	2.48
	10.0	1.47	1.66	1.89	20	53	80	1.89	18	46	1.89
90	0.5	3.28	4.16	5.53	40	118	200	5.53	39	116	5.53
	2.0	2.74	3.23	3.90	30	99	155	3.90	33	90	3.90
	5.0	2.21	2.51	2.82	20	80	120	2.82	27	70	2.82
	10.0	1.75	1.94	2.11	20	63	93	2.11	21	54	2.11

Ground motion	a, g	v, in/sec	d, in
alluvium	1.0	48	36
rock	1.0	28	12

Horizontal Design Spectra Bounds
Table A6-6. (reference 48)

Percentile	Damping percent	Amplification D	Amplification V	Amplification A	Faring frequency hertz	Spectrum bounds (alluvium) D in	Spectrum bounds (alluvium) V in/sec	Spectrum bounds (alluvium) A g	Spectrum bounds (rock) D in	Spectrum bounds (rock) V in/sec	Spectrum bounds (rock) A g
50	0.5	1.86	2.52	4.02	50	61	73	2.68	20	43	2.68
	2.0	1.65	1.97	2.80	50	54	57	1.87	18	33	1.87
	5.0	1.40	1.51	2.05	50	46	44	1.37	15	26	1.37
	10.0	1.16	1.17	1.59	50	38	34	1.06	13	20	1.06
75	0.5	2.48	3.39	5.46	50	82	98	3.64	27	58	3.64
	2.0	2.17	2.61	3.70	50	72	76	2.47	24	44	2.47
	5.0	1.81	1.97	2.57	50	60	57	1.71	20	33	1.71
	10.0	1.47	1.49	1.92	50	49	43	1.28	16	25	1.28
90	0.5	3.04	4.17	6.76	50	100	121	4.51	33	71	4.51
	2.0	2.63	3.18	4.51	50	87	92	3.01	29	54	3.01
	5.0	2.18	2.37	3.04	50	72	69	2.03	24	40	2.03
	10.0	1.75	1.78	2.22	50	58	52	1.48	19	30	1.48

Ground motion	a, g	v, in/sec	d, in
alluvium	2/3	29	33
rock	2/3	17	11

Vertical Design Spectra Bounds

Table A6-6. (continued) (Ref. 48)

TYPICAL PEAK ACCELERATIONS IN MULTI-STORY BUILDINGS

Accelerations in Fractions of g

No.	Building	Date	No. Stories	Ground, Basement or 1st Floor L	Ground, Basement or 1st Floor T	Ground, Basement or 1st Floor V	Intermediate Level L	Intermediate Level T	Intermediate Level V	Roof or Top Floor L	Roof or Top Floor T	Roof or Top Floor V
Reinforced Concrete												
1	15107 Vanowen	1970	7	0.11	0.11	0.11	0.24	0.23	0.20	0.36	0.40	0.17
2	8244 Orion	1967	7	0.25	0.15	0.19	0.21	0.25	0.24	0.40	0.34	0.26
3	1640 Marengo	1966	7	0.14	0.15	0.09	0.15	0.27	0.13	0.25	0.44	0.15
4	4680 Wilshire	1967	7	0.14	0.10	0.09	0.23	0.19	0.12	0.24	0.30	0.15
5	646 Olive*	1967	7	0.22	0.26	0.09	0.25	0.25	0.13	0.39	0.48	0.26
6	4687 Sunset	1966	8	0.20	0.18	0.15	0.31	0.24	0.15	0.45	0.47	0.22
7	2011 Zonal	1966	9	0.08	0.08	0.07	0.20	0.17	0.10	0.23	0.21	0.11
8	433 Oakhurst	1970	10	0.09	0.06	0.03	0.14	0.14	0.04	0.27	0.27	0.10
9	120 Robertson	1966	10	0.10	0.10	0.04	0.18	0.19	0.10	0.33	0.28	0.12
10	420 Roxbury	1969	10	0.21	0.17	0.05	0.21	0.24	0.11	0.30	0.22	0.14
11	7080 Hollywood	1966	11	0.11	0.11	0.08	0.21	0.13	0.16	0.21	0.13	0.22
12	3710 Wilshire*	1966	11	0.17	0.16	0.09	0.29	0.17	0.11	0.22	0.38	0.17
13	3470 Wilshire	1966	12	0.15	0.12	0.06	0.21	0.22	0.11	0.22	0.25	0.15
14	8639 Lincoln	1969	12	0.04	0.03	0.03	0.09	0.08	0.09	0.13	0.13	0.06
15	15250 Ventura	1971	12	0.17	0.24	0.11	0.25	0.28	0.16	0.18	0.30	0.18
16	2500 Wilshire	1969	13	0.11	0.13	0.06	0.14	0.16	0.07	0.20	0.20	0.15
17	6200 Wilshire	1970	16	0.12	0.13	0.03	0.29	0.17	0.05	0.28	0.26	0.08
18	4000 Chapman	1970	19	0.02	0.02	0.02	0.05	0.04	0.03	0.06	0.06	0.04
Steel Frame												
1	5260 Century*	1968	7	0.06	0.05	0.02	0.05	0.07	0.04	0.07	0.05	0.09
2	3407 Sixth	1966	8	0.17	0.20	0.06	0.22	0.22	0.10	0.28	0.22	0.27
3	1150 Hill	1970	10	0.12	0.09	0.05	0.10	0.12	0.09	0.12	0.12	0.15
4	900 Fremont	1971	12	0.14	0.14	0.07	0.14	0.15	0.12	0.18	0.15	0.17
5	L.A. Water and Power	1969	15	0.15	0.20	0.08	0.19	0.14	0.09	0.17	0.13	0.16
6	250 First	1967	15	0.10	0.14	0.06	0.21	0.17	0.09	0.17	0.18	0.20
7	1800 Century Park East	1970	16	0.08	0.11	0.08	0.23	0.25	0.16	0.28	0.28	0.33
8	800 First	1969	33	0.09	0.14	0.06	0.13	0.19	0.17	0.17	0.27	0.25

* Shear Wall

Table A6-7.
(reference 45)

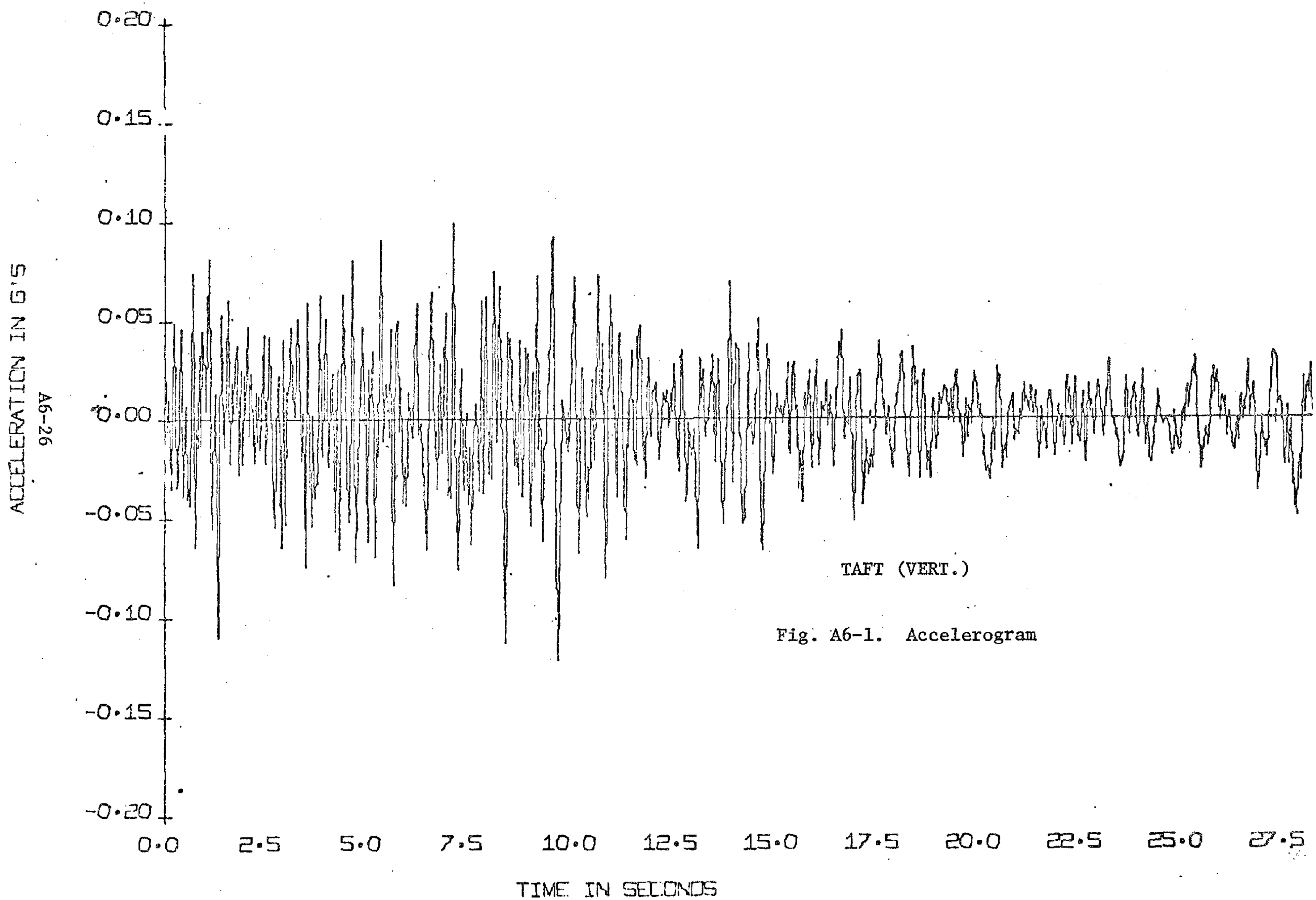

Fig. A6-1. Accelerogram

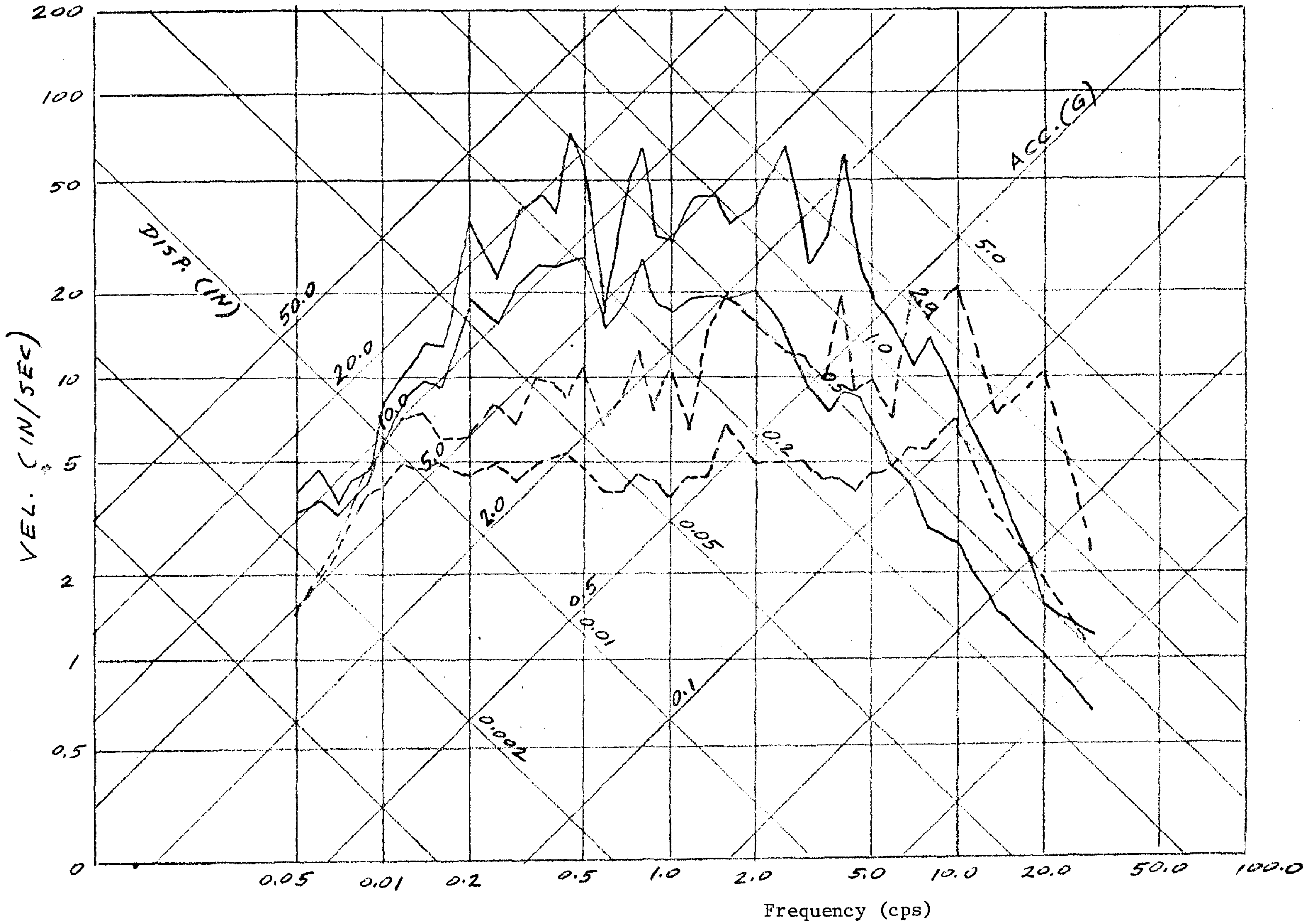

Fig. A6-2.
Response Spectra - El Centro, 5/18/40
——— EW, 0,5% Critical Damping
- - - - VERT, 0,5% Critical Damping

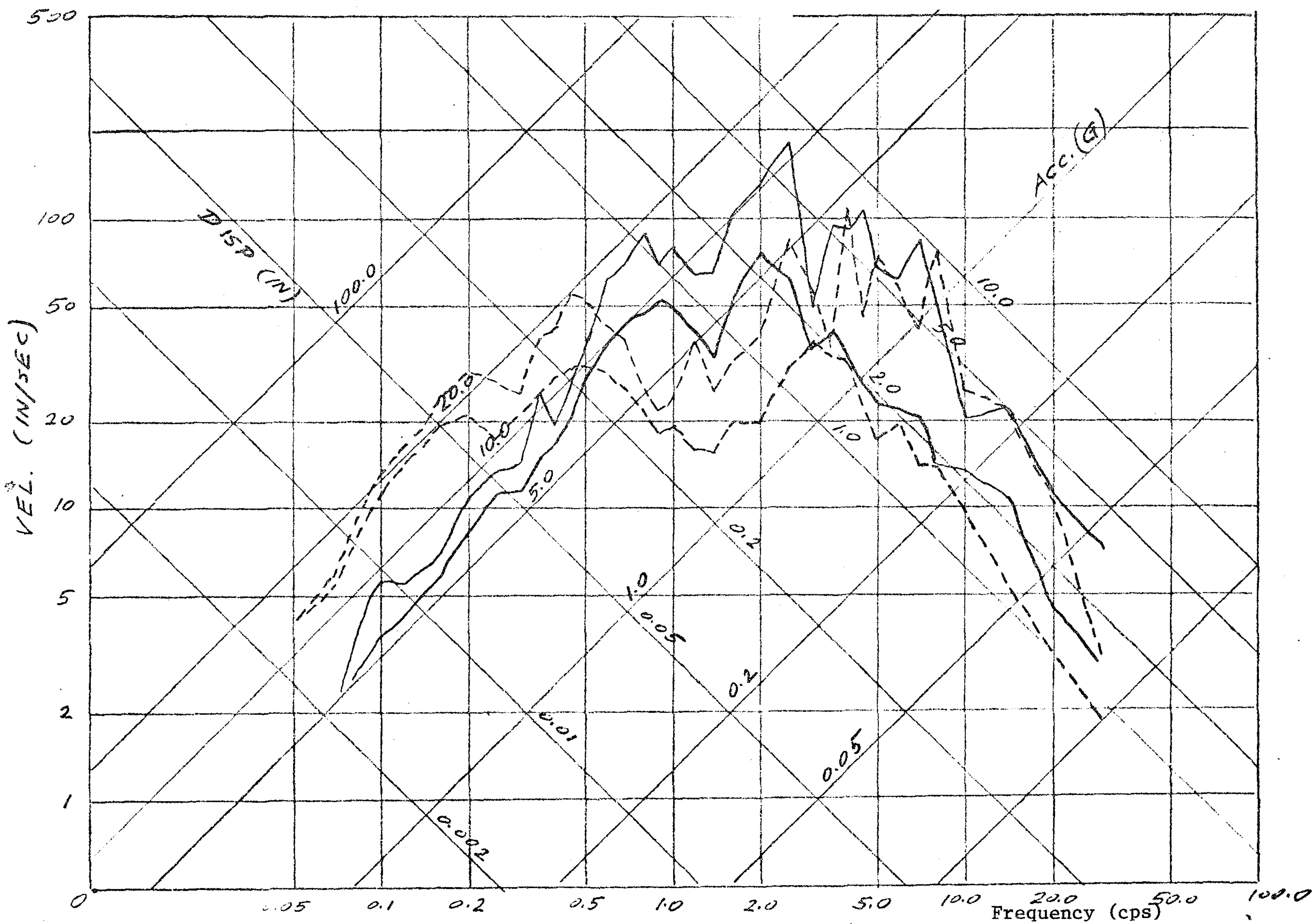

Fig. A6-3. Response Spectra - San Fernando, 2/9/71 - Pacoima

——— S74W, 0,5% Critical Damping

- - - - VERT, 0,5% Critical Damping

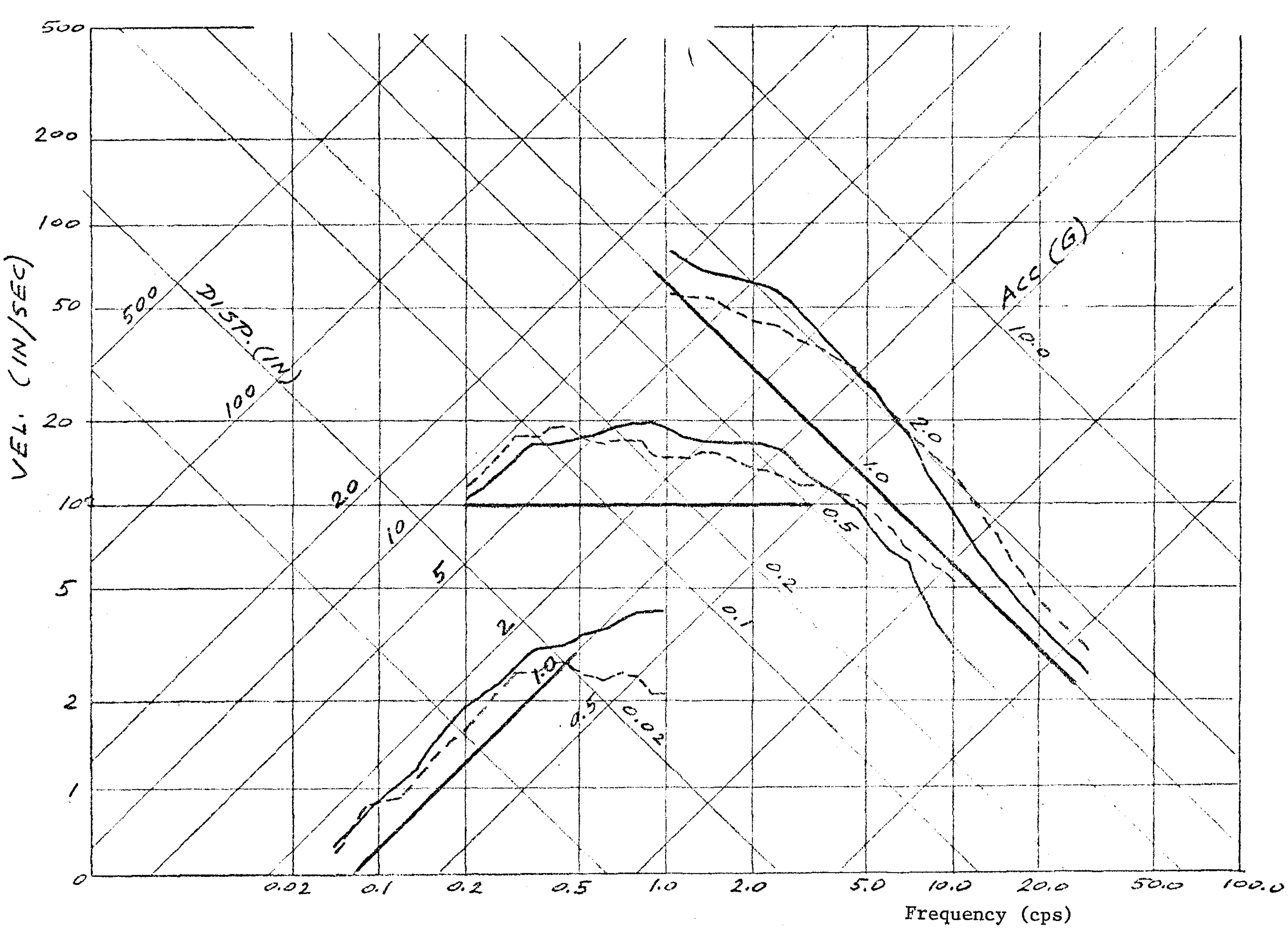

Fig. A6-4.
Mean Response Amplification, 5% Crit. Damping
——— Horizontal Components
- - - - Vertical Components

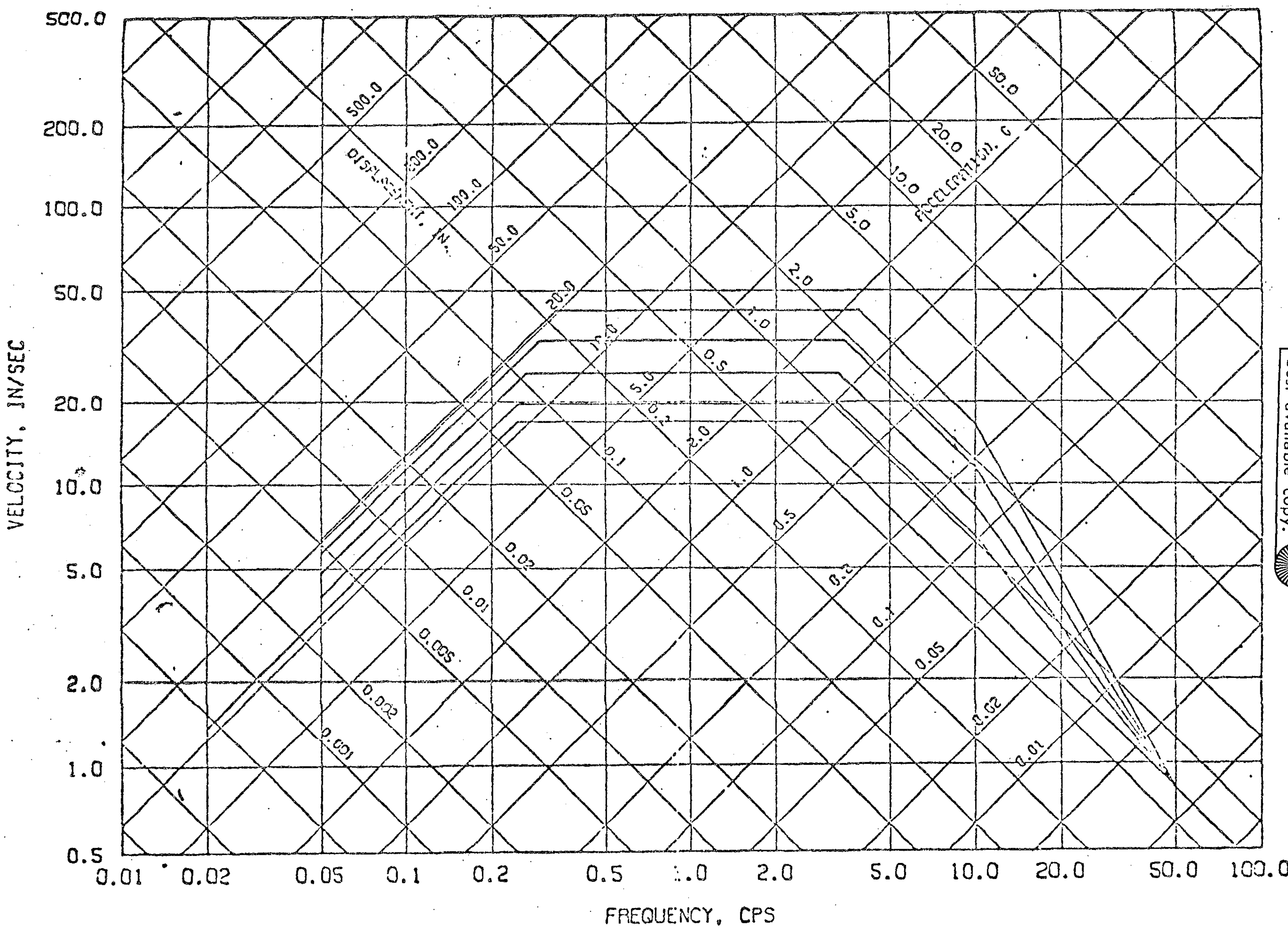

Fig. A6-5.
Design Spectra, Vertical Direction. Rock. 50 Percentile
0.5, 2, 5, & 10 percent critical damping (reference 48)

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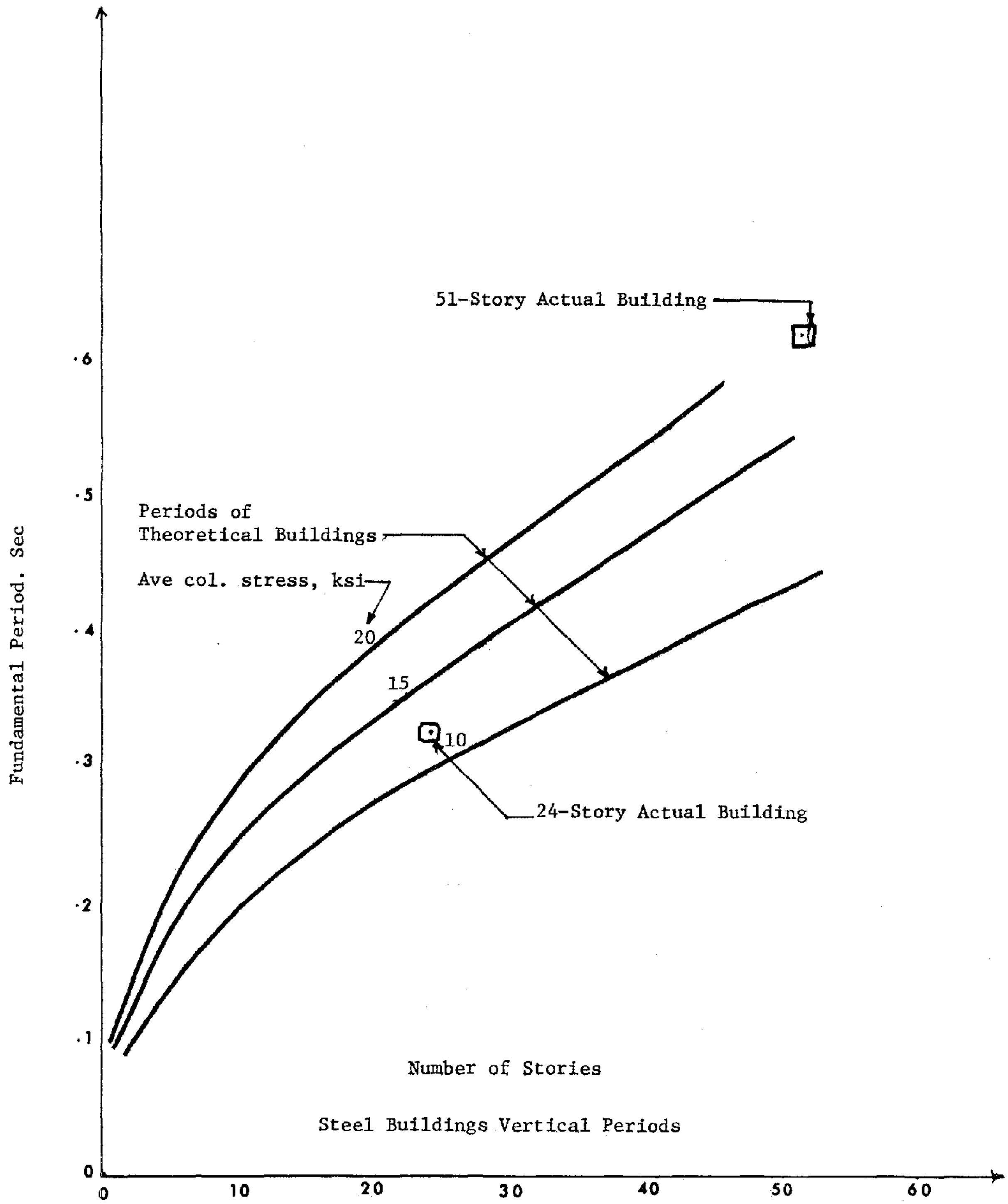


Fig. A6-6. (reference 46)

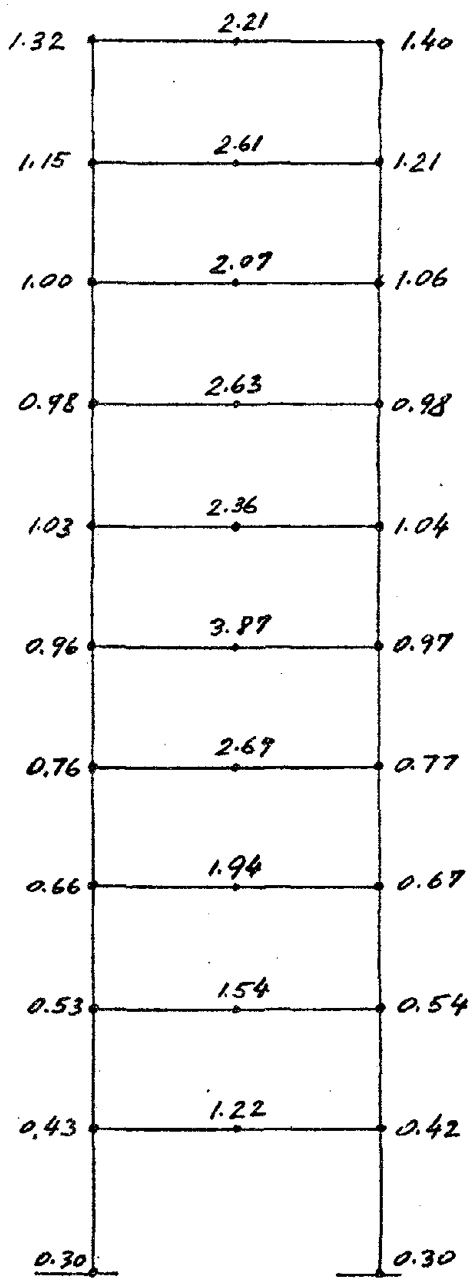
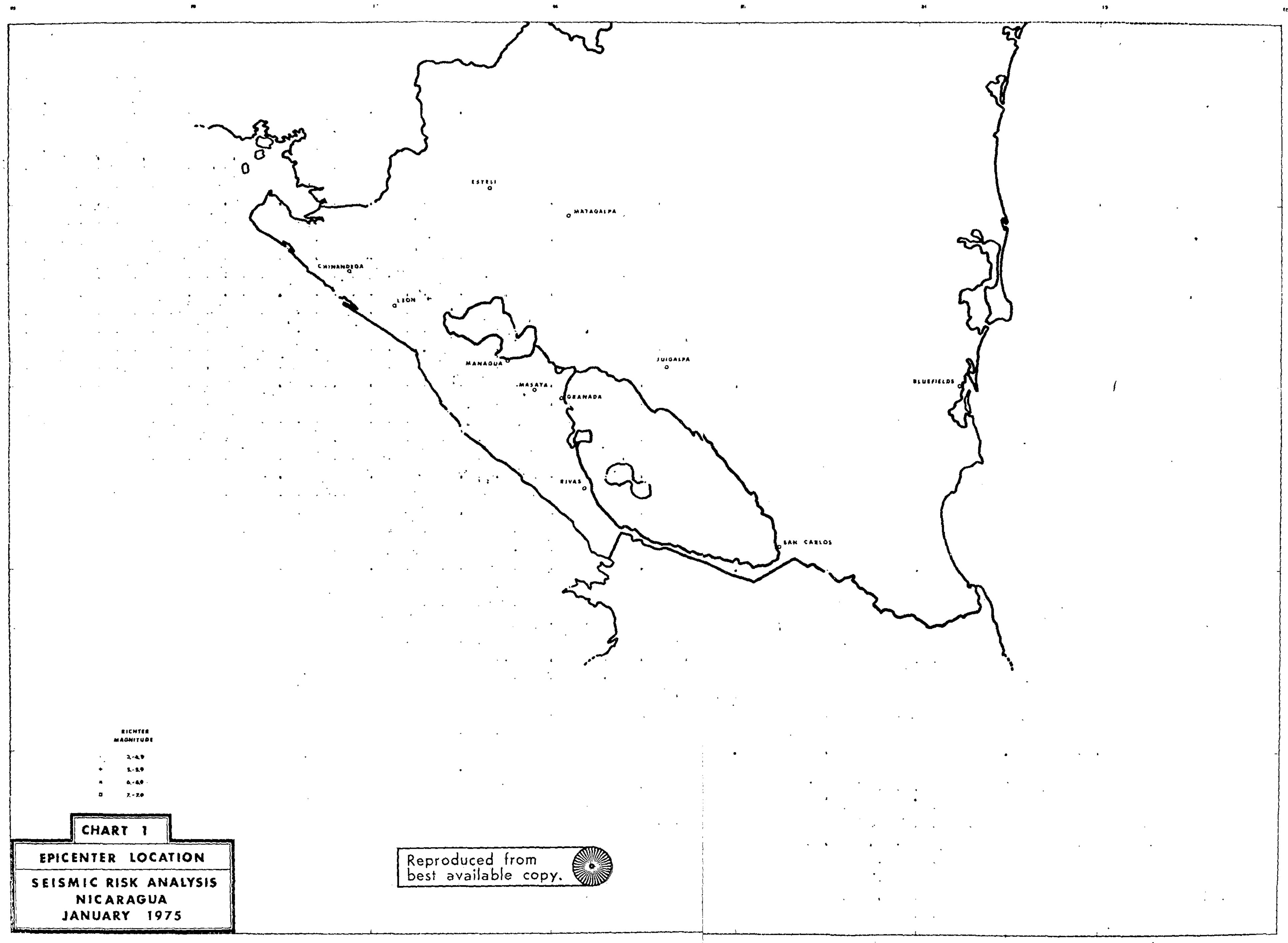


Fig. A6-7. Maximum Vertical Accelerations for Pacoima Dam Record (ref. 42)

CHARTS 1 THROUGH 13



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A7-2

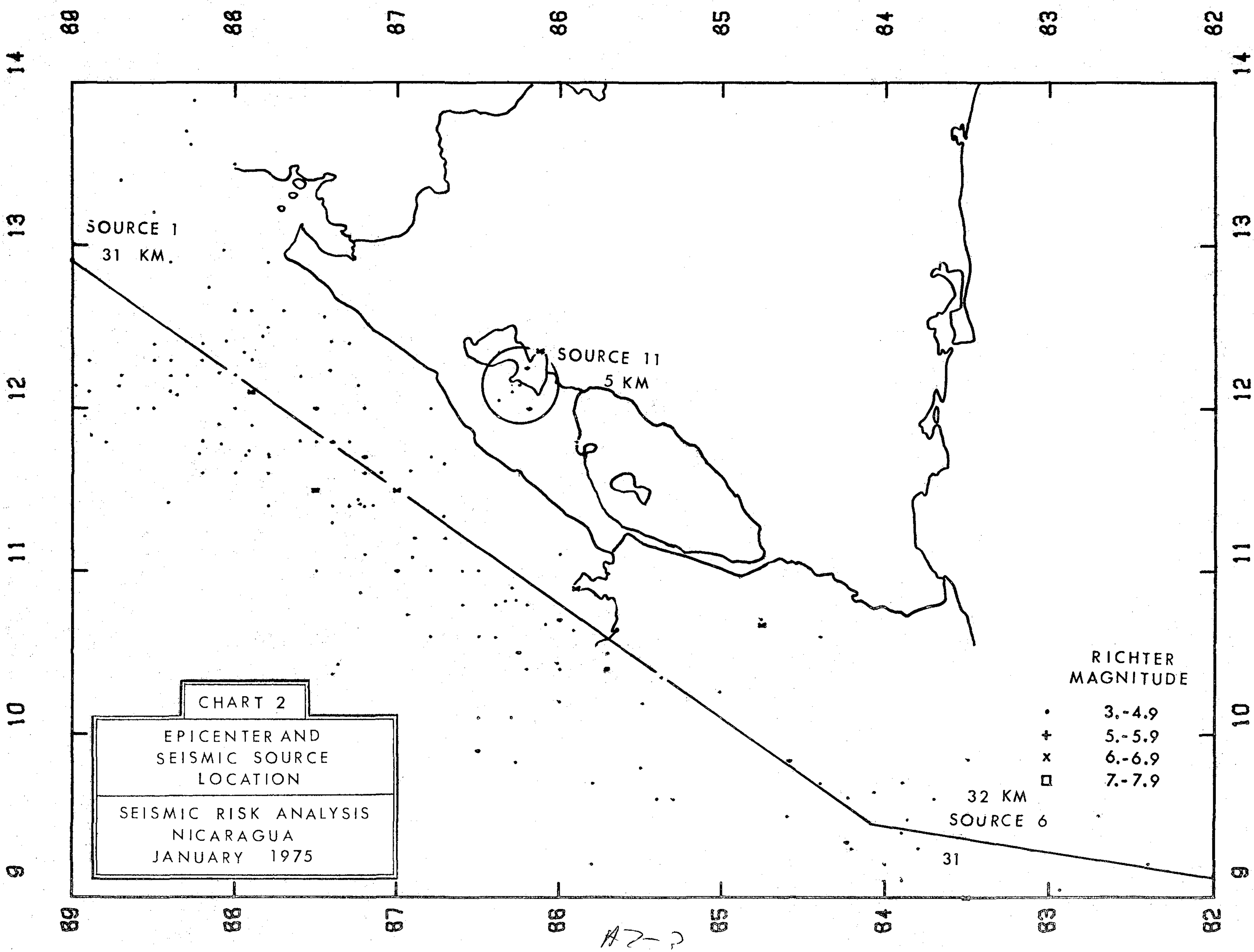

CHART 2

EPICENTER AND SEISMIC SOURCE LOCATION

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975

A7-?

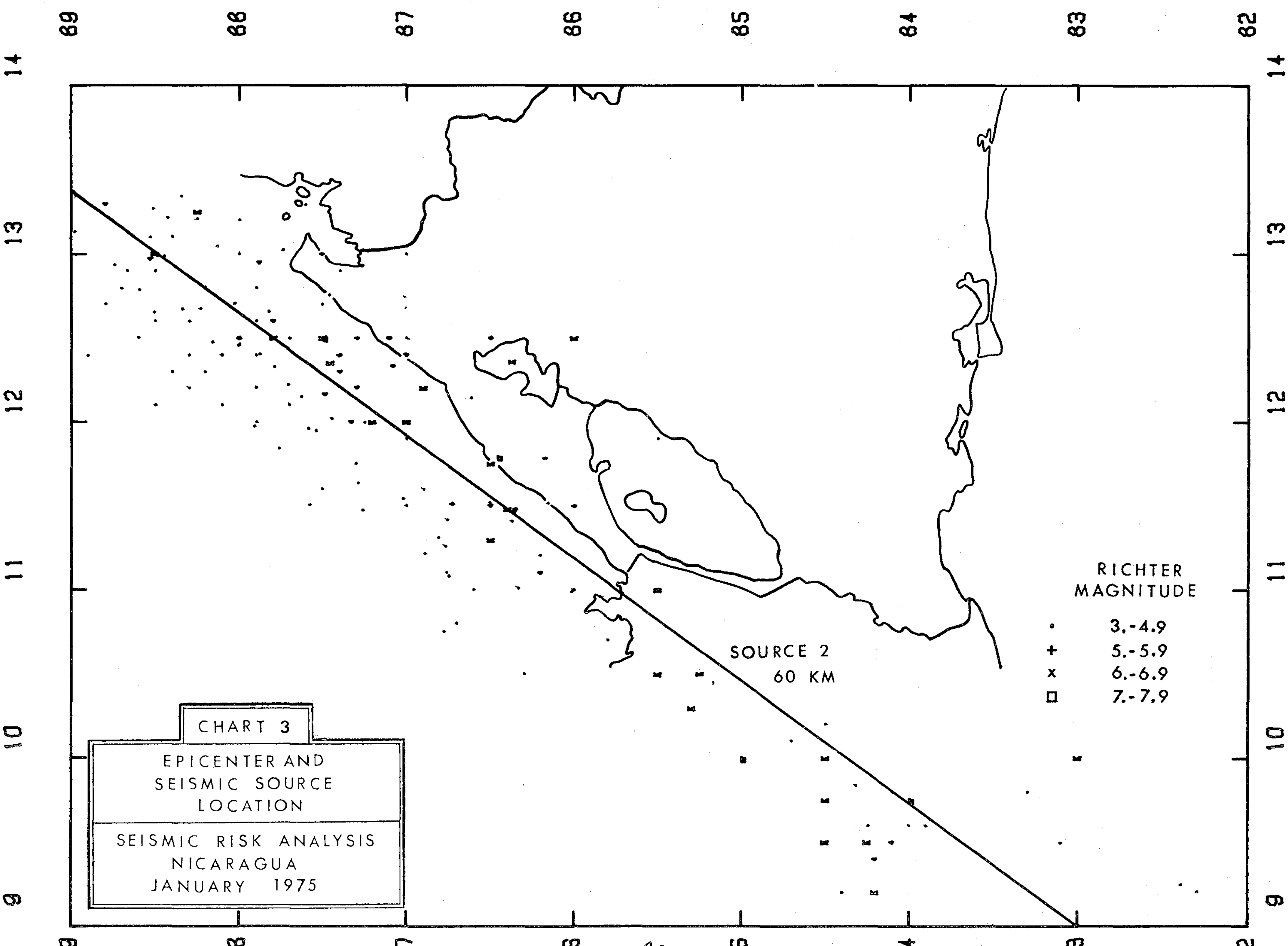

CHART 3

EPICENTER AND SEISMIC SOURCE LOCATION

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975

A7-4

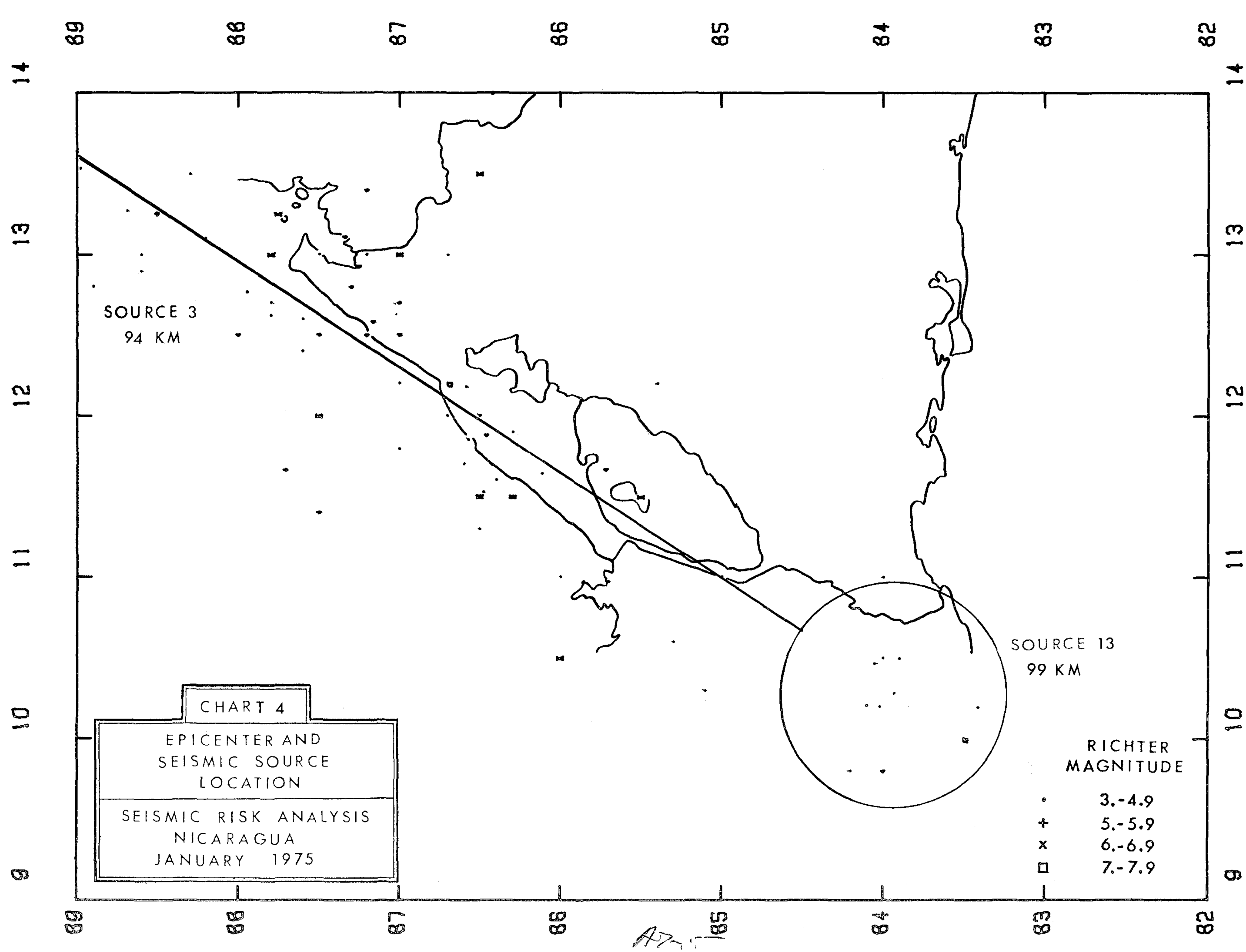

CHART 4

EPICENTER AND SEISMIC SOURCE LOCATION

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975

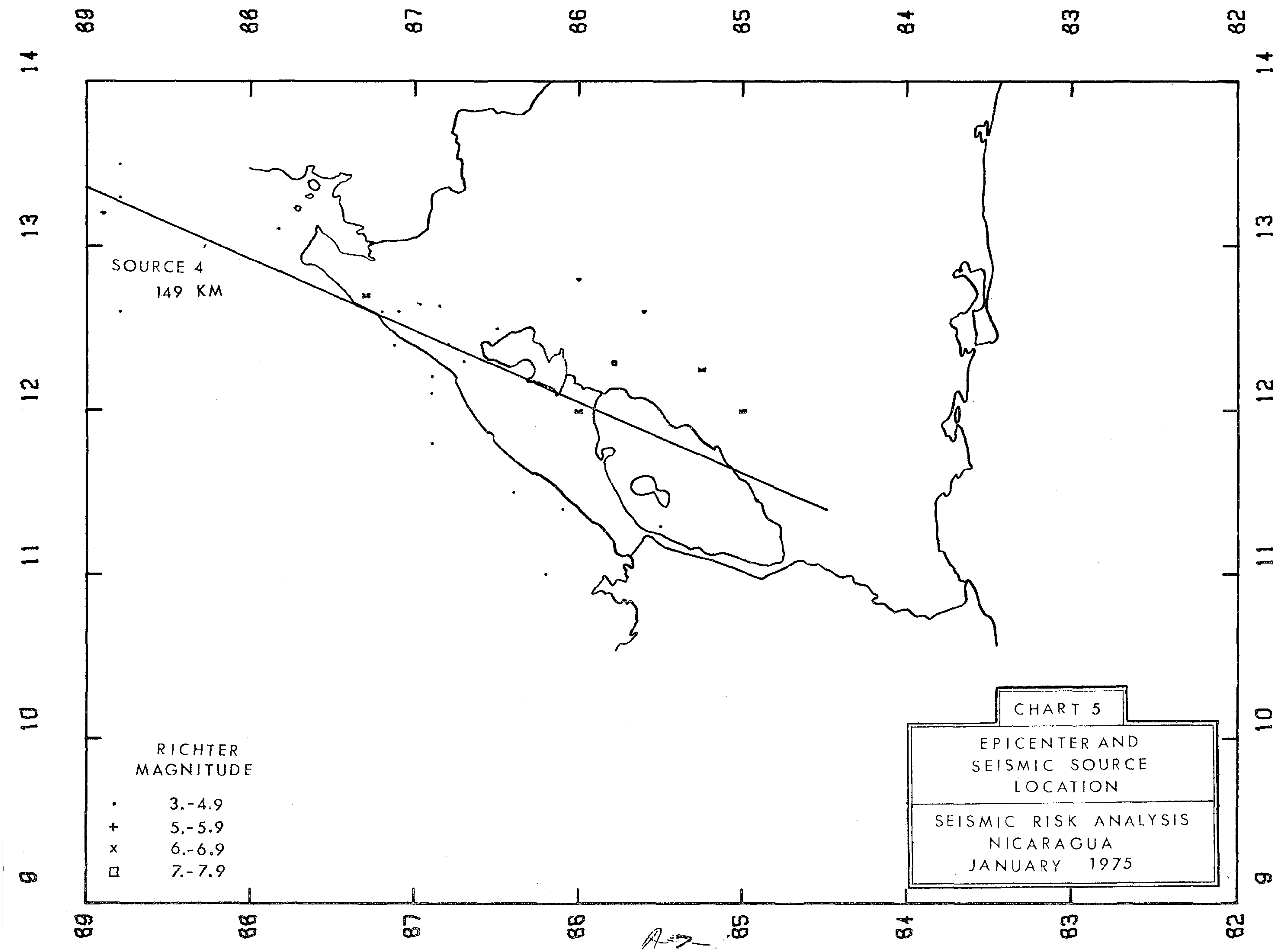
CHART 5
EPICENTER AND
SEISMIC SOURCE
LOCATION
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
SOURCE 4
149 KM
RICHTER
MAGNITUDE
3.-4.9
5.-5.9
6.-6.9
7.-7.9
89
88
87
86
85
84
83
82
14
13
12
11
10
9

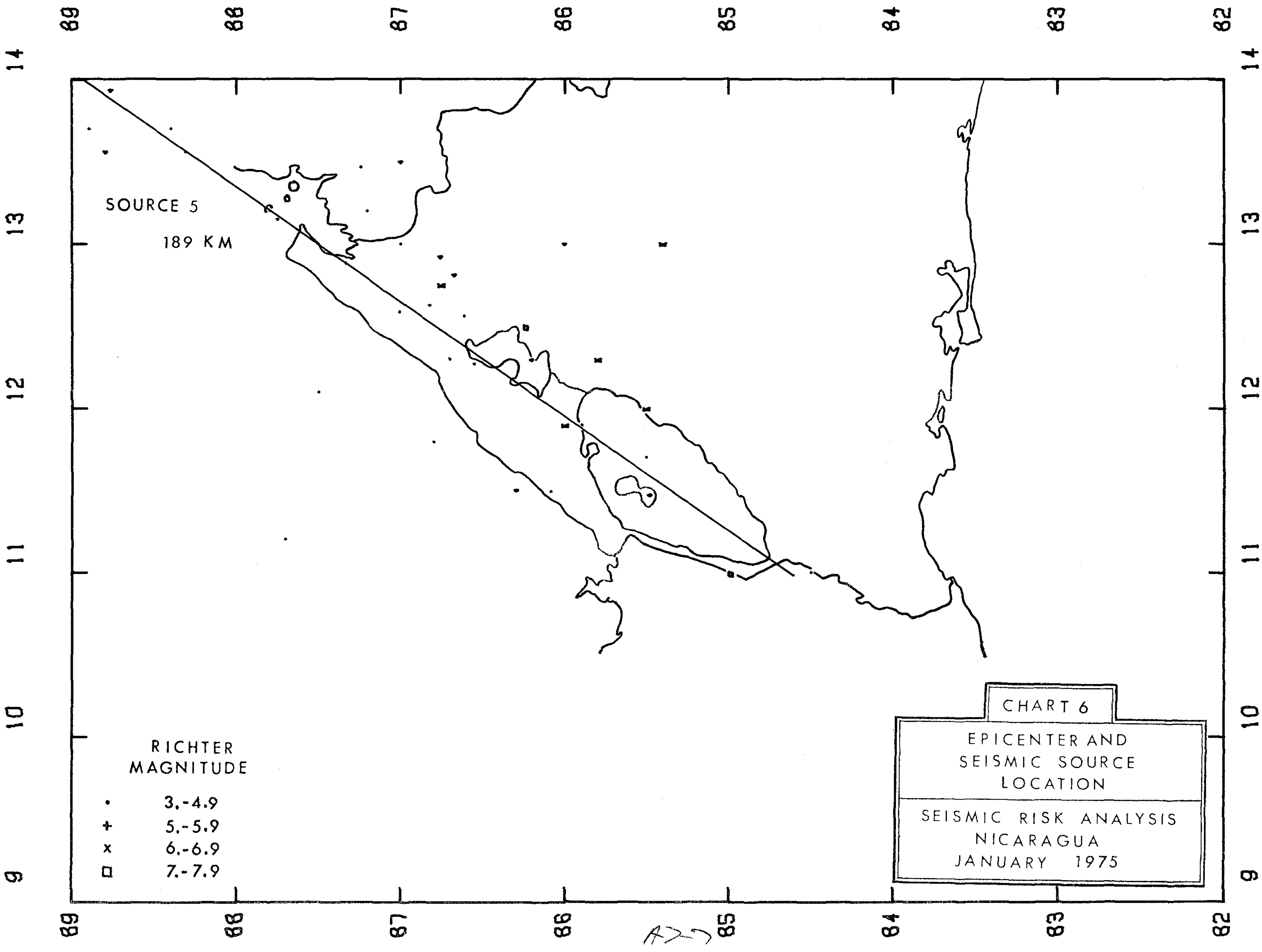
CHART 6
EPICENTER AND
SEISMIC SOURCE
LOCATION
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
RICHTER
MAGNITUDE
3.-4.9
5.-5.9
6.-6.9
7.-7.9
SOURCE 5
189 KM
89
88
87
86
85
84
83
82
14
13
12
11
10
9

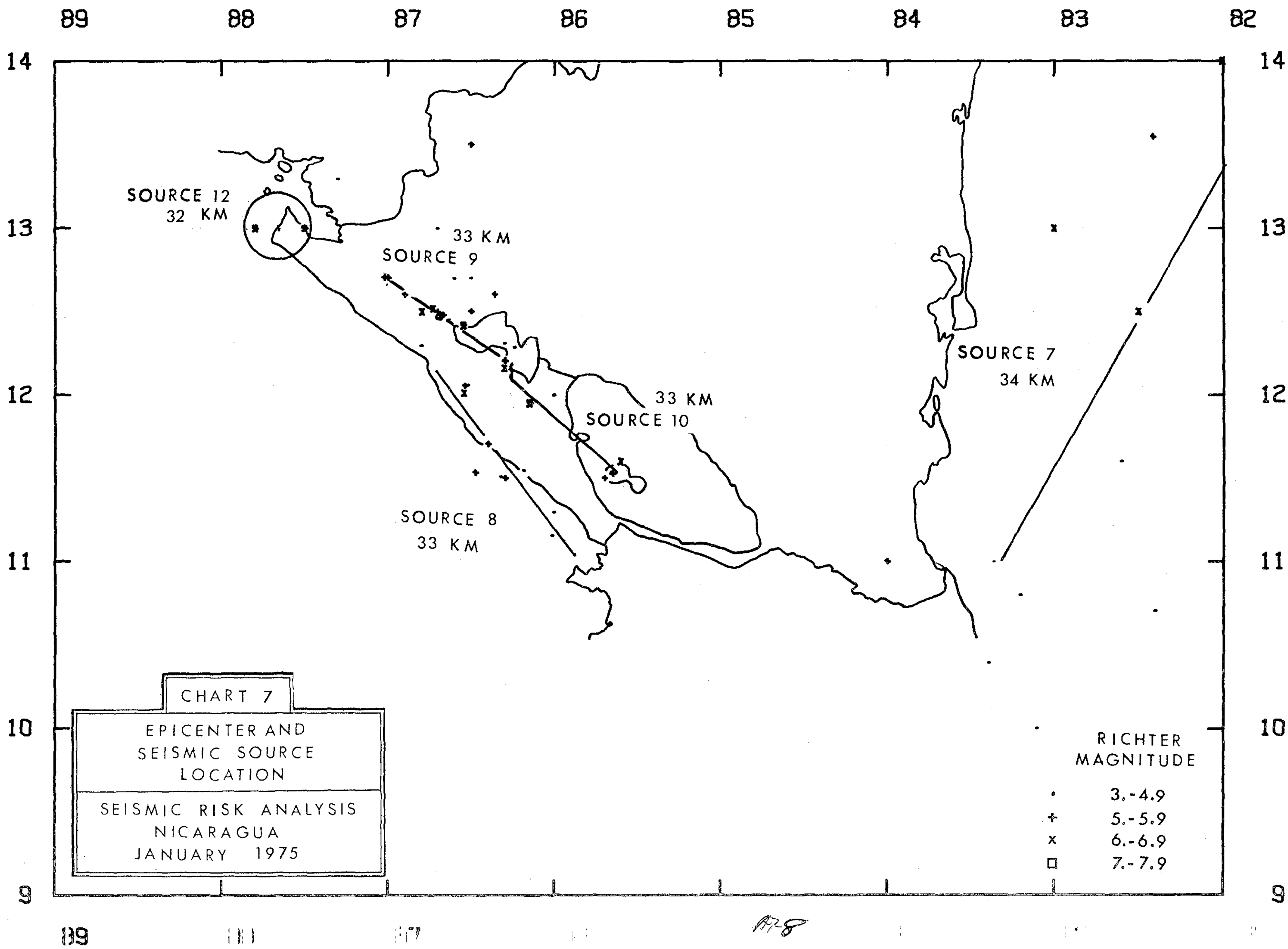

CHART 7

EPICENTER AND
SEISMIC SOURCE
LOCATION

SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975

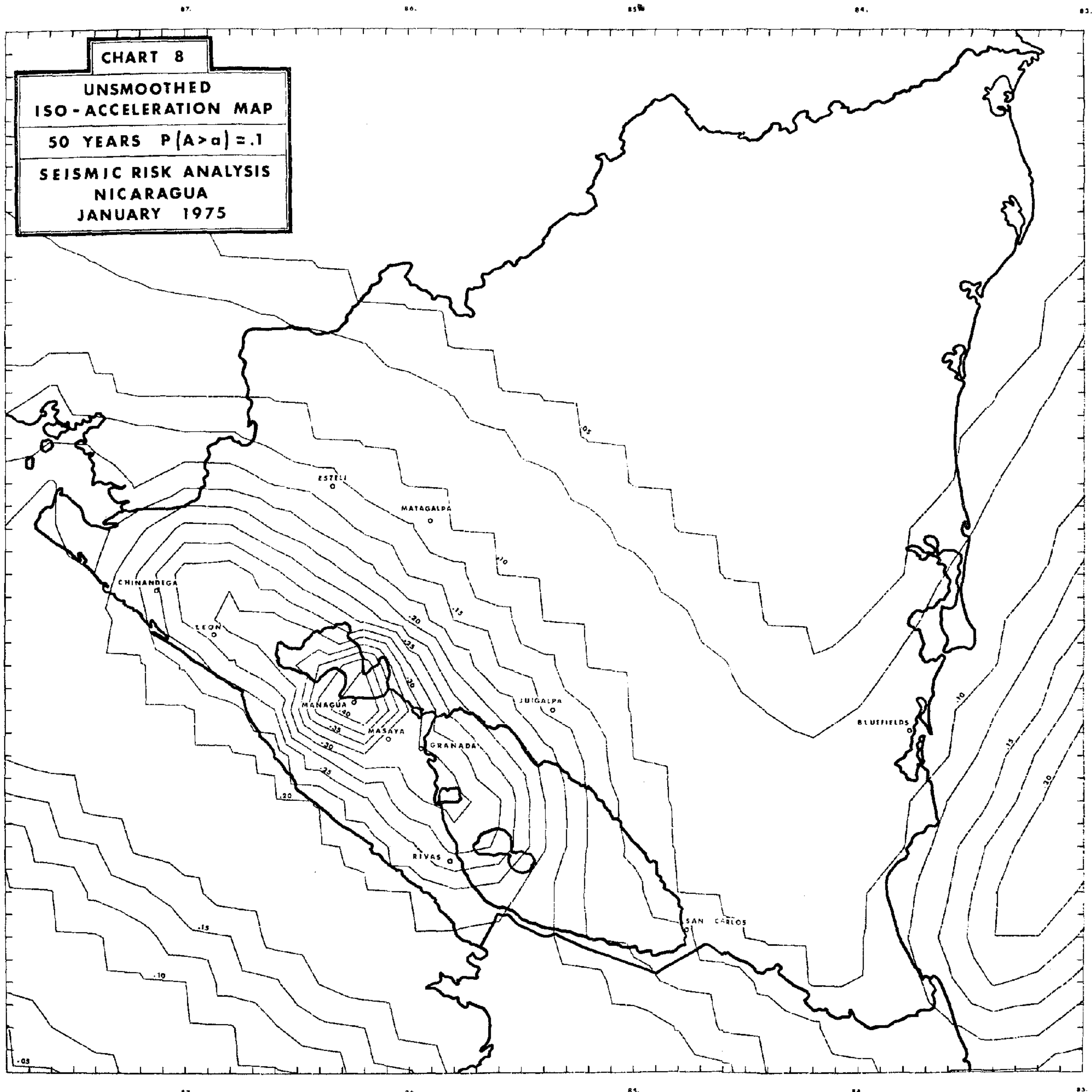
CHART 8
UNSMOOTHED
ISO-ACCELERATION MAP
50 YEARS P(A>a)=.1
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
ESTELI
MATAGALPA
CHINANDEGA
LEON
MANAGUA
MASAYA
GRANADA
JUIGALPA
RIVAS
SAN CARLOS
BLUEFIELDS
.05
.10
.15
.20
.25
.30
.35
.40
87.
86.
85.
84.
83.
15.
14.
13.
12.
11.

A7-9

A7-10

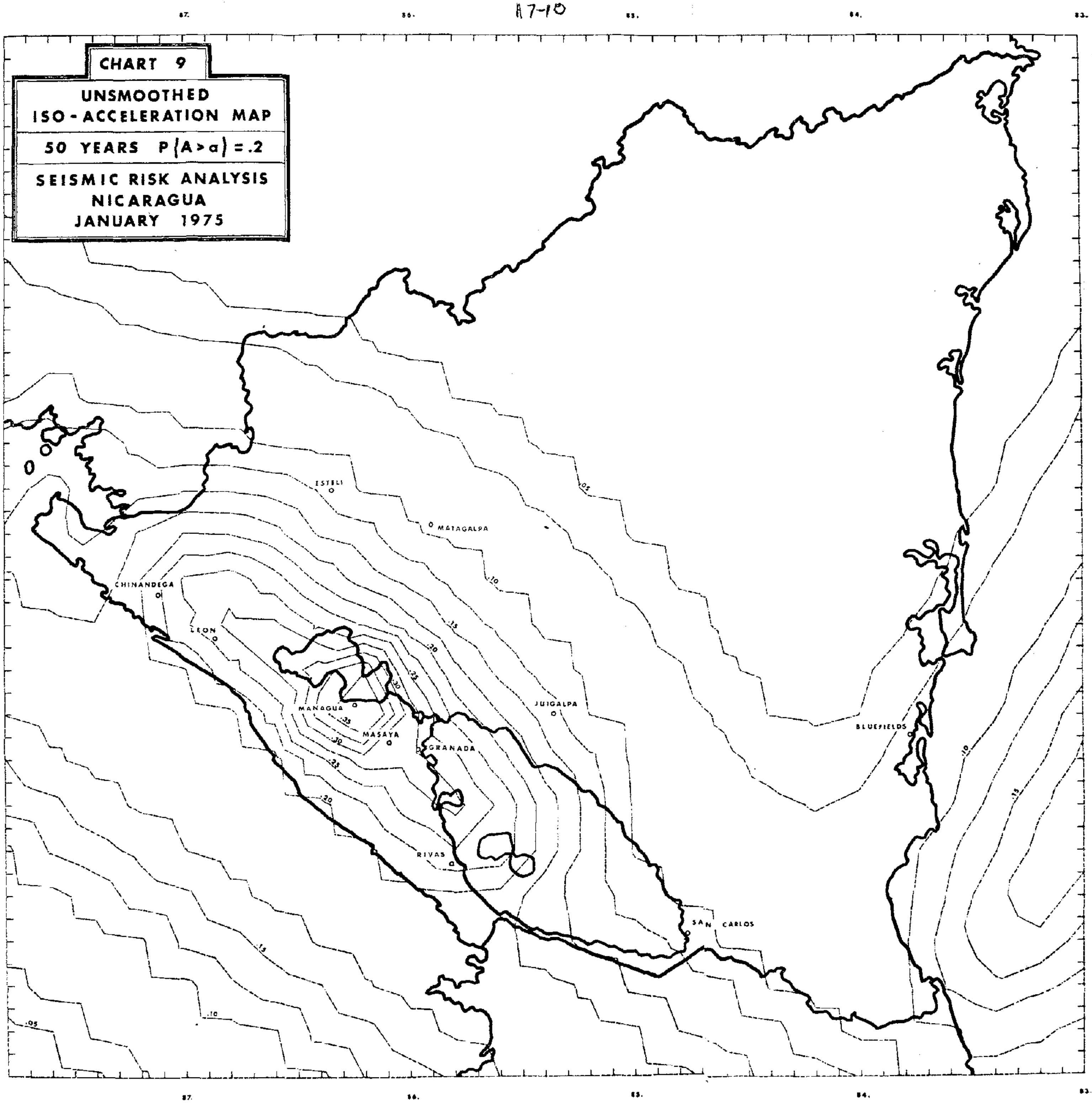

CHART 9
UNSMOOTHED
ISO-ACCELERATION MAP
50 YEARS P(A>a) = .2
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
ESTELI
MATAGALPA
CHINANDEGA
LEON
MANAGUA
MASAYA
GRANADA
JUIGALPA
RIVAS
SAN CARLOS
BLUEFIELDS
.05
.10
.15
.20
.25
.30
.35

A7-11

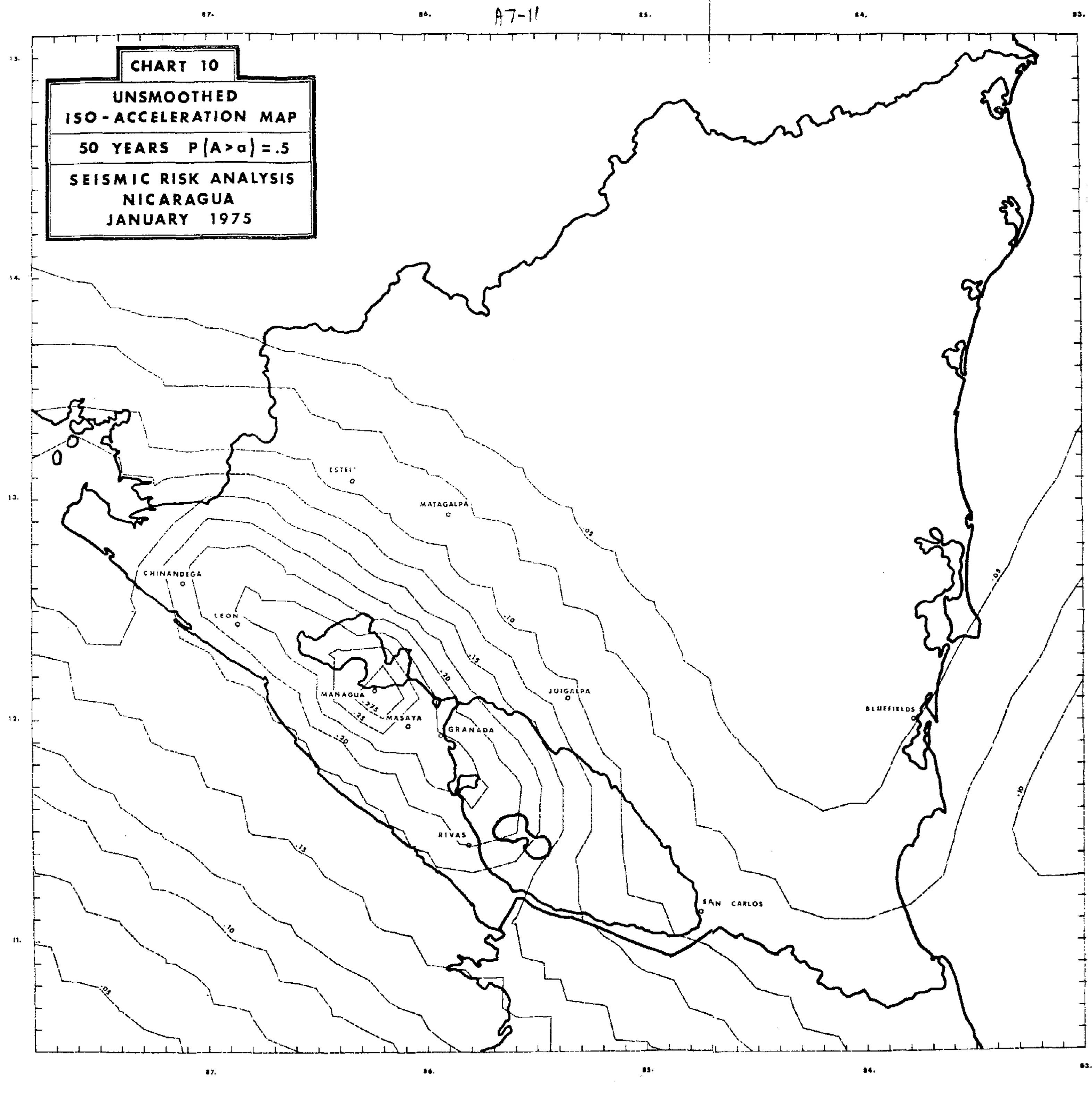

CHART 10
UNSMOOTHED
ISO-ACCELERATION MAP
50 YEARS P(A>a)=.5
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
ESTELI
MATAGALPA
CHINANDEGA
LEON
MANAGUA
MASAYA
GRANADA
JUIGALPA
RIVAS
SAN CARLOS
BLUEFIELDS
.05
.10
.15
.20
.25
.275

A7-12

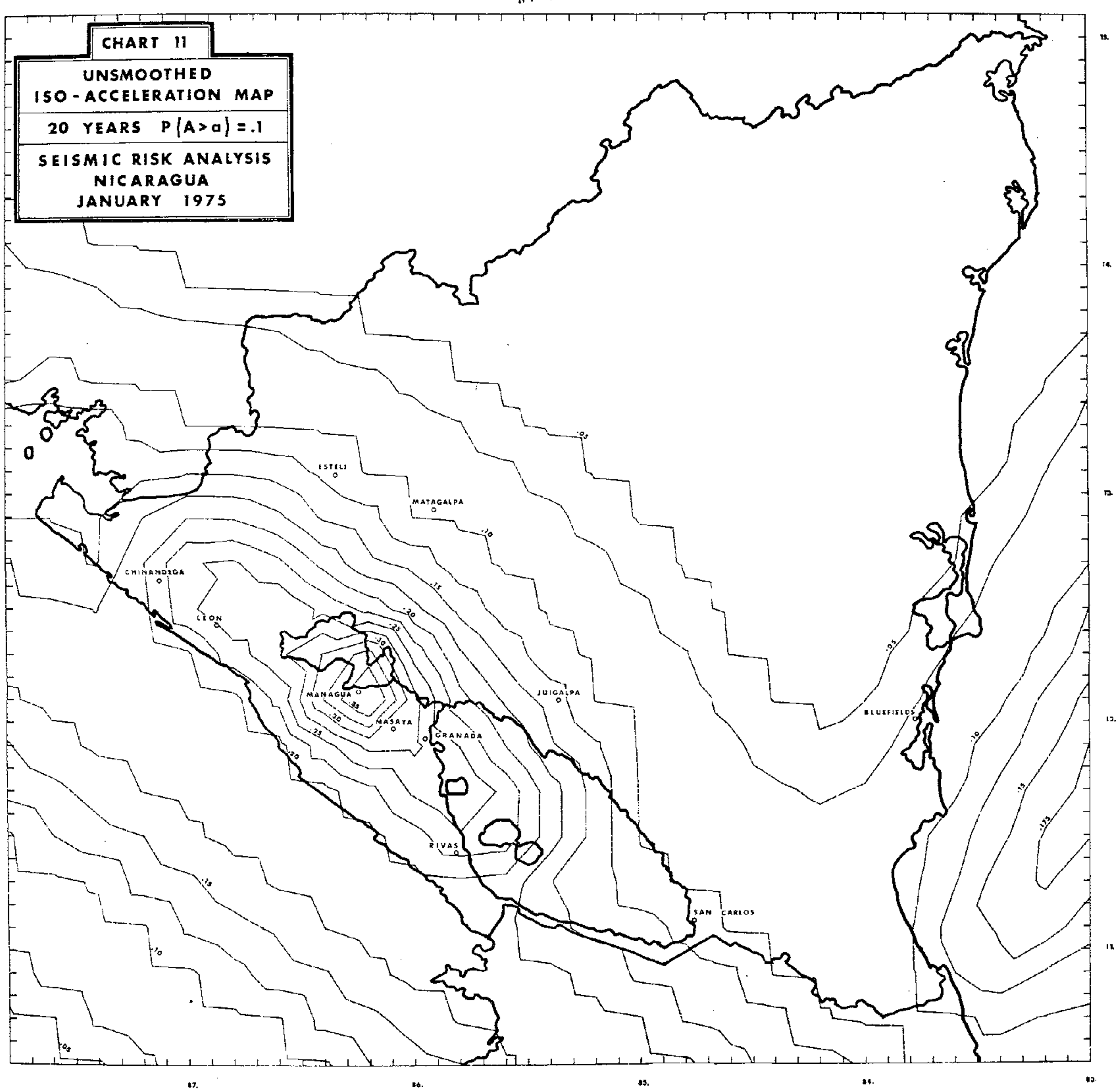
CHART 11
UNSMOOTHED
ISO-ACCELERATION MAP
20 YEARS P(A>a) = .1
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
ESTELI
MATAGALPA
CHINANDEGA
LEON
MANAGUA
MASAYA
GRANADA
JUIGALPA
RIVAS
SAN CARLOS
BLUEFIELDS

A7-13

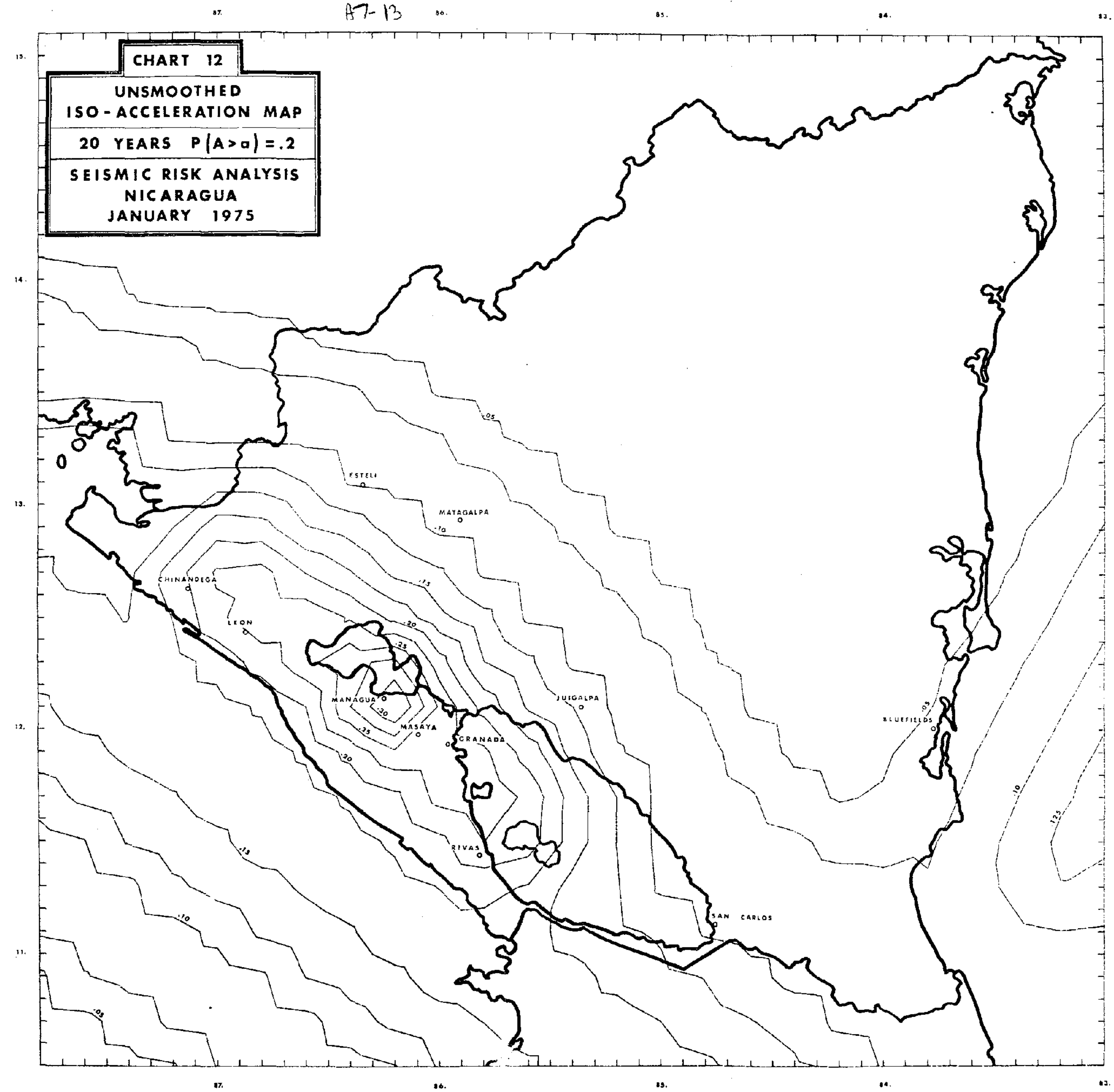
CHART 12
UNSMOOTHED
ISO-ACCELERATION MAP
20 YEARS P(A>a)=.2
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
ESTELI
MATAGALPA
CHINANDEGA
LEON
MANAGUA
MASAYA
GRANADA
JUIGALPA
RIVAS
SAN CARLOS
BLUEFIELDS
.05
.10
.15
.20
.25
.30
.125

A-7-14

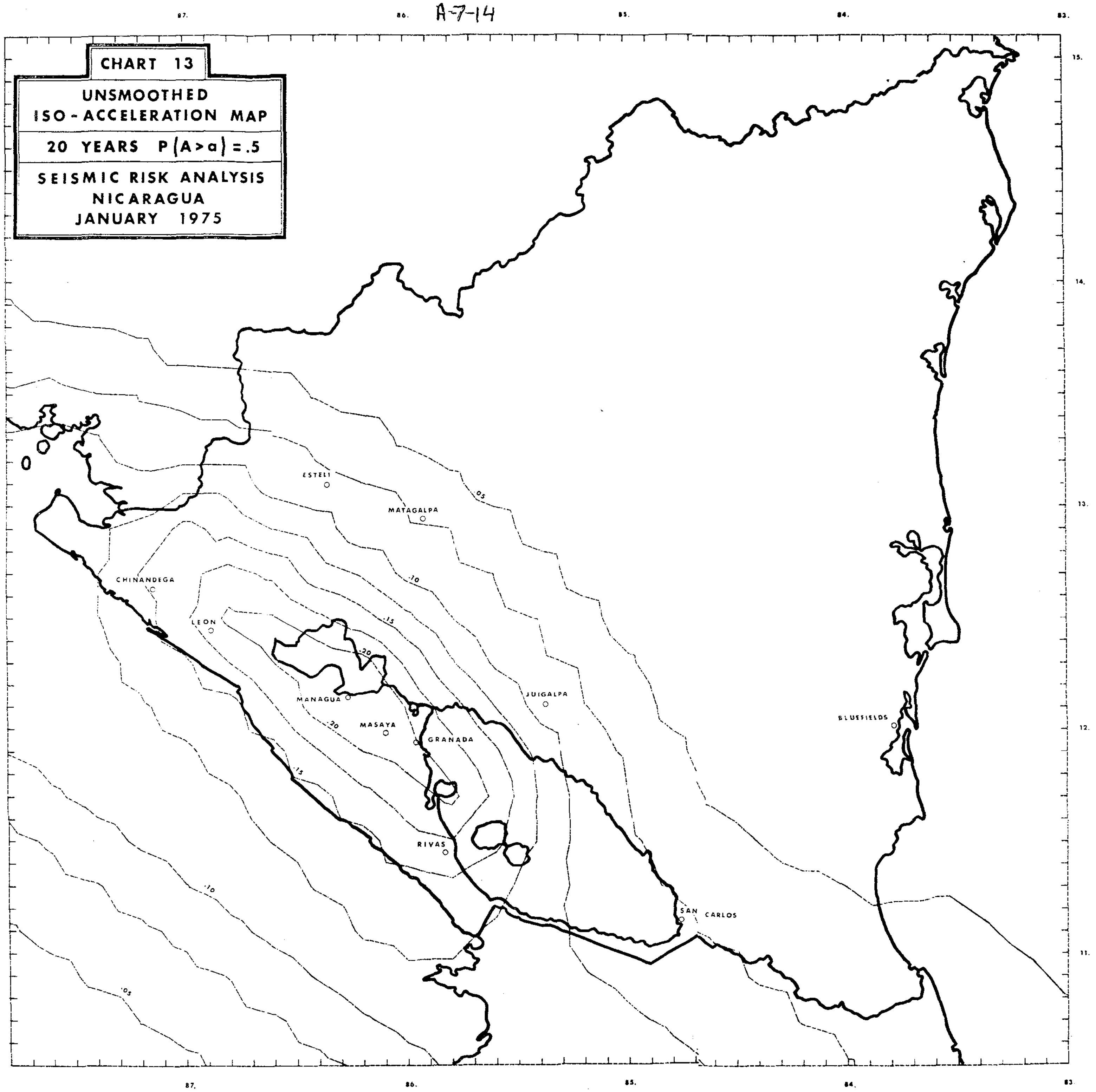
CHART 13
UNSMOOTHED
ISO-ACCELERATION MAP
20 YEARS P(A>a)=.5
SEISMIC RISK ANALYSIS
NICARAGUA
JANUARY 1975
87.
86.
85.
84.
83.
15.
14.
13.
12.
11.
ESTELI
MATAGALPA
CHINANDEGA
LEON
MANAGUA
MASAYA
GRANADA
JUIGALPA
RIVAS
SAN CARLOS
BLUEFIELDS
.05
.10
.15
.20