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FORECASTING THE RISK INHERENT IN EARTHQUAKE RESISTANT DESIGN

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I. INTRODUCTION

1.1 Statement of the Problem

An engineer designing a structure in an area which is seismically active must meet the minimum provisions of the applicable building codes. For seismic resistance this means designing for a statically applied horizontal base shear which is supposed to be equivalent to the dynamic loading of the design earthquake. Depending on the engineer's philosophy of what constitutes an earthquake resistant structure, the engineer may have provided substantially more resistance than the building code requires. Yet the building code does not provide the engineer with information or guidelines to evaluate the added safety against collapse or damage during an earthquake. Furthermore, if the engineer wishes to compare alternate designs of a proposed structure from the viewpoint of reduced danger against collapse or damage during an earthquake the building code is of little use. These problems are frequently encountered by structural designers. Consequently, this dissertation was written to go beyond the building codes and present analytical solutions to these problems.

There are two fundamental requirements to be met in designing an earthquake resistant structure. They cannot be met with certainty, but when analyzed probabilistically their interpretation is realistic. In the event of an earthquake there should be no loss of life or serious injury from the damage or collapse of the structure. Secondly, the cost of repairing the damage from the earthquake should not exceed the increased design, construction and financing costs which would have prevented damage, or collapse.

There are numerous design approaches to earthquake resistant structures. A possible design approach would be to design the structure to withstand the largest possible ground motion. Of course, the structure would be extremely costly to construct and this would place a severe economic burden on the financing of construction in the future. On the other hand, a more reasonable approach would be to provide earthquake resistance appropriate to the site conditions, the type of structure, and the regional economic conditions. If the soil conditions at a site are such that the expected damage level would be increased and if the past seismicity of the region indicates a high level of activity then additional cost to provide greater protection from an earthquake is warranted. Furthermore, a heavily industrialized region, desiring to protect its functioning economy, might want to require increased protection. Increased protection from earthquakes can be obtained at greater cost. Earthquake engineering attempts to reconcile the cost of greater protection with the additional safety it brings.

Implicit in this latter approach to earthquake engineering of structures is the acceptance of a certain level of damage during an earthquake. In fact, the earthquake provisions of the building codes in effect in different regions are statements by those communities regarding how much is to be spent to protect themselves from excessive earthquake damage. It is not implied that building according to the building codes will preclude damage from earthquakes.

Furthermore, the building code does not require that an evaluation of the potential hazard of a structure's damage or collapse in the event of an earthquake be made. Many engineers do perform these evaluations as part of a seismic analysis, but this is the exception. The problem here is not solely of an engineering nature. There is, in addition, the fact that the general public is not psychologically prepared to accept the expression of the safety of a building in terms of its probability to withstand earthquakes of a given magnitude. Consequently, engineers are reluctant to admit that a risk is accepted whenever a structure is built in a seismically active region.

The problem to which this dissertation is directed combines the engineering and economic considerations of designing a structure in the presence of an earthquake hazard. The associated risk is defined as a function of the seismicity of the area and the expected damage level. It attempts to go beyond the building code and illustrates methods of estimating the probable damage to a building constructed according to the applicable building code. It considers the added cost of earthquake protection beyond that required by the building code and indicates the benefits to be received from the added protection. Furthermore, a major portion of this study is an analysis of alternate designs to provide a required facility. It is recognized that optimum designs are possible only for a given region and economic situation.

Data are gathered for this study from the investigation of past earthquakes. Wherever possible the analysis is conducted on an empirical foundation. However, there are instances when meaningful data are not available and in these cases, engineering judgment must be exercised in order to generate the necessary information.

Figure 1.1 is a schematic of the problem. It indicates as the first step a macroregionalization of the country. The country should be divided into areas of investigation based on seismological and geologic factors. Investigation should be made of the extent of faulting, magnitudes of past earthquakes, and the soil conditions in the region before it is considered as a unit. Furthermore, the economy and population of the region might also be taken into consideration. Generally, this is not done. However, macroregionalization is done with the intent of establishing construction regulations for the region. Therefore, an area with a large industrial base would very likely want to protect its functioning economy and is therefore very likely to be willing to pay a greater cost for earthquake protection. The risk of damage or collapse of its facilities must be substantially smaller. In regions where there are large populations the chance of injury or death to the inhabitants is very great. This must also be considered relative to the risk accepted. Each area of macroregionalization must then be considered separately.

Microregionalization considers conditions at the actual building site. For each structure there is an intensity of ground shaking above which damage to the structure begins. As ground shaking intensifies damages increase and repairs become more costly. The ground motion of the building site is a function of the magnitude of the earthquake, the attenuation characteristics of the soil between the hypocenter and site, and the soil conditions at the site.

Each type of structure responds differently to the excitation of earthquake waves. The type of structure influences the expected damage

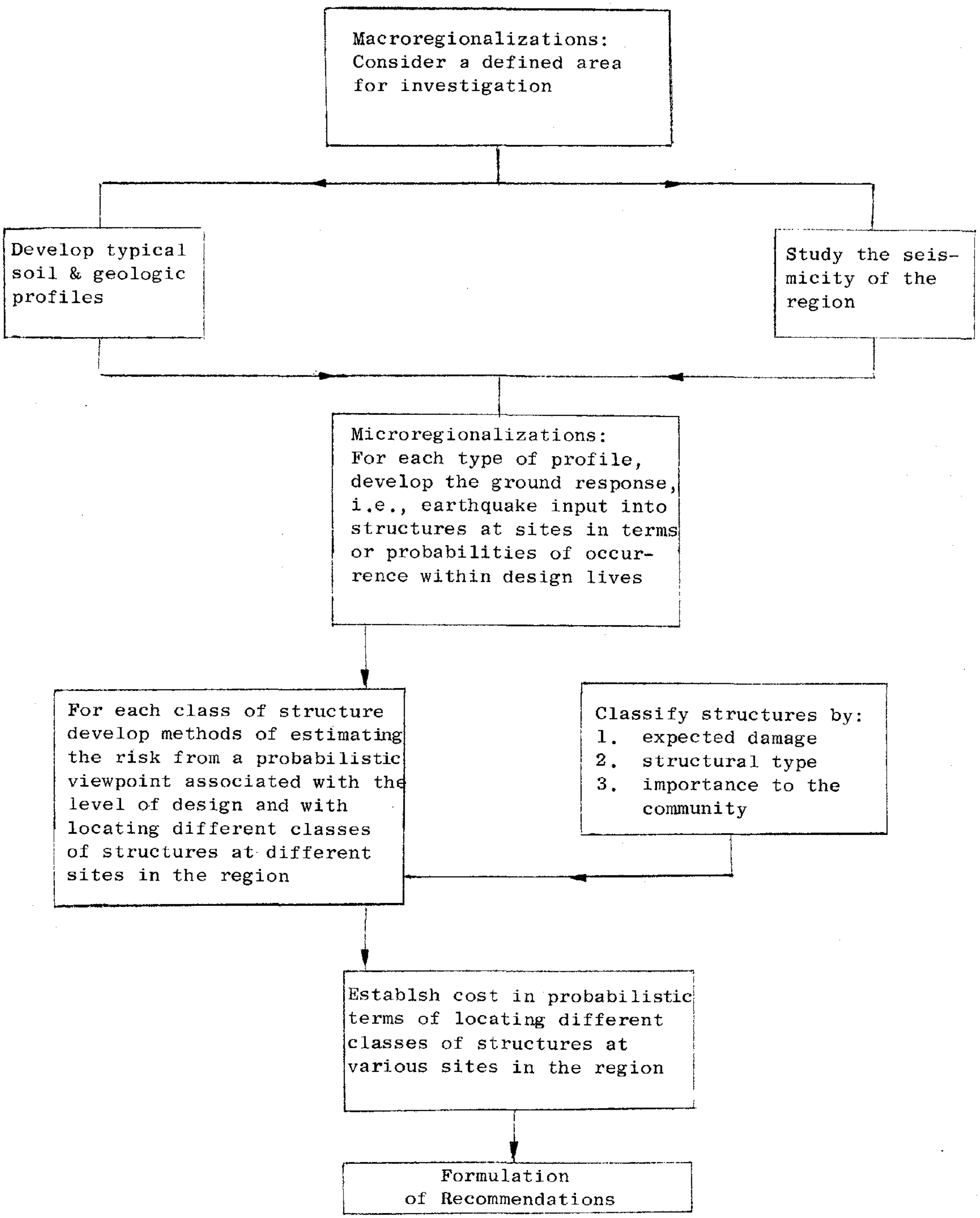

Figure 1.1 GENERAL PLANNING PURPOSES

level. Furthermore, the importance of the structure to the community will influence the level of risk which is acceptable. Power utilities, for example, might be built with much less chance of damage or collapse during an earthquake than perhaps industrial buildings.

The risk would be established by considering the past damage of these different types of structures and the probability of damage. The meaning of the term risk relates only to property damage in this case. After the engineer has investigated the geologic conditions, the effect of these geologic conditions on structures, and evaluated the consequences of their interaction, a decision must be made regarding the acceptability of the risk in a planned facility. This decision is of fundamental importance but very difficult to answer. The concept of private property is very deeply rooted in American society and no one will accept lightly being told that his proposed structure is a poor risk and cannot be built. However, the viewpoint taken here will be community planning by elected officials. It will be assumed that the risk analysis will be graciously accepted by property owners as beneficial to their welfare and the welfare of their fellow citizens.

1.2 Historical Background

The attempts to plan for the erection of structures in a seismically active region have been nearly non-existent. Heretofore, the main effort in earthquake engineering was to understand the phenomena of earthquakes and to quantify the effects of accompanying ground motion on structures. It is not until the phenomenology of earthquakes and a structures response to an earthquake is adequately understood that risk analyses can be undertaken. This explains the near absence in the literature of papers dealing with seismic risk analysis. Without a realistic risk analysis planning for the erection of structures in a seismic environment is nearly impossible.

In order to place the present dissertation in proper perspective, a review of the significant contributions to earthquake engineering is presented below. Most of the researchers commented upon have not contributed to risk analysis but their work is fundamental to understanding the role of risk analysis in earthquake engineering.

Richter [1,2] contributed a great deal to the understanding of the phenomena of earthquakes. It was his investigation which showed that earthquakes generally occur in particular regions of the world. They are random events only in the areas in which they occur. Furthermore, the introduction of the concept of earthquake magnitude--Richter Magnitude--helped to quantify the phenomena and offered a means of comparison at the origin of the earthquake. The Richter Magnitude is a measure of the energy released by an earthquake relative to a defined standard. It attempts to assign to each earthquake a single characteristic number based on measurement. It is a method depending on special charts and tables, and using recordings of instruments (seismographs) of standard type. The comparison of the amplitudes of recordings at a fixed distance from the earthquake's epicenter to a standard recording determines the magnitude of the earthquake. Generally, results from different stations are in good agreement.

It was recognized that the response of a structure to the elastic waves of an earthquake was primarily a problem in vibrations. Early attempts to solve this problem visualized the structure as being excited by an acceleration or displacement function applied at its base. The excitation was formed as a description of what was thought to be the earthquake elastic wave. Among the many notable investigators was Housner [4,12].

However, as additional knowledge was gained about the response of structures during earthquakes it came to be understood that a more realistic model of the problem could be made by considering the interaction between the base soil, foundation, and structure. The advent of digital computers made it possible to program the entire system and obtain a solution by iteration. Finite elements are used to model the soil, foundation, and structure. Using this method, the displacements, shears, and moments in the structure are calculated. In reference [25] a good description of this method is given and the contributions of chief investigators are discussed.

With the fundamental investigations into earthquake phenomenology and structural response to earthquakes, it became possible to design structures which when exposed to earthquakes suffer minor damage and whose probability of collapse is very low. Therefore, it is fitting that engineers began to think of the planning aspects of earthquake engineering. Among the many investigators only the chief contributors will be discussed here.

The probabilistic nature of planning for future earthquakes was recognized by Benjamin [14]. He proposed that a forecast be made of the probabilities of occurrence and the number of earthquakes of a given Modified Mercalli Intensity at the building site. The historical record was to form the basis for the calculations and Bayesian probability concepts were the basis for the statistical model. Damage statistics were decomposed into three categories--structural system, architectural system, and building contents. The probabilities of earthquake occurrence were multiplied by the estimated damage costs to obtain the expected damage costs at a site. The paper was not meant to be specific. Its purpose was to illustrate how probability concepts could be applied in planning.

Van Marcke and Diaz-Padilla [15] suggested a Markov Decision Model to estimate the risk involved in erecting a structure in a seismically active region. There were no calculations made with the model applied to a specific structure. An outline of the procedure for buildings with long design lives was presented. The earthquake occurrence model was Poisson. In this paper, it was proposed that a structure be modeled by a set of discrete states. Each state described the possible condition of a structure before and after an earthquake had occurred. A matrix of probabilities determined the transition of the structure from one state to another state. The structure was assumed to initially be in a given state. An expected cost associated with beginning in this initial state and later occupying other states was calculated. This expected cost reflected the level of seismic risk to which the structure was exposed.

Dalal [45] presented a seismic design methodology which incorporated phenomenology, structural behavior analysis and decision making. The emphasis was placed on probabilistic evaluation of seismic exposure. It was shown that structural response and potential damage and loss from seismic exposure could be incorporated in the design process.

Finally, it should be noted that Steinbrugge made many valuable contributions by compiling damage statistics for several significant earthquakes. Damage statistics are an indispensable part of a risk analysis. An example of his work is reference [19] which was used extensively in the preparation of this dissertation.

At the end of this dissertation is a list of references which were used in the preparation of this dissertation. They are also representative of the scope of past and current research in earthquake engineering.

1.3 Objective and Scope

The Markov Model will be used in this dissertation not only to model the structure but also to model earthquake occurrence. It will be shown that the first order Markov Chain has a property which is a reasonable fit to the mechanism thought to be behind shallow earthquake occurrences. The transition matrix for the structure will be based on a continuous-time concept. Furthermore, the risk will be estimated as a function of the time remaining in the life of the structure and a policy improvement routine will be included.

The objective is to estimate the cost of earthquake damage to a structure and the cost of a structure's collapse due to an earthquake during its design life. It will be shown that a statistics based decision model could be used to make these estimates. Furthermore, it is suggested that the method presented here could be incorporated into local building codes. Industries seeking to locate their plants in a seismically active region could balance the expected costs of earthquake damage to other benefits such as plentiful labor supply or proximity to shipping centers before actually making the decision to locate. The method may therefore be used in a general planning procedure.

However, there are further complications which are not considered here because they are in the domain of economic planning. If such a plan were put into effect now it might have tremendous effects on local communities. A community in a high risk area might find that industries may not want to locate there and that industries already there might want to relocate elsewhere. This could have adverse effects on a community's tax base. Furthermore, if it is shown that existing buildings are uneconomical because they represent a high risk and maybe a danger to the building's occupants should they be strengthened or torn down?

In order to carry out the stated objective and define risk as the cost of collapse or damage to a structure from an earthquake it is proposed that a Markov Decision Model be employed. This model can be used to quantify the uncertainty that exists in the phenomenological aspect of earthquake occurrence and in the quantitative effect of an earthquake on a structure. With this model, the available knowledge concerning earthquake occurrence and the past performance of structures exposed to earthquakes can be incorporated.

In Figure 1.2 a schematic is presented which illustrates the procedure that this dissertation will follow in reaching the stated objective. The region of interest is the San Francisco Bay Area. The seismicity of the region is discussed in terms of the past seismological record of the area. It is shown that the Markov Model is a reasonable model of the mechanism behind earthquake occurrence. After breaking down the earthquake record into four categories, for ease of adding data in the future, the Markov Model is used to calculate future probabilities of occurrence of earthquakes by category. These probabilities will be used later in decision making.

The area considered is then macroregionalized. A discussion of the soil conditions in the region is presented. Plots of the epicenters of past earthquakes are presented and it is shown how their location relates to the presence of faults in the region.

The fourth topic is microregionalization of the area. The discussion includes reference to how local soil conditions and geological features at a building site would affect the performance of a structure during an earthquake. The interaction between the waves of an earthquake and the soil conditions at a site in contributing to structural damage is a complex problem and cannot be treated exhaustively in this dissertation. Damage costs presented herein will be average values. Consequently, fluctuations in the damage costs due to soil conditions and other pertinent parameters will be assumed to take place about these values. The costs of damage are based on empirically obtained data.

The Markov Decision Model is introduced. This model permits quantification of the risk in earthquake design. The risk is calculated as an expected cost. The expected cost is dependent on the time remaining in the life of the structure.

The decision analysis will be applied to three cases. An improvement for the foundation for mobile homes will be discussed. It will be shown that damage to mobile homes during earthquakes could be substantially reduced if resistance to lateral motion was provided in the foundations of mobile homes. The second problem considers the risk associated with construction of modern high-rise buildings in a seismic environment. The risk values are calculated as a function of the number of stories in the building and as a function of the time remaining in the design life of the building.

Finally, a third problem, often discussed by structural engineers, will be treated. What benefit in terms of reduced damage levels or in prevention of structural collapse will be received if additional strength is provided to a structure in a seismically active region? This problem of cost-benefit tradeoff will be discussed in relation to light industrial buildings.

In the conclusion recommendations are made for continuing research in risk analysis. The problems still to be solved are enumerated and their significance to a risk analysis is discussed.

This dissertation is intended as a starting point for the introduction of risk analysis as a required undertaking in the design of structures in seismically active regions. It presents a statistical method to accomplish this undertaking. At the present time, the Markov Model appears to have definite advantages over other procedures.

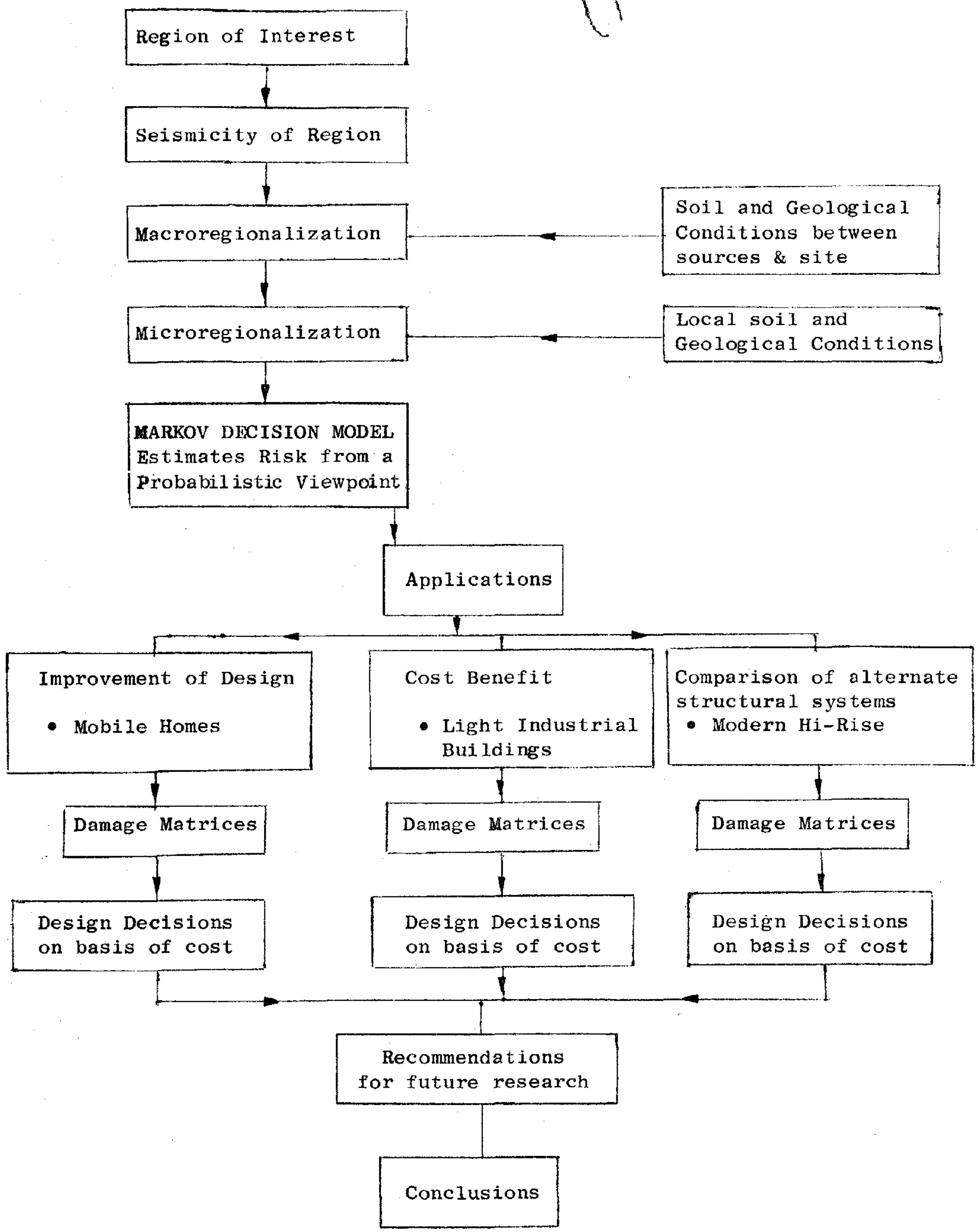

Figure 1.2 RISK ANALYSIS

II. CALCULATION OF EARTHQUAKE OCCURRENCE PROBABILITIES

2.1 Region of Interest

There are two major areas of population and industrial concentration in California: the San Francisco Bay Area and the Los Angeles Basin. Because of the difference in seismicity between the two regions, it is necessary that each region be investigated separately.

Historical evidence suggests that earthquakes have taken place regularly in California for at least the past 200 years [38]. It can be inferred that they have taken place for a much longer period; however, conclusive evidence is not available. In fact, there is really little known about earthquakes which occurred in California more than 70 years ago.

This dissertation will consider the risk associated with construction of a facility in the presence of an earthquake hazard only in the San Francisco Bay Area. This region contains three major faults. In the westernmost portion lies the San Andreas Fault. Parallel to the San Andreas Fault and located some 15 miles to the east is the Hayward Fault. Further to the east lies the third major fault--the Calaveras Fault. The largest recorded earthquake in this region is the San Francisco earthquake of April 18, 1906. Slippage along the San Andreas Fault probably caused this earthquake. It has been estimated that if this earthquake were measured by the Richter scale it would measure approximately 8.3. This is near the maximum value recorded for an earthquake.

Damage from the 1906 earthquake was considerable. However, the ensuing fire is known to have caused the major portion of the total damage. Estimates of damage were calculated to be in the millions of dollars. If an earthquake of this magnitude were to occur today, the greater concentration of property would probably mean an even larger amount of damage.

Damage in the 1906 earthquake extended from Eureka in the northern portion of the State of California to Salinas in the south. To the east, damage occurred in areas about halfway across the state.

The historical record of earthquake activity in the state was used as the foundation for the calculation of the probabilities of earthquake occurrence. An area large enough to include those earthquakes which would have an influence on the Bay Area had to be considered. Thus, it was decided to determine the boundaries of the area by the extent of the 1906 earthquake. It is assumed that if an earthquake of this magnitude occurred along the boundary of the 1906 earthquake its effects would be felt in the San Francisco Bay Area.

Thus, the region under consideration corresponds roughly to the area outlined in Figure 2.1. It is an area which might be described as the "Greater San Francisco Bay Area." In terms of map coordinates the area extends from a latitude of 36°-39°, while the longitude is from 120°-124°. The area consists of five counties and encloses segments of three major faults--the San Andreas, the Calaveras, and Hayward Faults.

The magnitude of the damage sustained at a site depends on at least the following eight factors.

1. The magnitude of the earthquake.
2. The distance of the site from the epicenter of the earthquake. This is the point immediately above the point of origin of the earthquake on the earth's surface.

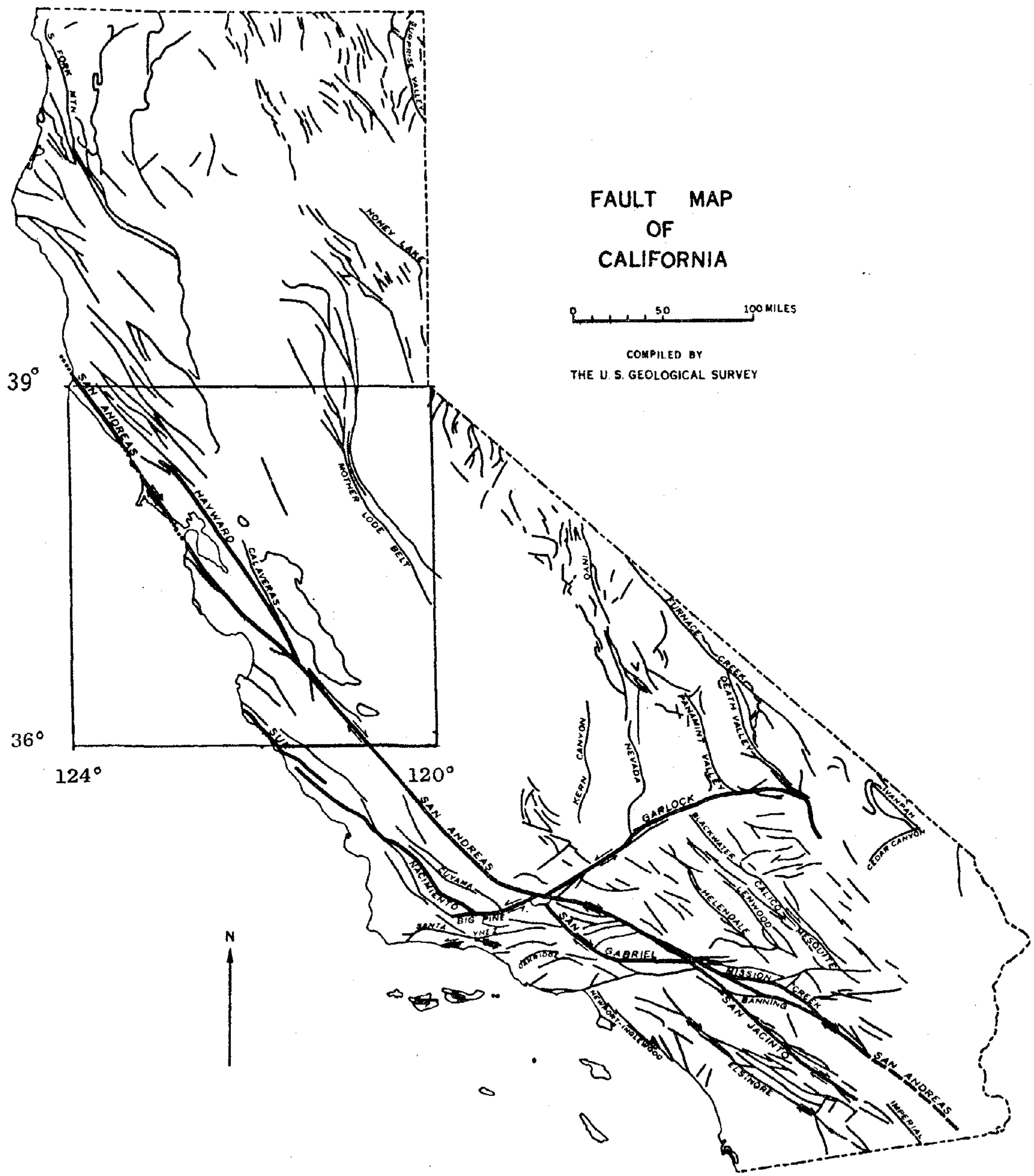
FAULT MAP
OF
CALIFORNIA
0 50 100 MILES
COMPILED BY
THE U. S. GEOLOGICAL SURVEY
39°
36°
124°
120°
N
S. FORK MTN
SURPRISE VALLEY
HONEY LAKE
SAN ANDREAS
HAYWARD
CALAVERAS
MOTHER LODE BELT
SUR
NACIMIENTO
CUYAMA
BIG PINE
SANTA YNEZ
KERN CANYON
NEVADA
FURNACE CREEK
DEATH VALLEY
PANAMINT VALLEY
GARLOCK
BLACKWATER
CALICO
LENWOOD
HELENDALE
MESQUITE
IVANPAH
CEDAR CANYON
SAN GABRIEL
MISSION CREEK
BANNING
SAN JACINTO
NEWPORT-INGLEWOOD
ELSINORE
IMPERIAL

FIGURE 2-1

3. Orientation of the earthquake locality relative to the fault line.
4. Depth of the earthquake.
5. Duration of the earthquake.
6. Geology of the intervening area between the considered site and the point of origin of the earthquake.
7. Local ground conditions at the site.
8. Extent of faulting.

To understand the seismicity of a region, each of the factors listed above must be investigated and their quantitative influence on damage estimates must be evaluated. This investigation must be as accurate as the empirical evidence will allow.

2.2 Earthquakes

A great deal of progress has been made in recent years in the understanding of the motions which take place between the large plates which constitute the earth's crust. For some time, it has been known that the earth's crust is composed of large contiguous plates--those plates forming a land mass were denoted as continental plates and those under the sea as oceanic plates. It has been shown that these plates are in motion relative to one another. Where it is possible, the relative displacement rates and the amount of displacement which has occurred in past years has been measured. It has been postulated that this velocity field imposed by the motion of the continental plates and ocean plates is the cause of tectonic earthquakes.

The relative motion between plates is sometimes exposed as a fault line. Along the fault line relative motion can easily be measured. It is always a shearing-type motion. But the shear motion can be in the vertical as well as the horizontal direction. If the motion along the fault line is permitted, the potential shear stresses are relieved and consequently there is no accumulation of strain representing elastic energy. Hence, the motion takes place uniformly and with no obstructions. But if the motion is prevented by locking along the fault line then large shearing stresses are generated along the fault and bending stresses at a distance away from the fault, indicating an accumulation of elastic strain energy. Eventually, the stressed material of the crust reaches a point where the large stresses can no longer be sustained. The crust fractures, releasing the accumulated energy in the form of elastic waves with accompanying vibrations. Some of the vibrations are of a low frequency and consequently they are in the audible range. This description is basically a description of the mechanism behind earthquake occurrence. It is generally known as the Elastic Rebound Theory. In summary, the Elastic Rebound Theory [2] describes the stress buildup in the earth's crust and its eventual release as elastic wave energy.

Since a fault line is only the trace of the fault on the surface of the earth and therefore the fault may extend miles into the earth's interior, terms have originated which qualitatively locate the point of earthquake occurrence. Origin of an earthquake occurs at the hypocenter or focal point. In reality, however, the earthquake occurs in a volume of the earth's crust and not at a point. Where the hypocenter projects on the earth's surface is the epicenter. This latter term is extremely useful in defining the location of an earthquake.

Focal depths of earthquakes are generally grouped into three classifications. Table 2.1 gives the classifications and the respective focal depths.

TABLE 2.1

Classification	Depth
Shallow	Surface to 38 miles
Intermediate	38 miles to 188 miles
Deep	188 miles to 440 miles

Shallow earthquakes are thought to be due to a fracture of the brittle rock in the earth's crust near its surface or the overcoming of the frictional forces in the rock which are locking the fault. This mechanism is described accurately by the Elastic Rebound Theory. Deep earthquakes are not fully understood but it is believed that they are associated with violent motions of the rock deep in the earth's interior. The Elastic Rebound Theory is not applicable in this case. Rocks deep in the earth's interior behave plastically because of the high temperature and high pressure they are exposed to. The mechanism behind intermediate earthquakes lies somewhere between the shallow and deep earthquake.

The waves associated with an earthquake are due to the motion of the shattering rocks along a fault as they oscillate back and forth before coming to rest. These waves are generated at the hypocenter and radiate in all directions away from this point. There are three categories of waves which are of interest.

1) The p or pressure wave travels with the largest velocity. It travels as a longitudinal wave, or sound. In the direction of its advance it produces a series of rarefactions and condensations. Its

velocity is given by Equation (2-1).

$$V_p = \frac{1}{s} \sqrt{\frac{G}{\varphi}} \quad . \tag{2-1}$$

The constants appearing in Equation (2-1) are defined as follows.

G = shear modulus of the soil

φ = mass density of the soil

s = a constant equal to $(1-2\nu)/2(1-\nu)$.

where

ν = Poisson's Ratio

2) The second type of wave is called the s or shear wave. The motion of the soil particles in the path of this wave is perpendicular to the direction of wave propagation. Furthermore, its velocity, V_s, given by Equation (2-2), is less than the velocity of the p-wave.

$$V_s = \sqrt{\frac{G}{\varphi}} \quad . \tag{2-2}$$

3) Along the surface of the earth two additional types of wave are known to occur. The Love wave is a surface wave which is distinguished by the absence of a vertical component of motion. It can be considered as a horizontally polarized shear wave in the upper layers of the earth's crust. It is propagated by multiple total reflections at the earth's surface. A second surface wave, called a Rayleigh wave, is a frequency dependent wave with a velocity near that of the shear wave. The velocity (V_R) of the Rayleigh wave is given by Equation (2-3).

$$V_R = (0.9)V_s \quad . \tag{2-3}$$

It is convenient to calculate the level of earthquake damage as a function of epicentral distance. The wave system generated by the earthquake has a salient feature by which it is possible to locate the epicenter. At any point away from the epicenter the first wave to arrive is the p-wave. It is followed by the s-wave and then the Rayleigh wave. The arrival of the various waves can be recorded and distinguished with instruments. The location of the epicenter is determined by recording the difference in time between the first arrival of the p-wave and s-wave. In effect, this recording permits calculation of the point along the fault where rupture begins. For earthquakes of small magnitude, the length of fault rupture is small and consequently the point at which the rupture begins gives a good approximation to the location of the epicenter. For large earthquakes, the length of fault rupture is large, as much as 80 miles. The energy of the earthquake might not be released equally along the fault. Most of the energy release could be concentrated at one point along the fault and this point could be far removed from the origin of the fault rupture. In this instance, the calculation of the epicenter of the earthquake would not be a good indication of its origin in the sense that it would not indicate where the maximum amount of energy was released.

It is clear from the evidence which has been gathered about the physical mechanism behind earthquake generation that earthquakes occur in the outer layers of the earth's crust. Deep earthquakes are still considered to occur in the earth's crust. The molten core of the earth, by reason of its fluidity, cannot sustain the shear and bending stresses necessary for earthquake generation. Consequently, there is no accumulation of strain energy and therefore no earthquakes can occur. Along the fault line on the earth's surface and deep into the fault zone there are different materials with varying strengths. The strength of the rock material puts a bound on the amount of strain energy which can be accumulated along the fault. This factor together with the length of fault available for rupture influence the magnitude of the earthquake.

Because of different rock strengths, the level of stress which can be sustained along the fault will vary considerably. Slip can occur along one segment of the fault releasing the energy stored there, but causing a stress buildup in other segments of the fault where motion is precluded. Small earthquakes could occur therefore where the material of the fault is weak but their occurrence would not imply that the straining of the earth's crust is diminishing. On the contrary, their occurrence may be straining other areas of the fault where the material is strongest. This could lead to an accumulation of a large amount of strain energy and its eventual release as an earthquake of large magnitude.

The points which have been discussed are very important to the understanding of the mechanism behind earthquakes. However, they are not the salient features which will provide a means of modeling earthquake occurrence by a probabilistic model. The following item provides the foundation for the probabilistic model. It is implied that the larger the time between earthquakes of a given magnitude--the interarrival time--the larger the earthquake. The larger interrarrival times afford greater time to accumulate strain energy along the fault. Short interarrival times imply that the strain energy is being accumulated and released rapidly.

The amount of energy released by an earthquake relative to a standard is measured by the Richter Magnitude. Richter Magnitude is the common logarithm of the ratio of amplitude of the earthquake measured on a specified apparatus to that of a standard trace amplitude.

The extent of damage will vary with site conditions for a given earthquake. Hence, a second scale--called an intensity scale--is needed. This scale measures an earthquake qualitatively in terms of human response. The Modified Mercalli is in common use for this purpose.

2.3 Historical Record

The major source of information to calculate the probabilities of earthquake occurrence in the region of interest must be the past historical data. Too little is known of the exact mechanism of earthquake occurrence, although the Elastic Rebound Theory offers a good description on a macro scale. Unfortunately, the available historical record is very short. It was not until the year 1933 that a fairly complete record of earthquake activity in the State of California was kept. It must be kept in mind that California was nearly a frontier state as late as 1900. Data on earthquake activity can be accumulated from first-hand accounts before this year (1933) but it is often inaccurate except for large and moderate earthquakes. Consequently, a great deal of caution should be exercised in evaluation and use of the historical record prior to 1933.

It is very likely that the short record available is indicative of the future occurrence of earthquakes. At least, for a short time into the future its use is warranted. Nature does not make rapid changes.

The Seismological Station at the University of California, Berkeley, has compiled an accurate record of earthquake occurrence beginning in the year 1933. Prior to this year, the staff of the Seismological Station has gathered what information is available in publications and journals concerning the earlier earthquake activity in the State of California. This information together with theinformation gathered since 1933 has been put on a computer tape.* It is the source of information for the tables of earthquake activity which follow.

The computer tape was run on the IBM 360-67 computer at Stanford University. All earthquakes having a Richter Magnitude assigned to them and lying within the area bounded by 36°-39° latitude and 120°-124° longitude were taken off the tape. The data were analyzed and it was found that the two large earthquakes having occurred in 1906 and 1911 respectively were reasonably well-documented. Consequently, these two entries were kept. There are two entries from the year 1926. This was before accurate records were kept. But here again they seem to be well-documented and consequently they are also included. Thus, the data from this tape constitute the historical record.

From past experience of the damage to structures caused by earthquakes, it is generally believed that earthquakes with a Richter Magnitude below about 3.5 cause little or no damage. Even to structures near the epicenter of an earthquake with a Richter Magnitude of 3.5 or smaller very little damage will result. This lower bound is not precise and perhaps extraordinary conditions might lead to even considerable damage with earthquakes of this low a magnitude. However, the lower bound of the historical record will be set at 3.5.

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* Our appreciation is extended to Dr. B. A. Bolt of the Seismological Station for making this tape available to the Department of Civil Engineering at Stanford University.

Furthermore, it will be shown that the available record of earthquakes below 3.5 on the Richter Scale would not influence the calculation of probabilities because if their total energy was released in a single shock, it would not influence any of the transition matrices.

In order to use the proposed model for earthquake occurrence, it is necessary to sort the historical record by Richter Magnitude. There are two reasons for doing this. First, it is necessary that the formulation can incorporate future information in a systematic way. If the earthquakes are grouped, future sorting and grouping for probabilistic calculations can be simplified. Secondly, examination of all the past records indicate that there is a very definite demarcation between different magnitude earthquakes.

The following earthquake designation is used for the grouping of the past earthquake data.

TABLE 2.2

Richter Magnitude	Designation
6.5 and greater	large
5.5 - 6.4	moderate
4.5 - 5.4	small
3.5 - 4.4	very small

To give some idea of the relative amounts of energy released, Table 2.3 is reproduced. The amount of energy released does not always indicate the magnitude of the resulting damage. If the energy of the earthquake is released over a large area rather than at a point then the effect of the earthquake is diminished. Thus, it appears that the resulting stress drop along the fault during the earthquake has a great deal to do with the resulting ground motion.

TABLE 2.3

Richter Magnitude	Energy Released (ergs)
3.0 - 3.9	9.5×10^{15} - 4.0×10^{17}
4.0 - 4.9	6.0×10^{17} - 8.8×10^{18}
5.0 - 5.9	9.5×10^{18} - 4.0×10^{20}
6.0 - 6.9	6.0×10^{20} - 8.8×10^{21}
7.0 - 7.9	9.5×10^{22} - 4.0×10^{23}
8.0 - 8.9	6.0×10^{23} - 8.8×10^{24}

Listings 1,2,3, and 4 of Appendix A give the past data on earthquake occurrences. Figures 2.2 through 2.6 show graphically the number of occurrences based on these lists.

For the region under study, no attempt is made to correlate the event with the causative fault. However, it is noted that some faults are more active than others. If probabilities of future occurrences for a given site are based on causative faults, their lumping the total region for data analysis may not give accurate results.

2.4 Macroregionalization

In this section, the entire area denoted as the San Francisco Bay Area will be described with respect to its salient geological and seismic features.

There are three major fault zones in this region--the San Andreas, Hayward, and Calaveras Faults. Figure 2.1 gives the relative location of these faults. All three faults are active. Generally, in this area and for that matter in the entire State of California earthquakes are classified as being shallow (see Section 2.2). For example, along the San Andreas Fault the focal depths are usually less than 15 miles.

In the area under study and in the entire State of California, strain energy appears to be entering the ground at a very rapid rate [20]. The historical record in Section 2.3 indicates that generally this energy is being released rapidly by small or very small earthquakes. The rapid release of energy precludes the large accumulation of strain energy necessary for a large shock. The strain energy accumulation occurs to a depth of about 13 miles. In fact, the major portion of strain energy accumulation is probably within 10 miles of the surface. Below about 13 miles, the strength of the rocks is such that slip along the fault occurs by a creep-type process. This action precludes strain energy accumulation. However, these processes are not uniform. There are regions above 10 miles where slipping occurs but this is an exception.

Much has been said and written about the effect of soil conditions on the damage suffered by structures during earthquakes. This is certainly an important factor. However, in this dissertation only the average conditions will be reflected in the damage costs presented. The region as a whole will be discussed and the probable effect of soil conditions will be indicated, but the effect of different soils will not be considered explicitly.

The predominance of soft alluviums in the San Francisco Bay Area makes the area more hazardous than areas where the soils are "firm." Thus, the damage to be expected in this area will be much larger. This is especially true of the areas along the shore of San Francisco Bay.

In this respect, the San Francisco Bay Area is more hazardous than the Los Angeles Basin. C. F. Richter [3] gives the following table to describe the seismicity for the Los Angeles Basin and its vicinity (Table 2.4).

TABLE 2.4

Geological Character in the Region	Probable Maximum MM Intensity
Granite	VI
Alluvium	IX

The situation is expected to be much worse in the Bay Area.

At a site, where no construction has taken place, it is known that a soft soil will amplify the shear amplitude of the earthquake wave [8]. In fact, there is evidence that the amplitude of displacement increases linearly with the thickness of the alluvium [9]. In 1906 structures located on bedrock suffered little or no damage while those on bay fill were heavily damaged. It appears that local geology has its greatest effect on the amplitude of the transverse portion of the earthquake wave [7,8].

2.5 Microzonation

Microzonation considers the conditions at the building site. Thus, it considers the geologic and seismic properties where the building is to be constructed.

The map in Figure 2.1 presents the area under investigation and the fault zones lying in this area. This map indicates that not every building or facility within this region will be exposed to the same seismic risk. Therefore, the need to microzone the area in order to give the conditions at a site is evident. Proper microzonation would give the type of response to be expected of a given structure at a given site.

Microzonation of a given region is an involved and complex undertaking. Because of its nature microzonation is considered to be out of the scope of this dissertation. Damage costs presented here will be functions of the Richter Magnitude of the earthquake and the distance from the earthquake's epicenter. The damage costs will be average values, but it is possible to adjust them for conditions other than average.

Consequently, for the purpose of this dissertation the likely points of earthquake origin will be determined. This determination is based on the historical record of earthquake occurrence. In Figures 2.8, 2.9, 2.10, 2.11 and 2.12 the recorded historical record for earthquake occurrences is plotted. Table 2.5 defines the symbols representing the category to which the earthquake belongs.

TABLE 2.5

Symbol	Earthquake Category
	Large
	Moderate
	Small
	Very Small

It can be seen from the figures that most of the earthquakes occur near the San Andreas and Hayward faults. Therefore, for our purposes, microzonation of the region might be to calculate the distance from the nearest fault and use this value in the figures to be presented later to obtain the cost of damage after an earthquake has occurred.*

Although the costs of damage from earthquakes are given as a function of Richter Magnitude and distance from the earthquake epicenter, there is much more to this problem. The following discussion will consider this problem further.

The vibration of the structure to the excitation imposed by the earthquake waves does not depend solely on its structural properties. The allurium near a building site has its own natural period of vibration. Therefore, when considering the response of a structure to an earthquake the soil beneath the structure and structure's foundation must be considered as integral parts of the structure. The interaction between the

*My appreciation is extended to Mr. William Buckland and Mr. Charles Kircher, graduate students at Stanford University, for the preparation of these drawings.

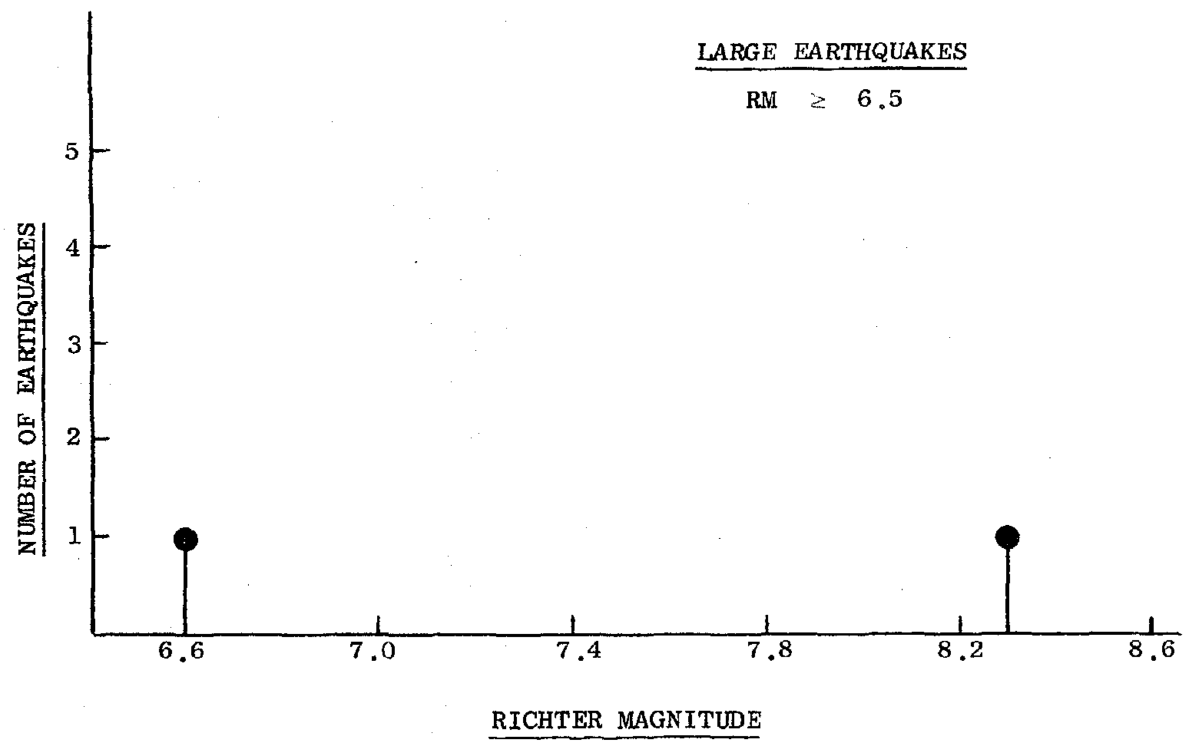
structure, its foundation and the underlying soil constitute the vibration system. Reference [25] contains a description of this problem.

Furthermore, where the frequency of vibration of the ground might constitute an additional hazard because of a resonate condition in the structure the distance from the epicenter is still more important. The intervening soil between building site and earthquake epicenter can act as a filter, thus filtering out certain wavelengths of the excitation wave. Far from the earthquake epicenter the long periods of vibration predominate. Generally, a soil where the long periods of vibration predominate and where the long periods have large amplitudes of vibration, one will have what are classified as poor soils. The long period excitation wave are generally closer to the natural periods of structures. A resonance between soil and structure is therefore possible. The shear wave of the earthquake excitation will increase its amplitude of vibration as it passes through a soft soil. The amplification can be increased by as much as a factor of five [8]. The softer the soil the greater the amplification. Thus, it can be seen that the soil conditions at the building site and in the vicinity of the building may work to increase the amount of damage.

Generally, the worst soil conditions are soft alluvial soils. Unfortunately, this type of soil predominates in the San Francisco Bay Area. Since the expected intensity of ground motion varies principally with the soil conditions at the site, this type of soil contributes greatly to the earthquake hazard by increased levels of damage.

Soil conditions constitute additional problems which must be evaluated at the particular building site. The probability of a landslide must be evaluated if the site is in the vicinity of slopes where a landslide is possible. Another behavior of a soil which can increase the level of damage is liquefaction. Under the oscillating stresses of the earthquake waves the soil, if the water table is high, has a tendency to lose its strength because of the high pore water pressures that may develop. The chief indicators of a propensity to liquefaction are a high water table and granular soils.

The past record of damage to structures by type can furnish estimates to future damages during earthquakes. Unfortunately, extensive damage records are not available to investigate the effect of soil conditions quantitatively. Generally, damage statistics are given with only a cursory reference to the soil conditions in the vicinity of the building. To be accurate, soil conditions at the proposed construction site should be compared to the soil conditions where the damage occurred. If the soil conditions are significantly different, the damage statistics must be modified to reflect this fact. The cost figures presented here will reflect "average" soil conditions.



LARGE EARTHQUAKES
RM ≥ 6.5
NUMBER OF EARTHQUAKES
5
4
3
2
1
6.6
7.0
7.4
7.8
8.2
8.6
RICHTER MAGNITUDE

FIGURE 2-2

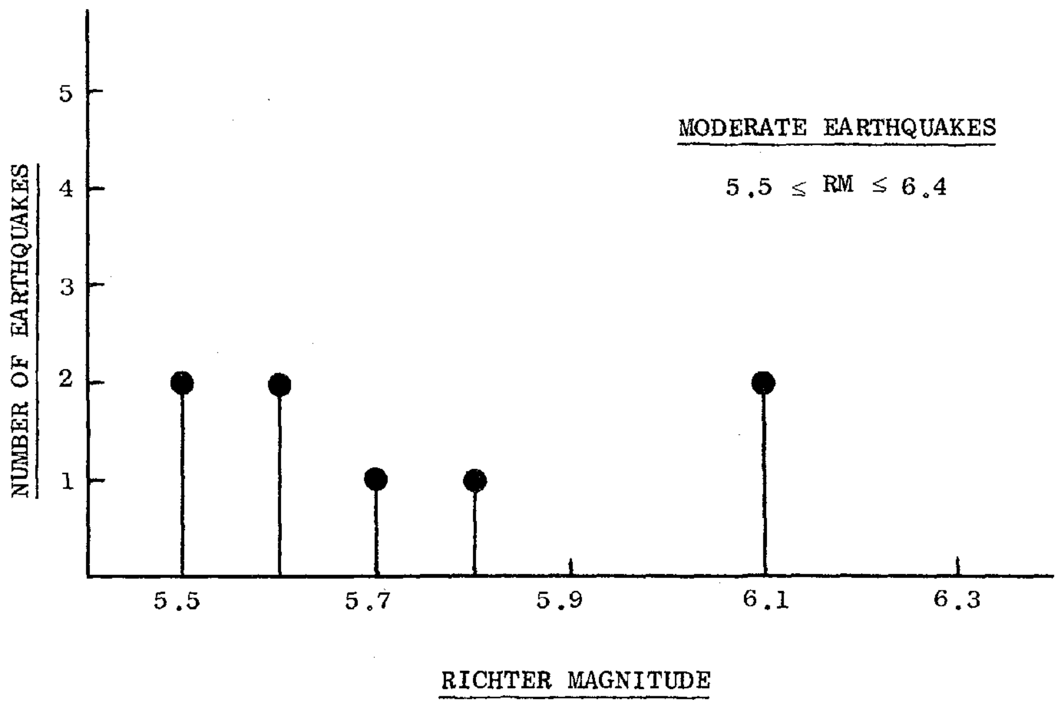

MODERATE EARTHQUAKES
5.5 ≤ RM ≤ 6.4
NUMBER OF EARTHQUAKES
1
2
3
4
5
5.5
5.7
5.9
6.1
6.3
RICHTER MAGNITUDE

FIGURE 2-3

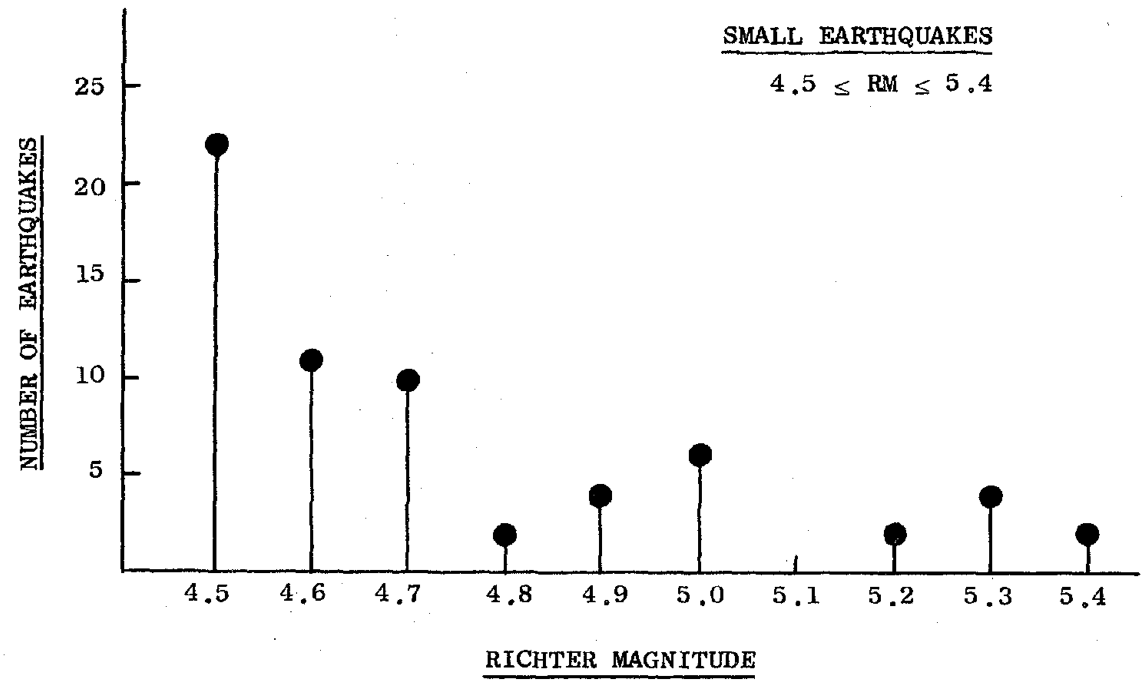

SMALL EARTHQUAKES
4.5 ≤ RM ≤ 5.4
NUMBER OF EARTHQUAKES
25
20
15
10
5
4.5
4.6
4.7
4.8
4.9
5.0
5.1
5.2
5.3
5.4
RICHTER MAGNITUDE

FIGURE 2.4

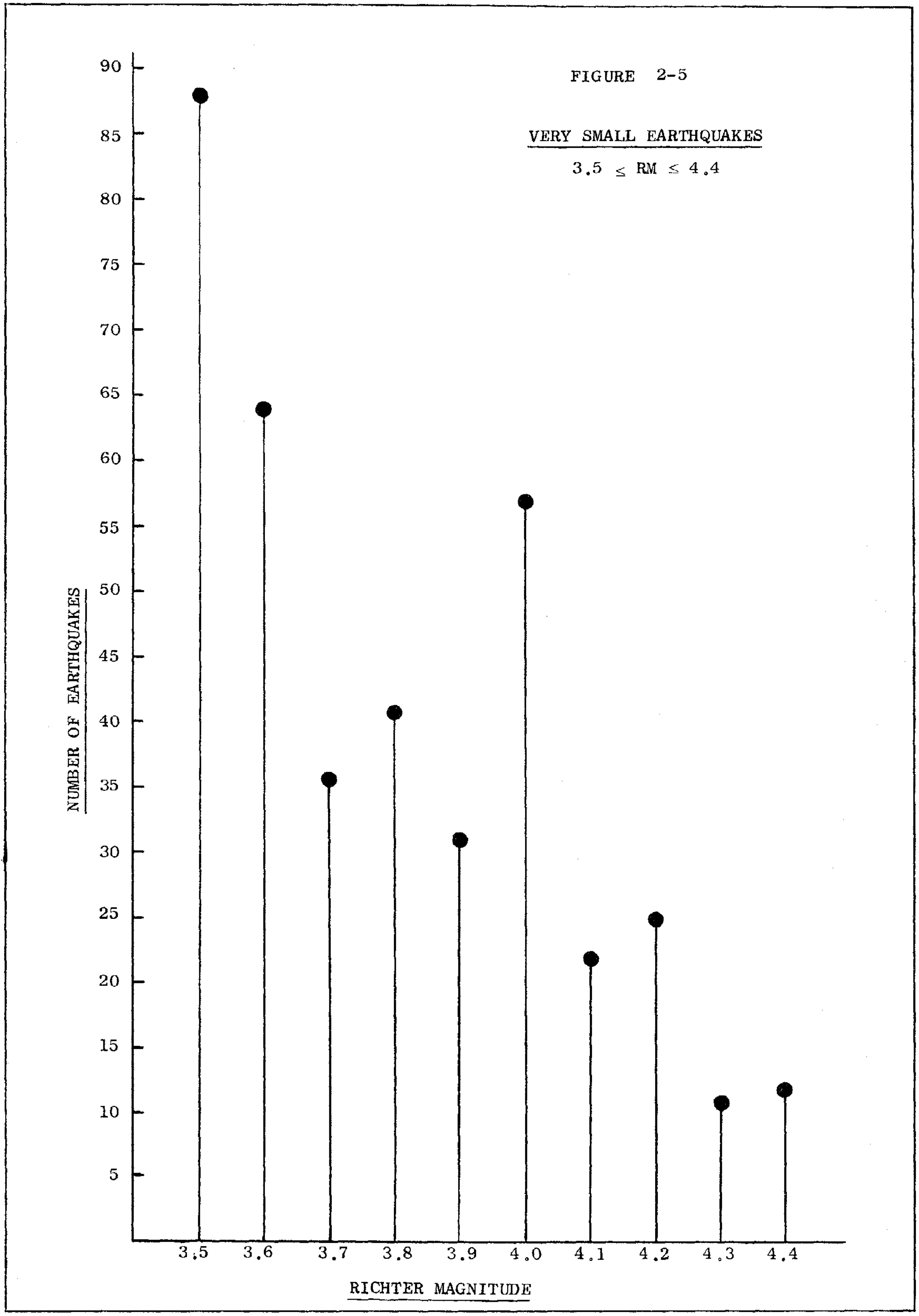


FIGURE 2-5
VERY SMALL EARTHQUAKES
3.5 ≤ RM ≤ 4.4
NUMBER OF EARTHQUAKES
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
3.5
3.6
3.7
3.8
3.9
4.0
4.1
4.2
4.3
4.4
RICHTER MAGNITUDE

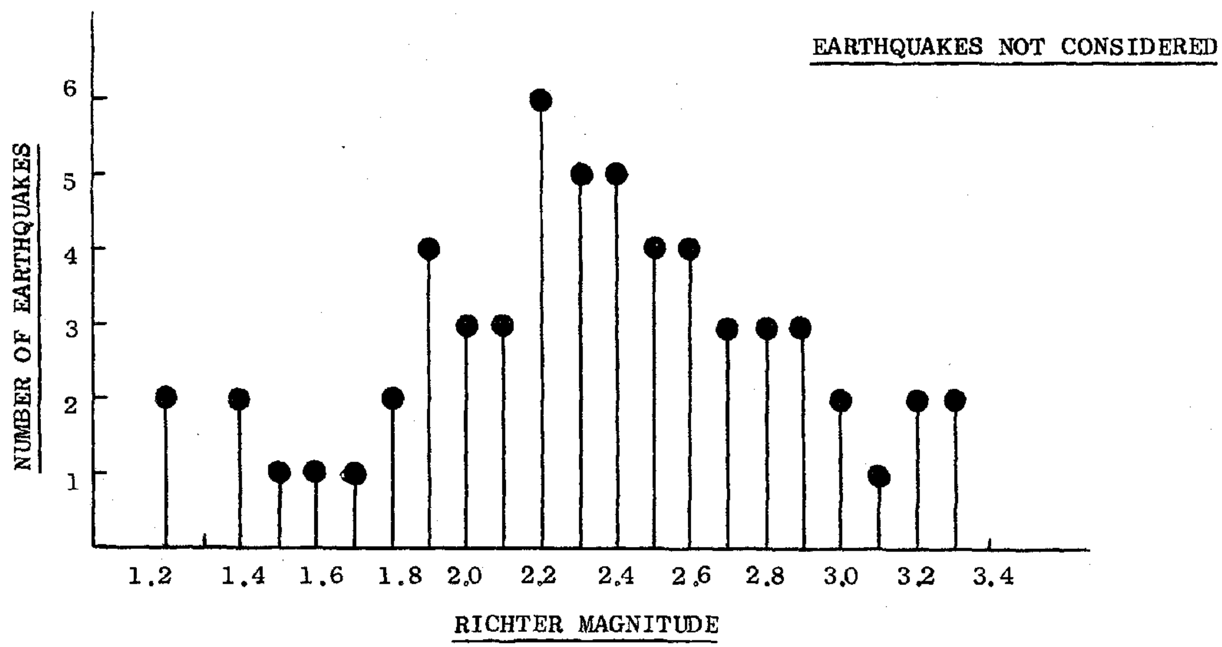

EARTHQUAKES NOT CONSIDERED
NUMBER OF EARTHQUAKES
1
2
3
4
5
6
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
RICHTER MAGNITUDE

FIGURE 2-6

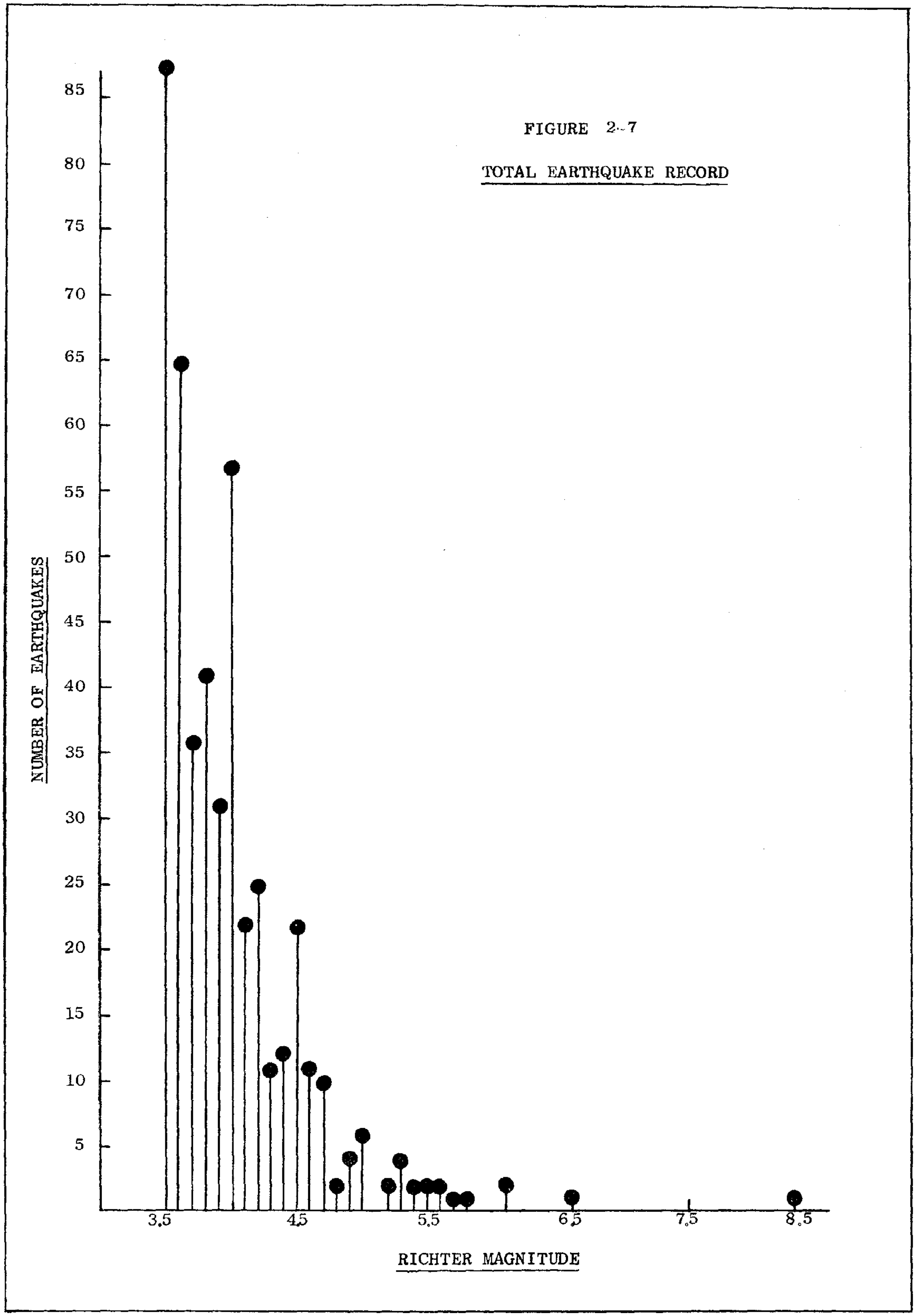
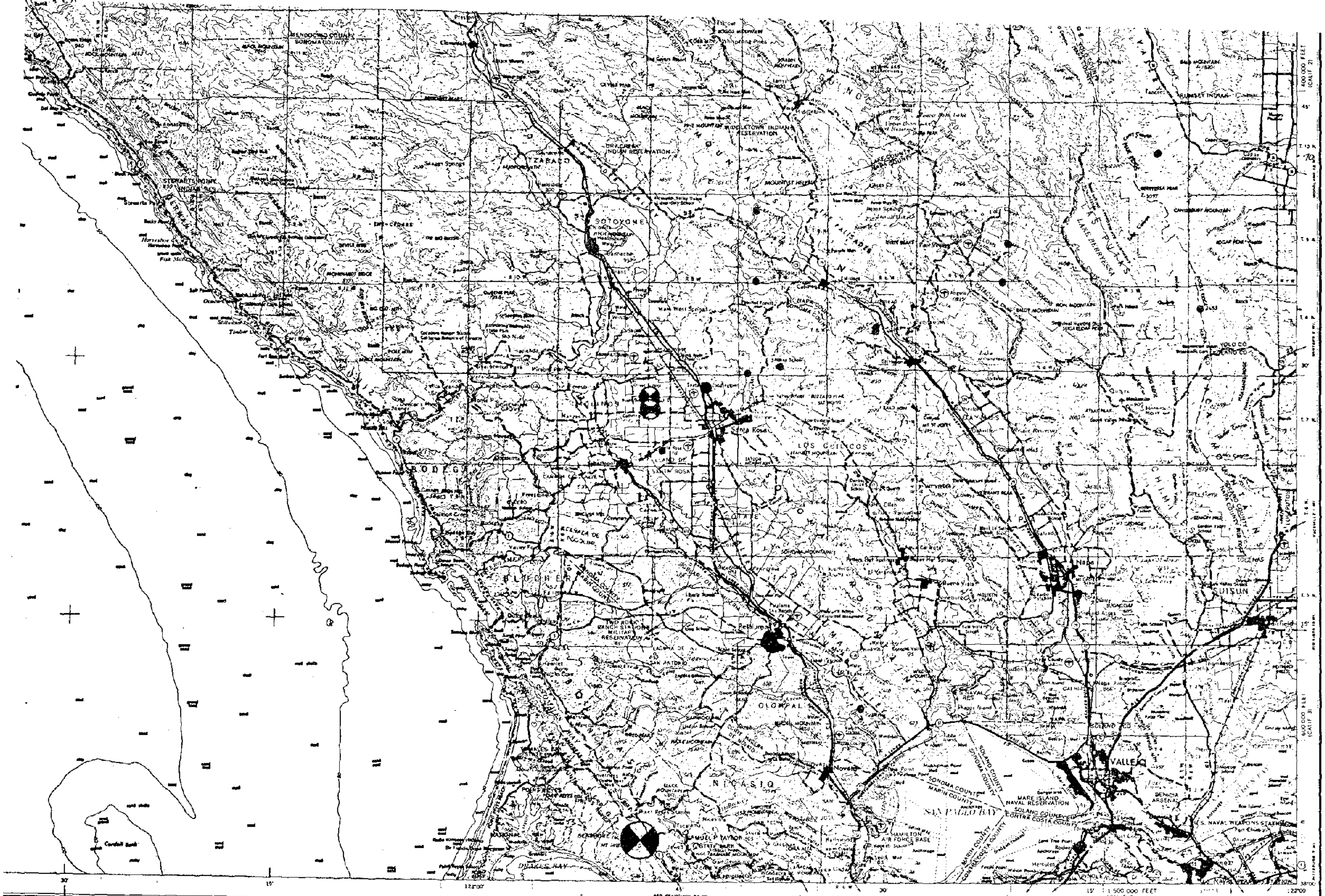


FIGURE 2-7
TOTAL EARTHQUAKE RECORD
NUMBER OF EARTHQUAKES
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
3.5
4.5
5.5
6.5
7.5
8.5
RICHTER MAGNITUDE



Scale 1:250,000

CONTOUR INTERVAL 200 FEET
WITH SUPPLEMENTARY CONTOURS AT 100 FOOT INTERVALS
TRANSVERSE MERCATOR PROJECTION

FOR SALE BY U.S. GEOLOGICAL SURVEY, DENVER, COLORADO 80225, OR WASHINGTON, D.C. 20242

LOCATION DIAGRAM

RELIABILITY DIAGRAM

INTERIOR GEOLOGICAL SURVEY, WASHINGTON, D.C.—1958

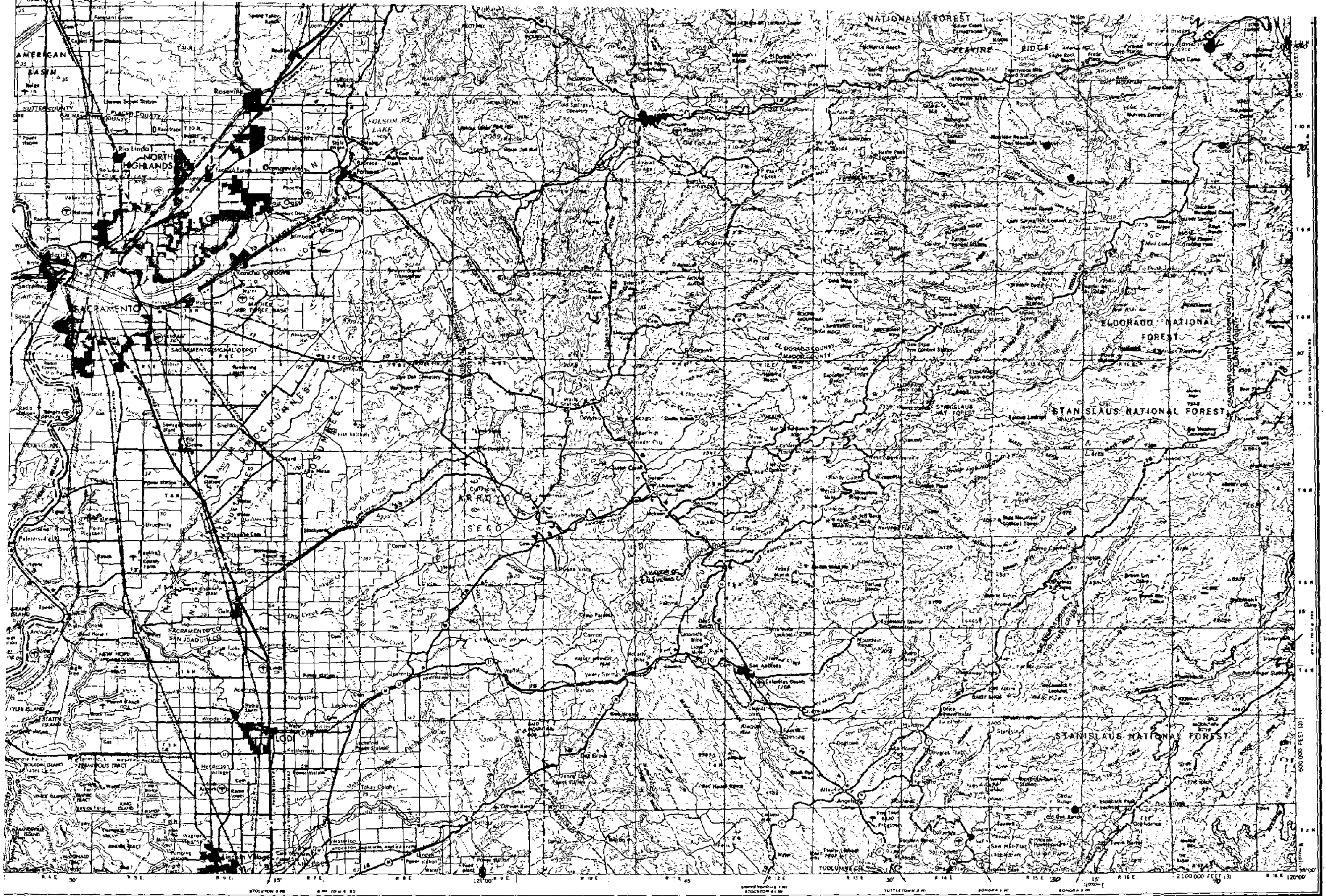
TOWNSHIP OR RANGE LINE
LAND GRANT BOUNDARY

SECTIONIZED TOWNSHIP

6	5	4	3	2	1
7	8	9	10	11	12
18	17	16	15	14	13
19	20	21	22	23	24
30	29	28	27	26	25
31	32	33	34	35	36

40

SANTA ROSA, CALIFORNIA
1958
LIMITED REVISION 1967



LEGEND

Scale 1:250,000

CONTOUR INTERVAL 200 FEET
WITH SUPPLEMENTARY CONTOURS AT 100 FOOT INTERVALS
TRANSVERSE MERCATOR PROJECTION

FOR SALE BY U.S. GEOLOGICAL SURVEY, DENVER, COLORADO 80225, OR WASHINGTON, D.C. 20242

LOCATION DIAGRAM

RELIABILITY DIAGRAM

TOWNSHIP OR RANGE LINE
LAND GRANT BOUNDARY

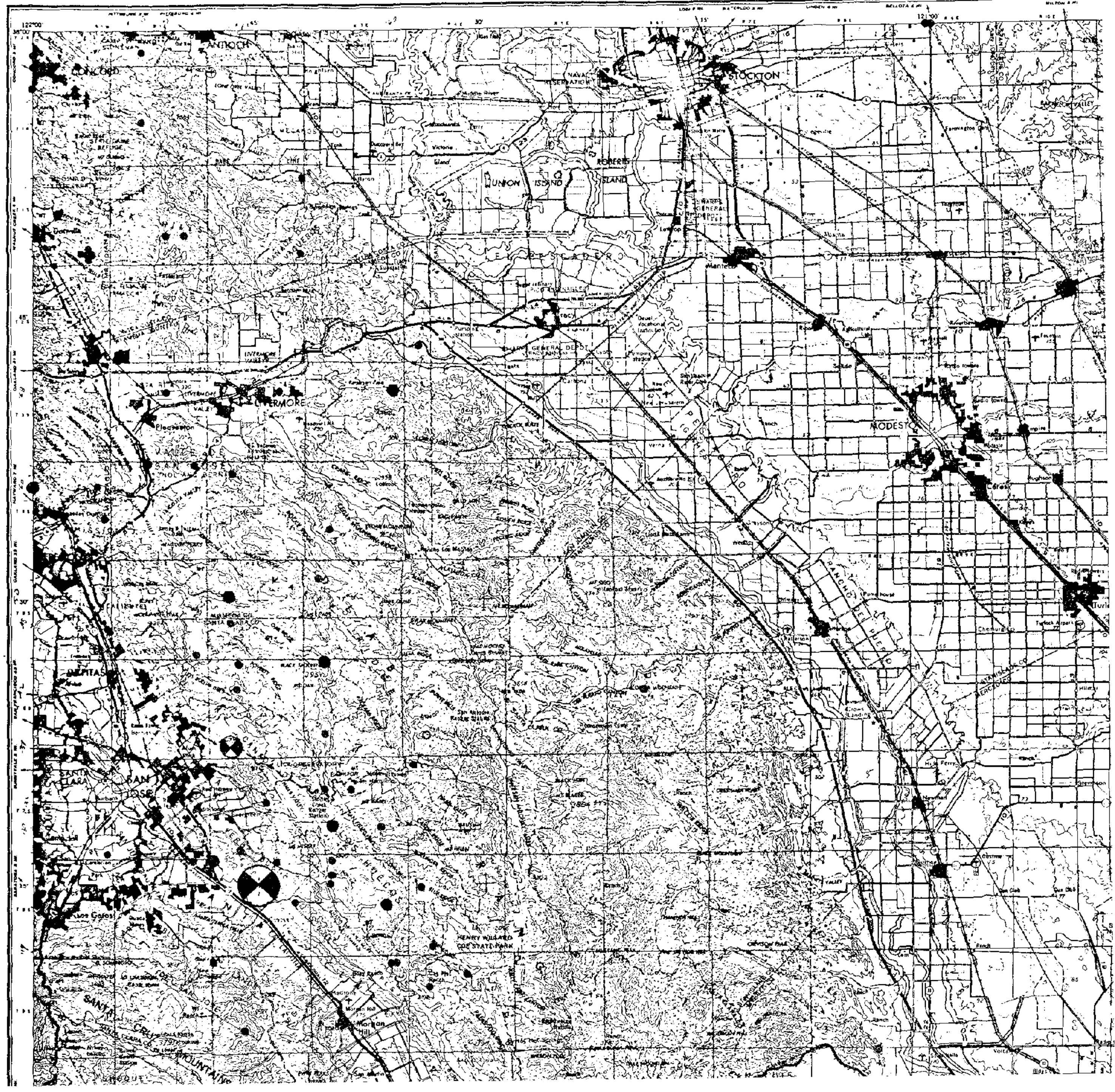
SECTIONIZED TOWNSHIP

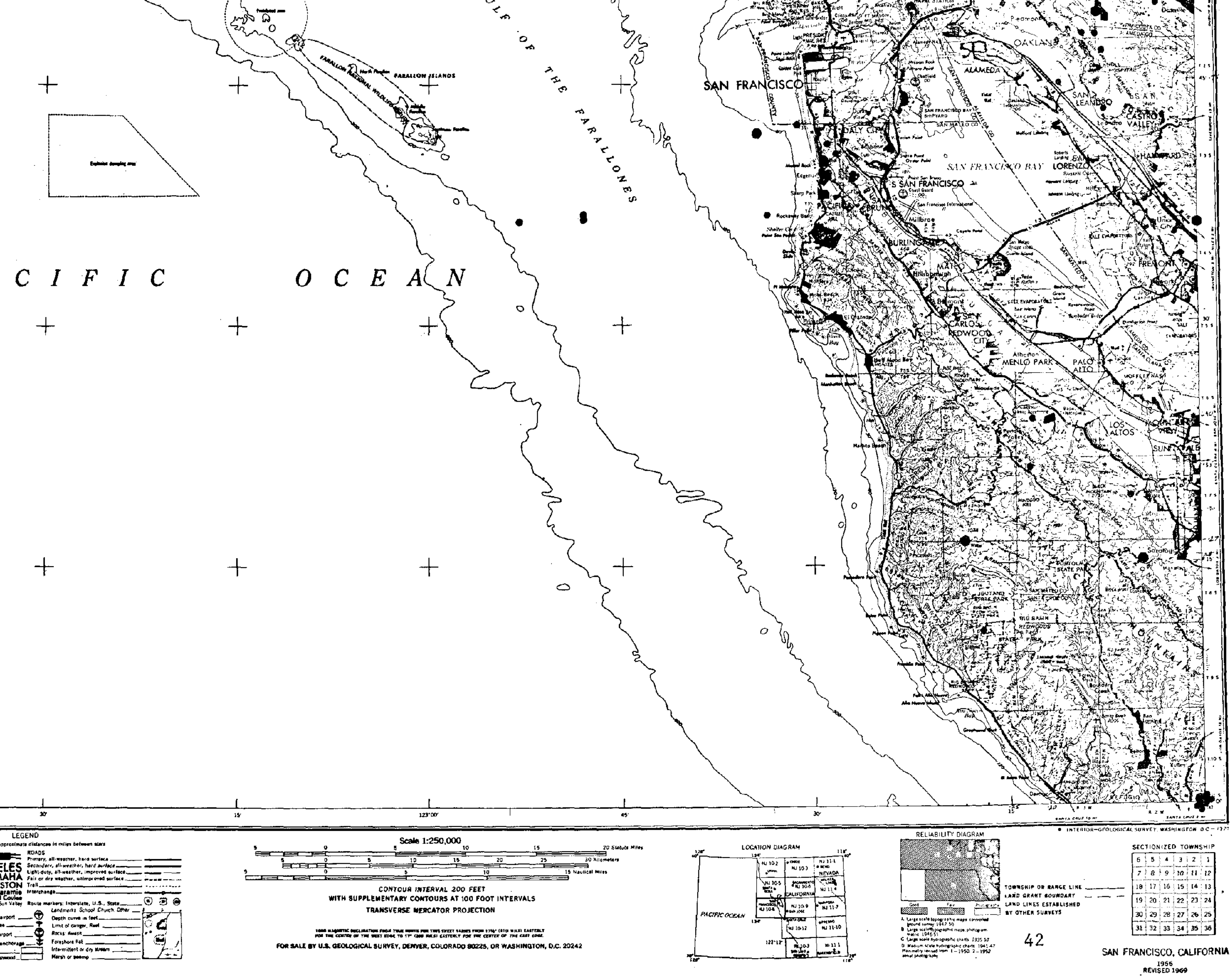
SACRAMENTO, CALIFORNIA
1957

WESTERN UNITED STATES, 1:250,000

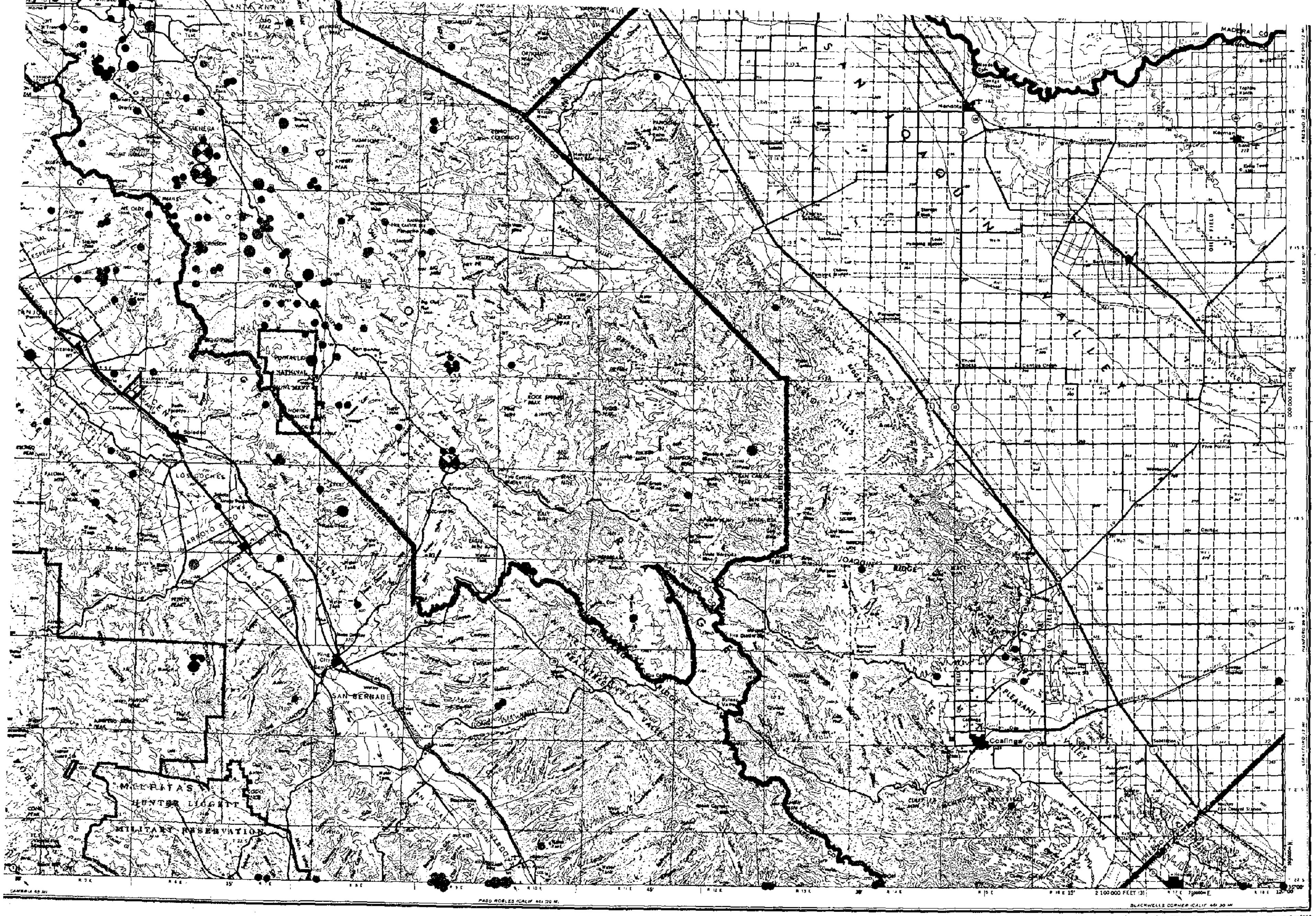
Figure 2.11

SAN JOSE





FARALLON ISLANDS
FARALLON NATIONAL WILDLIFE REFUGE
North Farallon
Explosives dumping area
GULF OF THE FARALLONES
PACIFIC OCEAN
SAN FRANCISCO
OAKLAND
ALAMEDA
SAN LEANDRO
CASTRO VALLEY
HAYWARD
SAN LORENZO
DALY CITY
SAN FRANCISCO BAY
S SAN FRANCISCO
PACIFICA
Millbrae
BURLINGAME
SAN MATEO
Hillsborough
Belmont
SAN CARLOS
REDWOOD CITY
Atherton
MENLO PARK
PALO ALTO
FREMONT
LOS ALTOS
MOUNTAIN VIEW
SUNNYVALE
Saratoga
BIG BASIN REDWOODS STATE PARK
PORTOLA STATE PARK
REFUGIO
30'
15'
123°00'
45'
30'
15'
R 3 W
R 2 W
INTERIOR—GEOLOGICAL SURVEY, WASHINGTON, D.C.—1977
LEGEND
Approximate distances in miles between stars
ROADS
Primary, all-weather, hard surface
Secondary, all-weather, hard surface
Light-duty, all-weather, improved surface
Fair or dry weather, unimproved surface
Trail
Interchange
Route markers: Interstate, U.S., State
Landmarks: School Church Other
Depth curve in feet
Limit of danger, Reef
Rocks: Awash
Foreshore flat
Intermittent or dry stream
Marsh or swamp
Scale 1:250,000
20 Statute Miles
30 Kilometers
15 Nautical Miles
CONTOUR INTERVAL 200 FEET
WITH SUPPLEMENTARY CONTOURS AT 100 FOOT INTERVALS
TRANSVERSE MERCATOR PROJECTION
FOR SALE BY U.S. GEOLOGICAL SURVEY, DENVER, COLORADO 80225, OR WASHINGTON, D.C. 20242
LOCATION DIAGRAM
PACIFIC OCEAN
NEVADA
CALIFORNIA
NJ 10-2
NJ 10-3
NJ 11-1
NJ 10-5
NJ 10-6
NJ 11-4
NJ 10-8
NJ 10-9
NJ 11-7
NJ 10-12
NJ 11-10
NJ 10-3
NJ 11-1
RELIABILITY DIAGRAM
Good
Fair
Photographic
A. Large scale topographic maps controlled ground survey 1947-50
B. Large scale topographic maps, photogrammetric 1946-51
C. Large scale hydrographic charts 1935-53
D. Medium scale hydrographic charts 1941-47
Planimetry revised from 1—1950, 2—1952 aerial photography
TOWNSHIP OR RANGE LINE
LAND GRANT BOUNDARY
LAND LINES ESTABLISHED BY OTHER SURVEYS
SECTIONIZED TOWNSHIP
42
SAN FRANCISCO, CALIFORNIA
1956
REVISED 1969



SCALE 1:250,000

5 0 5 10 15 20 STATUTE MILES

5 0 5 10 15 20 25 30 KILOMETERS

CONTOUR INTERVAL 200 FEET
DOTTED LINES REPRESENT 100-FOOT CONTOURS
DATUM IS MEAN SEA LEVEL
DEPTH CURVES IN FEET—DATUM IS MEAN LOWER LOW WATER
SHORELINE SHOWN REPRESENTS THE APPROXIMATE LINE OF MEAN HIGH WATER

1966 MAGNETIC DECLINATION FOR THIS SHEET VARIES FROM 16°45' EASTERLY FOR THE CENTER OF THE WEST EDGE TO 16°30' EASTERLY FOR THE CENTER OF THE EAST EDGE. MEAN ANNUAL CHANGE IS 0°02' WESTERLY

THIS MAP COMPLIES WITH NATIONAL MAP ACCURACY STANDARDS

FOR SALE BY U. S. GEOLOGICAL SURVEY, DENVER, COLORADO 80225, OR WASHINGTON, D. C. 20242

A FOLDER DESCRIBING TOPOGRAPHIC MAPS AND SYMBOLS IS AVAILABLE ON REQUEST

INDEX TO ADJOINING SHEETS

SAN FRANCISCO NJ 10-8	SAN JOSE NJ 10-9	MARIPOSA NJ 11-7
PACIFIC OCEAN	SANTA CRUZ NJ 10-12	FRESNO NJ 11-10
	SAN LUIS OBISPO NI 10-3	BAKERSFIELD NI 11-2

44

ROAD CLASSIFICATION

Heavy-duty — Light-duty
Medium-duty — Unimproved dirt
Trail
Road interchange
Interstate Route U.S. Route State Route

SANTA CRUZ, CALIF.
N3600—W12000/60X132
1965

2.6 First Order Markov Process

In the description of the Elastic Rebound Theory it was pointed out that for an earthquake to occur strain energy had to be accumulated in the material along the fault. If strain energy was present then the possibility of an earthquake in the near future was nearly certain. If an earthquake had occurred recently then it was assumed that the accumulated strain energy had been released and thus another would not occur until the strain energy could again accumulate in the region. Thus, what will occur in the near future depends on the state of the region now.

It is proposed here that a first order Markov Model be used to model earthquake occurrence because its mathematical properties are analogous to the Elastic Rebound Theory. A discrete parameter stochastic process described by the function $X(t)$ for the values of $t=0,1,2,3...$ is said to be a first order Markov Chain if the conditional probability of $X(t)$ depends only on $X(t-1)$, the previous value. Analytically, the condition is written in Equation (2-4).

$$p[X(t)/X(1),X(2)...X(t-1]\ =\ p[X(t)/X(t-1)] \quad . \qquad (2\text{-}4)$$

Generally, a process governed by such an equation is said to have a "one-step memory."

A discrete Markov Chain models a system by considering the system as a set of discrete states. The system is observed at regularly spaced intervals. The trajectory of the system during any unit time interval is described by a transition matrix $[P]$.

The transition matrix $[P]$ contains the probabilities of the system moving among the defined states. The initial state of the system is defined by a probability row vector $\pi(n)$. Premultiplication of the transition matrix $[P]$ by the initial state row vector gives the state of the system in terms of probabilities at the end of that interval. In matrix notation the governing equation is written in Equation (2-5).

$$\pi(n+1) = \pi(n) \cdot [P] \qquad \text{for } n=0,1,2,3... \tag{2-5}$$

For the k states of a system there is the requirement that the probabilities of the row vectors add to one. Mathematically, this condition is given in Equation (2-6).

$$\sum_{i=1}^{k} \pi_i(n) = 1 \quad . \tag{2-6}$$

Since by recursion

$$\begin{aligned} \pi(1) &= \pi(0)[P] \\ \pi(2) &= \pi(1) \cdot [P] = \pi(0) \cdot [P]^2 \\ &\vdots \end{aligned} \tag{2-7}$$

a general expression can be written in terms of the initial state probabilities.

Equation (2-8) is this general expression.

$$\pi(n) = \pi(0) \cdot [P]^n \qquad (2\text{-}8)$$

for n=0,1,2,3...

A salient observation is that the transition matrix $[P^n]$ represents the probabilities of moving among the various states in n steps. This matrix contains the probabilities which define the trajectory of the system from some state i at the present time to a state j in the future in n steps.

2.6.1 Two State Markov Chain

Consider a two state Markov Chain and its application to modeling earthquake occurrences. Since there are four defined levels of earthquake magnitude (refer to Section 2.3) a transition matrix [P] must be constructed separately for each.

Since there are only two states for this model, the system must be in either one state or the other at any given instant in time. Generally, the states are denoted success and failure, respectively. The definition of the terms success and failure is left to the individual doing the modeling. For example, success could be defined as the occurrence of an earthquake in a given time period while failure might be defined as no occurrence in the same time interval. Figure 2.13 describes the transition between the two states graphically.

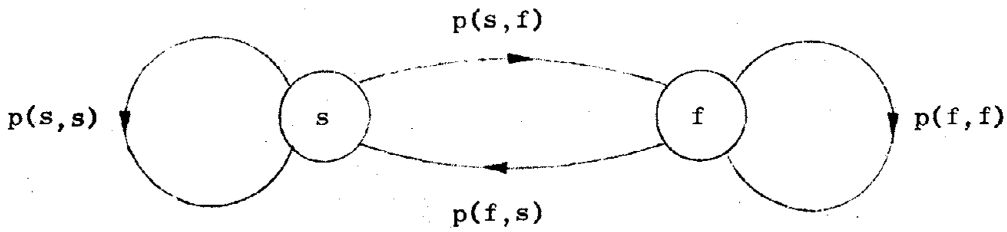


FIGURE 2.13

The symbol s denotes the first state which is the success state and the symbol f the failure state. The quantity p(s,s) represents the probability that a success is followed by a success in one step. For example, this implies that an earthquake is followed by a second earthquake in one step. The probability that no earthquake is followed by an earthquake is designated p(f,s). Similar descriptions can be made for the quantities p(s,f) and p(f,f).

Thus, the one step transition matrix can be defined by Equation (2-9).

$$[P] = \begin{bmatrix} p(s,s) & p(s,f) \\ p(f,s) & p(f,f) \end{bmatrix} . \qquad (2\text{-}9)$$

Using Equation (2-5) and (2-9) and knowing the initial state row vector $\pi(0)$ it is possible to calculate the probability of future earthquake occurrences. One is not limited to a one step forecast. Use of Equation (2-8) permits a forecast into the future of as many steps as are desired.

The n^{th} step transition matrix

$$[P]^n = \begin{bmatrix} p(s,s) & p(s,f) \\ p(f,s) & p(f,f) \end{bmatrix}^n \qquad n=0,1,2,3\ldots \tag{2-10}$$

can be obtained in closed form [16]. With the stipulation that

$$[p(s,s) + p(f,f) - 1] < 1 \tag{2-11}$$

the closed form of $[P]^n$ can be written

$$[P]^n = \frac{1}{2-p(s,s)-p(f,f)} \begin{bmatrix} 1 - p(f,f) & 1 - p(s,s) \\ 1 - p(f,f) & 1 - p(s,s) \end{bmatrix}$$

$$+ \frac{[p(s,s)+p(f,f)-1]^n}{2-p(s,s)-p(f,f)} \begin{bmatrix} 1 - p(s,s) & -(1-p(s,s)) \\ -(1-p(f,f)) & 1-p(f,f) \end{bmatrix} . \tag{2-12}$$

This transition matrix contains conditional probabilities dependent on the initial state of the system. The transition probabilities describe the probability of transition to a state on the n^{th} step conditional on the initial state of the system.

2.6.2 Calculation of the Two State Markov Transition Matrix from the Historical Earthquake Record

In Appendix A is presented the available historical record of earthquake occurrence in the Greater San Francisco Bay Area. This record is shown after the entries have been sorted according to Richter Magnitude into the four categories discussed in Section 2.3. For each of the four categories the two state transition matrix [P] given in Equation (2-13) will be calculated. With this calculation, a forecast of the probability of future earthquake occurrence can be made.

$$[P] = \begin{bmatrix} p(s,s) & p(s,f) \\ p(f,s) & p(f,f) \end{bmatrix} . \qquad (2\text{-}13)$$

Each earthquake occurrence releases a given amount of energy measured by its Richter Magnitude. Thus, the historical matrix [P] is a measure of the rate at which strain energy is being released in a given region. In addition, it is a measure of the rate at which strain energy is accumulated provided that large amounts of strain energy are not stored for a long period of time.

The procedure used to calculate the Markov transition matrix [P] will be illustrated here. Only one calculation will be presented--the calculation for the category of large earthquakes--but the transition matrices for the other categories will be given.

The available historical record for large earthquakes is reproduced from Appendix A in Table 2.6. Two entries constitute the entire record.

TABLE 2.6

Year	Month	Day	Hour	Min	Sec	Longitude	Latitude	Richter Magnitude
1906	4	18	13	12	0	122.80	38.05	8.3
1911	7	1	22	0	0	121.75	37.25	6.6

The first entry is the well-known San Francisco Earthquake. It took place along the San Andreas Fault. A second entry took place along the Calaveras Fault. It is a much smaller earthquake than the San Francisco Earthquake.

It is recognized that the small amount of data and the short period of recorded time complicates considerably the calculation of the transition matrix. However, large earthquakes occur infrequently and the available record might be representative of the rate at which earthquakes of this magnitude occur.

The interval of time used in the calculaton is one year. This choice is arbitrary and immaterial except where multiple occurrences exist within the time interval. In such cases, a loss of information will occur. For example, if the definition of the term s is the occurrence of one earthquake in one year and the historical record contains instances where two earthquakes occurred in one year it is probably better to reduce the time interval so that all of the data will be used. Thus, the transition matrix will represent the rate at which strain energy is released.

For the category of large earthquakes, the following definitions of the terms s and f are satisfactory.

s(success) = the occurrence of one earthquake having a Richter Magnitude within the designated range of large earthquakes in a time interval of one year.

f(failure) = no occurrence in the time interval.

Examination of the data in Table 2.6 shows that there were two years when the definition of success was met--1906 and 1911. In both cases, the earthquake was not followed in the succeeding year by an earthquake with Richter Magnitude in the category of large earthquakes.

Thus, the probability of a large earthquake followed by a large earthquake, which is indicated symbolically by $p(s,s)$, is zero. Each row of quantities in the matrix $[P]$ must add to one. This requirement is written mathematically in Equation (2-14).

$$\begin{aligned} p(s,s) + p(s,f) &= 1 \\ p(f,s) + p(f,f) &= 1 \end{aligned} \quad . \qquad (2\text{-}14)$$

From the first of these relations, the quantity $p(s,f)$ can be calculated since $p(s,s)$ is known. The probability $p(s,f)$ represents an earthquake followed by no earthquake.

$$p(s,f) = 1 - p(s,s) = 1 \quad . \qquad (2\text{-}15)$$

With the result of Equation (2-15) the upper row of the Markov transition matrix $[P]$ has been calculated.

The second row of the matrix [P] can be calculated by considering the two instances where an earthquake was preceded by no earthquake. The year pairs 1905-1906 and 1910-1911 form this portion of the calculation. Thus, there are two occurrences. To complete this calculation a count must be made of the number of times no earthquake in a year interval was followed by no earthquake in the succeeding year. The record was begun in 1905 and ended in the year 1971. Consequently, the total length of record is 66 years and in 64 of these years no earthquake occurred and no earthquake occurred in the following year. Equation (2-16) presents the calculation of the quantities p(f,s) and p(f,f), respectively.

$$p(f,s) = \frac{2}{66} = .0303$$

$$p(f,f) = 1-p(s,s) = .9697 \qquad (2\text{-}16)$$

The historical transition matrix [P] can now be constructed for the category of large earthquakes and in Equation (2-17) the matrix is illustrated.

LARGE EARTHQUAKES

$$\left[P_L\right] = \begin{bmatrix} 0.0000 & 1.0000 \\ 0.0303 & 0.9697 \end{bmatrix} . \qquad (2\text{-}17)$$

A salient feature of the one-step transition matrix of Equation (2-17) is the presence of a trapping state at p(s,f).

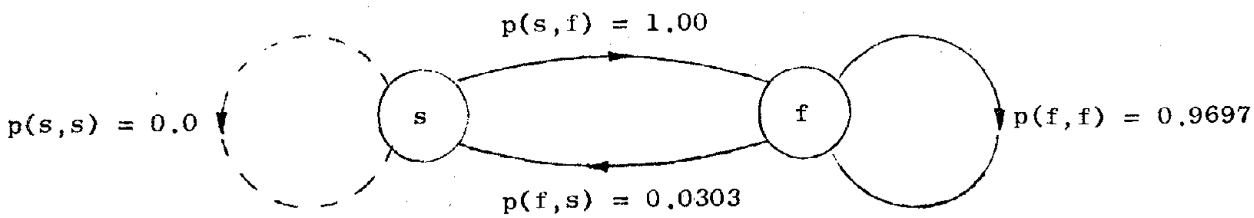


FIGURE 2-14

When the success state is entered the system does not remain there. It moves immediately to the failure state. This feature is illustrated in Figure 2-14.

An identical procedure is followed for the calculation of the transition matrices for the remaining three categories of earthquakes. Tables II, III, and IV from Appendix A provide the historical data for the calculation. The calculated transition matrices are displayed for each category. A commentary is made on the conspicuous features of each matrix.

Table II in Appendix A contains the historical record for moderate earthquakes. The table consists of eight entries. In 1926, 1961, and 1969 there were two occurrences which happened within hours of one another. Consequently, there is no possibility of using a smaller time interval so that all of the occurrences will be counted. The previous definition for the letters s and f is retained. Equation (2-18) is the Markov transition matrix for moderate earthquakes.

$$\left[P_M\right] = \begin{bmatrix} 0.0000 & 1.0000 \\ & \\ 0.0758 & 0.9242 \end{bmatrix} . \qquad (2\text{-}18)$$

Examination of Table III of Appendix A, which is the historical record for small earthquakes, reveals a nearly complete record of occurrences between the years 1933 and 1970. Before 1933 there was no record kept of earthquakes with this range of values of Richter Magnitude. Indeed, prior to 1933 the region of interest was sparsely settled and the probability of an earthquake with a Richter Magnitude in this range causing significant damage was small; consequently, no record was kept of these earthquakes. However, there is good reason to believe that earthquakes in this category occurred at the same rate before 1933 as after that year. Natural processes do not change abruptly but evolve slowly in time.

The total time period used for the transition matrix in Equation (2-19) is 39 years. In some years, there have been multiple occurrences of earthquakes in this category. Relative to large and moderate earthquakes the energy release of an earthquake in this category is small. Consequently, in order to compare the probabilities of earthquake occurrence between the four categories the same definitions for the symbols s and f have been retained.

$$\left[P_s\right] = \begin{bmatrix} 0.8000 & 0.2000 \\ & \\ 0.7500 & 0.2500 \end{bmatrix} . \qquad (2\text{-}19)$$

Finally, from Table IV of Appendix A, which is the historical record for very small earthquakes, the Markov transition matrix can be calculated. The total length of record considered is 40 years. Again, the definitions of the symbols s and f remain the same.

$$\left[P_{v.s.}\right] = \begin{bmatrix} 0.9744 & 0.0256 \\ \\ 1.0000 & 0.0000 \end{bmatrix} . \qquad (2\text{-}20)$$

2.6.3 Transform Analysis of the Discrete Markov Transition Matrix

A very useful calculation is to decompose the transition matrix of Section 2.6.1 into a steady-state matrix and a transient matrix. For a two-state matrix which is ergodic there will be one steady-state matrix and one transient matrix. No information is lost in this calculation.

For ergodic probability matrices, the rows of the steady-state matrix will be equal and each row will sum to one. The transient matrix will have rows which sum to zero. The steady-state matrix will be independent of the step but the transient matrix will not. One can think of the transient matrix as being a perturbation superimposed on the steady-state matrix. After many steps the elements of the transient matrix will approach zero. Further discussion of this procedure is given in Reference [17].

The historical matrices presented in Section 2.6.1 can be decomposed into the steady-state and transient portions with the use of the z-transform [13]. Transformation of the historical matrix is made by taking the z-transform of each element of the matrix individually. The transformed portion is then separated into a portion which does not depend on the index of the step, and a portion which does. Inverse transformation of the two parts gives the steady-state matrix and transient matrix, respectively.

In general, the procedure for decomposition of the historical matrix is as follows. The discrete Markov Chain is defined by Equation (2-21).

$$\pi(n+1) = \pi(n)\,[P] \qquad n=0,1,2,3\ldots \tag{2-21}$$

It follows by recursion that a multi-step transition can be calculated from Equation (2-22).

$$\pi(n+1) = \pi(n)[P] \quad \text{or} \quad \pi(n) = \pi(o)[P]^n \qquad n=0,1,2,3\ldots \tag{2-22}$$

Transformation of Equation (2-22) will lead to decomposition of the transition matrix $[P]^n$ into its two component parts.

The z-transform $\pi(z)$ of a function is defined by Equation (2-23).

$$\pi(z) = \sum_{n=0}^{\infty} \pi(n) z^n \quad . \tag{2-23}$$

For the analysis of the historical matrices only three z-transform pairs are required. Table 2.7 gives the needed pairs.

TABLE 2.7

z-Transform Pairs

Time Function $n \geq 0$	z-Transform $\pi(z)$
$\pi(n+1)$	$z^{-1}[\pi(z)-\pi(0)]$
α^n	$1/(1-\alpha z)$
1(unit step)	$1/(1-z)$

Transformation of Equation (2-21) directly, results in Equation (2-24).

$$z^{-1}[\pi(z) - \pi(0)] = \pi(z)[P] \quad . \tag{2-24}$$

This equation can be rearranged and written as given in Equation (2-25).

$$\pi(z) = \pi(0)\Big[[I] - z[P]\Big]^{-1} \quad . \tag{2-25}$$

If the matrix $\Big[[I] - z[P]\Big]^{-1}$ is calculated and then the inverse transformation made the results are the steady-state and transient matrices. For the four categories of earthquakes, calculation of this matrix and inverse transformation follow.

The historical matrix $[P_L]$ is the first to be analyzed. Using Equation (2-17), the matrix $\Big[[I] - z[P]\Big]^{-1}$ can be calculated. For large earthquakes this matrix is written in Equation (2-26). The transformed matrix is presented separated into the steady-state and transient components.

$$\left[[I] - z[P_L]\right]^{-1} = \frac{1}{1-z}\begin{bmatrix} 0.0294 & 0.9706 \\ 0.0294 & 0.9706 \end{bmatrix} + \frac{1}{1+0.0303z}\begin{bmatrix} 0.9706 & -0.9706 \\ -0.0294 & 0.0294 \end{bmatrix} . \tag{2-26}$$

The inverse transform of Equation (2-26) gives the decomposed transition matrix. This matrix is denoted $H(n)$ and plays the same role as the matrix $[P]^n$. For large earthquakes the matrix $H_L(n)$ is given by Equation (2-27).

$$H_L(n) = \begin{bmatrix} 0.0294 & 0.9706 \\ 0.0294 & 0.9706 \end{bmatrix} + (-0.0303)^n \begin{bmatrix} 0.9706 & -0.9706 \\ -0.0294 & 0.9294 \end{bmatrix} \qquad n=0,1,2,3... \tag{2-27}$$

The first matrix in Equation (2-27) is independent of the index n. The second matrix represents the transient component. Note that the coefficient before the transient matrix in this case is negative. Thus, the transient matrix at odd numbered steps subtracts from the steady-state matrix and at even numbered steps it adds. The pertubation is oscillatory. Ultimately, for large values of n, the transient matrix's elements approach zero and the limiting probabilities are given by the steady-state matrix.

For the remaining three earthquake categories the calculation is the same. Consequently, only the decomposed transition matrices H(n) will be given.

Equation (2-28) presents the transition matrix for moderate earthquakes.

$$H_M(n) = \begin{bmatrix} 0.0704 & 0.9296 \\ 0.0704 & 0.9296 \end{bmatrix} + (-0.0758)^n \begin{bmatrix} 0.9296 & -0.9296 \\ -0.0704 & 0.0704 \end{bmatrix}$$

$$n=0,1,2,3... \qquad (2\text{-}28)$$

Equations (2-29) and (2-30) are the transition matrices for small and very small earthquakes.

$$H_s(n) = \begin{bmatrix} 0.7895 & 0.2105 \\ 0.7895 & 0.2105 \end{bmatrix} + (0.0500)^n \begin{bmatrix} 0.2106 & -0.2106 \\ -0.7895 & 0.7895 \end{bmatrix}$$

$$n=0,1,2,3... \qquad (2\text{-}29)$$

$$H_{v.s.}(n) = \begin{bmatrix} 0.9750 & 0.0250 \\ 0.9750 & 0.0250 \end{bmatrix} + (-0.0256)^n \begin{bmatrix} 0.0250 & -0.0250 \\ -0.9750 & 0.9750 \end{bmatrix}$$

$$n=0,1,2,3... \qquad (2\text{-}29)$$

Consequently, a method for separating the transition matrix into its component parts has been presented. By this method, the contribution of the transient portion to the total matrix can be evaluated. If its contribution is small, it may be better to use only the steady-state portion and thus, simplify the calculations. The formulation of the two state Markov Model can be approached differently from that given in Section 2.6.1. The resulting model is, of course, identical, but the designation is more descriptive. The second formulation is called the Markov Dependent Bernoulli Trials model. A discussion of the model is included for completeness. The model is Markov since the memory aspect of a Markov Process is present. The term Dependent is best explained by considering a chain of events. Note that emphasis is placed on events and not on the index n, denoting time intervals, as in previous discussions. Let the integer k, having values from 1 to $(n-1)$, be the index of the $(k+1)$ events. Therefore, the events are $A_1, A_2, \ldots A_{k+1}$, depending respectively on the first, second, ...(k+1) trial. The trials are said to be dependent if Equation (2-30) holds.

$$P[A_{k+1}/A_k, A_{k-1}, \ldots A_1] = p[A_{k+1}/A_k] \quad . \qquad (2\text{-}30)$$

Thus, each succeeding event depends on the event immediately before it. Whenever a probability model is described as having Bernoulli Trials it is implied that there are only two possible outcomes. In the case considered, the two outcomes are denoted by the symbols s and f for success and failure.

A detailed derivation of the governing equations for this model is presented in Appendix B. The results are given in Equation (2-31). These equations are identical to the closed form representation of the transition matrix $[P]^n$ given in Equation (2-12). The probabilities $p_k(s,s)$, $p_k(s,f)$, $p_k(f,s)$ and $p_k(f,f)$ are the components of this transition matrix.

$$p_k(s,s) = \frac{1-p(s,s)}{2-p(s,s)-p(f,f)}\left[p(s,s)+p(f,f)-1\right]^k + \frac{1-p(f,f)}{2-p(s,s)-p(f,f)} \qquad (2\text{-}31)$$

$$p_k(f,f) = \frac{1-p(f,f)}{2-p(s,s)-p(f,f)}\left[p(s,s)+p(f,f)-1\right]^k + \frac{1-p(s,s)}{2-p(s,s)-p(f,f)}$$

$$p_k(s,f) = 1-p_k(s,s)$$

$$p_k(f,s) = 1-p_k(f,f) \quad .$$

The input probabilities p(s,s), p(s,f), p(f,s) and p(f,f) are obtained from the transition matrices calculated in Section 2.6.2. For each designation of earthquake, there are these four components of the transition matrix These probabilities are considered to be independent of time and the number of the trial in the calculations.

The definitions of the states of success and failure are the same as those used in Section 2.6.1. With input as given by Table 2.8 the probabilities for the model are calculated for a period of 30 years Complete results are given in Appendix C.

TABLE 2.8

Earthquake Designation	p(s,s)	p(f,f)
Large	0.000	0.970
Moderate	0.000	0.924
Small	0.800	0.250
Very Small	0.974	0.000

The computer outputs in Appendix C have two properties in common. All of the components of the matrices converge rapidly to a steady-state probability. Secondly, all the matrices are homogeneous, that is, the rows are identical. The steady-state probabilities are given in Table 2.9.

TABLE 2.9

Calculated Steady-State Probabilities of the

Markov Dependent Bernoulli Trials Model

Earthquake Designation	$p_k(s,s)$	$p_k(s,f)$
Large	0.02940	0.97059
Moderate	0.07042	0.92958
Small	0.78947	0.21053
Very Small	0.97500	0.02500

Earthquake Designation	$p_k(f,s)$	$p_k(f,f)$
Large	0.02940	0.97059
Moderate	0.07042	0.92958
Small	0.78947	0.21053
Very Small	0.97500	0.02500

The probabilities given in Appendix C are calculated to six decimal places, but are accurate only to three figures because the input data were to three significant places.

Convergence of the calculated probabilities given in Appendix C is rapid, occurring by the sixth step in all cases. The limiting value is reached rapidly because the term given in Equation (2-32) is approaching zero rapidly with increasing values of the index k.

$$[p(s,s) + p(f,f) - 1]^k \quad . \qquad (2\text{-}32)$$

Consequently, the equations for the quantities $p_k(s,s)$, $p_k(f,f)$, $p_k(s,f)$ and $p_k(f,s)$ are approaching the constant values given by Equation (2-33).

$$p_k(s,s) \rightarrow \frac{1-p(f,f)}{2-p(s,s)-p(f,f)} \tag{2-33}$$

$$p_k(f,f) \rightarrow \frac{1-p(s,s)}{2-p(s,s)-p(f,f)}$$

$$p_k(s,f) \rightarrow \frac{1-p(s,s)}{2-p(s,s)-p(f,f)}$$

$$p_k(f,s) \rightarrow \frac{1-p(f,f)}{2-p(s,s)-p(f,f)} .$$

If the generated probabilities are examined closely, the remarkable similarity between the values of the historical transition matrix and the calculated probabilities is obvious. It is obvious that the calculated probabilities are very dependent on the accuracy of the input. Here then, is a point where the short available earthquake record may have a telling effect

The historical record for large and moderate earthquakes differs remarkably from the record for small and very small earthquakes. This difference is reflected in the calculation of the probabilities of the transition matrix [P] and in the probabilities calculated for the Markov Dependent Bernoulli Trials model. Note that in both cases for large and moderate earthquakes $p(s,s)$ is zero or nearly zero and

$p(f,f)$ is one or nearly one. On the other hand, for small and very small earthquakes the situation is reversed. It appears that the physical mechanism for the generation of large and moderate earthquakes is different from the mechanism for small and very small earthquakes. For example, if all four categories conformed to the Elastic Rebound Theory, the probability $p(s,s)$ should be small. For large and moderate earthquakes it is, but for small and very small earthquakes it is not.

It may be presumptuous to attempt to explain the difference in the calculated probabilities. However, it may be that strain energy is entering the region at a very rapid rate but being released almost as rapidly. Only in rare instances is strain energy accumulated leading to a large or moderate earthquake.

2.7 Time to First Arrival of an Earthquake

In the preceding sections, the probability of the occurrence of an earthquake in a given year was calculated. Once these probabilities are known it is possible to calculate the average time to the next occurrence This calculation can be made dependent on the present state of the system. Later, in the Markov decision process, it will be necessary to have this information for calculation of the continuous time transition matrices.

First arrivals of earthquakes will generally depend on the present state of the system. In the discussion of the mechanism behind earthquake occurrence in Section 2.2 the significance of this factor was pointed out. However, in Section 2.6 it was pointed out that after a small number of transitions the memory of the system is lost and the transition probabilities approach limiting values. For example, consider the category of large earthquakes. From Equation (2-26), it is seen that after a small number of transitions the initial state of the system no longer influences the transition probabilities. The probability of an occurrence becomes constant. For large earthquakes, this probability is given by Equation (2-34).

$$p_k(s,s) = p_k(f,s) = 0.0294 \quad . \qquad (2\text{-}34)$$

The transitions are now independent of time.

Let the letter p indicate the probability of an earthquake occurrence. Then the probability of no occurrence is (1-p). If the letter N denotes the number of trials to the first occurrence, the probability distribution function is given by Equation (2-35).

$$P[N=n] = (1-p)^n p \qquad (2\text{-}35)$$

This distribution is called the Geometric. Its first moment, which is tantamount to an average value in some instances, is given by Equation (2-36).

$$E(N) = \frac{1}{p} \quad . \qquad (2\text{-}36)$$

Since each trial represents a period of one year the reciprocal of p is measured in years. Table 2.10 gives the expected time to the next occurrence of an earthquake by category.

TABLE 2.10

Earthquake Designation	p	1/p
Large	0.0294	34.0
Moderate	0.0704	14.2
Small	0.7895	1.27
Very Small	0.9750	1.02

For the categories of moderate through very small, calculation of the simple average will not agree with the calculation from the Markov transition matrix. This fact is due to the definition of the states of success and failure. For example, success was defined as the occurrence of at least one earthquake in a given year. The calculation of the simple average will include all occurrences and not just the first in a given year.

Furthermore, this calculation is based on the steady-state probabilities. Once this point is reached the occurrence of earthquakes can be assumed to be independent of one another. This is due to the fact that the probabilities have reached constant values.

III. MARKOV DECISION ANALYSIS

3.1 Introduction

In this chapter it will be shown how the Markov Process can be used to develop a decision model for risk analysis. The material presented here is part of the theory developed by Howard [17,21]. Only the basic concepts needed for a seismic risk analysis are discussed.

In Chapter II, it was stated that a discrete Markov Process makes its transition at a uniformly spaced time interval. For example, given the initial state of the system $\pi(0)$ the probable state of the system at the n^{th} step can be calculated by postmultiplying the initial state vector by the n^{th} step transition matrix.

$$\pi(n) = \pi(0) \cdot [P]^n \qquad n=0,1,2,3... \qquad (3\text{-}1)$$

The integer n refers to the number of uniform time intervals which are assumed to have passed in calculating the state of the system $\pi(n)$ in the future. The discrete nature of time makes it possible to calculate probabilities of the trajectory of the process only at the integer value of the time interval. Although the length of the time interval can have any value, the calculation of the probabilities is restricted to multiples of the fundamental time interval.

It is apparent that if this restriction could be removed a more realistic model of a process could be constructed. The continuous-time Markov Process removes this restriction. It imposes other restrictions which are, however, much less confining.

The continuous-time Markov Process involves solution of the following differential equation.

$$\frac{d\pi_j(t)}{dt} = \pi_i(t) \cdot [A] \qquad t \geq 0 \qquad (3\text{-}2)$$

$\pi_j(t)$ -- final state vector

$\pi_i(t)$ -- initial state vector

$[A]$ -- transition rate matrix

The continuous-time Markov Process permits calculation of the probability of transition at a random time.

The solution of the governing differential equation follows.

$$\pi_j(t) = \pi_i(t) \cdot e^{[A]t} \qquad t \geq 0 \qquad (3\text{-}3)$$

Noteworthy is the similarity between the equations of the discrete and continuous Markov Processes. It is immediately evident that the discrete transition matrix $[P]^n$ is analogous to the continuous quantity $e^{[A]t}$. The interpretation of the quantity $e^{[A]t}$ is not immediately obvious. It is to be interpreted in the form of an infinite series.

$$e^{[A]t} = I + tA + \frac{t^2}{2!} A^2 + \ldots \qquad (3\text{-}4)$$

The immediate problem is to obtain a closed form solution to Equation (3-4) and thereby illustrate the nature of the transition matrix.

3.2 Salient Restrictions of the Continuous-Time Markov Process

To be considered a Markov Process the continuous-time Markov Process must satisfy two important requirements. These two requirements depend on an understanding of the concepts of holding time and waiting time. If two particular states i and j are defined, the holding time is the interval which the process spends in state i before making a transition to state j. Holding times are positive valued random variables described by a probability mass function. If the system is presently in state i, the time before a transition is made to any state j is called the waiting time. Therefore, the distinction between holding and waiting times is that the former considers the time between transition from a state i to a particular state j while the latter considers the time spent in state i before transition to any successor state j.

First, the state presently occupied must be the sole determination of the future trajectory of the process. Secondly, the length of time that this state has been occupied must be irrelevant in predicting the final state and in assigning the probability distribution to the remainder of the holding time in its present state. Otherwise, the holding time function will not be a description of the time to transition.

The first requirement precludes different holding times for transitions out of the same state. The holding time functions are identical for the transition from the same state, but different for different states. The second requirement is that the time that the state has been occupied must not affect the holding time remaining until the next transition. Mathematically, this requirement can be written as follows.

$$\text{Probability} \left\{ \tau_i > t+\Delta / \tau_i > t \right\} = g(\Delta) \quad . \qquad (3\text{-}5)$$

This requirement that the probability that the waiting time in state i, denoted τ_i, is greater than the time t plus an additional interval of time Δ given that τ_i is greater than t is a function only of the time Δ.

3.3 Holding and Waiting Time Functions

Understanding the concepts of holding and waiting time is essential to understanding the continuous-time Markov Process. Basically the time to the transition is considered. The time variable is considered a random variable.

The time spent in a state i is considered to be described by an exponentially distributed probability density function. The exponential distribution is given by the following function.

$$h(\tau) = \lambda e^{-\lambda\tau} \qquad \tau \geq 0 \tag{3-6}$$

The average of this function is $1/\lambda$ and the variance is $1/\lambda^2$. Before the transition occurs from state i to state j the process is said to hold for a time in state i. This holding time is denoted τ_{ij}. The probability density function of τ_{ij} is denoted $h_{ij}(\tau)$ and is exponentially distributed.

$$h_{ij}(\tau) = \lambda e^{-\lambda\tau} \qquad \tau \geq 0 \tag{3-7}$$

Then the probability that τ_{ij} is greater than some arbitrary time t is given by the following integral.

$$p\left\{\tau_{ij} \geq t\right\} = \int_t^\infty h_{ij}(\tau)dt \quad . \tag{3-8}$$

The waiting time density function $\omega_i(\tau)$ is related to the holding time density function $h_{ij}(\tau)$. It is the summation of the holding time functions for state i weighted by the probability that they occur. Thus,

$$\omega_i(\tau) = \sum_{j=1}^{N} p_{ij}\, h_{ij}(\tau) \quad . \tag{3-9}$$

Consider the possible transitions from state i to state j. The process could remain in state i, that is, state i and state j are synonymous. Thus, the process remains in state i for the time t and its transition occurs after the time t. If states i and j are different states and the transition is made from state i to state j at least one transition must be made in the time t. In fact, a number of transitions could be made in time t. The process could make a transition from state i to some state k and then by a series of transitions from state k to other states until finally state j is reached at time t. The probability that the waiting time in state i, denoted τ_i, will be greater than some value t is denoted $\bar{\omega}_i(\tau)$. Equation (3-10) defines this quantity mathematically.

$$\bar{\omega}_i(\tau) = P\left\{\tau_i > t\right\} = \int_t^{\infty} dt \sum_{j=1}^{N} p_{ij}\, h_{ij}(\tau) \quad . \tag{3-10}$$

The transition rate matrix will be denoted by the symbol Φ. The elements of this matrix are denoted by $\phi_{ij}(t)$. Each element $\phi_{ij}(t)$ is the probability that the process occupied state j at time t given that it began in state i at time zero. Recall that the future trajectory of the process depends only on the state occupied at time zero.

The fundamental equation governing calculation of the transition matrix can be developed by considering the possible transitions from state i to state j during a time interval of arbitrary length t. The process begins in state i at time zero. The length of time that the process has been in state i does not affect the future trajectory of the process.

The transitions of the system are described in Equation (3-11) Hence, one may write

$$\phi_{ij}(t) = \delta_{ij} \; \overline{\omega}_i(t) + \sum_{k=1}^{N} p_{ik} \int_0^t d\tau \, h_{ik}(\tau) \cdot \phi_{kj}(t-\tau)$$

for $i=1,2\ldots N$ $\quad j=1,2\ldots N$ $\quad t\geq 0$ (3-11)

The symbol δ_{ij} is the Kronecker delta function and is defined as follows.

$$\delta_{ij} = \begin{cases} 1 & i=j \\ 0 & i\neq j \end{cases}$$

The first term in Equation (3-11) is the probability that the transition occurs after some time t while the second term describes the systems transition from state i to state j.

The interval transition rate matrix $\Phi(t)$ is composed of the elements denoted $\phi_{ij}(t)$ and the matrix [H] includes the elements h_{ij}. Equation (3-12) defines the transition matrix $\Phi(t)$.

$$\Phi(t) = \bar{\omega}(t) + \int_0^t d\tau \, [P \square H \cdot \Phi(t-\tau)] \qquad . \qquad (3\text{-}12)$$

Solution of this equation can be done by exponential transform. The symbol $\square$ implies multiplication of the corresponding elements of [P] and [H], where [P] is the matrix of probabilities p_{ij} and [H] is the matrix of holding time functions $h_{ij}(\tau)$.

Solution of this equation can be found in reference [21]. The outline of the solution will be presented here.

The transition rate matrix is defined by the matrix relation

$$[A] = [\Lambda]([I]-[P]) \tag{3-13}$$

where $[I]$ is the identity matrix and $[\Lambda]$ is a diagonal matrix containing the coefficients λ_i of the waiting time functions. For a system with n possible states, $[\Lambda]$ will be an nxn matrix. The waiting time in each state i is given by:

$$\omega_i(\tau) = \lambda_i e^{-\lambda_i t} \tag{3-14}$$

for $i=1,2,...n$.

The matrix $[P]$ is the discrete time transition matrix which describes the transitions from state i to state j for the system.

To obtain the closed form transition matrix $\Phi(t)$ it is necessary to perform an inverse exponential transform Equation (3-12).

$$\Phi^e(s) = \left[s[I]-[A]\right]^{-1} \tag{3-15}$$

The matrix $\Phi^e(s)$ is the exponential transform of $\Phi(t)$. Transforming $\Phi^e(s)$ gives

$$\Phi(t) = e^{[A]t} \tag{3-16}$$

The matrix $e^{[A]t}$ can be decomposed into two parts--a steady state and a transient portion.

This development of the continuous time transition is important to understanding the system which is to be modeled. The salient features of the system are to be modeled probabilistically. By calculating the continuous time transition matrix for the system one can determine if the model is an accurate description of the system.

3.4 Governing Equations

The governing equations of the Markov Decision Model contain all that is known about the system as a function of time. For this model the variable time (t) is defined as the remaining time in the life of the structure. This manner of looking at time is somewhat different from what one is accustomed to. In this case, when the time t is large the structure is far from having expended its useful life, and when time t is small the structure is near the end of its useful life.

The proposed model is applicable to systems in which the termination point is remote or in which there is a finite life. For structures whose design life is very large and where there are many possible states for the structure to be in, the asymptotic or steady-state solution to the governing equations is the best approach. In this case, interest is focused away from the point of termination where the asymptotic solution is as good as the exact solution. Near the point of termination the exact solution is preferred.

The Markov Decision Model considers how the events to which the system is exposed affect the cost of operating the system. It includes the information concerning the trajectory of the system obtained from calculation of the continuous time transition matrix but goes further in that now the costs associated with a transition are also considered.

3.5 Markov Decision Equations

In order to appreciate the applicability of the Markov Decision Model to estimating the probable risk of constructing a building at a site or planning a community in a seismically active region a description of the model is presented here. The emphasis is on the physical meaning of the equations and how various interpretations of the equations permit different models to be formulated. The risk is measured in monetary units and reflects the seismicity of the region, the type of building and its susceptibility to earthquake damage. The model can be used for planning on a regional or local level. The comparison criterion is that all future monetary costs are brought to a common point in time for comparison.

A realistic comparison of incomes or expenses must take into account the fact that the value of money is time dependent. The time dependence is twofold. Inflation contributes to a loss in value of money. The rate of inflation varies with time. Secondly, the value of a sum of money to be paid out in the future has a smaller value today because the smaller sum of money placed at interest will generate the larger sum with the passage of time.

Both of these factors can be accounted for by appropriately selecting a discount factor β. To account for inflation increase the discount factor β over what its value was to account for the interest rate.

The choice for discounting is continuous compounding at the rate beta (β). Selection of a continuous type of discounting facilitates development of the model. Continuous compounding implies that a unit sum of money to be received after a short interval of time Δt has a present worth of $(1-\beta \cdot \Delta t)$.

There is another interpretation of a discount factor. A continuous time expense or income function $f(t)$ discounted continuously at the rate beta (β) has as its present value the amount

$$\int_0^{+t_0} f(t) \cdot e^{-\beta t} dt \qquad 0 \leq t \leq \infty \qquad (3\text{-}17)$$

At time $t=0$, the process has reached the terminal state. At this point the boundary conditions can be imposed. If the structure is to be sold or torn down, the income or cost of either eventuality can be included in the model. In fact, each state of the model can have an independent boundary condition.

The quantity $C_i(t)$ is defined as the total expected cost accumulated in the time t, where t is the time remaining in the life of the structure, given that the system started in state i.

The Markov Decision Model is based on the following first order differential equation.

$$\frac{dC_i(t)}{dt} + \beta \cdot C_i(t) = q_i + \sum_{j=1}^{N} a_{ij}C_j(t) \qquad (3\text{-}18)$$

$$i=1,2,\ldots N$$

The derivation of this equation is given in Appendix D. Basically, this equation associates a cost with the possibility of transition by the system.

The quantity $C_i(t)$ is the quantity of interest. If the time of interest is near to termination it is best to solve the differential equations. There are as many equations as there are possible states. These equations are coupled and their solution can be obtained by LaPlace Transformation. If there are many equations or if the time to termination is remote it is best to solve the asymptotic form of the equations. The solution of these equations approaches a steady-state value very quickly. The steady-state solution can be obtained by solving the following set of algebraic equations.

$$\beta C_i(t) = q_i + \sum_{j=1}^{N} a_{ij}C_j(t) \qquad (3\text{-}19)$$

$$i=1,2,\ldots N$$

Again there are as many equations as there are possible states. The discount factor beta was discussed earlier.

The quantity q_i is called the earning rate of the system when the structure is in state i. It can be a positive or negative quantity depending on whether it is an income or an expense. This quantity q_i is a characteristic of the system and can be used for comparison purposes in a policy improvement consideration. Definition of q_i is as follows.

$$q_i = c_{ii} + \sum_{\substack{j \neq i \\ j=1}}^{N} a_{ij} c_{ij} \qquad (3\text{-}20)$$

The terms c_{ii} and c_{ij} have different units. The quantity c_{ii} is the cost associated with being in state i and is a cost per unit time. On the other hand, τ_{ij} is the fixed cost of moving to state j from state i for the system.

The associated expected cost $C_i(t)$ is a future cost. Knowledge of its value at time t represents knowledge of what the expected cost in the future would be over the remaining time t. Suppose it is desired that improvement be made to the structure and that their cost if known. If one knows the burdens of the future, one can make adjustments now.

The time variable t is considered a continuous variable. It is therefore appropriate that the probabilities defining the transition of the structure to the various states be continuous. This implies that a transition rate matrix defined by the letter A is required here.

The elements of the matrix A are denoted a_{ij} where the transition is from state i to state j, $i \neq j$. The diagonal elements are obtained from the following relationship.

$$a_{ii} = -\sum_{\substack{i \neq j \\ j=1}}^{N} a_{ij} \tag{3-21}$$

Thus, the rows of the matrix add to zero instead of one for the discrete time Markov Transition Matrix.

This transition rate matrix A forms a part of the subject of continuous-time Markov Processes. The procedure for constructing this matrix was discussed in the first part of this chapter.

3.6 Markov Models

The models which will be used in this dissertation will be discussed in this section. The generality of the Markov approach is illustrated.

3.6.1 Models Involving a Single State of Unserviceability

This model is to be used where two requirements can be met. The system must be in one of two states--undamaged state or damaged state. It must begin in the first state and move to the second state. Once it reaches the second state the process has come to a halt and must begin again.

The defining equations would be for $i=1,2$ and $j=1,2$.

$$\frac{dC_1(t)}{dt} + \beta C_1(t) = q_1 + a_{11}C_1(t) + a_{12}C_2(t)$$

$$\frac{dC_2(t)}{dt} + \beta C_2(t) = q_2 + a_{21}C_1(t) + a_{22}C_2(t) \qquad . \qquad (3\text{-}22)$$

The two differential equations are coupled together, requiring that the equation be solved simultaneously.

The earning rates of the system

$$q_1 = c_{11} + a_{12}c_{12}$$
$$q_2 = c_{21} + a_{22}c_{22} \qquad (3\text{-}23)$$

contain the cost information of the system. Denote the individual costs c_{ij} by the matrix symbol $[C]$.

$$[C] = \begin{bmatrix} c_{11} & c_{12} \\ c_{21} & c_{22} \end{bmatrix} \qquad . \qquad (3\text{-}24)$$

There are many interpretations which can be given to the quantities C_{ij} for a two state system. If it is assumed that there is no replacement of the facility upon reaching state 2 (damaged state) the following interpretation is applicable. The quantity c_{11} is the cost of the remaining in state one. It is a fixed cost and could possibly represent the insurance premium.

Then the cost c_{12} would be the cost of restoring the system to state 1 after it had arrived in state 2. At this point, the process would be assumed to begin again. The other costs would be set equal to zero for this problem.

$$c_{21} = c_{22} = 0 \quad . \tag{3-25}$$

The transition matrix A whose elements are a_{ij}, would be calculated from the past record if it is available. Otherwise estimates of [A] would have to be made. For the two state system being considered here the quantities a_{21} and a_{22} are equal to zero. The solution of the following first order coupled differential equations represents the exact solution to the problem. Solution of two coupled equations which are first order can be easily obtained using LaPlace Transformation.

$$\frac{dC_1(t)}{dt} = q_1 + (a_{11}-\beta)C_1(t) + a_{12}C_2(t)$$

$$\frac{dC_2(t)}{dt} = q_2 - \beta C_2(t) = -\beta C_2(t) \quad . \tag{3-26}$$

Solutions are

$$C_1(t) = \frac{q_1}{\beta-a_{11}} + \left[C_1(0) - \frac{q_1}{\beta-a_{11}} + \frac{a_{12}}{a_{11}} C_2(0)\right] e^{-(\beta-a_{11})t} - \frac{a_{12}}{a_{11}} e^{-\beta t} C_2(0)$$

$$C_2(t) = C_2(0)e^{-\beta t} \quad . \tag{3-27}$$

The system is assumed to begin in state 1; hence, $C_2(t)$ must be put equal to zero. Thus, only one solution is useful, that for $C_1(t)$.

For large values of time the equations in (3-27) reduce to constant values

$$C_1(t) = \frac{q_1}{\beta - a_{11}}$$

$$C_2(t) = 0 \quad . \tag{3-28}$$

These are the asymptotic or steady-state solutions which could have been obtained directly by solution of the algebraic equations in (3-19)

The boundary conditions are represented by the quantities $C_1(0)$ and $C_2(0)$. These are the values at the termination point if the system had begun in state 1 and state 2 respectively. For the particular model considered here $C_2(0)$ is equal to zero since the system begins in state 1.

$$C_1(t) = C_1(0)e^{-(\beta - a_{11})t} + \frac{q_1}{\beta - a_{11}}\left[1 - e^{-(\beta - a_{11})t}\right]$$

$$C_2(t) = 0 \quad . \tag{3-29}$$

Applications of this model could be found in the insurance field. An insurance company could set its earthquake insurance rate by adjusting c_{11} in such a way that together with the expected cost of damage and the probabilities of its occurrence it will obtain an equitable reward $C_1(t)$. Thus, depending on the risk evidenced by the seismicity of the region and on the value of c_{12} the insurance company could establish its rates.

For the case under consideration, the costs c_{21} and c_{22} are both zero. From equation (3-23) the earning rates are given by

$$q_1 = c_{11} + a_{12}c_{12}$$
$$q_2 = 0 \quad . \tag{3-30}$$

The transition rate probabilities and cost c_{12} would be obtained from historical records.

A second application of this model would be to consider a structure where state 2 represents a failure and where the structure is replaced immediately by an identical structure when failure occurs. In this model the cost quantities c_{12} and c_{21} are equal. They represent the cost of replacement.

It may be possible that in both models, the expected total cost of damage $C_1(t)$ is unacceptable. In such a case, the cost of the insurance premium would make the construction of the building uneconomical. This may lead to the conclusion that the building site is unacceptable because the risk is high.

Equation (3-34) has a further interesting property. Note that the value of the average cost of damage c_{12} does not enter the calculations directly. It must be multiplied by the transition probability a_{12}. There are statistical variations in the quantities c_{12} and a_{12}. Consequently, both quantities could contribute to random error in $C_1(t)$.

3.6.2 Models Involving the Cost-Benefit Relation

Another case of interest is the optimumization of the cost-benefit relation in aseismic design. This problem can be analyzed using the Markov Decision Model proposed herein.

Consider a three-state model. The states which the structure could occupy are listed below.

TABLE 3.1

State	Description
1	Structure is undamaged
2	Structure suffers a given amount of damage
3	Structure has collapsed

Suppose that the transition probabilities of interest consider the cases where the structure remains in state one or during an earthquake moves to state two or state three. In this case, three probabilities, a_{11}, a_{12}, and a_{13} are of interest.

Each of the three transition probabilities are dependent on the intensity of ground motion at the site. This relation could be determined analytically or empirically through statistical analysis. Equation (3-31) indicates the functional dependence.

$$a_{11} = g_1(I)$$

$$a_{12} = g_2(I) \qquad (3\text{-}31)$$

$$a_{13} = g_3(I) \quad .$$

The magnitude of transition probabilities a_{12} and a_{13} can be reduced by providing additional strength to the structure. This assumption follows the philosophy of most building codes which imply that greater strength provides greater safety. In turn, the intensities of ground motion can be related to the Richter Magnitude of an earthquake and the distance from the earthquake's epicenter. Consequently, a design basis ground motion can be selected and probabilistically designed for.

The additional cost of providing earthquake resistance to the structure above code requirements will be designated C'. Therefore, the transition probabilities are functionally related to this quantity.

Equation (3-34) presents this functional relationship.

$$a_{12} = g_4(C')$$

$$(3\text{-}32)$$

$$a_{13} = g_5(C') \quad .$$

This functional relationship would be very difficult to determine. However, as a first choice one might assume that it is a proportional relation. The reason behind this assumption would be the same argument given for assuming that the transition probabilities are proportional to ground intensity.

If the structure begins in the undamaged state the quantity of interest is $C_1(t)$. The optimizing procedure might be to add the quantities C' and $C_1(t)$ together. This determines the minimum of the combined function with respect to the transition probabilities a_{12} and a_{13}.

To carry out this procedure define a function $F(a_{12}, a_{13}$ by Equation (3-33).

$$F(a_{12}, a_{13}) = C'(a_{12}, a_{13}) + C_1(a_{12}, a_{13}, t) \qquad (3\text{-}33)$$

If the function F has a minimum it can be found by the methods of the calculus. Briefly stated, the requirements are that the function F have continuous first and second derivatives with respect to the quantities a_{12} and a_{13}, respectively. Then for a point denoted by the symbols $\bar{a}_{12}$ and $\bar{a}_{13}$ Equation (3-34) must hold.

$$\frac{\partial F(\bar{a}_{12}, \bar{a}_{13})}{\partial a_{12}} = 0 \qquad \frac{\partial F(\bar{a}_{12}, \bar{a}_{13})}{\partial a_{13}} = 0 \quad . \qquad (3\text{-}34)$$

A relative minimum exists at the point if Equations (3-35) hold.

$$\frac{\partial^2 F}{\partial a_{12} \partial a_{13}} - \left(\frac{\partial^2 F}{\partial a_{12}^2}\right)\left(\frac{\partial^2 F}{\partial a_{13}^2}\right) < 0$$

$$\left[\frac{\partial^2 F}{\partial a_{12}^2} + \frac{\partial^2 F}{\partial a_{13}^2}\right] > 0 \quad . \tag{3-35}$$

Once the point $\bar{a}_{12}$, $\bar{a}_{13}$ has been found the risk for the structure $C_1(t)$ can be calculated and the added cost of protection C' will be known.

3.7 Conclusion

The two models discussed here will be applied to engineering problems in the following chapters. A situation where there are two possible states for a system will be found in Chapter IV. In Chapter V this model is generalized to include four possible states. The cost-benefit relation is applied to an engineering problem in Chapter VI.

IV. MOBILE HOMES

4.1 Introduction

A form of shelter gaining greater acceptance among Americans is the mobile home. In 1970, it was estimated that 2 % of the population of the United States lived in mobile homes. By 1980 it is expected that this figure will reach at least 10% [24].

Mobile home living in California is quite different from that of the rest of the Nation. Nationwide, it is estimated that the average cost of a mobile home is $6,500 while in California the average cost is nearly $16,000. This difference is due predominantly to one major factor. In California, over 75% of all mobile homes are owned by people over 55 years old. Nationwide, this age group owns only 18% of all mobile homes. Therefore, it appears that in California the mobile home affords shelter for those who are retired or about to retire. Furthermore, 50% of all mobile homes sold in California are double widths (24 feet wide) while 48% are single widths (12 feet wide). Only 2% of sales are in the smaller sizes. This implies that the mobile home dweller in California is more affluent and wants the accommodations which a large mobile home can provide.

There are some 5,500 mobile home parks in California in which are placed about 200,000 mobile homes. It is estimated that about 500,000 people live in these mobile homes. Thus, one can see that the large number of mobile homes and their rapidly growing numbers present a potentially costly hazard during an earthquake. Most mobile home parks are in Southern California.

4.2 Definition of a Mobile Home

A mobile home is a portable unit designed to be moved from site to site by towing. Its chassis consists of a rigid frame from which two to six wheels are suspended. When the unit is attached to utilities it provides year-round living space.

The term 'mobile home' applies to units wider than eight feet and longer than 32 feet. Units providing less than this amount of floor space are termed 'recreational vehicles.'

Originally, the mobile home was considered a mobile dwelling. Consequently, for purposes of taxation it was considered a chattel and not taxed as real property. Recent studies have shown that once the mobile home is put in place it is rarely moved. Consequently, it is very likely that the method of taxation will be modified in the near future. However, at the present time, the mobile home is taxed as a vehicle by the state at the rate of eleven dollars per year and at the local level as property at the rate of 2% of market value.

Most mobile homes which are to be used as permanent dwellings are at least 10 feet wide and 40 feet long. From a practical standpoint a mobile home smaller than this size is not able to provide the necessary accommodations.

4.3 Type of Construction

Construction of a mobile home follows a rather standard practice. On a suspension consisting of from two to six wheels, two channel or I-beam sections are placed extending the entire length of the mobile home. A stiff, but light in weight, frame outlining the contour of the home is attached to the sections. Over this frame, a thin skin of aluminum is riveted which forms the outer shell of the home. The thin skin is left unpainted so that its rolled brightness aids in reflecting heat and, therefore, aids in reducing air conditioning costs. Inside the vehicle various types of facades are installed to form the inner walls. The purchaser of the mobile home can select the level of quality and amount of furnishings to be placed in the vehicle. Facilities generally include a stove, refrigerator, toilet, and air-conditioning.

Because of the light construction a mobile home is simply not as sturdy as a permanent dwelling. The thin exterior skin is particularly susceptible to damage. Therefore, it is to be expected that the life of a mobile home as a dwelling would be less than that of a permanent home. Generally, it is estimated that a mobile home can be used as a dwelling for about 15 years. At the conslusion of this period of time a mobile home is usually converted to other uses or used only as a recreation or second home.

4.4 Insurance and Depreciation

When the owner of a mobile home purchases insurance, earthquake protection is generally included in the policy. On the average an insurance policy will cost $200 for a period of three years. If the mobile home owner decides that earthquake protection is not necessary, approximately $15 is deducted from the insurance premium. A $50 deductible is nearly standard.

In California, the average cost of a mobile home is about $16,000 with a range of about $12,000 to $25,000. Depreciation of the home is fairly rapid. It is not uncommon for a mobile home to depreciate by as much as 40% by the end of the fifth year from the purchase date.

4.5 Purpose of the Investigation

The large number of mobile homes in California and their concentration in mobile home parks represents a potentially costly earthquake hazard. It will be shown that a large amount of the expected damage could be avoided by proper design of the foundation of the mobile home. Basically, the foundation must be constructed such that lateral stability is provided to the mobile home. An alternate type of foundation, which will provide lateral stability, is considered to be placed at two different points in time. The first point is when the coach is placed in the mobile home park and the second point after the coach has been in place for a period of time. The efficacy of the new foundation will be demonstrated by comparing the risk values associated with each type of foundation.

Secondly, the quantitifcation of the level of risk associated with a mobile home in a seismic environment should be useful to an insurance company. The level of risk is the prime factor in determining the cost of the insurance premium. Without a long history of seismic exposure an insurance company would find it difficult to establish a rate for mobile homes.

4.6 Damage Estimates

The plotted data in Figure 4.1 was taken directly from reference [19]. The data represents the experience of one insurance company during the San Fernando Earthquake of 1971. Values of the damage are averages of the total paid loss at a particular mobile home park. Thus, more than one mobile home is included in each damage value.

Noteworthy in this data is the large amount of dispersion. It is known that two significant factors contributed to this dispersion. Soil conditions at the site represent one significant factor. For example, the data points labeled one and two in Figure 4.1 are much larger than what would normally be expected. This is thought to be due to the high water table at both of these sites. The second factor is associated with the flatness of the terrain at the site. The typical mobile home is supported by concrete blocks upon which jacks are placed to level the coach. The State code requires a minimum 12-inch clearance everywhere under the coach. In order to obtain the minimum clearance on sloping ground one end of the coach is considerably higher off the ground than the other end. This requirement contributes to the poor performance of mobile homes exposed to a lateral motion. Unfortunately, information which would identify the data points subject to this condition is not available.

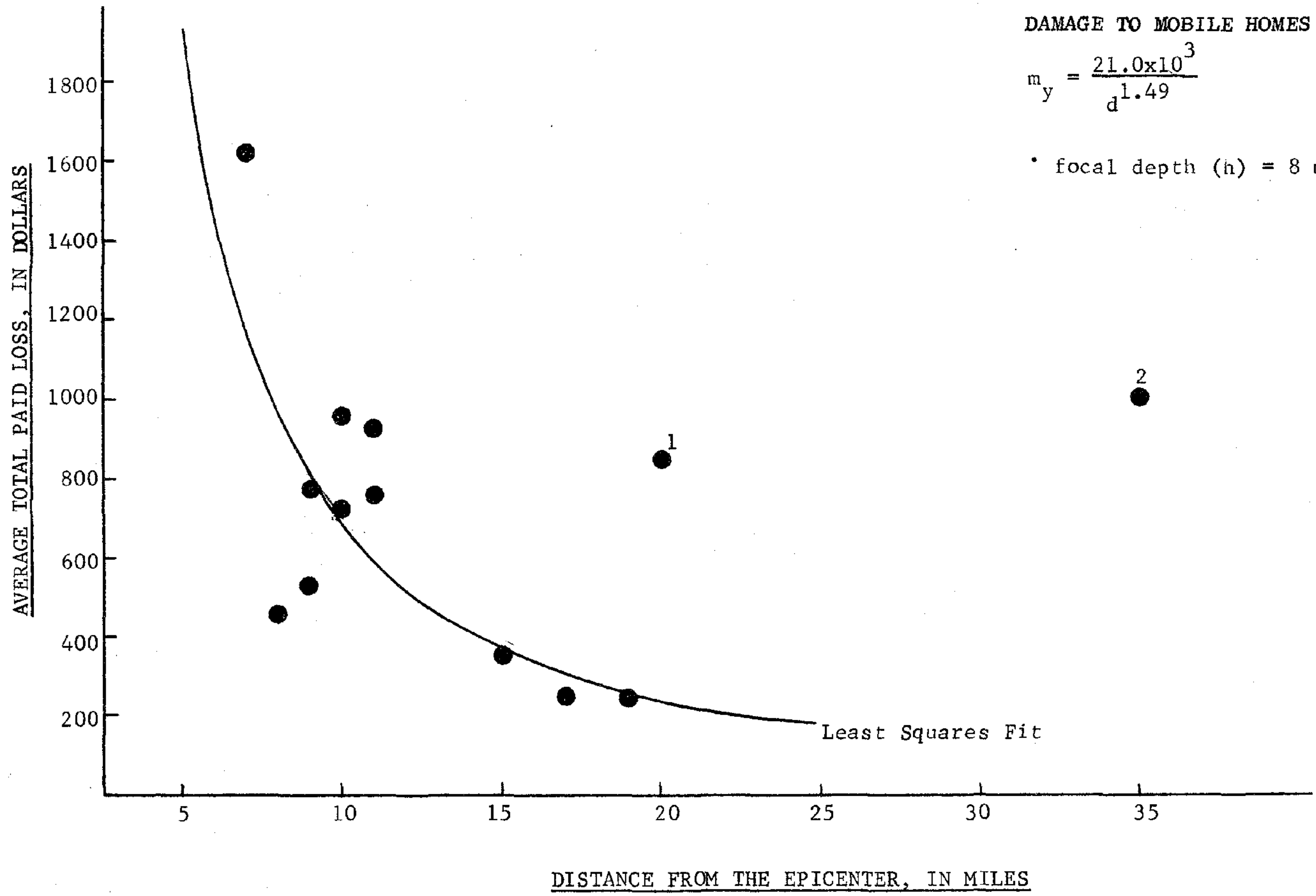
Figures 4.2, 4.3, 4.4 and 4.5 illustrate the damage to mobile homes by category. The four significant categories are damage to the coach itself, damage to mobile home contents, damage to the decorating skirt and awning, and finally the cost of releveling the mobile home after it had been displaced from its foundation by the motion associated with an earthquake. Damage to the coach is primarily related to buckling and wrinkling of the thin aluminum skin forming the exterior covering. On occasion, the concrete blocks supporting the mobile home penetrated through

FIGURE 4-1

DAMAGE TO MOBILE HOMES

$$m_y = \frac{21.0 \times 10^3}{d^{1.49}}$$

• focal depth (h) = 8 miles



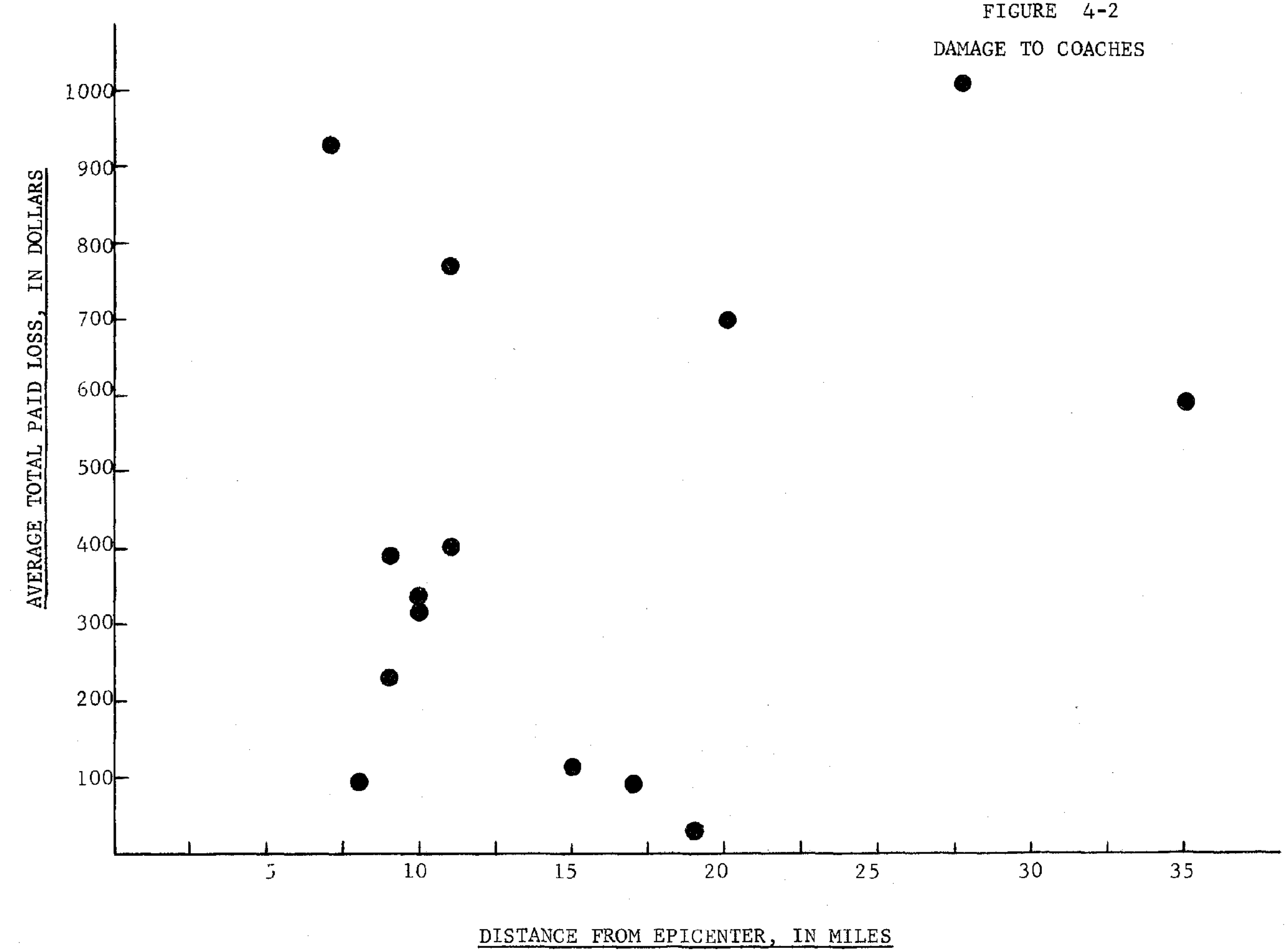


FIGURE 4-2

DAMAGE TO COACHES

FIGURE 4-3

MOBILE HOME CONTENTS

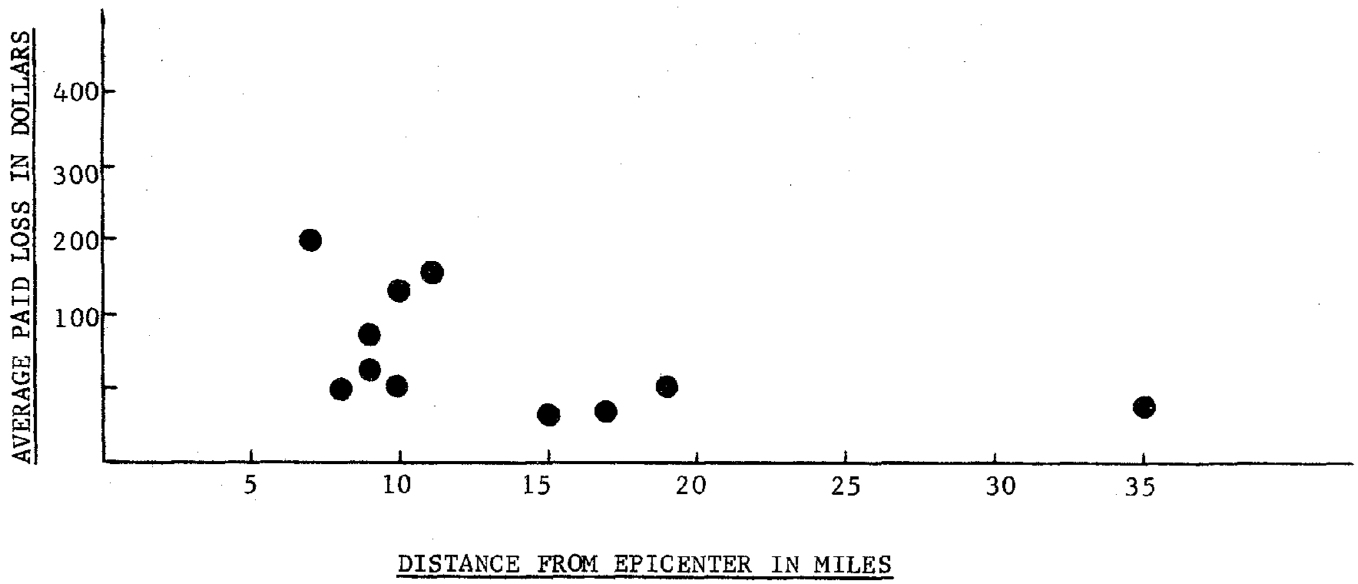
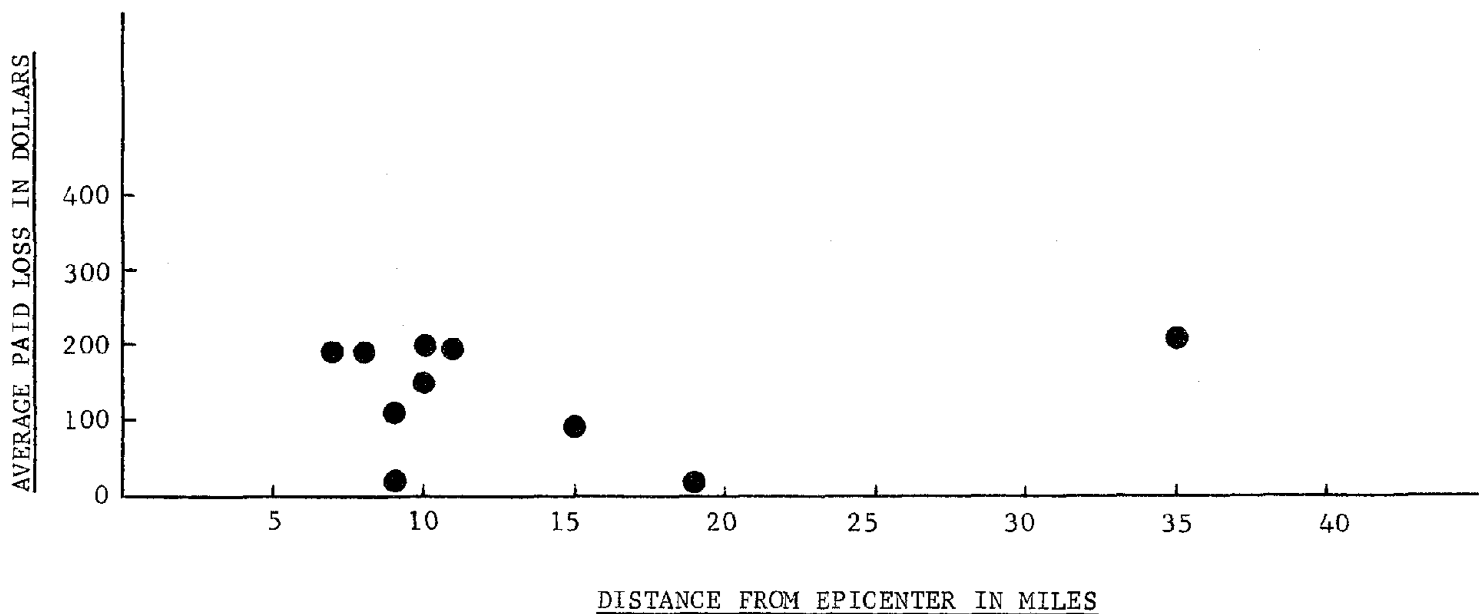


FIGURE 4-4

AWNING AND SKIRT



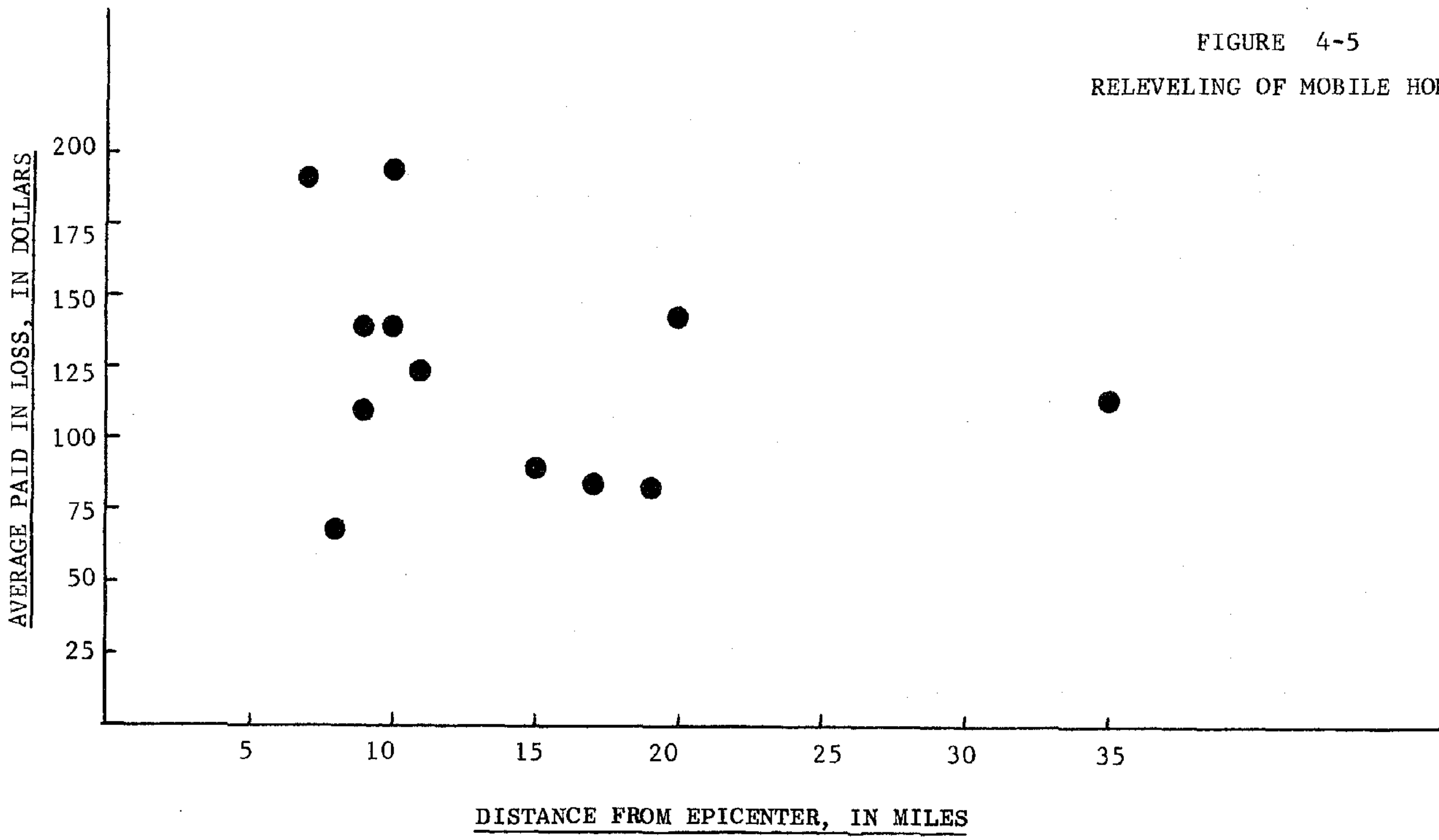


FIGURE 4-5
RELEVELING OF MOBILE HOME

the floor after the coach was displaced. The category, mobile home contents, refers to the furnishings and utilities in the dwelling. Around the bottom of the coach a flexible aluminum skirt is generally placed for decorative purposes. It hides the chassis of the mobile home and gives a permanent appearance to the dwelling. During the motion of the coach this skirt was always damaged or destroyed. Its cost is approximately one dollar per linear foot. In addition, displacement of the mobile home generally results in damage to the awnings which are attached to the exterior of the coach. This damage cost represents category three. The final category is the cost of releveling the coach after it has been displaced from its foundation. This is a fixed cost and is approximately $130 per coach. Table 4.1 summarizes the paid loss by category.

TABLE 4.1

MOBILE HOME DAMAGE BY CATEGORY

Percent of Paid Loss	Damage Category
43	Coach
23	Coach Contents
17	Relevel Coach
17	Awnings and Skirt

4.7 Interpretation of the Damage Estimates

The damage values given in Figure 4.1 are average values and are not expressed in relation to the initial cost of the mobile home. It appears reasonable to assume that a more expensive mobile home would suffer proportionately more costly damage. This would especially be true when the cost of damage to the coach and the contents of the coach is considered. Therefore, it is necessary that a method be found wherein damage to individual coaches can be estimated.

It is possible to overcome this shortcoming in the available data by considering the damage level as a standardized random variable and the cost of the coach as a standardized random variable. The ratio of the two standardized, independent random variables can be dealt with as a single random variable. It is assumed that both random variables are normally distributed in the absence of any data to the contrary. This assumption is based on the Central Limit Theorem.

The standardized variable denoting the cost of damage to a particular coach will be denoted by the letter C.

$$C = \frac{Y - m_y}{\sigma_y} \qquad (4\text{-}1)$$

where

m_y - average cost of damage or mean of Y

Y - damage to a coach, a random variable

σ_y - standard deviation of Y

The average values of damage m_y are known and can be obtained from Figure 4.1. The standard deviation will be estimated. The coefficient of variation, defined as the ratio of the standard deviation to the mean, will be used to obtain an estimate of the standard deviation. Equation (4-2) defines the coefficient of variation V.

$$V = \frac{\sigma_y}{m_y} \quad . \tag{4-2}$$

The standardized random variable for the cost of a mobile home will be denoted by C_o. It is defined by Equation (4-3).

$$C_o = \frac{X - m_x}{\sigma_x} \tag{4-3}$$

where

X - cost of the mobile home, a random variable

m_x - average cost of a mobile home in California

σ_x - standard deviation of the cost of a mobile home.

The density function of the ratio of two standardized normally distributed random variables is known to be a Cauchy distribution. The variable of the Cauchy distribution is denoted by the letter z. Therefore, the variable z is defined by Equation (4-4).

$$z = \frac{C}{C_o} \quad . \tag{4-4}$$

The distribution function is written in Equation (4-5) and plotted in Figure 4.6.

$$f_Z(z) = \frac{1}{\pi} \frac{1}{1+z^2} \quad -\infty \leq z \leq \infty \quad . \tag{4-5}$$

It is not possible to calculate the expected value or variance of this distribution. However, from the symmetry of the distribution about the origin it can be seen that the mean value is zero.

It is necessary to determine the value of z such that it is less than some constant a with a required probability. The choice of the value for the probability is arbitrary. But a reasonable value is one which includes most of the values of the distribution. Consequently, a probability value of 0.9 has been selected. In order to obtain the required value of z it is necessary to integrate Equation (4-5) over the interval $(-\infty, a)$. This calculation has been carried out. The integration is presented in Equation (4-6).

$$p(z \leq a) = \int_{-\infty}^{a} \frac{1}{\pi} \left(\frac{1}{1+z^2}\right) dz = 0.9 \quad . \tag{4-6}$$

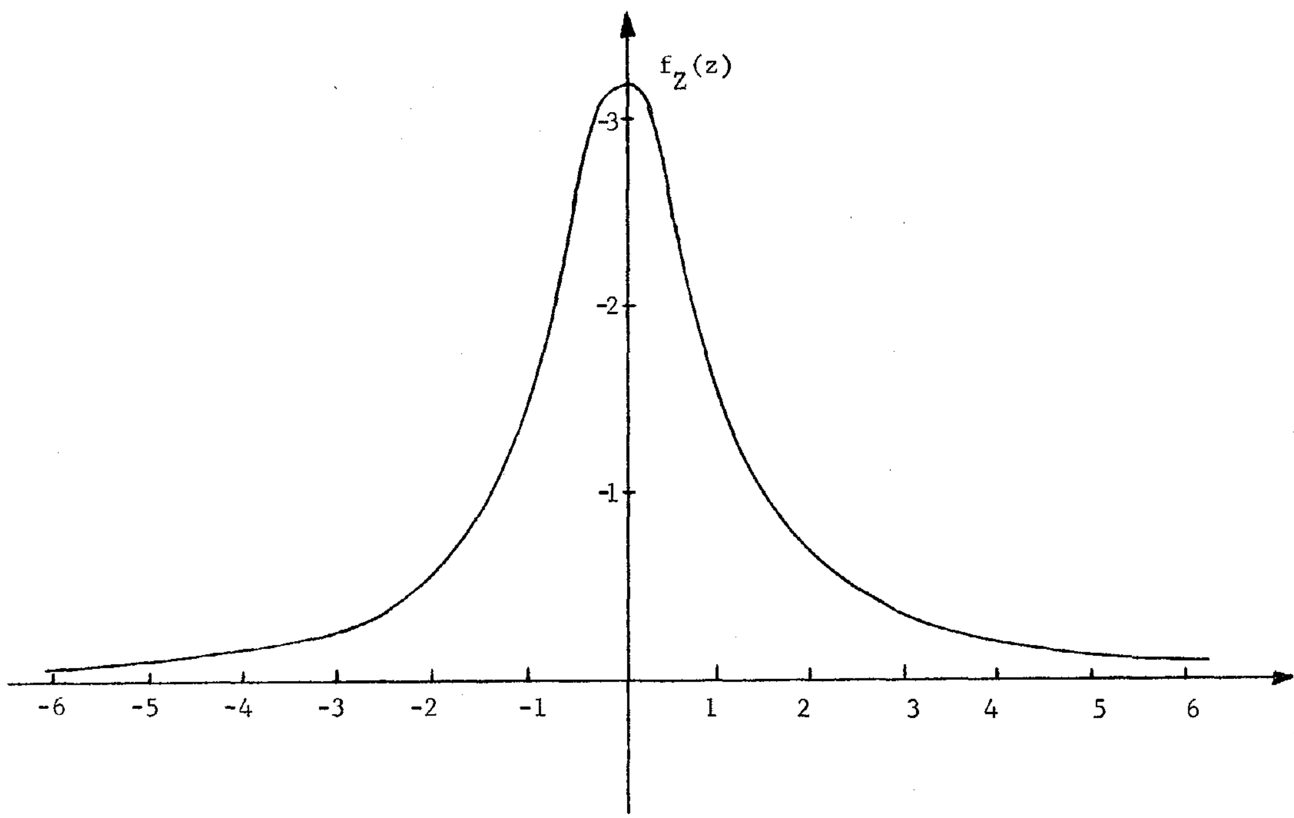
Solving for the constant a from Equation (4-6), one obtains the value given by Equation (4-7).

$$a = 3.078 \quad . \tag{4-7}$$

Thus, the Cauchy variable given by Equation (4-4) can be replaced by the value of the constant a in Equation (4-7).

FIGURE 4-6

CAUCHY DISTRIBUTION



At times, a smaller or larger confidence may be required in the value of the parameter z. Table 4.2 contains a list of the probabilities and the respective value of the parameter a.

TABLE 4.2

CAUCHY PROBABILITIES

$p(z \leq a)$	Value of the parameter a
0.80	1.376
0.90	3.078
0.95	6.314
0.99	31.820

$$3.078 = \frac{(Y-m_y)\ \sigma_y}{(X-m_x)/\sigma_x} \quad . \qquad (4\text{-}8)$$

The quantities appearing in Equation (4-8) were all discussed earlier. At this point it has not been shown how one would go about calculating the quantity m_y. This procedure will be discussed in the next section. Following is a summary of what has been developed concerning the quantities in Equation (4-8).

m_y - average cost of damage for a given earthquake category.

σ_y - standard deviation associated with Y and assumed equal to 1/4 m_y.

m_x - average cost of a mobile home in California which is \$16,000.

σ_x - standard deviation associated with the quantity X. (It is estimated to be (1/4) m_x or \$4,000).

If these quantities are substituted into Equation (4-8) a linear equation results. Equation (4-9) is the relation between the ratio of the actual cost of damage to a mobile home divided by the average cost of damage read from Figure 4.7 for a particular earthquake category and the actual cost of the mobile home.

$$X = 5195 \left(\frac{Y}{m_y}\right) + 10{,}800. \qquad (4\text{-}9)$$

For ease of computation this equation is plotted in Figure 4.7.

4.8 Analysis of the Damage Data

The following discussion concerns the determination of the quantity m_y the average cost of damage to a mobile home, as a function of the earthquake magnitude and distance from the earthquake epicenter. The available data are from the San Fernando Earthquake of 1971. It will be shown that this data can be used and applied in the San Francisco Bay Area.

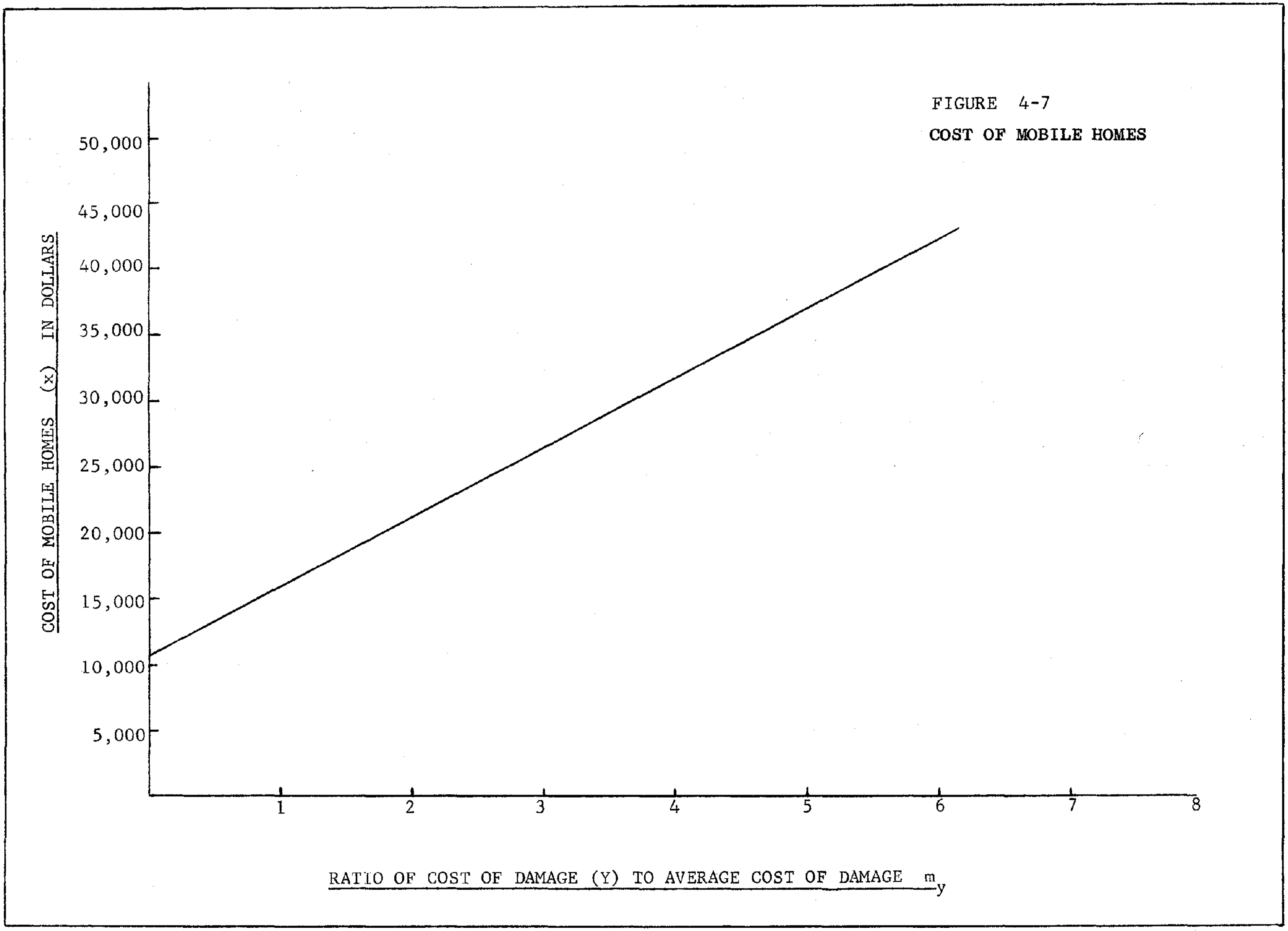


FIGURE 4-7
COST OF MOBILE HOMES

Examination of the damage data in Figure 4.1 indicates that the relationship between the average amount of damage to a mobile home and its distance from the epicenter of an earthquake is not a linear one. If the values at 20 miles and 35 miles (numbered points 1 and 2 in Figure 4.1) are discounted, a curve depending on the reciprocal of the distance raised to a power from the earthquake epicenter can be visualized passing through the remaining points. In fact, if a least squares fit of a curve of the form given in Equation (4-10) is tried, the plotted curve in Figure 4.1 results.

$$m_y = A(d^B) \quad . \tag{4-10}$$

In Equation (4-10), the quantities A and B are arbitrary constants to be determined by least squares computation, the quantity d is the distance from the earthquake's epicenter, and of course, the quantity m_y is the average cost of damage to a mobile home. The values of the quantities A and B can be calculated by least squares analysis. The resulting equation is given in Equation (4-11).

$$m_y = 21{,}081(d)^{-1.49} \quad . \tag{4-11}$$

Equation (4-11) allows calculation of the average amount of damage as a function of epicentral distance for only one Richter Magnitude of earthquake; namely, the San Fernando Earthquake of 1971. In order to

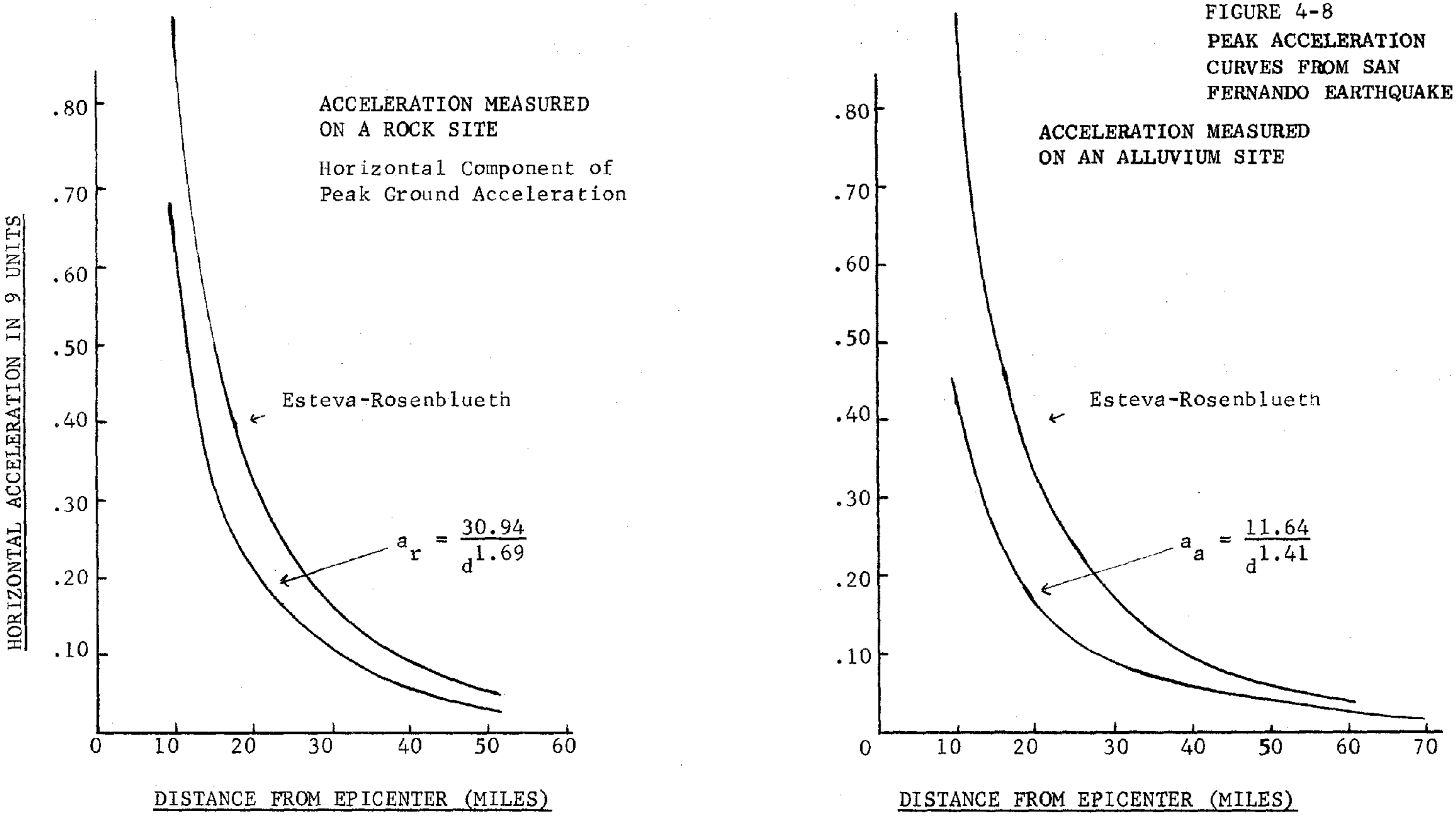


FIGURE 4-8
PEAK ACCELERATION
CURVES FROM SAN
FERNANDO EARTHQUAKE

calculate the average cost of damage as a function of Richter Magnitude, which is necessary for a decision analysis, it is necessary to use additional engineering analysis of the problem.

Intuitively, it would seem that peak ground acceleration would be the aspect of ground motion to consider as being the primary cause of the damage to mobile homes. The mobile home suffers its worst damage when it is knocked off its foundation. Since for the type of foundation used to support mobile homes there is small resistance to lateral motion, it would appear that ground acceleration is the cause of moving the coach off its foundation.

In Figure 4.8, the least squares fit to the available data for peak ground acceleration from the San Fernando Earthquake is plotted. Note that the damage data and acceleration data are from the same earthquake. The peak acceleration data were read from graphs obtained in Reference [26]. These values of acceleration were the largest values which appear on the acceleration record obtained at different distances from the earthquake's epicenter.

For comparison with analytical relations, the Esteva-Rosenblueth relation is plotted with the empirical data. This relation is based on a regression analysis of all available data throughout the world. It relates Richter Magnitude and hypocentral distance to peak ground acceleration for a firm site. The relation is given in Equation (4-12).

$$a = \frac{0.778 \exp(0.8 x RM)}{R_h^2} . \qquad (4\text{-}12)$$

The quantities appearing in the relation are defined as follows:

a = peak ground acceleration in g units

RM = Richter Magnitude

R_h = hypocentral distance in miles

The hypocentral distance can be related to epicentral distance and focal depth through Equation (4-13).

$$R_h = (R^2+h^2)^{1/2} \quad . \tag{4-13}$$

The quantities R and h are defined as follows:

R = epicentral distance in miles

h = focal depth in miles

The Esteva-Rosenblueth relation overestimates the peak ground acceleration data from the San Fernando Earthquake. This is especially true near the origin of the earthquake. However, it appears from Figure 4.8 that the shape of the curve is a reasonably good fit to the data.

Therefore, a least squares fit to the peak acceleration data will be made using a relation having the same shape as Equation (4-10). For the analysis of the data obtained in bedrock Equation (4-14) results.

$$a_r = 30.94(d)^{-1.69} \quad . \tag{4-14}$$

For the peak acceleration data obtained in alluvium Equation (4-15) results.

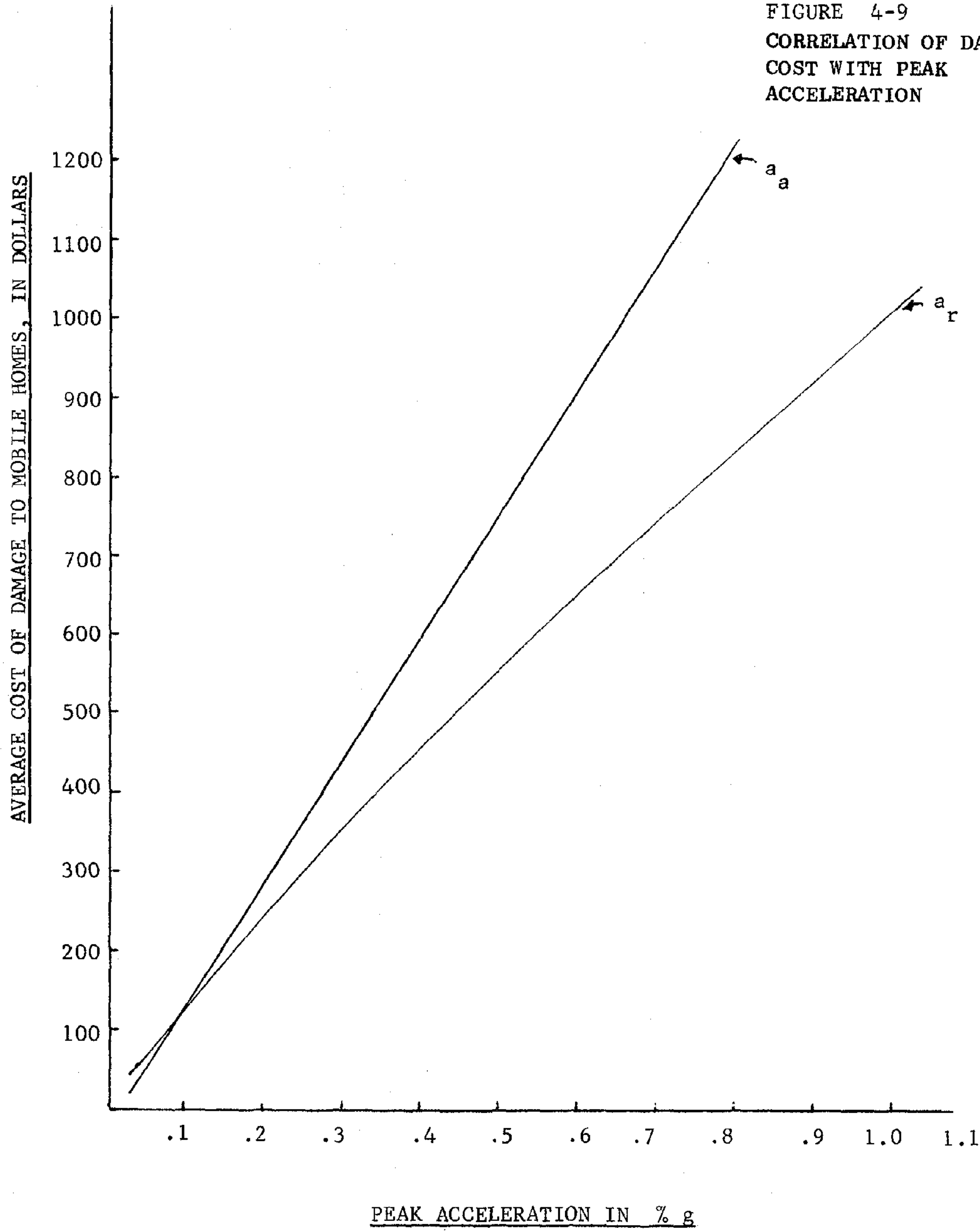
$$a_a = 11.64(d)^{-1.41} \quad . \qquad (4\text{-}15)$$

The quantities a_r and a_a are the peak ground acceleration in g units for bedrock and alluvium, respectively. Note that the exponent B for the damage data (see Equation (4-11)) and for the peak acceleration (see Equations (4-16) and (4-17)) are very close in magnitude. In order to investigate this observation further, average damage values are plotted versus peak ground acceleration in bedrock and alluvium in Figure 4.9. For the plot against peak acceleration in alluvium a nearly straight curve results. Thus, the average cost of damage to mobile homes in linearly related to peak ground acceleration in alluvium and nearly linearly related to peak ground acceleration in bedrock.

Note that peak ground acceleration at a given distance from the earthquake epicenter is greater in bedrock than in alluvium. Perhaps the alluvium deposit is filtering out the frequencies in the ground motion which correspond to the larger peak ground acceleration values.

Reasonable confidence has been established in the relation between average cost of damage to a mobile home and peak ground acceleration. Consequently, part of the Esteva-Rosenblueth relation given in Equation (4-12) will be used to develop the average cost of damage curves for the four categories of earthquakes. Equation (4-11) was calculated for an earthquake with Richter Magnitude of 6.6. To use this equation for other

FIGURE 4-9
CORRELATION OF DAMAGE COST WITH PEAK ACCELERATION



earthquake magnitudes it is necessary to first divide it by $\exp[(0.8)(6.6)]$ and then multiply by $\exp[(0.8)RM]$, where the appropriate value of RM has been substituted. When this is done Equation (4-16) results.

$$m_y = \frac{21,081}{d^{1.49}} \left(\frac{\exp(0.8)(RM)}{\exp(0.8)(6.6)}\right) \quad . \qquad (4\text{-}16)$$

Using this expression and the average values of Richter Magnitude in the range for each earthquake category Figure 4.10 can be constructed. The selected value of RM, the Richter Magnitude, for each earthquake category is given in Table 4.3.

TABLE 4.3

Category	Range of Richter Magnitude	Selected Value of Richter Magnitude
Large	$RM \geq 6.5$	7.0
Moderate	$5.5 \leq RM \leq 6.4$	6.0
Small	$4.5 \leq RM \leq 5.4$	5.0
Very Small	$3.5 \leq RM \leq 4.4$	4.0

The curves constructed in Figure 4.10 are assumed to be applicable to the San Francisco Bay Area. Since they are average values and there are numerous factors, notably soil conditions, influencing the damage values, one may expect that they are reasonable approximations to the values which would be produced by an earthquake occurring in the San Francisco Bay Area.

FIGURE 4-10
TOTAL AVERAGE PAID LOSS AS A FUNCTION OF EPICENTRAL DISTANCE

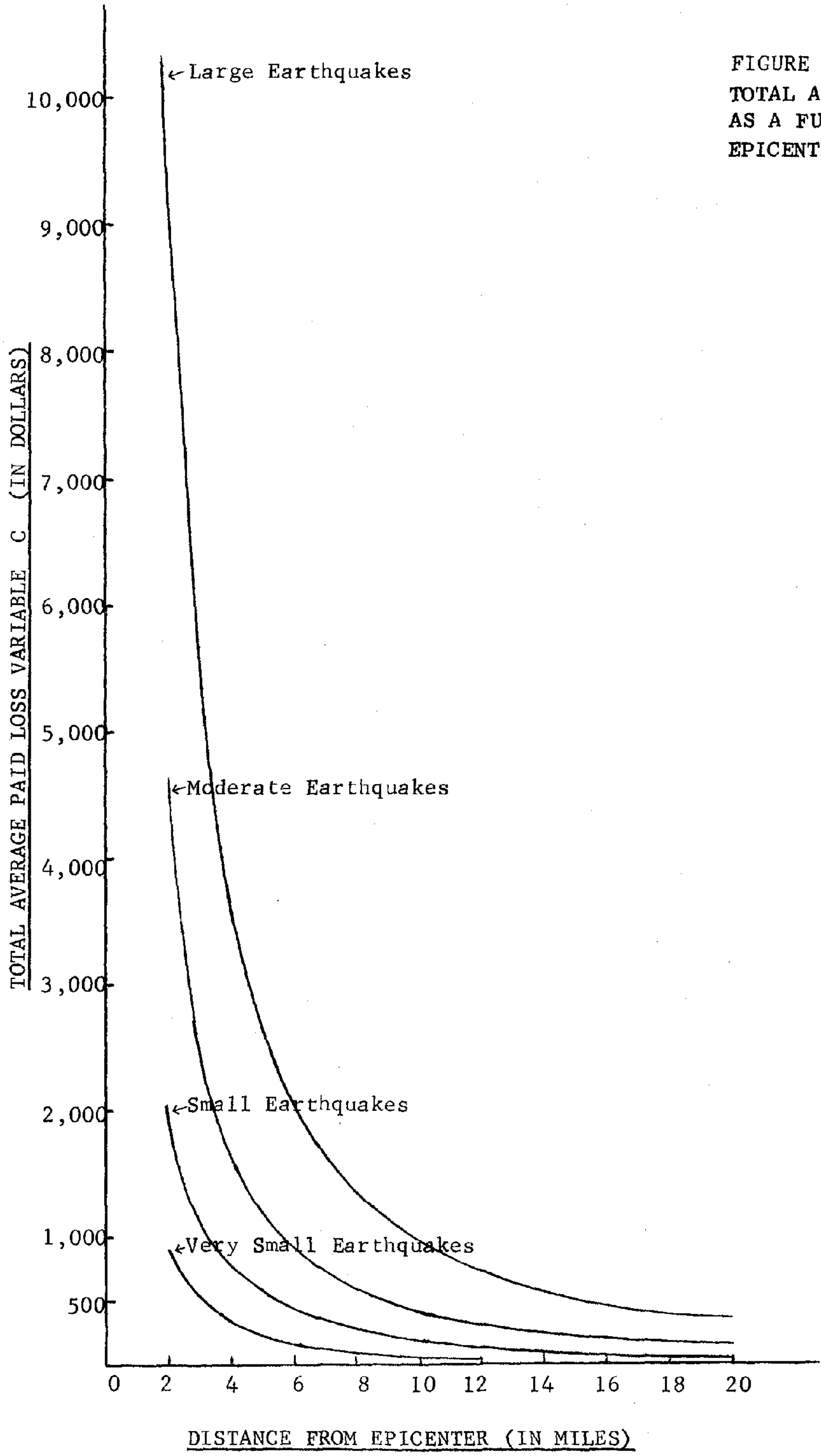


Figure 4.11 is a plot of the average total paid loss versus Richter Magnitude. As might be expected, the average amount of damage increases with increasing values of the Richter Magnitude.

These damage values include damage to the coach, coach contents, awning and skirt and releveling. The values of total damage have been scaled down for earthquakes of smaller magnitudes but there is still a contribution to the total damage from each category. It seems reasonable to assume that for the small and very small earthquake categories that the mobile home would not be knocked off its foundation. Hence, these latter two curves probably overestimate the total average damage.

4.9 Transition Rate Probability Matrix

With the completion of the damage data analysis it is time to consider the trajectory of the mobile home in its seismic environment. Thus, the probability that the mobile home moves from the undamaged state to the damaged state for each category of earthquake will be calculated. The discussion here will consider the calculation of the transition rate matrix A. The procedure for this calculation was presented in Chapter III. There it was shown that the matrix $[A]$ could be calculated from Equation (3-13) which is repeated below.

$$[A] = [\Lambda]([P]-[I]) \quad .$$

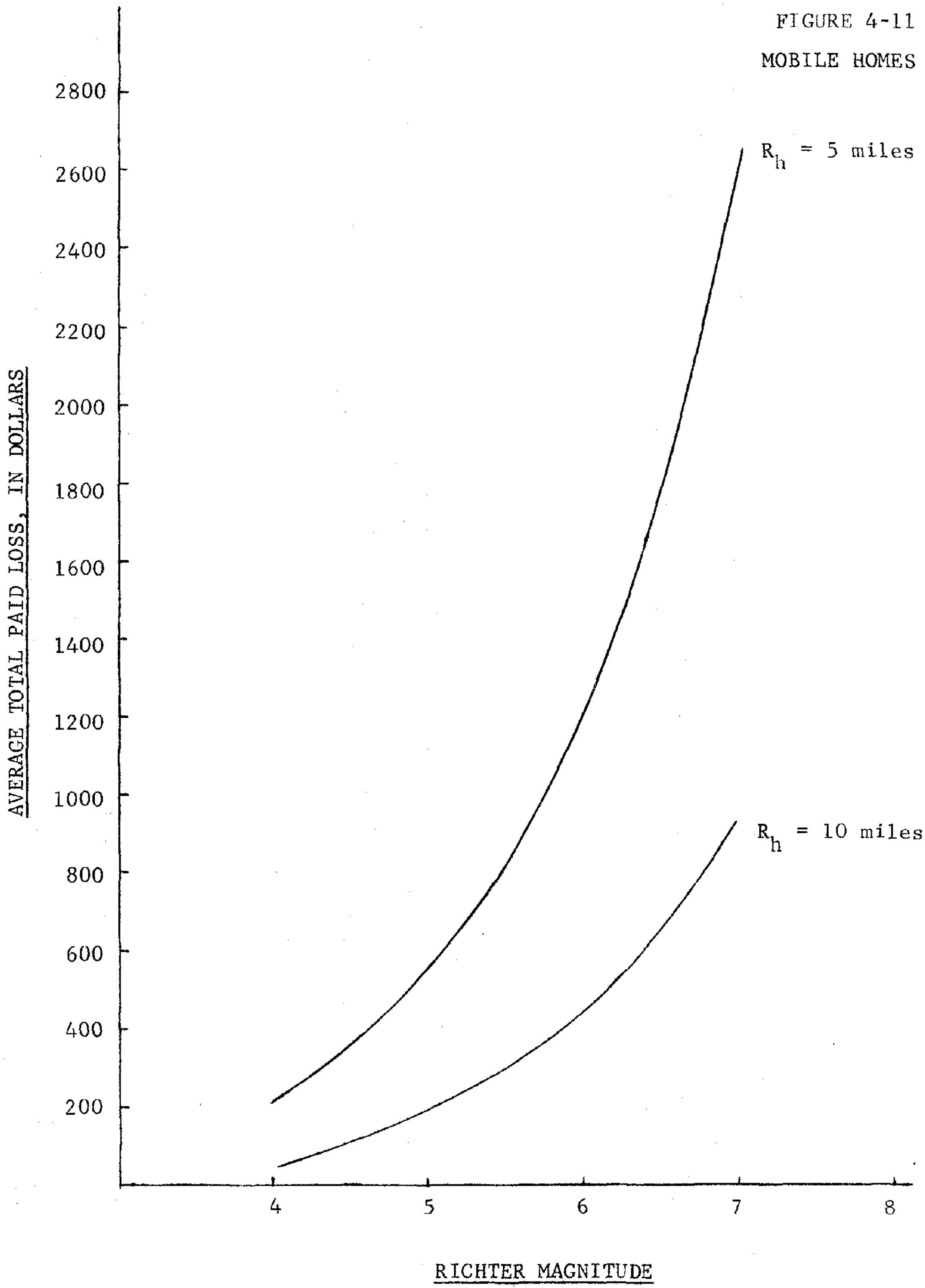


FIGURE 4-11
MOBILE HOMES
R_h = 5 miles
R_h = 10 miles
AVERAGE TOTAL PAID LOSS, IN DOLLARS
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
4
5
6
7
8
RICHTER MAGNITUDE

The discrete time transition matrix $[P]$ will be considered first. For the two-state model considered here $[P]$ is a two-state matrix given by Equation (4-17).

$$[P] = \begin{bmatrix} 1-p_{12} & p_{12} \\ 1-p_{22} & p_{22} \end{bmatrix} . \qquad (4\text{-}17)$$

The rows of this matrix must add to 1.

The probability of interest is p_{12}. This quantity is the probability of moving from state one--the undamaged state--to state two--the damaged state. These transition probabilities are considered to be independent of the distance from the earthquake's epicenter and earthquake's magnitude. There are two reasons for making this assumption. First, the average damage values vary with epicentral distance and secondly, the available data in Reference [19] indicates that such an assumption is warranted.

TABLE 4.4

Epicentral Distance	p_{12}
7	.750
8	.625
9	.882
9	.500
10	.575
10	.700
11	.667
11	.333
15	.625
19	.286
17	.147
20	.200
35	1.000

Table 4.4 was prepared from the data in Reference [19]. Note that the data vary considerably. On the average the transition probability p_{12} is about 0.56. If the value at 35 miles is neglected, and this is reasonable since it represents only one insured unit, the value of p_{12} is 0.52. If the values at 20 miles and 35 miles are neglected then the value for p_{12} is about 0.55. This latter value seems most reasonable for the available data.

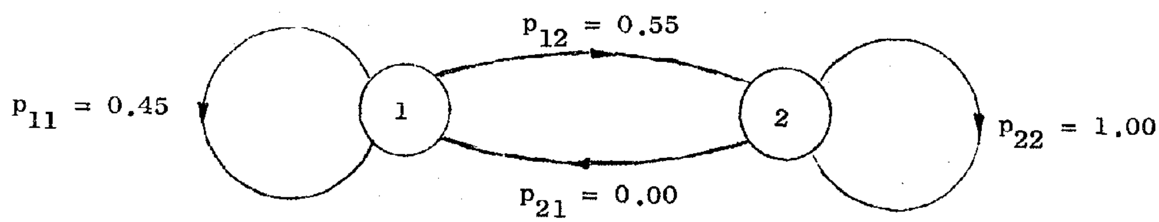
This two-state model considers the process up to the point where the mobile home enters the damaged state which is called state 2.

The mobile home remains in this state until it is repaired. At this time the process must begin anew.

The transition probability matrix given by Equation (4-18) is calculated from Equation (4-17)

$$[P] = \begin{bmatrix} 0.45 & 0.55 \\ 0 & 1.00 \end{bmatrix} . \qquad (4\text{-}18)$$

These probabilities are conditioned on the fact that an earthquake occurs. A schematic of the transition matrix is given in Figure 4.12.



(4-19)

FIGURE 4.12

It is assumed that the transition to the damaged state takes place randomly and is exponentially distributed.

Using Equation (3-15) the continuous-time transition matrices for the mobile home will be calculated. The first calculation is for the category of large earthquakes.

The transition rate matrix [A] can be calculated from Equation (3-13).

$$A = [\Lambda] \left([I]-[P]\right) .$$

The matrix $[\Lambda]$ is a diagonal matrix of the coefficients λ_i from the waiting time functions. The waiting time functions describe the time that the system remains in the state before making the transition. They were defined by Equation (3-14)

$$\omega_s = \lambda_s e^{-\lambda_s \tau}$$

$$\omega_f = \lambda_f e^{-\lambda_f \tau} \quad .$$

The subscripts refer to whether the earthquake has occurred recently--success or whether it has not--failure. Thus, the exponentially distributed time to the first transition of the system is determined by the time to the first occurrence for the earthquake. For the case in question the waiting times are the same irrespective of the starting state.

The average value of the waiting time density functions ω_i is $1/\lambda_i$. Hence, for large earthquakes the coefficients are equal to those given in Equation (4-20). These values were calculated initially in Table 2.7.

$$\lambda_s = \lambda_f = 0.029 \tag{4-20}$$

Consequently, for large earthquakes the transition rate matrix $[A]$ can be calculated and is given by Equation (4-21).

$$[A] = \begin{bmatrix} .029 & 0 \\ 0 & .029 \end{bmatrix} \begin{bmatrix} -.55 & .55 \\ 0 & 0 \end{bmatrix} = \begin{bmatrix} -.016 & .016 \\ 0 & 0 \end{bmatrix} . \tag{4-21}$$

Knowing the transition rate matrix [A] one can calculate the trajectory of the system in time. The exponential transform of the continuous-time transition matrix is given by Equation (3-15) and is repeated below.

$$\Phi^e(s) = (s[I]-[A])^{-1} \quad .$$

The quantity in brackets can be calculated using the results in Equation (4-21).

$$s[I]-[A] = \begin{bmatrix} s + .016 & -.016 \\ 0 & s \end{bmatrix} \quad .$$

The inverse of this equation is calculated using the methods of the algebra of matrices.

$$\Phi^e(s) = \begin{bmatrix} \dfrac{1}{s + .016} & \dfrac{.016}{s(s + .016)} \\ 0 & \dfrac{1}{s} \end{bmatrix} \quad . \qquad (4\text{-}22)$$

Taking the exponential transform of this equation and writing the result as the sum of two matrices, a steady-state and transient state matrices, Equation (4-23) results.

$$\Phi(t) = \begin{bmatrix} 0 & 1 \\ 0 & 1 \end{bmatrix} + e^{-.016t} \begin{bmatrix} 1 & -1 \\ 0 & 0 \end{bmatrix} \quad . \qquad (4\text{-}23)$$

The first matrix is the steady-state matrix and the second matrix is the transient or time dependent matrix. For large values of time the second matrix will approach zero. The system will reach the damaged state ultimately and cannot return to the undamaged state.

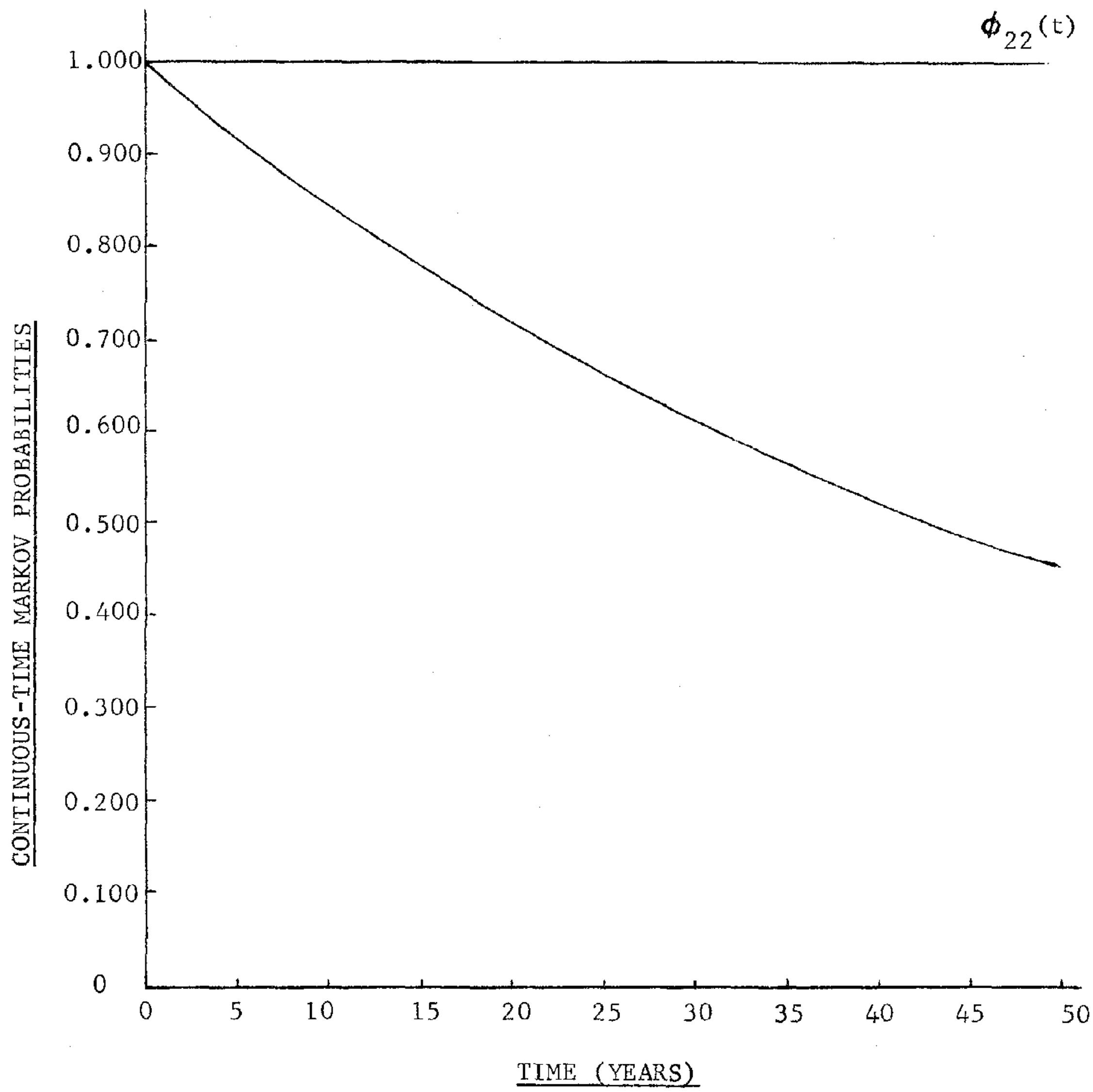
This matrix $\Phi(t)$ is only for the consideration of large earthquakes. Note that $\Phi(t)$ reduces to the identity matrix at time t equal to zero. In Figure 4.13 the probabilities $\phi_{11}(t)$ and $\phi_{22}(t)$ are plotted. For example, consider the probability $\phi_{11}(t)$. With the passage of time this probability decreases. Consequently, the mobile home will eventually make a transition out of the undamaged state--state one, to the damaged state--state two. The time required for this transition depends on the seismicity of the region. The probability $o_{22}(t)$ is equal to one. Therefore, state two is a trapping state. When the mobile home reaches this state it remains there.

The same procedure can be carried out for the other three categories of earthquakes. The resulting plots of the diagonal probabilities are comparable to the plot for large earthquakes. Only the components $\phi_{11}(t)$ and $\phi_{22}(t)$ of the transition matrix are plotted since their values determine the entire matrix $\Phi(t)$.

For moderate earthquakes the transition rate matrix $[A]$ is given by Equation (4-24).

$$[A] = \begin{bmatrix} .070 & 0 \\ 0 & .070 \end{bmatrix} \begin{bmatrix} -.55 & .55 \\ 0 & 0 \end{bmatrix} = \begin{bmatrix} -.0385 & .0385 \\ 0 & 0 \end{bmatrix} . \qquad (4\text{-}24)$$

FIGURE 4-13
TRANSITION PROBABILITIES
FOR LARGE EARTHQUAKES



The resulting transition matrix follows in Equation (4-25)

$$\Phi(t) = \begin{bmatrix} 0 & 1 \\ 0 & 1 \end{bmatrix} + e^{-.0385t} \begin{bmatrix} 1 & -1 \\ 0 & 0 \end{bmatrix} . \tag{4-25}$$

For small earthquakes the transition rate matrix [A] is given by Equation (4-26).

$$[A] = \begin{bmatrix} .789 & 0 \\ 0 & .789 \end{bmatrix} \begin{bmatrix} -.55 & .55 \\ 0 & 0 \end{bmatrix} = \begin{bmatrix} -.434 & .434 \\ 0 & 0 \end{bmatrix} . \tag{4-26}$$

The continuous-time transition matrix is given by Equation (4-27).

$$\Phi(t) = \begin{bmatrix} 0 & 1 \\ 0 & 1 \end{bmatrix} + e^{-.439t} \begin{bmatrix} 1 & -1 \\ 0 & 0 \end{bmatrix} . \tag{4-27}$$

Finally, the category of very small earthquakes is considered. Equation (4-28) gives the transition rate matrix [A].

$$[A] = \begin{bmatrix} .975 & 0 \\ 0 & .975 \end{bmatrix} \begin{bmatrix} -.55 & .55 \\ 0 & 0 \end{bmatrix} = \begin{bmatrix} -.536 & .536 \\ 0 & 0 \end{bmatrix} . \tag{4-28}$$

The continuous-time transition matrix follows from Equation (4-28).

$$\Phi(t) = \begin{bmatrix} 0 & 1 \\ 0 & 1 \end{bmatrix} + e^{-.536t} \begin{bmatrix} 1 & -1 \\ 0 & 0 \end{bmatrix} . \tag{4-29}$$

If in Equations (4-25), (4-27), and (4-29) the elements $\phi_{11}(t)$ and $\phi_{22}(t)$ are plotted a graph similar to that of Figure 4.12 is obtained. Only the components $\phi_{11}(t)$ and $\phi_{22}(t)$ are plotted since their values determine the entire matrix $\Phi(t)$. Note that in each case the damaged state-state two is a trapping state.

4.10 Calculation of the Risk

In this section, the risk associated with owning a mobile home located in a seismically active region will be calculated. The mobile home will be considered as being in one of two possible states. Either it is in state one--the undamaged state--or it is in state two--the damaged state. After the mobile home is damaged by an earthquake it is assumed that it is repaired. The risk analysis at this point must begin again from the undamaged state.

The analysis will be applied to a mobile home which is located at an epicentral distance of 20 miles. The focal depth will be taken as eight miles. This depth is representative of earthquakes in the San Francisco Bay Area. Furthermore, with this choice of focal depth Figure 4.10 can be used directly. With increasing distance from the earthquake's epicenter the level of the risk will decrease.

The amount of risk will be quantified by the quantity $C_1(t)$. The meaning of this quantity was discussed in Chapter III. It will suffice to define it here. For a system which begins in state one, the quantity $C_1(t)$ is the expected cost in the time t remaining. Since there are four categories of earthquakes and an earthquake in any category can damage a mobile home it is necessary to calculate the quantity $C_1(t)$ for each. The total risk is obtained by summing the four values of $C_1(t)$ at any time t.

The first calculation will consider only the cost c_{12}. This is the cost of damage associated with a transition from state one to state two during an earthquake. Discussion of the significance of this quantity was given in Section 3.6. Once the mobile home has entered state two it is assumed to remain there. It is then repaired and it is assumed that the repair restores it to its original condition. Consequently, the boundary conditions are put equal to zero.

Table 4.5 summarizes the needed input to this model for each category of earthquake. The probability a_{12} is obtained in Section 4.9. The average cost m_y was calculated from Equation (4-16) From Equation (4-9) the actual cost c_{12} was calculated. In Equation (4-9) the cost c_{12} is denoted by the letter Y.

TABLE 4.5

Earthquake Category	Probability a_{12}	Average Cost m_y (dollars)	Actual Cost c_{12} (dollars)
Large	0.0160	334.45	463.21
Moderate	0.0385	150.28	208.14
Small	0.4340	67.52	93.52
Very Small	0.5360	30.34	42.00

The governing equations are obtained from Equation (3-29) after the appropriate quantities have been set equal to zero; also note that $a_{11} = -a_{21}$ in this case. Equations (4-30) and (4-31) are the governing equations for the problem. It is assumed that the system begins in state one--the undamaged state--and therefore, the quantity $C_2(t)$ is set equal to zero.

$$C_1(t) = \frac{c_{11}+a_{12}c_{12}}{\beta+a_{12}} \left\{1-\exp[-(\beta+a_{12})]t\right\} \qquad (4-30)$$

$$C_2(t) = 0 \quad . \qquad (4-31)$$

For the first calculation of risk the quantity c_{11} is set equal to zero. This implies no insurance. Equation (4-30) must be calculated for each category of earthquake. A plot of these equations is presented in Figure 4.13.

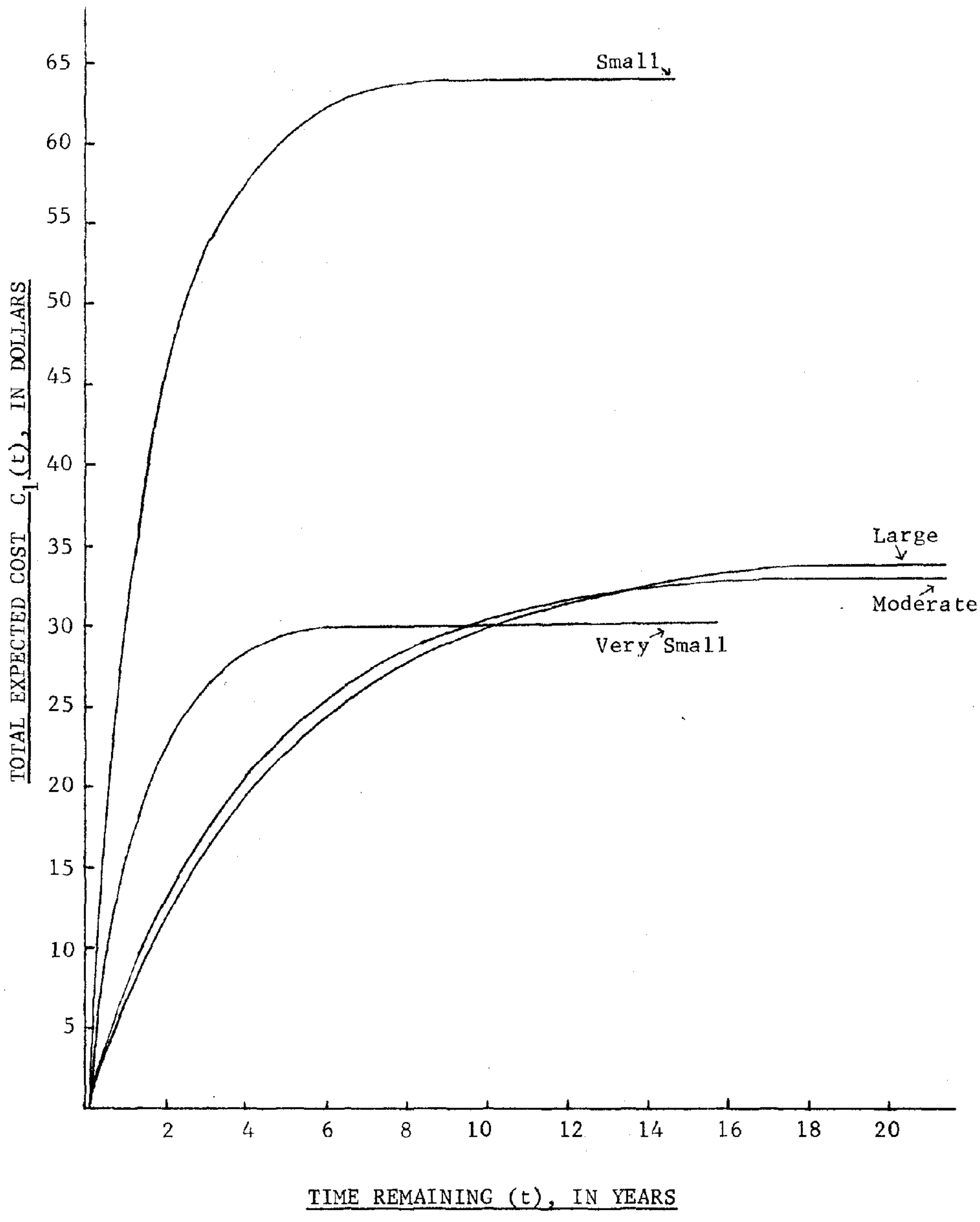
Figure 4.13 is a plot of the risk facing a mobile home owner as a function of the time remaining in the useful life of the mobile home. The risk curves are plotted for each earthquake category. In order to obtain the total risk the ordinates at a particular time must be added together. For example, with a period of 15 years remaining the mobile home owner faces a total risk of about \$162.52. This works out to be about \$10.83 per year. For times close to the termination point, the total risk will be smaller. A plot of the total risk is presented in Figure 4.15.

For large values of time the curves in Figure 4-14 approach limiting values. Equation (4-30) shows that for large values of time, the exponential function is nearly zero. Hence, the value of the quantity $C_1(t)$ will approach a limiting value given by Equation (4-32).

$$C_1(t) = \frac{c_{11}+a_{12}c_{12}}{\beta+a_{12}} \quad . \qquad (4-32)$$

In view of the amount of risk taken, the insurance cost of \$15.00 for a period of three years (See Section 4.4) is a very reasonable value.

FIGURE 4-14
SEISMIC RISK BY
EARTHQUAKE CATEGORY



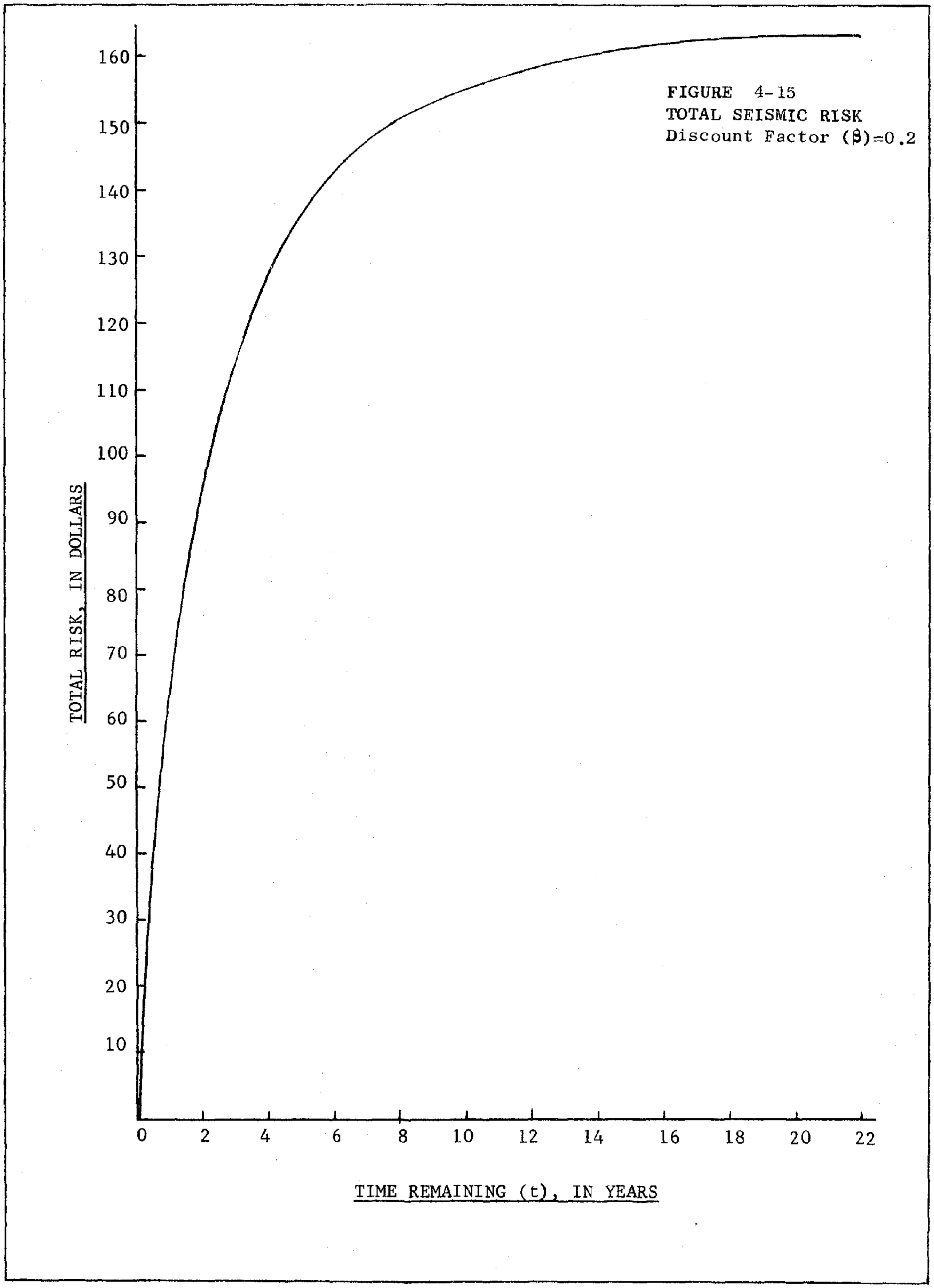


FIGURE 4-15
TOTAL SEISMIC RISK
Discount Factor (β)=0.2
TOTAL RISK, IN DOLLARS
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
2
4
6
8
10
12
14
16
18
20
22
TIME REMAINING (t), IN YEARS

Recall that Figure 4.14 is for a mobile home located 20 miles from the epicenter of an earthquake. For mobile homes located nearer to the earthquake's epicenter the risk will be larger. If the insurance premium for earthquake protection were placed at normal interest rates it could not generate the value of the risk.

Figure 4.16 presents two curves. Each curve is the risk associated with large earthquakes. One curve is with a discount factor (β) of 0.2 and a second curve for a discount factor of 0.4. The value of the discount factor has a large influence on the magnitude of the risk. This implies that money to be paid out in the future has a smaller value at the present time for larger discount factors. This factor is a consequence of the rate at which interest is paid. For example, for high discount rates and hence high interest rates, a sum of money placed at interest at the present time would generate a larger sum of money.

Consider the effect of improving the foundation of the mobile home so that resistance to lateral forces could be developed. In the majority of cases of damage to mobile homes, the mobile home was displaced from its foundation. Concrete piers supporting the mobile home are not anchored in the ground nor are they anchored to the frame of the mobile home. The concrete piers provide the means of support to the coach since its wheels are removed once it has been located. If screwjack fasteners at the top of the pier were made to fasten to the coach's frame and the concrete pier was made longer so that a portion of its length could be sunk into the ground, lateral force resistance could be developed. This would insure that the mobile home remains on its foundation during an earthquake.

Table 4.1 shows that 43% of the damage from earthquakes was to the

FIGURE 4-16
EFFECT OF THE DISCOUNT FACTOR ON SEISMIC RISK (LARGE EARTHQUAKES)

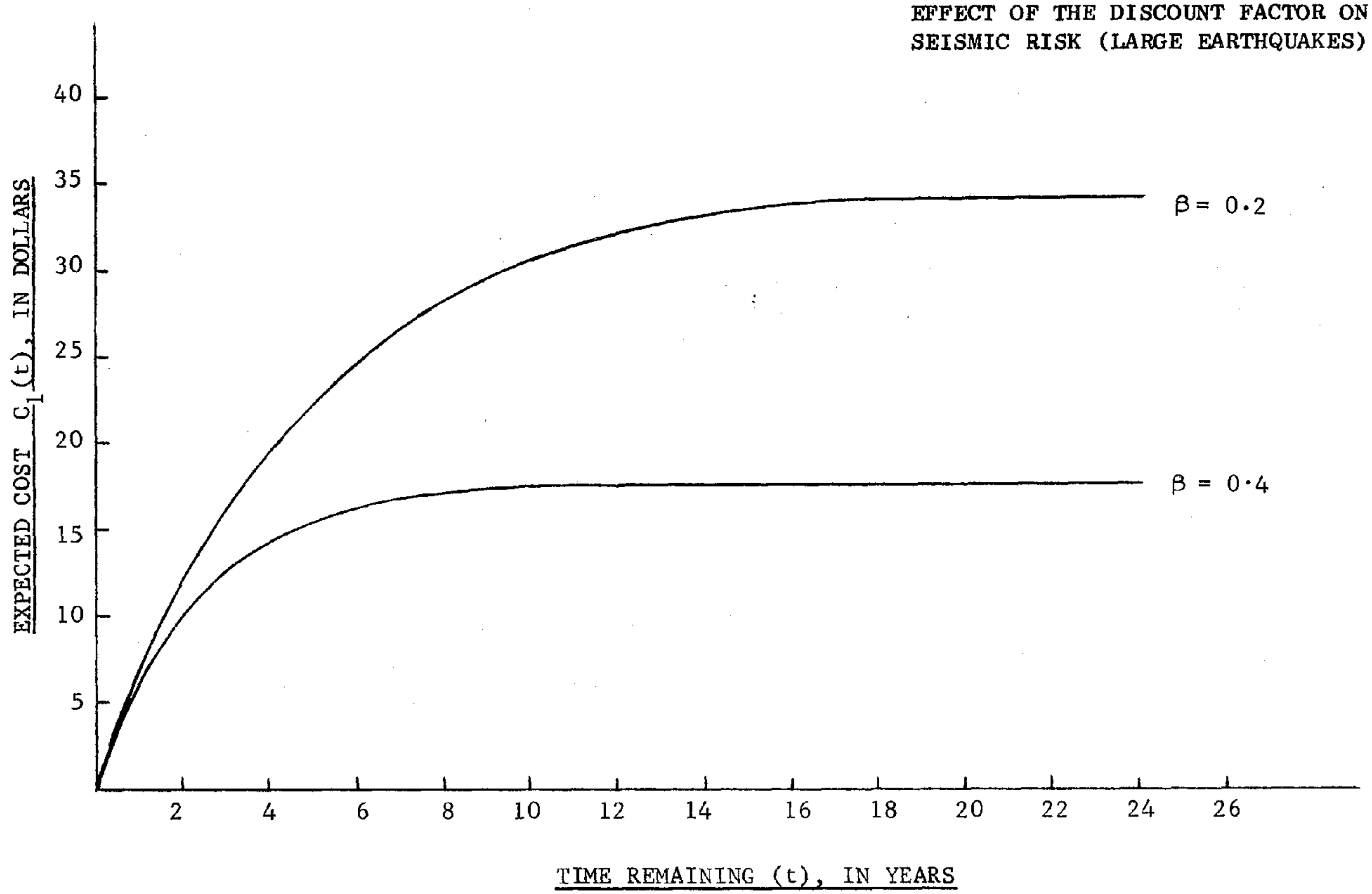
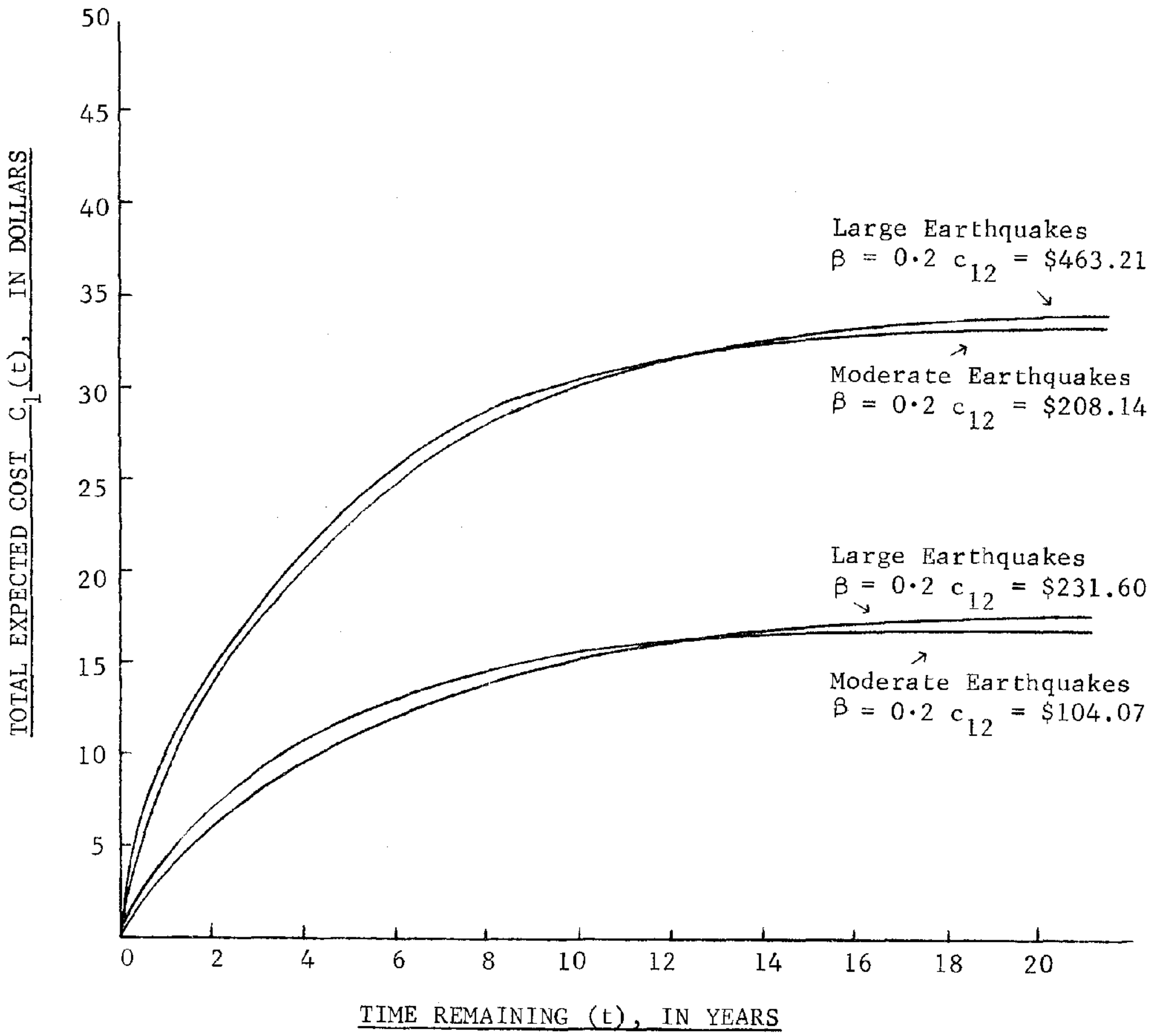


FIGURE 4-17
SEISMIC RISK WITH
IMPROVED FOUNDATION



coach itself. Part of this damage was caused by the concrete piers penetrating the plywood floor of the coach when it toppled off its foundation. In addition, if the coach was displaced from its foundation wrinkling of its thin aluminum skin generally resulted. Furthermore, it is not unreasonable to suppose that damage to the contents of the mobile home and its awning and skirt would be reduced. Probably, costs of releveling a coach could be eliminated. A 50% reduction in damage might not be an unreasonable amount.

The cost of producing such a foundation would probably be about $50.00 per coach. The concrete pier with fastener could be made standard and used with any type or size of mobile home. The cost of the improved foundation could be borne by the owner of the mobile home or by the owner of the mobile home park.

If each of the costs of damage c_{12} were reduced by a factor of one-half the risk would be reduced. In this case, the risk would be reduced by one-half. For a period of 15 years remaining, the risk would be $81.26. This would amount to $5.42 per year.

In Figure 4.17, the risk for large and moderate earthquakes is presented with two values for the cost of damage. Note that the risk has been reduced by one-half for the problem considered. Additional plots for the categories of small and very small earthquakes could also be made with an equivalent reduction in the level of risk.

4.11 Conclusion

In this chapter an analytical method for the calculation of the risk associated with mobile homes in a seismic environment has been illustrated.

It has also been shown how factors such as the discount rate (β) affect the level of risk.

This analytical calculation of risk could best be used by an insurance company attempting to set its rates for insuring a mobile home in a seismic environment. No longer would it be necessary to depend on the past history of performance of a mobile home to establish the insurance premium. An analytical investigation, such as the one presented here, could be used to estimate the risk. Furthermore, changes in the discount rate and foundation design could be incorporated rapidly in the risk calculation and these changes reflected immediately in the cost of the insurance premium.

The cost of the insurance premium for earthquake protection could be put equal to the risk. For example, if there were no improvements made to the foundation of the mobile home, the insurance premium should be about $10 per year. This rate is about twice the value which is presently charged.

V MODERN HIGH-RISE CONSTRUCTION

5.1 Introduction

Where the cost of land is high or where land is not available and where a large amount of floor space in a building is required, the solution is to construct a multi-story building. Generally, the design requires that a choice be made between a reinforced concrete structure or a steel-framed structure. The basis for such a decision is economics. The choice of a structural system is generally determined by the one which allows construction of the building at the lowest cost.

The analysis will be carried out using the Markov Decision Model presented in Chapter III. The transition matrix will be independent of the distance from the earthquake and the magnitude of the earthquake. These factors will be reflected in the costs of damage. A great deal of variation can exist between seemingly identical buildings when exposed to an earthquake. They may have the same structural system but engineers have different ideas about building design and different philosophies on how a structure should resist seismic loadings. Although every structure will meet the provisions of the applicable building code, the difference in the design philosophies used in construction of the building could appreciably affect a structure's performance during an earthquake.

It should be noted that design alternatives generally include more than simply the choice of structural scheme. The engineer has a choice between different structural materials or a combination of them. It must be assumed, in order to make the problem tractable, that the quality of materials used in the construction of the investigated buildings is comparable.

5.2 Cost of Construction

The cost of a modern high-rise building varies considerably. A reasonable range would be $25.00 to $35.00 per square foot of building floor space. These figures are applicable to buildings with steel frames or reinforced concrete frames.

The variability in the cost of construction of a high-rise building depends on the type of lighting fixtures, facades installed and whether or not air-conditioning is installed.

5.3 Type of Damage

From inspection of the many photographs in references [19,28,29] showing damage to reinforced concrete structures, the following observations were made.

The investigated buildings were all designed to be earthquake resistant and met the basic building code philosophy. However, there are instances, especially in reinforced concrete structures, where vital structural elements failed. This might imply that either the ground motion at the site was severe or that the building code is unconservative.

There is no design requirement for vertical motion. The building is designed to support its weight with a margin of safety measured by the factor of safety. This is generally considered to be more than adequate. However, during an earthquake displacements can be large.

In this case, large moments will be created by the fact that the forces created by the weight of the building above the first floor are no longer plumb with the structural axis of the first floor columns. This is particularly true if the duration of the earthquake is large. A resonance condition could develop in this case leading to large displacements.

In this section, the costs of damage for multi-story buildings exposed to earthquakes is developed. Again only average costs of damage will be considered. However, it should be recognized that there are four significant factors which could increase or decrease the costs of seismic damage. These factors are the following.

1. The distance between the origin of the earthquake and the construction site.
2. The depth and type of soil under the structure.
3. The duration of the earthquake. This factor has a large influence on whether a resonance in the structure can be developed.
4. The value of the fundamental period of vibration and the degree of damping in the structure.

Ideally, a statistical correlation between the response of the structure and the associated cost of seismic damage should be obtained. The lack of sufficient data precludes such an investigation at this time. Consequently, only average values of seismic damage will be calculated.

When a structure sustains seismic damage a portion of the monetary investment in the structure is lost. Additional resources must be expended to repair the damage. This represents a tangible loss to the owners of the structure. In addition, there is an intangible loss. There may be injury to the occupants of the structure or death. Furthermore, repairs to the building may never restore it to its pre-damaged state. Only the tangible loss to structures will be discussed here. However, intangible losses could be included if they could be estimated. The viewpoint taken is directed to long-term planning.

5.4 Scope of the Investigation

This investigation will be based on the published findings from two earthquakes. The earthquakes considered here are listed below.

1. Venezuela Earthquake of 1967
2. San Fernando Earthquake of 1971

Damaged structures will be classifed by story height. This is necessary because building height is one of the important factors which determines the fundamental period of vibration. Table 5.1 lists the classifications. Sufficient data are available for buildings between five and nineteen stories only.

TABLE 5.1

Group	Story Height
A	5-8
B	9-14
C	15-19

Methods of construction depend on the available materials, the climate of the region and the expertise of the labor force. These factors make it extremely difficult to compare similar structures constructed in dissimilar regions. Consequently, construction will be classified in a general fashion by noting whether workmanship or quality of materials was qualitatively acceptable.

The design strategy will be classified using the UNIFORM BUILDING CODE classifications. Thus, a building will be classified as having been constructed using a Zone 1, 2, or 3 strategy.

It is well-known that the response of a multi-storied building depends on three basic factors. They are the stiffness and mass of the building, the type of foundation, and the physical properties of the underlying soil. Stiffness and mass of the building require no comment. The type of foundation refers to whether the foundation is shallow or deep and whether piles are used. The properties of the underlying soil are important because it has been shown that during an earthquake the response of the structure is not determined by its properties alone. The integrated soil-foundation-structure system must be considered. This is an extremely complex problem.

It is not possible to consider all of these factors here. Only an average cost of damage will be considered. The factors discussed are implicitly included in the damage costs. It is assumed that since these factors are random variables and random in any given region it is possible to calculate an average value which will characterize any given region. The usual practice of architects and architect-engineers in estimating the potential cost of a structure is to first categorize the costs. Four categories are generally sufficient to define potential costs. Categories including structural, architectural, utilities and contents, may be combined to obtain the total cost.

A discussion of each of the two earthquakes given earlier will be discussed. Particular attention will be given to the type of construction prevalent in each area. A rather long discussion of the Caracas, Venequela Earthquake of 1967 is presented because of the different design and construction techniques used in this region. Furthermore, a discussion of the seismic features of the region are included for comparison with the information presented in Chapter II concerning the San Francisco Bay Area.

1967 CARACAS VENEZUELA EARTHQUAKE

Northern Venezuela is a seismically active region. Its past history contains numerous earthquake occurrences. A branch of the system of faults forming the Circum-Pacific Earthquake Belt extends across Columbia into Venezuela and hence, continues under the Caribbean Sea. The major fault of this system in Venezuela is known as the Bocono Fault. It begins in the Andes Mountains and extends across northwest Venezuela to the

Caribbean Sea. Upon entering the sea it continues parallel to the coast and it is called the Sebastian Fault. There is a large number of smaller faults which complement the larger Bacono Fault in Venezuela. They are situated in the vicinity of the city of Caracas composing the Avila Fault Zone. Generally, only small earthquakes occur in this zone.

The city of Caracas, founded in 1567, is the capital of Venezuela. Caracas today has a population in excess of 2 million people. The city is modernizing rapidly. This is particularly true of the construction taking place in the city. Numerous high-rise buildings have been built in the past 10 years. Presently, there are about 1,000 high-rise buildings between 10 and 30 stories in the city.

Caracas is situated in a valley surrounded by moderately high mountains. Draining water from the mountains has formed some large alluvial fans in the valley. The Sierra de Avila mountains lying to the north of the valley have been the source for most of the deposits. The composition of the alluvial fans vary. At the northern edge of the valley they are composed of large rocks, boulders, sand and clay. The deposits at the southern edge are composed of a smaller aggregate with larger portions of silt and clay.

It has been suspected that the large earthquakes which have occurred in the vicinity of Caracas in the past have occurred on the Sebastian Fault. It is also believed that the July 29, 1967 earthquake occurred on the Sebastian Fault. The location of the epicenter of this earthquake at 10.56° north latitude and 67.26° west longitude strongly suggests that this was the case. This point is on the Sebastian Fault six miles off Venezuela's coast and about 30 miles northwest of the capital city of Caracas.

Major building damage to Venezuela was centered in the city of Caracas. Only minor damage occurred elsewhere. Significant characteristics of the earthquake are well-known. The Richter Magnitude was about 6.5. It is estimated that the modified Mercalli intensity associated with this earthquake varied between VI and VII in the city of Caracas. Total duration of the earthquake was perhaps 60 seconds with strong motion lasting between 15 and 20 seconds. The focal depth was about 10 miles, about what would be expected in California. The damage to structures resulted from building vibration, and there were no instances of foundation failure.

It is estimated that there are about 10 thousand multi-story buildings in Caracas. Of these the vast majority are constructed of reinforced concrete. Only a few multi-story steel-framed buildings exist. No steel-framed building suffered significant damage.

Venezuela's building regulations and standards are very similar to American practices. To obtain an insight into how to apply the experience gained in the 1967 Caracas earthquake to the San Francisco Bay Area it is appropriate that materials, construction, and design methods be discussed.

The concrete used in construction is of excellent quality. Local mines produce the cement and a dense coarse aggregate. Expected compressive strengths of the concrete varies between 2500 lbs/in^2 to 4300 lbs/in^2. Furthermore, the climate is moist and temperatures nearly constant, varying from 56°F to 80°F, so that shrinkage and curing are not major problems.

Steels used as reinforcement are also produced from locally mined ores. The typical reinforcing steel corresponds to the American intermediate grade. Its yield stress is 34,000 psi and a working stress of 17,000 psi is generally used. A high strength reinforcing steel bar is also produced by deforming the bar prior to installation. A high strength steel reinforcing bar is "heliacero." Minimum yield stress of this material is 56,000 lbs/in^2. The working stress is generally taken as 28,000 lbs/in^2. The high strength material is generally used for the main reinforcement while the intermediate grades are used for the ties and stirrups.

Interior partitioning walls and the outside walls are made of an unreinforced hollow clay tile. The size of the tile varies considerably. One style is approximately the size of ordinary American brick, but has two central holes extending its length. The mortar used in construction of these tiles is of good quality with a compressive working stress between 1500 and 2000 lbs/in^2. In general, very little wood is used in construction, hollow clay tiles being used as a substitute.

The design of multi-story buildings in Venezuela is similar to American practices. The analysis includes lateral as well as vertical forces but there is no seismic requirement for buildings below four stories. Seismic design for Caracas corresponds to a zone 2 approach in American terms.

Generally, only the frame of the building is considered to resist the vertical and lateral forces. However, there are a small number of buildings which utilize concrete walls around the staircase and elevator shafts to provide lateral support. Interior and exterior walls of clay tile are not considered to contribute to the structural strength of the building. Their weight is considered as part of the vertical force. The position of the tile walls could make torsion of the structure an important design consideration. Torsional effects are rarely considered in the design.

Column design in Venezuela follows accepted procedures and is similar to American practice. The supporting beams of the frame are of two types; the deep girder and the flat beam type. In the former, a supporting concrete girder is cast with the floor slab, but below the slab. In the latter, all resistance is provided by the floor slab alone.

Generally, Venezuela construction practices and design are good. Noteworthy in the design is the assumption that the concrete frame acts independent of the hollow tile, interior and exterior walls.

This is possibly the only unrealistic portion of the design. This assumption fails to consider that the rigidity of the tile walls increases the stiffness of that portion of the structure, and hence, more load during an earthquake might be concentrated there. The tile walls could also cause localized stresses in the frame while resisting lateral forces. A further complication could arise when some stories have tile walls and others do not. The difference in the period of vibration for different stories could complicate the response of the structure to the earthquake. On the other hand, the tiled walls do absorb energy during an earthquake which is beneficial.

Structural damage was generally confined to the supporting columns of the building. Only a few instances of beam failure were noted, but broken tile walls were common. The damage potential in Caracas is considerably reduced due to the absence of air conditioning and heating systems. The mild climate of Caracas makes these conveniences unnecessary.

The city of Caracas must be divided into two parts when considering the probabilities of sustaining damage and the level of damage. The division is necessary because of the large variation in the depths of alluvium under different areas of the city. For example, the depths of alluvium in the Los Palos Grande region ranged up to 450 feet while in the rest of the city the alluvium was very shallow. In fact, for the region outside of the Los Palos Grande district the buildings were essentially founded on bedrock.

It has been mentioned previously that alluvium intensifies seismic ground motion. This factor is certainly in evidence during this earthquake. In the Los Palos Grande area four major buildings collapsed, while there were no instances of building collapse outside of this region.

Table 5.2 gives the definitions of the four states defining the level of damage. The definition of damage was taken directly from Reference [30].

TABLE 5.2

State	Damage Category	Average Damage/ Cost Ratio
1	No Damage	0
2	Light	0.005
3	Heavy	0.200
4	Structure Requires Replacement	1.000

Likewise the transition matrices are taken directly from Reference [30]. There is not sufficient information available for an independent analysis. The transition matrices are given in Table 5.3.

A building may have been exposed to many seismic events prior to the occurrence of the event which significantly damages it. Prior to being damaged the strength, natural period of vibration and damping in the structure may have been significantly changed. The changes in these factors could have made the structure more susceptible to seismic damage. But the available statistical data are too meager to allow the structure to be placed in its true state prior to sustaining seismic damage. Therefore, it must be assumed that the structure began in state one--the undamaged state--and from this state the probabilities of transition to other states must be calculated. The transition matrix is a four by four matrix since four states are used to define the possible states of the structure. However, since the structure begins in state one only the first row of the transition matrix will have entries different from zero. Equation (5-1) displays the general form of the transition matrix.

$$[P] = \begin{bmatrix} p_{11} & p_{12} & p_{13} & p_{14} \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} \qquad (5\text{-}1)$$

The probabilities of transition given in Table 5.3 are the elements of the first row of the matrix in Equation (5-1).

TABLE 5.3

TRANSITION PROBABILITIES

Los Palos Grandes

	Number of Stories		
State	5-8	9-14	15-19
1	0.91	0.75	0.23
2	0.08	0.13	0.50
3	0.01	0.07	0.22
4	0.00	0.05	0.05

Non-Los Palos Grandes

State	5-8	9-14	15-19
1	0.30	0.73	0.52
2	0.60	0.22	0.45
3	0.10	0.05	0.03
4	0.00	0.00	0.00

These are the transition probabilities from the damage data. For example, a building of between five and eight stories would have the following probabilities associated with a transition among the various states: the probability of remaining in state one is 0.91, the probability of being lightly damaged is 0.08, heavily damaged is 0.01 and the probability of collapse is zero.

1971 SAN FERNANDO EARTHQUAKE

Prior to the Long Beach earthquake of 1933, there were no engineering requirements for earthquake resistant structures in the City of Los Angeles. Buildings were limited to a height of 13 stories or 150 feet after this earthquake and until the year 1956. In this year the height limit was removed for structures with a steel frame. Later, reinforced concrete structures were allowed if the structure met certain engineering requirements.

The San Fernando Earthquake of February 9, 1971 was the first significant test of the earthquake resistant structures built since 1933. With a Richter Magnitude of 6.6 and an epicenter in the San Gabriel Mountains north of the City of Los Angeles, severe shaking resulted in metropolitan Los Angeles. The Modified Mercalli Intensity varied from seven to eight in the severely shaken areas. The construction in the region is typical of American practice. The discussion of the resulting damage in Section 5.3 applies to this area.

Tables 5.1 and 5.2 continue to apply here. References [19] and [31] are the chief sources of information. It appears at this time that there were no significant differences between the damage of steel-framed and reinforced concrete buildings.

Reference [31] provides a compilation of damage to modern high-rise buildings which is not available elsewhere. Eight states of damage are given in the reference. The number of possible states is reduced to four because the purpose here is to present the method of analysis. Reinforced concrete structures and steel framed structures are combined. Buildings are designed corresponding to a UBC zone 3 strategy.

Table 5.4 represents an abridgment of the information given in Reference [31].

TABLE 5.4

State	Description	Ratio of Damage to Present Cost
1	No Damage	0 - .0005
2	Non-Structural Damage	.0005 - .035
3	Structural Damage	.035 - 0.65
4	Building Condemned or Collapsed	1.0

The damage probabilities are calculated on buildings constructed since 1947. Thus, all of the structures were intended to be earthquake resistant. The transition matrices are presented by the number of stories. Table 5.5 contains the probability matrices. These transition probabilities are the first row of a transition matrix for the structure such as the one given in Equation (5-1).

TABLE 5.5

TRANSITION PROBABILITIES

BUILDING HEIGHT 5 TO 7 STORIES

Damage State	Modified Mercalli Intensity VI	VII	VIII
1	1.0	0.2	0.308
2	0.0	0.767	0.539
3	0.0	0.033	0.077
4	0.0	0.0	0.077
Number of Buildings	3	30	12

BUILDING HEIGHT 8 TO 13 STORIES

Damage State	Modified Mercalli Intensity VI	VII	VIII
1	0.0667	0.268	0.091
2	0.333	0.676	0.728
3	0.0	0.056	0.182
4	0.0	0.0	0.0
Number of Buildings	18	71	11

BUILDING HEIGHT 14 TO 18 STORIES

Damage State	Modified Mercalli Intensity VI	VII	VIII
1	0.5	0.500	0.0
2	0.5	0.445	0.0
3	0.0	0.056	0.0
4	0.0	0.0	0.0
Number of Buildings	4	18	

5.5 Analysis of the Damage Data

The damage statistics to buildings of varying story height have been analyzed to produce the transition matrices presented in Tables 5.3, and 5.5. Calculation of the seismic risk associated with high-rise buildings can be made using this data. For purposes of illustration consider the construction of a building of between five- and seven-stories in the San Francisco Bay Area. The results presented here are applicable to either steel-framed or reinforced concrete structures. Furthermore, recall that the damage levels represent average values. Therefore, it is expected that fluctuations or variations in these values have an equal likelihood of being larger or smaller.

As an example, suppose that the seismic risk associated with a proposed high-rise structure in the San Francisco Bay Area is required. The level of risk is required for large earthquakes which it is believed will cause a ground motion of perhaps VII on the Modified Mercalli Intensity scale.

In order to calculate the seismic risk the quantity $C_1(t)$ must be calculated. The governing equation for the calculation of this quantity can be obtained from Equation (3-18). Recall that the system is to begin in state one. Hence, the governing differential equation is displayed in Equation 5-2.

$$\frac{d\,C_1(t)}{dt} + (\beta - a_{11})\,C_1(t) = q_1 \quad . \tag{5-2}$$

The solution of this differential equation follows in Equation (5-3).

$$C_1(t) = C_1(0) \cdot e^{-(\beta - a_{11})t} + \frac{q_1}{(\beta - a_{11})} \left[1 - e^{-(\beta - a_{11})t}\right] \tag{5-3}$$

If it is desired to compare the risk only, the boundary condition can be put equal to zero. Therefore, the solution of Equation (5-3) reduces to that given by Equation (5-4).

$$C_1(t) = \frac{q_1}{\beta - a_{11}} \left[1 - e^{-(\beta - a_{11})t}\right] \quad . \tag{5-4}$$

This calculation will not consider the costs associated with remaining in state one--the undamaged state. Thus, costs such as earthquake insurance will not be considered and the quantity c_{11} will be put equal to zero. The quantity q_1 can thus be calculated from Equation (5-5).

$$q_1 = a_{12}\, c_{12} + a_{13}\, c_{13} + a_{14}\, c_{14} \quad . \tag{5-5}$$

The discount factor (β) will be assumed to be equal to 0.4.

The initial cost of construction of the structure will be denoted by the symbol $\overline{C}$. Since the following discussion will consider a building of between five and seven stories without regard to its initial cost, it is best to non-dimensionalize Equation (5-4) by dividing it by the initial cost of the building $\overline{C}$. Consequently, the risk curves calculated here are applicable to any modern high-rise building whose story height is within the stipulated limits.

After non-dimensionalizing Equation (5-4) the result is given by Equation (5-6).

$$\frac{C_1(t)}{\overline{C}} = \frac{q_1/\overline{C}}{\beta - a_{11}} \left[1 - e^{-(\beta - a_{11})t}\right] \quad . \qquad (5\text{-}6)$$

This is the equation which was used to plot Figures 5.1 and 5.2.

The Caracas Venezuela Earthquake with ground motion corresponding to a Modified Mercalli Intensity of VII and the data for this intensity from the San Fernando Earthquake are the input used in Equation (5-6). The seismic occurrence data are for large earthquakes.

The procedure to follow is to first calculate the transition rate matrix $[A]$ from Equation (3-13). The analysis is identical to that given in Section 4.9 of Chapter IV. For example, the transition rate matrix $[A]$ for the Los Palos Grandes area in Caracas, Venezuela is given in Equation (5-7).

$$[A] = \begin{bmatrix} -0.0026 & 0.0023 & 0.0003 & 0.0 \\ 0.0 & -0.029 & 0.0 & 0.0 \\ 0.0 & 0.0 & -0.029 & 0.0 \\ 0.0 & 0.0 & 0.0 & -0.029 \end{bmatrix} \qquad (5\text{-}7)$$

After non-dimensionalization the quantity $\frac{q_1/\bar{C}}{\beta - a_{11}}$ has the value given in Equation (5-8).

$$\frac{q_1/\bar{C}}{\beta - a_{11}} = 35.3\text{x}10^{-5} \quad . \qquad (5\text{-}8)$$

Substitution of this value given in Equation (5-8) into Equation (5-6) and plotting this equation as a function of the time remaining in the design life of the structure results in the lower curve in Figure 5.1 which represents the risk associated with high-rise structures in the Los Palos Grandes area of Caracas. The same procedure can be applied to the remaining data. This permits construction of Figures 5.1 and 5.2.

The risk calculation based on the data from the San Fernando Earthquake is about four times the level of risk calculated from the Non Los Palos Grandes area and nine times the risk from the Los Palos Grandes area of Caracas, Venezuela. There could be many possible explanations for the difference. Basically, it can be said that the data are not strictly comparable. The data for the Venezuela Earthquake are based on buildings without air conditioning and the facadas which are common to structures erected in the United States. Furthermore, further investigation is required into the soil conditions in each area. This degree of difference in the values of the risk indicates that the damage data from Venezuela must be adjusted before it can be used in the San Francisco Bay Area.

For the typical structure erected in the San Francisco Bay Area the associated risk would be best illustrated by the calculation based on the San Fernando Earthquake. The structures in these two regions would be comparable insofar as materials, structural design and decorative facades are considered. Furthermore, based on the information given in Reference [3] the soil conditions are similar. It is implied in this reference that the San Francisco Bay Area could expect higher damage levels than in the Los Angeles Area.

5.6 Conclusion

Consequently, Figure 5.2 could be used as the basis for determining the seismic risk to high-rise buildings between five- and seven-stories high. For example, consider a five-story building costing ($\bar{C}$) about $800,000. From Figure 5.2, the steady-state risk is about $2600 for the time remaining. If the building is designed for 20 years the risk per year would be about $130. An insurance company insuring such a structure against the occurrence of an earthquake might want to use this value as the cost of the insurance premium.

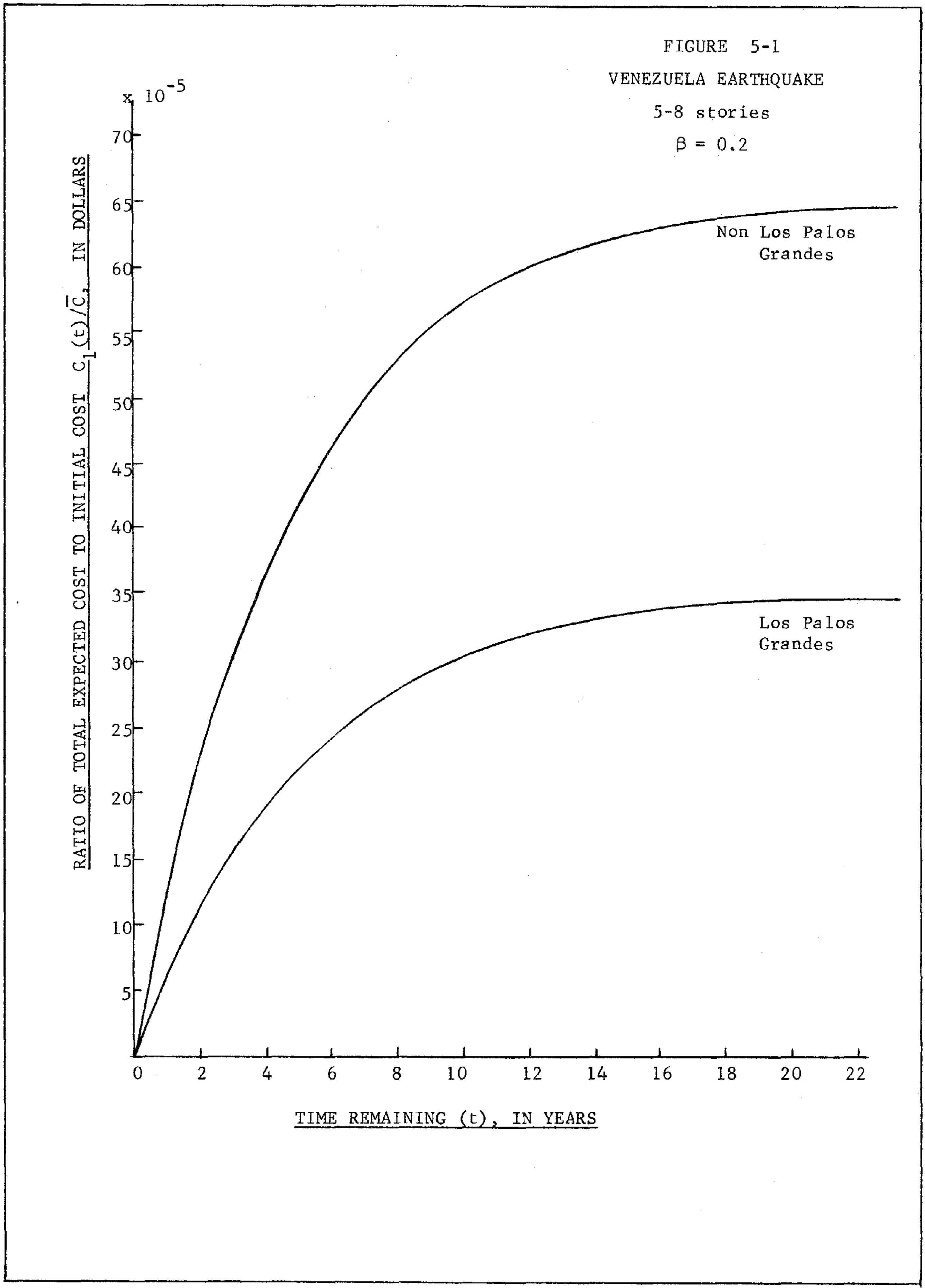
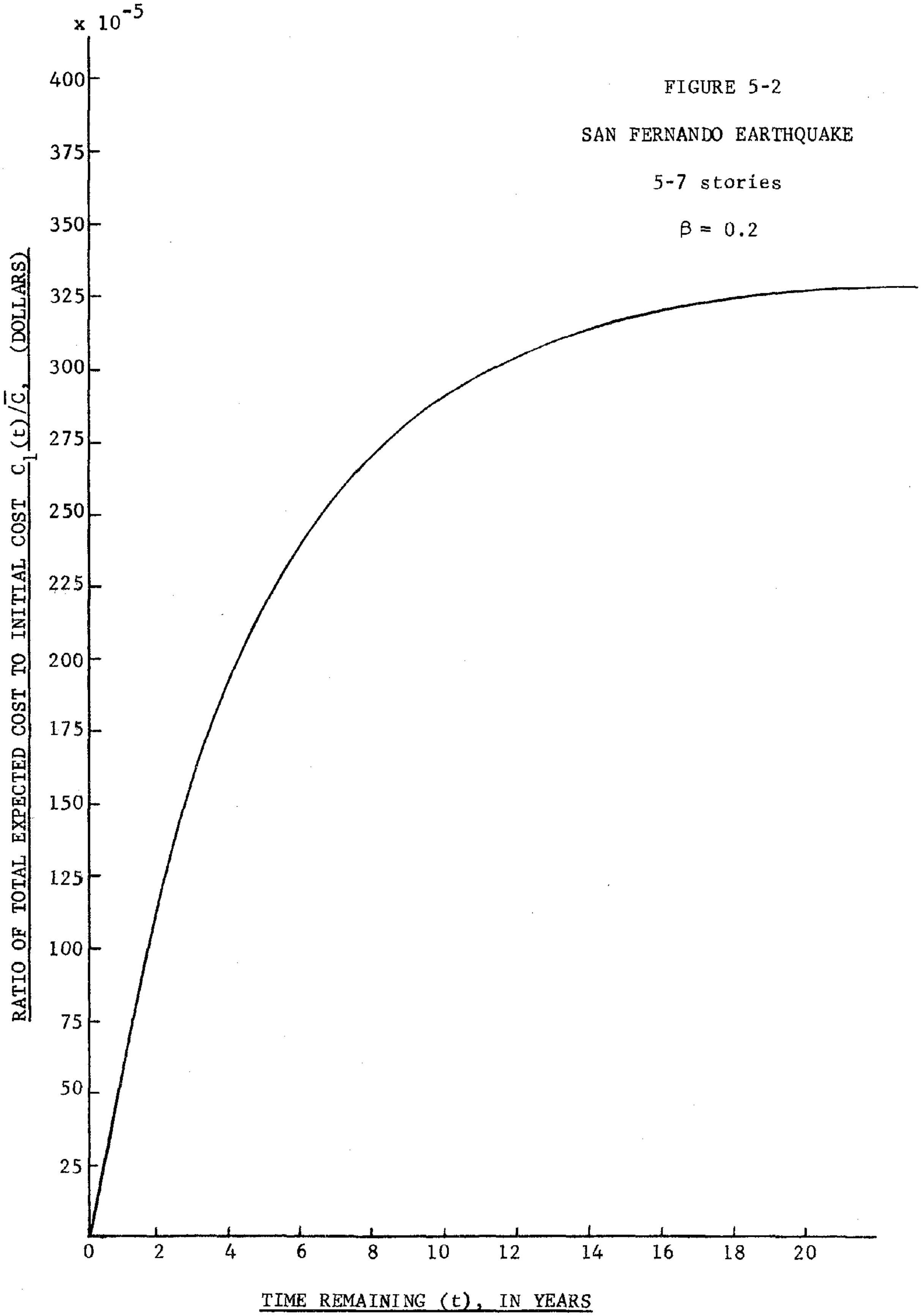


FIGURE 5-1
VENEZUELA EARTHQUAKE
5-8 stories
β = 0.2
x 10^-5
70
65
60
55
50
45
40
35
30
25
20
15
10
5
RATIO OF TOTAL EXPECTED COST TO INITIAL COST C1(t)/C̄, IN DOLLARS
Non Los Palos Grandes
Los Palos Grandes
0
2
4
6
8
10
12
14
16
18
20
22
TIME REMAINING (t), IN YEARS


x 10^-5
400
375
350
325
300
275
250
225
200
175
150
125
100
75
50
25
0
2
4
6
8
10
12
14
16
18
20
RATIO OF TOTAL EXPECTED COST TO INITIAL COST $C_1(t)/\bar{C}$, (DOLLARS)
TIME REMAINING (t), IN YEARS
FIGURE 5-2
SAN FERNANDO EARTHQUAKE
5-7 stories
$\beta = 0.2$

VI LIGHT INDUSTRIAL BUILDINGS

6.1 Introduction

One of the goals of earthquake engineering is to design structures which when exposed to seismic motions sustain damages whose repair would not exceed the increased cost of design, construction, and financing which would have prevented damage. In this chapter, this question will be investigated and a method of approach to this problem will be presented. References [15,46] also discuss this problem.

The level of protective design to be provided to obtain a reasonable degree of safety from excessive damage or collapse of a structure during an earthquake combines economics with the engineering aspects of the design. The economics of the problem involve estimating the added cost of design and construction of the building to control the damage level and prevent its collapse. Damage or collapse of the structure requires a probabilistic description. Thus, additional strength in the building reduces the probability that the structure will be damaged or will collapse during an earthquake. The choice of the values of these probabilities depends upon the designer. From the engineering standpoint, the engineer must have a reasonable estimate of the expected ground motion at the construction site during a given earthquake. In addition, the engineer must understand the structural system of the proposed facility so that the forces of the earthquake can be withstood.

A structure constructed in a seismic region is required to function in an environment characterized by random earthquake occurrences and magnitudes. Consequently, due to these two factors and due to the soil conditions between the origin of the earthquake and the construction site, the resulting ground motion is a random variable. The dependence of the transition probabilities on the ground motion is evident. However, the functional dependence is complex and will not be considered here. The approach taken here will be empirical and will be based on the available data.

It will be assumed that additional structural strength results in a safer structure. This is the philosophy which predominates in building codes. For example, the UNIFORM BUILDING CODE requires a larger lateral force for seismic design in regions where the probability of seismic motions is greater. Thus, greater protection from seismic motions is required and increased protection is obtained by increasing the strength of the structure. The transition probabilities, which describe the transition of a given structure from the undamaged state to the damaged state or to the collapse state, will be assumed proportional to the structural strength of the structure.

6.2 Construction Costs of Light Industrial Buildings

Light industrial buildings are fairly common in the shopping centers and industrial parks of California. This type of building represents an attempt to provide a large enclosed floor area with a light and inexpensive structure. Single story buildings predominate.

Construction of this type of building generally begins by pouring a large reinforced concrete slab to function as the floor of the building.

Unit masonry or tiltup walls are then set in a foundation and attached to the floor by dowels concreted in place. Generally, the roof of the building is of plywood or steel. It is attached to the walls by bolts. In addition, laminated beams or a light truss work provide additional support to the roof. This structure, which has been described, constitutes the shell of the building.

The structural resistance of this type of building is provided by what is commonly called a box system. In such a system, there is no load carrying space frame. The vertical loads are carried by the walls or by columns cast integrally or separately from the walls. Resistance to lateral loadings are provided by the walls, which are designed to resist lateral loads by shear deformation. For additional strength, the masonry or concrete walls are generally reinforced with steel. The roof of the building acts as a diaphragm when stressed by lateral loads. The forces are carried from the roof to the walls and hence, to the foundation.

The roof of a light industrial building is usually flat or convex. The columns provided along the walls of the building provide convenient attachment points between the roof and the walls. This connection is extremely important. The roof is particularly important to the strength of the building and it can only be effective if the roof to wall connections are of adequate strength.

The cost of constructing a light industrial building varies in the United States. The proximity to the source of materials and the prevailing labor rates in the chosen region influence the costs. In the Greater

San Francisco Bay Area the cost ranges from about $5.00 per square foot to about $15.00 per square foot. This latter figure is representative of a fairly well-equipped building. A well-equipped building is one with air conditioning and perhaps facades on the exterior walls or interior walls to make the building more attractive. In addition, if the building is to be used as office space requiring partitioning walls, more elegant lighting fixtures and perhaps a lounge to go with the toilet facilities the figure of $15.00 per square foot may be exceeded.

Note that the cost figures given above do not include the cost of the land upon which the building is sited or the cost of landscaping. Both of these factors vary considerably from one area to another.

6.3 Damage Sustained by Light Industrial Buildings During Earthquakes

The dynamic characteristics of light industrial buildings can be found analytically quite simply. The buildings are generally regular in shape and usually they are a single story. Because of the simple manner in which the structural system is designed, the values of the stiffness and weights for each individual segment of the building is precisely known. Hence, the vibration characteristics of the building such as the fundamental period of vibration, the modes of vibration, and to a lesser extent the degree of damping can be calculated.

It is not the purpose here to go into the analysis of the structural characteristics of light industrial buildings. It will suffice if only the range of values for the fundamental period of vibration and the degree of damping are known. Of course, the values of these quantities will vary for different buildings. But in general, one can expect the fundamental period of vibration to be about one second and the degree of damping to be less than one percent of the critical value.

The fundamental period of vibration and the degree of damping are functions of the amplitude of vibration. For small amplitudes of vibration all of the structural components of the building are being stressed. For example, one would not expect the partitioning walls in a building to contribute to its stiffness unless the amplitude of vibration was very large.

The structural characteristics of a building determine its response to a base excitation. For light industrial buildings, it is reasonable to assume that the magnitude of a base acceleration is the critical factor causing seismic damage. Because of the shear stiffness of the building and the low amount of damping, the motion input to the base of the building will carry through to the top of the building. Therefore, large acceleration at the base will result in large forces being transmitted throughout the building without diminution.

Light industrial buildings, being characteristically stiff in shear, would probably not oscillate in the fashion of a high-rise structure. Unless the duration of the earthquake was excessively long, there would be little opportunity for a resonant condition to develop. Consequently, the assumption that forces due to the seismic base acceleration are the major cause of damage to the structure appears reasonable.

Damage to light industrial buildings is confined to particular locations. The continuity of connections in these types of buildings definitely needs improvement. The connection between the roof and wall is a critical connection. If the roof is joined to the wall by the plywood-to-ledger type of connection, the design should be based on a greater force than is currently required by building codes. During the inspection of the damage to light industrial buildings exposed to the ground motion of the San Fernando Earthquake of 1971, this connection had failed in almost every building.

In addition, it also appears that requirements for joining pilasters to walls, for those which are not integral, are inadequate. The connection of the masonry or tiltup wall to the floor slab by dowelling is another instance where the strength required during an earthquake is underestimated by the building code. Failure of these connections was observed during inspection of buildings exposed to the San Fernando Earthquake.

There are two additional causes of damage which were noted during the recent San Fernando Earthquake which require comment. Failure to anchor equipment securely in the building represents a significant contribution to damage. For example, storage closets and book shelves should be anchored securely to prevent their upset during an earthquake. The building facades and particularly walls require better and stronger anchoring if their destruction is to be prevented. The second factor which must be provided for is the relative displacement between adjacent buildings during an earthquake. Buildings should not be constructed so close together that out of phase displacement between them results in hammering.

6.4 Earthquake Damage Data

The available damage data is from the Los Angeles area and represents the performance of light industrial buildings during the San Fernando earthquake of 1971. Information obtained during this earthquake can be applied to the San Francisco Bay Area.

Generally, the building sites for shopping centers and industrial parks are chosen in areas which are flat. Flat areas are commonly alluvial flood plains in California. This factor is due to the presence of mountainous areas surrounding the population centers. If the building site is not flat excavation and fill is required to provide the necessary flatness. In either case, it is common to find light industrial buildings constructed on soft cohesionless soils. This was the case with the buildings inspected after the San Fernando Earthquake.

Structural engineers, who are well acquainted with earthquake damage to light industrial buildings believe that the aspect ratio (length to width) of the roof diaphragm has a great deal to do with the amount of damage sustained by this type of structure. This may be due to the different vibration quantities such as the period of vibration which depends on the roof's area.

Although it is generally held that the aspect ratio influences the level of damage, the following figures do not bear this belief out. Hence, as far as this dissertation is concerned, no use will be made of this aspect of the problem.

The following histograms (Figures 6.1, 6.2, and 6.3) give the damage experienced by buildings in two industrial tracts during the San Fernando Earthquake of February 1971. The hatched blocks represent buildings with floor areas greater than 40,000 ft^2, those blocks which are plain have floor areas below 40,000 ft^2.

There were two tracts during the San Fernando Earthquake which contained light industrial buildings and which were exposed to significant ground motion. The Arroyo Tract contained 33 buildings and was exposed to a Modified Mercalli Intensity of about 7. The Bradley Tract contained 23 buildings and was exposed to a Modified Mercalli Intensity greater than 7 and perhaps as high as 8. The majority of these buildings had floor areas less than 40,000 square feet. The buildings in the Bradley Tract were more expensive than those in the Arroyo Tract. They averaged about $55,500 per building compared to $23,900 per building for the Arroyo Tract. However, in both cases the average cost of damage was about 56¢ per square foot.

Figure 6.4 indicates the expected curve relating the average cost of damage to the Modified Mercalli Intensity. Insufficient statistical data precludes a complete description. However, examination of data from other earthquakes indicates that the shape of the curve representing the functional relation is similar to that pictured here.

During the San Fernando Earthquake there was no instance of a total collapse of a light industrial structure. There were four cases where partial collapse occurred. Three partially collapsed buildings were in the Bradley Tract and one in the Arroyo Tract.

FIGURE 6-1

ROOF DIAPHRAGM

Aspect Ratio 1.0 to 1.5

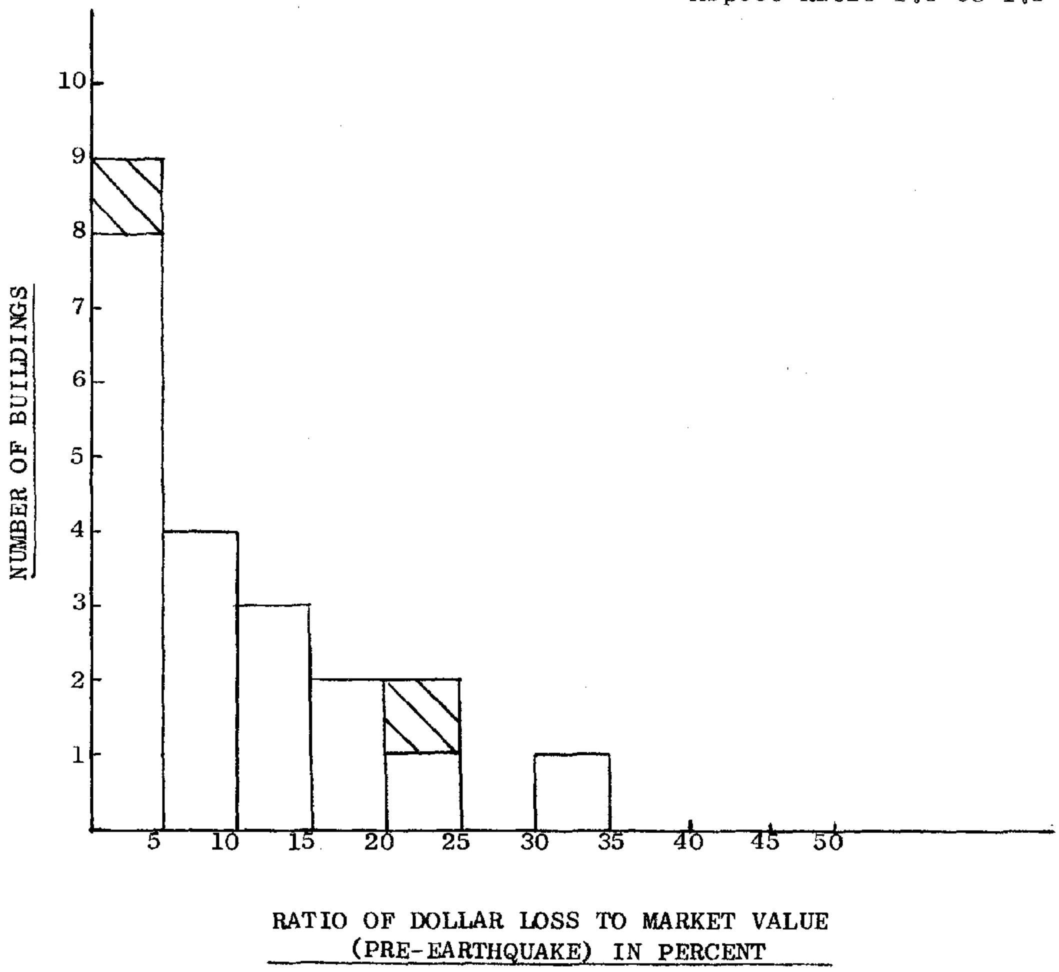
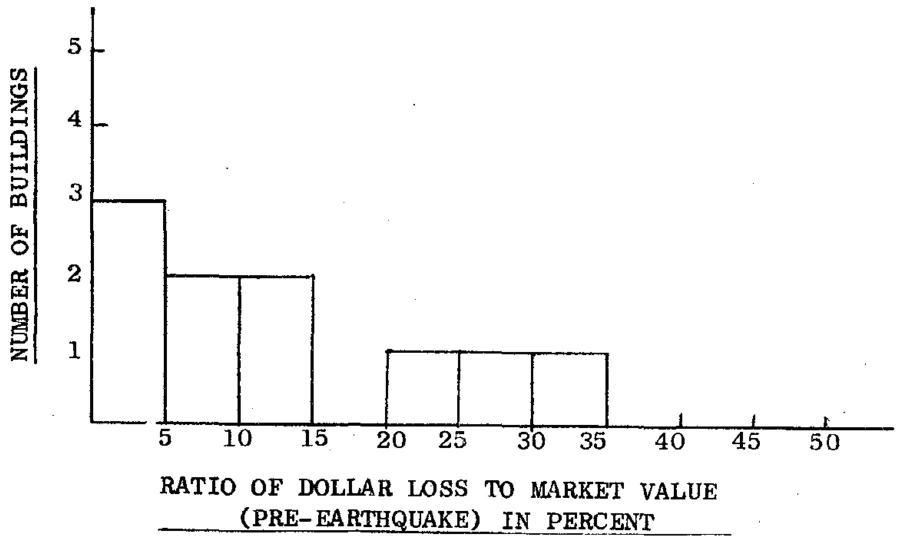


FIGURE 6-2

ROOF DIAPHRAGM
Aspect Ratio
2.0 and over



Aspect Ratio
1.5 to 2.0

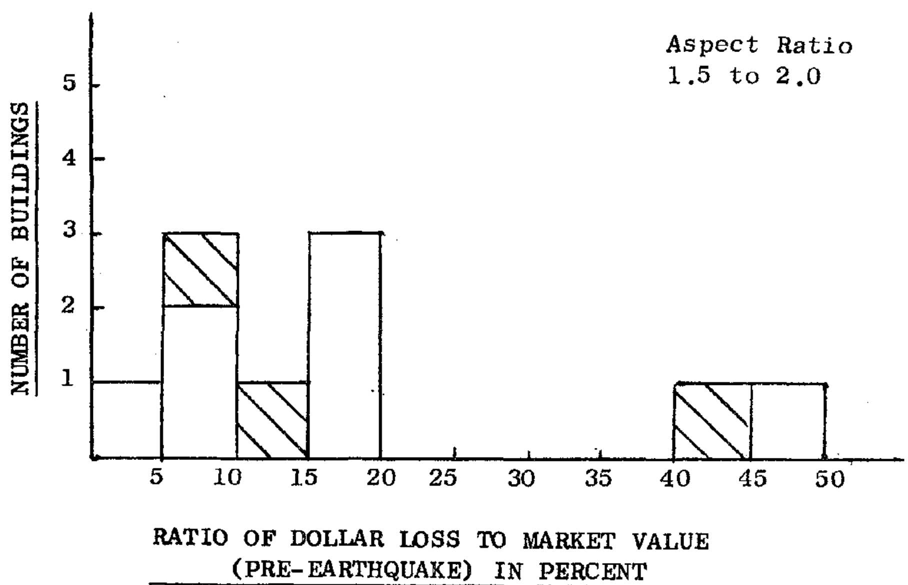
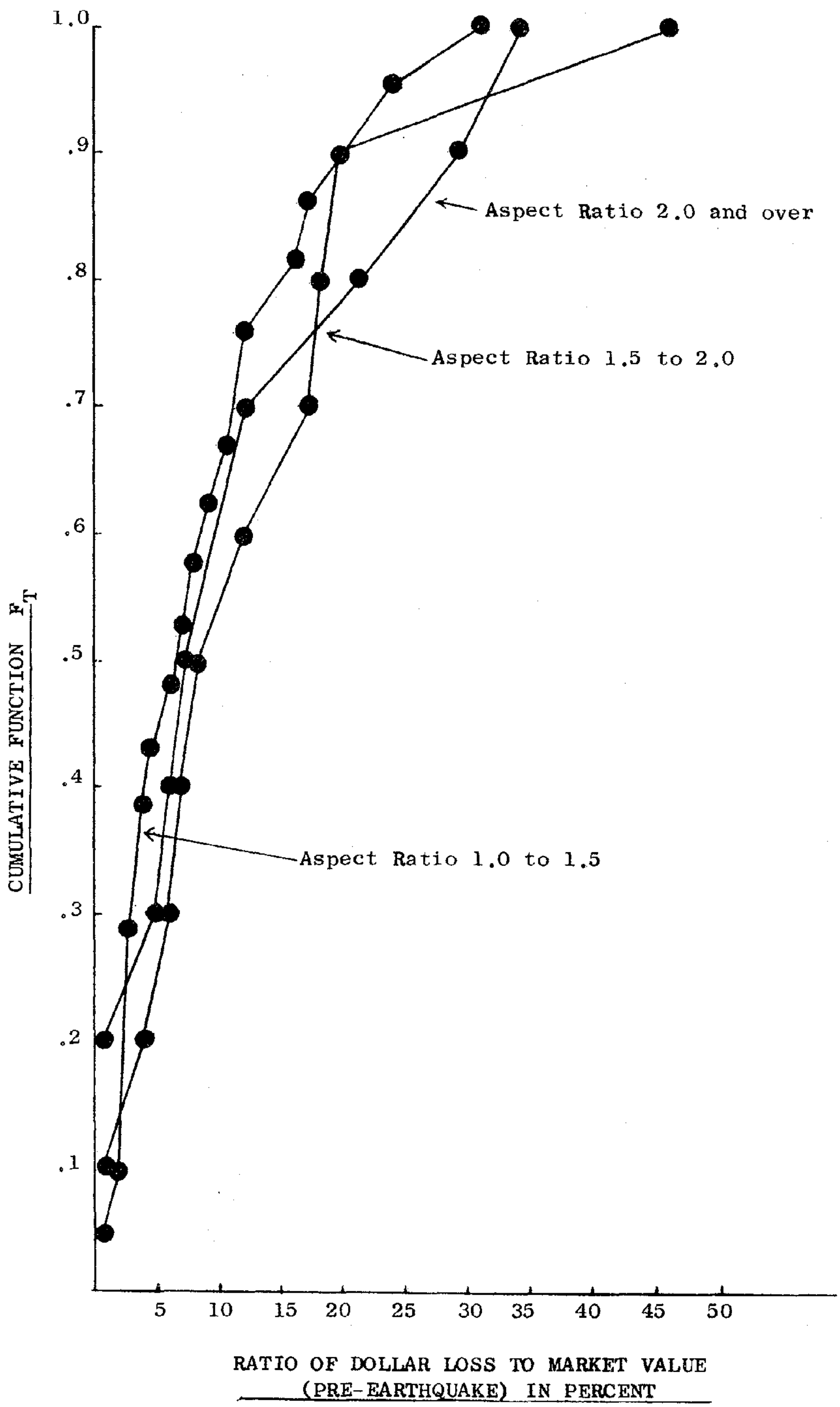


FIGURE 6-3


1.0
.9
.8
.7
.6
.5
.4
.3
.2
.1
CUMULATIVE FUNCTION F_T
Aspect Ratio 2.0 and over
Aspect Ratio 1.5 to 2.0
Aspect Ratio 1.0 to 1.5
5 10 15 20 25 30 35 40 45 50
RATIO OF DOLLAR LOSS TO MARKET VALUE
(PRE-EARTHQUAKE) IN PERCENT

6.5 Analysis of the Cost-Benefit Relation

Consider a Markov Decision Model such as the case described in Section 3.6.2. In the present investigation only two possible states will be considered. State one will represent the undamaged state and state two the damaged state. If the light industrial building is damaged it will be assumed to sustain the average amount of damage calculated in Section 6.4 This figure was fifty-six cents per square foot.

Some clarification of the concept of average cost is in order. By an average value of damage it is implied that variations in the value of the damage sustained because of soil conditions, differences in the quality of materials used in the building's construction or perhaps the size of the statistical sample, take place about this average value. Consequently, variations about the average value will also occur if the figures are applied to different regions. For example, the data obtained from Los Angeles will be applied in this example to the San Francisco Bay Area.

In Sections 6.3 and 6.4 the type of damage and location of damage to light industrial buildings was discussed. At this point one wants to investigate how this damage might be minimized by expending additional resources. From the San Fernando Earthquake a boundary condition can be obtained. The transition probability a_{12} for buildings exposed to Modified Mercalli Intensities of seven to eight was one. These buildings were designed according to the UNIFORM BUILDING CODE with a zone 3 strategy.

In this section, the cost-benefit relation for a particular light industrial building is investigated. In order to prepare this example, a building typical of this type of construction will be analyzed. The guideline for the design of this structure will be the 1970 edition of the UNIFORM BUILDING CODE. The building must be classified according to the code in order to determine the allowable floor area.

The classification of these buildings is by their fire rating. This involves using accepted practices to construct a fire resistant building in any designated fire zone of a locality. The fire rating of the walls is required to be higher than it is for the roofs and floors. Generally, a two-hour fire rating is required in industrial parks and this rating can be achieved with a four- to six-inch thick reinforced concrete wall.

The majority of these buildings are given either G, F, or E occupancy ratings in the UNIFORM BUILDING CODE. The typical rating is a F-2 and includes stores, warehouses, and office buildings.

These structures are generally a Type III building. The type includes a plywood roof and tiltup or masonry walls.

By Table 5C in the UNIFORM BUILDING CODE buildings of this type are allowed only 13,500 ft^2 if constructed in fire zones Nos. 1 and 2. If the structure is located in fire zone No. 3 this basic area may be increased by one third. By Table 5D the maximum height allowed is 65 feet. The example will consider a building in fire zone three where the allowed area is 18,000 ft^2.

The constructure procedure is straightforward. A floor slab is poured in place. The floor slab serves as a useful foundation to pour the walls. While the walls are curing, the foundations for the walls are constructed. This permits the walls to cure and shrink. It is assumed that two-thirds of the shrinkage is out before the walls are tilted in place.

A drilled-in-place footing is used to support the walls. The walls are then tilted into place and attached by dowels to the floor slab. The walls are joined together with poured-in-place columns.

The roof is the final component to be fabricated. A wood glulam beam is chosen with a plywood roof. A ledger is bolted to the wall and the roof attached to the ledger.

The cost considers strengthening the structure above what is required by the building code for the particular zone in question. For the San Francisco Bay Area a zone three design strategy is required. This cost is based on adjusted labor rates and material costs given in references [41,42]. The labor rates have been adjusted to reflect the present situation in the San Francisco Bay Area. Material costs are also estimates of the cost to be expected in this area.

Changes in labor rates and material costs can be expected to change the numerical results of this investigation. However, the method is generally applicable.

A light industrial building could be exposed to the ground effects of a number of earthquakes before it makes a transition from the undamaged to the damaged state. The following sequence of earthquakes could occur in the vicinity of the building: moderate, small, small, moderate, large and finally a small earthquake before transition. Consequently, it is reasonable to consider transitions probabilistically.

In calculating the additional cost C' only the field costs of labor and the cost of the materials has been included. This implies that the costs of insurance, taxes, office overhead and permit fees are not included.

Figure 6.5 is a plot of the cost of added protection versus the probability of being damaged. This plot has been estimated from the data on costs of light industrial buildings given in References [41,42]. Labor costs and material costs have been adjusted to reflect the increase in these quantities. It is supposed that the transition probability a_{12} can at first be reduced by improving the roof to wall ledger joint and the floor slab to wall connection. In each case the number of attachments are increased. As additional strength is required all thickness and wall reinforcing must be increased.

Equation (6-1) gives the asymptotic value of the total expected risk $C_1(t)$.

$$C_1(t) = \frac{c_{12}}{\beta/a_{12}+1} \quad . \qquad (6\text{-}1)$$

The asymptotic value is obtained from Equation (6-2) for large values of the time t,

$$C_1(t) = \frac{c_{12}}{\beta/a_{12}+1} \left[1-e^{-(\beta+a_{12})t}\right] \quad . \qquad (6\text{-}2)$$

The value of c_{12} is assumed to remain constant. At this point, Equation (3-33) from Section 3.6.2 can be formed. For the case considered here the function F, given in Equation (6-3), is a function of the transition probability a_{12}.

$$F(a_{12}) = C'(a_{12}) + C_1(a_{12}, t)$$

This function can now be optimized with respect to the transition probability a_{12}. Figure 6-5 shows this operation graphically. The dashed vertical line in this figure corresponds to the minimum value of the function F. The minimum occurs at a value of the transition probability a_{12} equal to 0.85. Furthermore, this result implies that about four cents per square foot should be spent for added earthquake protection.

Now that the optimum value of a_{12} has been calculated the seismic risk for this problem can be determined. Using Equation (6-2) with the optimized value of the transition probability a_{12} and a value of the discount factor (β) equal to 0.2 the curve in Figure 6.6 can be constructed. This curve gives the value of the risk after adding the additional earthquake protection.

The difference in the steady-state risk before and after optimization is about 1.4 cents per square foot. For a building of perhaps 20,000 ft^2 with 10 years remaining in its design life, this would amount to $28 per year on the average. Of course, for a larger building the difference in the level of risk would be greater.

FIGURE 6-4

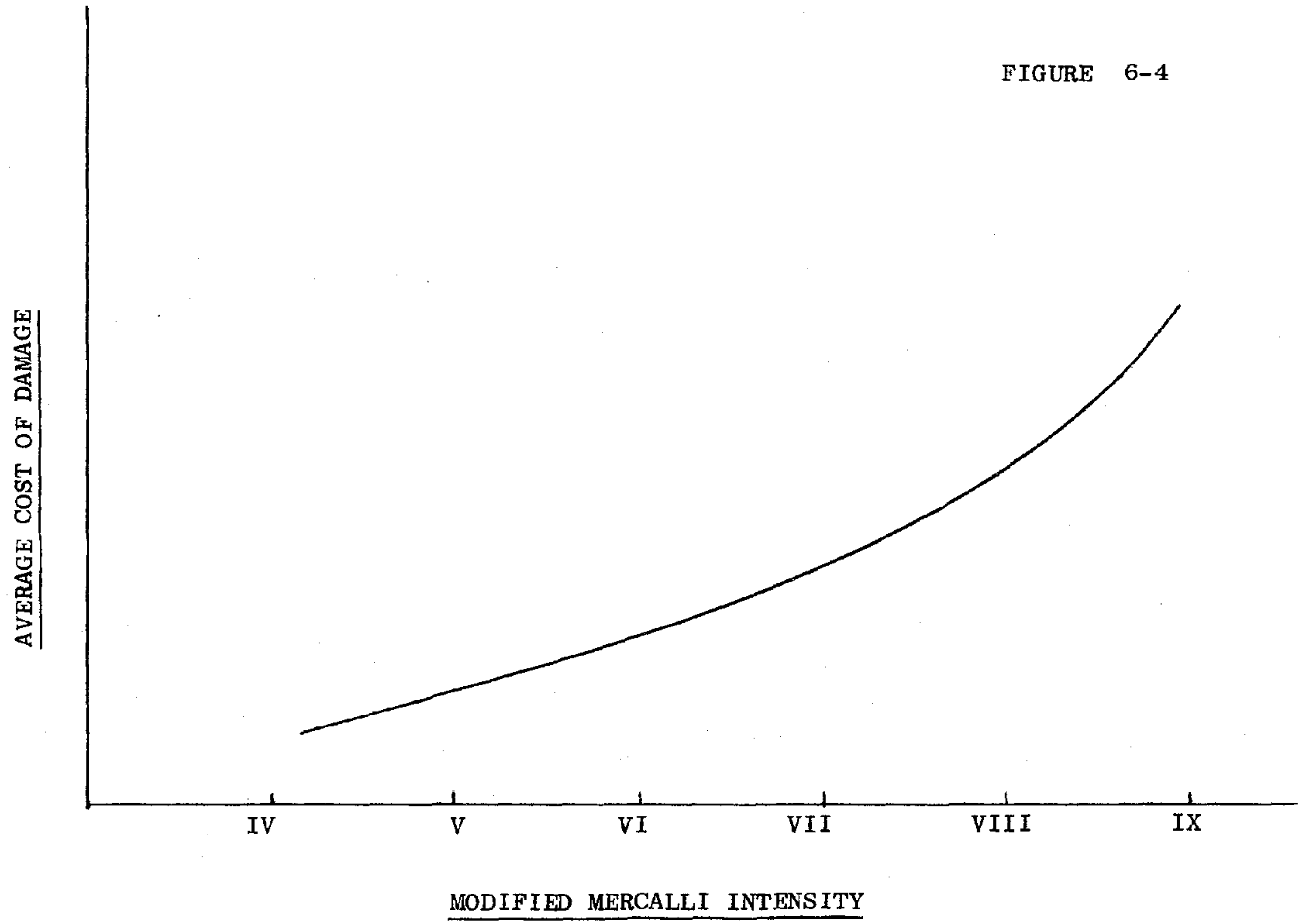

AVERAGE COST OF DAMAGE
IV
V
VI
VII
VIII
IX
MODIFIED MERCALLI INTENSITY

FIGURE 6-5

LIGHT INDUSTRIAL BUILDINGS

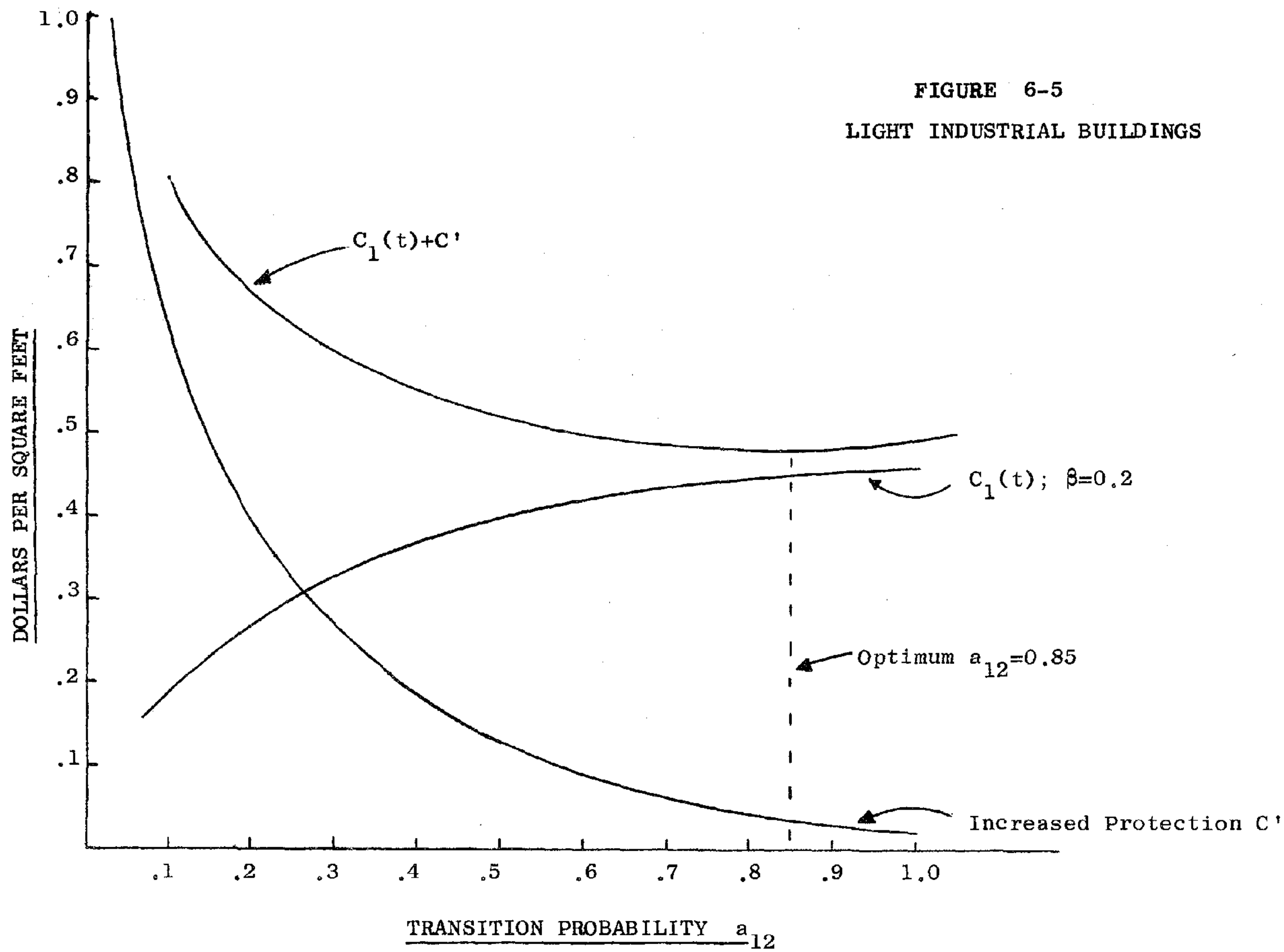
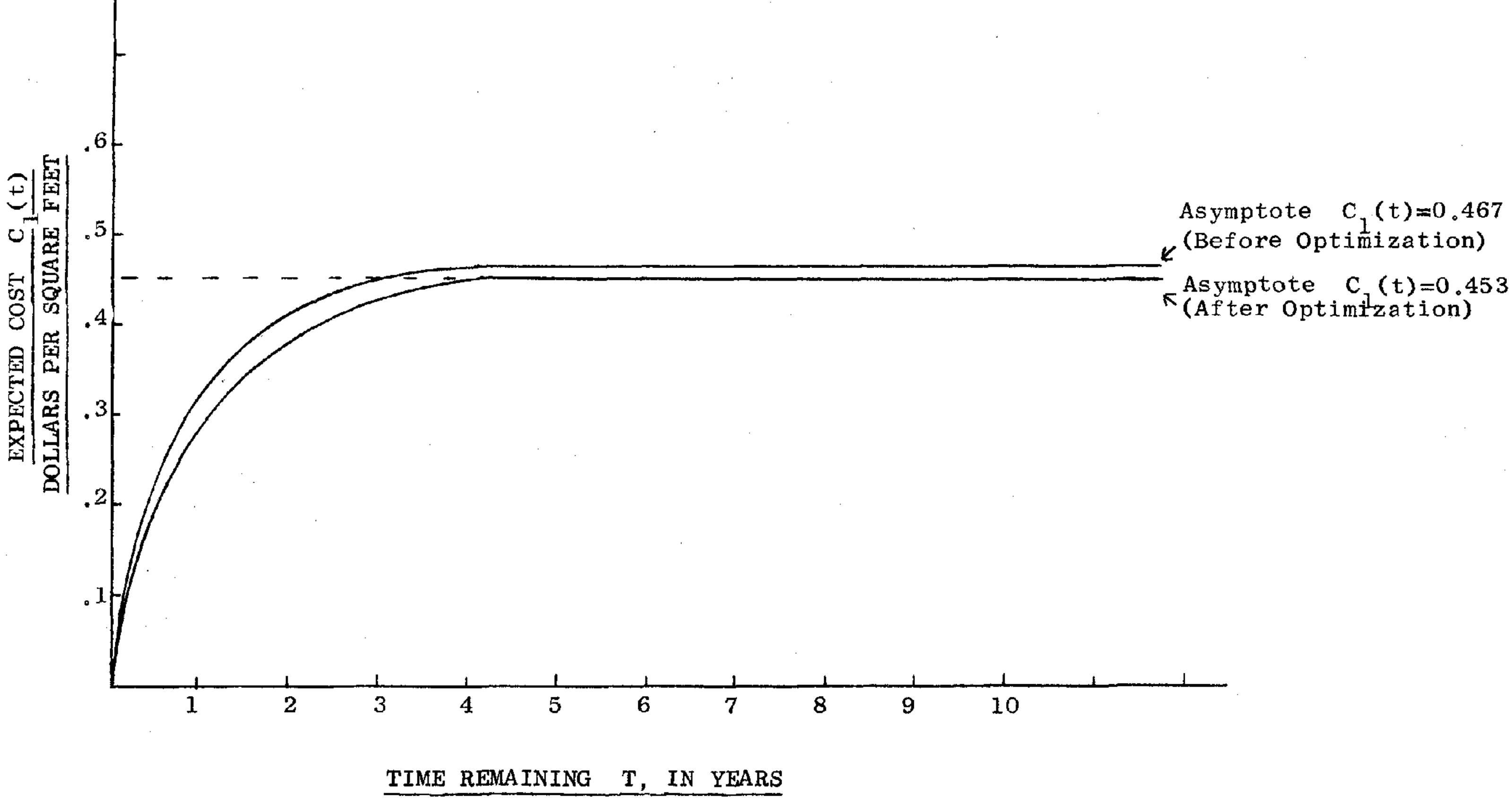


FIGURE 6-6
LIGHT INDUSTRIAL BUILDINGS



6.6 Conclusion

In this chapter, the analytical model for the cost-benefit problem, discussed previously in Section 3.6.2, has been applied to a practical problem. It has been shown how an engineer could determine the optimum value of the transition probability a_{12} and use this value to calculate the seismic risk associated with the structure. The seismic risk is ultimately the determining factor in establishing the insurance premium for earthquake protection.

A reduction in the level of seismic risk requires an expenditure for the added protection from earthquake damage. From Figure 6.5 note that to achieve the optimum transition probability (a_{12}) it is required that three cents per square foot be spent for added earthquake protection (C'). For this expenditure the level of seismic risk is reduced by 1.4 cents per square foot and probability of suffering earthquake damage by 15%.

VII SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE RESEARCH

7.1 Summary

A method of performing a risk analysis for structures exposed to seismic occurrences has been presented. A risk analysis combines the probability of earthquake occurrence with the associated consequence which is the level of property damage. Risk has been quantified as a cost, but it is not an absolute quantity and its measure is most useful when risk values are being compared.

The seismicity of a region must be investigated before the level of risk associated with construction of a facility can be quantified. Seismicity considers primarily the probability of an earthquake's occurrence. For the purpose considered herein, the seismicity of a region--the Greater San Francisco Bay Area--was investigated. Investigation of the available historical earthquake record indicated that earthquakes generally occurred at shallow depths. For shallow focal depths the Elastic Rebound Theory is a good description of the mechanism behind earthquake occurrence.

Thus, it was necessary to find and apply a probability model which mathematically described the salient features of the ELASTIC REBOUND THEORY. In particular, the memory aspect of earthquake occurrence had to be described by the probability model. This property was possessed by a first order Markov Process. The Markov Model permits calculation of a conditional probability for a future earthquake, many years removed from the present time, but dependent on whether or not an earthquake has occurred at present.

The input to the model is the past historical earthquake record. Data have been obtained which are nearly complete between the years 1933 and 1969. Prior to this interval of time the record of earthquake occurrence is not precisely known. However, where enough information was available to reasonably classify a past earthquake, the earthquake was included in the record. The calculated probabilities are applicable to the entire region and consequently, they are to be considered average values.

The decision model is also Markovian in formulation. The model possesses features which make it very applicable to risk analysis. A decision model should incorporate the judgment of the investigators systematically. This factor is important because the results of the risk analysis should be reproducible and investigators working independently should obtain basically the same result.

A second criterion is that the measure of risk should be a quantity readily understandable to individuals who are not engineers. Planning in a seismically active region is not entirely the responsibility of engineers. Generally, competent engineers are asked for advice but the final responsibility rests with elected representatives. Consequently, it is assumed that quantification of risk should be made in monetary units.

Furthermore, the variable time should be considered a continuous variable. This feature is much more realistic than a discrete time approach because the calculations are not limited to prior selected points in time.

In addition, the seismicity of the region is included in the calculations. From the investigation of earthquake occurrence using a Markov Chain an estimate of the time to occurrence of earthquakes of given magnitude is made. This quantity is put directly into the decision model.

The value of money is time dependent. The idea is that a sum of money placed at interest today will generate a larger amount in a given time interval. The Markov Decision Model includes a factor which reflects the interest and inflation rate directly. Thus, a separate calculation is not necessary.

The Markov Decision Model incorporates all of these salient features directly. The only quantity left which must be calculated is the cost of the expected damage associated with the occurrence of an earthquake. The damage states for the structure are discrete. They are selected by the investigator so that the possible states of the structure after the earthquake can be adequately described.

The cost of the damage sustained by the structure is estimated as a function of the Richter Magnitude of the earthquake and the distance of the facility from the earthquake's epicenter. The damage costs obtained in this fashion are average values. No attempt has been made to correlate the cost of damage with a particular site. The estimates of damage for cohesionless soils, liquefaction, landslides, or fault rupture are difficult to obtain and are out of the scope of this dissertation. With the damage costs, input into the Markov Decision Model can be made and the associated level of risk can be calculated.

Three examples of risk calculations are presented. In each example, a different type of risk calculation is made. The goal is to illustrate the generality of the Markov Decision Model by applying it to problems confronting design engineers.

The first example of a risk calculation considers the investigation of an improved foundation for mobile homes. The improved foundation offers better protection during an earthquake. It is shown that a more stable foundation would be of definite advantage for mobile homes in a seismic environment. This conclusion is arrived at by comparing the risk to the mobile home before and after the improved foundation is used.

A second example evaluates the seismic risk associated with modern high-rise buildings. The seismic risk is calculated as a function of the building's story height.

Finally, the third example considers the cost-benefit concept in structural design. This example investigates a frequent problem faced by the design engineer. The engineer must balance the cost of added protection against earthquakes and the actual benefit in the form of reduced damage levels or a reduction in the probability of collapse of the structure. This example illustrates the application of the Markov Decision Model to this problem. Light industrial buildings are selected for investigation.

Thus, the contribution of this dissertation is to the field of risk analysis. It presents a comprehensive application of Markov Processes to estimating the risk in aseismic design. The conspicuous feature of the method presented is in its incorporation of the salient factors of seismic risk analysis. No proposed method, appearing in the literature, exhibits the versatility of the method presented here or the property of encompassing all of the important factors of a seismic risk analysis in a single calculation.

7.2 Conclusion

An earthquake resistant structure is a combination of good engineering and good construction in the field. Generally, several design alternatives are present during the planning of a facility. If the facility is to be exposed to seismic occurrences then consideration of the performance of the facility during an earthquake should be considered. This consideration should be of the capability of the structure of resist the imposed forces and the damages sustained by the structure during the earthquake. Good field construction is a function of the materials and equipment available for the job and the provided supervision to insure that construction follows the design.

This dissertation has been written to aid the engineering in estimating the risk associated with a particular design in a seismic environment. Generally, the optimum alternative is not immediately obvious. A mathematical procedure is required which considers the pertinent factors and quantifies the amount of risk. After each alternate design has been evaluated a comparison of the level of risk of each can be made. This will establish the optimum design from the standpoint of the risk from earthquake damage.

The attempt to correlate damage to structures from earthquake occurrence is, at this point, rudimentary. Undoubtedly, more sophisticated analyses will be made in the future after additional data have been collected and analyzed. However, it is extremely important that these attempts be made so that each succeeding attempt to construct an earthquake resistant structure builds on the preceding one and finally, a point is reached where the level of damage to a structure can be correlated to earthquake occurrence with confidence. Furthermore, it is by this means that engineers may be able to tell if progress towards an economical earthquake resistant strucure is being made. If in time the levels of damage to particular types of structures decrease for a given earthquake magnitude then the indication is that progress is being made toward earthquake resistant design.

7.3 Recommendations for Future Research

Although the Markov Decision Model presented incorporates all of the pertinent factors of a seismic risk analysis, it must be borne in mind that the analysis is only as good as the accuracy of the input data. There are two factors where significant error could exist.

The first source of major error would be in the calculation of the probabilities of earthquake occurrence. The historical earthquake represents only a short interval of time. It had to be assumed for the calculation presented that the available record represented the seismicity of the region for any period of time. This is tantamount for assuming that the rate of earthquake occurrence is stationary, which is probably not strictly true. In this case, there is no way to improve the available record and the assumption of stationarity must be accepted.

A second factor which could possess a large error is the calculation of the costs of damage to structures exposed to earthquakes. In this dissertation, an empirical foundation was used to develop these values. However, methods of construction and the materials used in construction evolve with time. Furthermore, the design of comparable buildings varies according to the design philosophies of individual engineers. These factors may appear to be inconsequential, but they may have a large effect on the performance of a structure during an earthquake. This factor needs further investigation. The effect of structural stiffness, fundamental period of vibration and type of foundation on the earthquake damage level should be determined.

APPENDIX A

This appendix contains the historical record of earthquake activity in the region of interest. This record was taken from the tape provided by the U. C. Seismological Station.

The data in the record appears in the following order: number, code, year, month, day, hour, min, sec, secten, longitude, latitude, quality, Richter Magnitude, stations and felt. Our interest is only in the year, longitude, latitude, and Richter Magnitude. From this data the historical two-state probability matrices are constructed. These matrices are used in the generation of the Markov Dependent Bernoulli Trials probabilities.

LARGE EARTHQUAKES

NUMBER	KODE	YEAR	MCNTH	DAY	HOUR	MIN	SEC	SECTEN	LAT	LCNG	QUAL	MAG	STNS	FELT
1995	1	1906	4	18	13	12	0 .		38.05	122.80	D	8.3	0	F
2957	1	1911	7	1	22	0	0 .		37.25	121.75		6.6	0	F

MODERATE EARTHQUAKES

NUMBER	KODE	YEAR	MONTH	DAY	HOUR	MIN	SEC	SECTEN	LAT	LONG	QUAL	MAG	STNS	FELT
4163	1	1926	10	22	12	35	7	.	36.61	122.35	A	6.1	4	F
4165	1	1926	10	22	13	35	22	.	36.57	122.18	A	6.1	4	F
7374	1	1939	6	24	13	2	0	.	36.40	121.00	C	5.5	0	F
16871	1	1955	9	5	2	1	18	.	37.37	121.78	B	5.8	0	F
20454	1	1961	4	9	7	25	41	.	36.70	121.30	D	5.5	0	F
20453	1	1961	4	9	7	23	16	.	36.68	121.30	A	5.6	0	F
23687	1	1969	10	2	4	56	46	.5	38.47	122.69		5.6	0	F
23690	1	1969	10	2	6	19	57	.1	38.46	122.69		5.7	0	F

SMALL EARTHQUAKES

NUMBER	KODE	YEAR	MONTH	DAY	HOUR	MIN	SEC	SECTEN	LAT	LONG	QUAL	MAG	STNS	FELT
.349	1	1932	4	16	18	48	10	.	36.67	121.22	C	4.5	1	F
4877	1	1932	6	14	9	44	17	.	37.25	122.08	D	4.5	1	F
4838	1	1932	2	26	16	58	0	.	36.00	121.00		5.0	0	F
5000	1	1933	5	16	11	45	26	.	37.60	122.00	D	4.5	1	F
6611	1	1936	5	27	19	55	0	.	36.50	121.17	C	4.5	0	
7000	1	1937	2	17	3	33	0	.	36.50	121.58	C	4.5	0	F
7025	1	1937	3	5	12	47	0	.	36.70	121.70	D	4.5	0	F
7028	1	1937	3	8	10	31	0	.	37.80	122.20	D	4.5	0	F
7174	1	1937	8	6	3	24	0	.	38.80	120.10	D	4.5	0	F
7271	1	1937	10	27	15	53	0	.	37.00	122.00	D	4.5	0	F
7364	1	1938	2	12	20	0	14	.	37.00	122.00	D	4.5	0	F
7426	1	1938	5	10	10	32	0	.	36.20	121.30	D	4.5	0	F
7608	1	1938	12	1	16	17	0	.	37.50	121.80	D	4.5	0	F
7562	1	1938	9	27	12	23	0	.	36.30	120.90	C	5.0	0	F
7911	1	1939	7	17	9	25	0	.	36.85	121.68	D	4.5	0	F
8340	1	1940	9	7	13	2	6	.	36.50	121.50	D	4.5	0	F
8533	1	1941	5	28	6	23	18	.	37.00	122.00	D	4.5	0	F
9478	1	1943	10	26	4	50	33	.	37.43	121.68	B	4.9	0	F
10116	1	1945	8	27	9	13	4	.	37.27	121.80	C	4.5	0	F
10035	1	1945	5	17	15	6	47	.	36.82	121.37	B	4.6	0	F
9908	1	1945	1	7	22	25	33	.	36.73	121.20	B	4.7	0	F
10478	1	1946	5	29	17	51	3	.	36.77	121.42	C	4.5	0	F
10438	1	1946	5	2	1	26	12	.	37.68	121.60	A	4.6	0	F
11021	1	1947	6	22	23	30	0	.	37.00	121.77	A	4.7	0	F
10763	1	1947	2	5	6	14	0	.	36.23	120.65	B	5.0	0	F
.515	1	1948	3	28	22	45	0	.	36.85	121.57	C	4.5	0	F
11514	1	1948	3	28	22	38	3	.	36.85	121.57	A	4.6	0	F
11950	1	1949	1	1	1	17	54	.	36.90	121.62	B	4.5	0	F
12270	1	1949	6	10	3	6	40	.	37.30	121.67	B	4.6	0	F
12497	1	1949	10	22	21	45	20	.	36.58	121.17	C	4.7	0	F
12059	1	1949	3	9	12	28	39	.	37.02	121.48	B	5.2	0	F
13881	1	1951	10	31	20	58	19	.	36.90	121.42	A	4.8	0	F
13752	1	1951	8	6	9	5	2	.	36.62	121.22	B	4.9	0	F
13726	1	1951	7	29	10	53	45	.	36.58	121.18	B	5.0	0	F
16348	1	1954	12	17	7	8	58	.	37.72	122.13	B	4.5	0	F
15827	1	1954	4	25	20	33	28	.	36.93	121.68	B	5.3	0	F
16673	1	1955	5	7	11	50	39	.	38.93	122.87	B	4.6	0	F
16550	1	1955	3	2	15	59	1	.	36.00	120.93	B	4.8	0	F
16982	1	1955	11	2	19	40	6	.	36.00	120.92	A	5.2	0	F
16945	1	1955	10	24	4	10	44	.	37.97	122.05	A	5.4	0	F
17681	1	1956	7	23	8	3	48	.	36.30	121.30	D	4.7	0	F
18365	1	1957	9	28	21	4	39	.	36.60	121.23	B	4.5	0	F
18028	1	1957	3	22	19	44	21	.	37.67	122.48	A	5.3	0	
18895	1	1958	9	21	7	24	55	.	36.35	121.12	C	4.6	0	F
19016	1	1958	12	11	9	52	27	.	37.70	122.57	A	4.7	0	F
19344	1	1959	5	26	15	58	1	.	36.72	121.62	C	4.6	0	F
19749	1	1959	12	29	2	32	53	.	36.90	121.48	B	4.7	0	F
19143	1	1959	3	2	23	27	17	.	36.98	121.60	B	5.3	0	F
19780	1	1960	1	20	3	25	53	.	36.78	121.43	B	5.0	0	F
21076	1	1962	4	15	8	41	2	.3	36.42	120.62	B	4.7	23	F
590	1	1963	5	22	22	41	4	.8	37.27	122.31		4.6	12	F
21763	1	1963	9	14	20	28	11	.2	36.92	121.65		4.6	18	F
21761	1	1963	9	14	19	46	17	.0	36.87	121.63		5.4	19	F
22233	1	1964	11	16	2	46	41	.7	37.06	121.69		5.0	17	F
22618	1	1965	9	10	21	28	34	.3	38.01	121.82		4.9	16	F

22845	1	1966	5	13	17	25	55	.9	36.92	121.57	4.5	12	F
23247	1	1967	9	7	12	39	17	.2	37.03	120.80	4.7	18	F
23259	1	1967	9	28	15	38	36	.1	37.23	121.61	4.9	15	F
23309	1	1967	12	18	17	24	32	.0	37.00	121.78	5.3	17	F
395	1	1968	4	25	19	49	45	.2	38.48	122.73	4.6	10	F
23703	1	1969	10	27	10	59	42	.8	36.79	121.39	4.6	18	F
23844	1	1970	3	31	7	2	28	.6	36.86	121.36	4.7	18	F
23951	1	1970	8	4	4	14	21	.4	36.65	122.19	4.7	11	F

VERY SMALL EARTHQUAKES

NUMBER	KODE	YEAR	MONTH	DAY	HOUR	MIN	SEC	SECTEN	LAT	LONG	QUAL	MAG	STNS	FELT
4646	1	1931	1	6	23	28	40.		36.60	122.40	D	4.0	1	F
4823	1	1932	1	29	4	14	8.		36.83	121.42	D	3.5	0	F
4825	1	1932	2	2	20	51	47.		36.56	120.60	C	3.5	1	F
4892	1	1932	7	19	23	35	0.		36.67	121.16	D	3.5	1	F
4815	1	1932	1	8	18	16	59.		36.67	121.33	D	4.0	1	F
4855	1	1932	4	22	0	8	16.		35.78	120.75	C	4.0	1	
4885	1	1932	6	27	5	17	25.		36.00	122.00	D	4.0	0	
5306	1	1934	4	23	21	20	0.		36.75	121.40	D	3.5	0	F
5303	1	1934	4	23	16	8	0.		37.00	122.00	D	4.0	0	F
5422	1	1934	6	16	23	3	0.		36.50	121.00	D	4.0	0	F
5576	1	1934	10	2	20	20	0.		37.60	122.80	D	4.0	0	F
5577	1	1934	10	2	20	30	0.		37.60	122.80	D	4.0	0	F
6189	1	1935	8	9	17	14	0.		36.17	120.98	C	3.5	0	F
6098	1	1935	6	18	4	15	0.		37.00	122.00	D	4.0	0	F
6136	1	1935	6	30	23	28	0.		36.00	121.00	D	4.0	0	F
6309	1	1935	10	25	20	56	0.		36.90	121.75	D	4.0	0	F
6513	1	1936	3	17	1	55	0.		36.50	120.92	C	4.0	0	F
6606	1	1936	5	23	4	41	0.		36.17	120.92	C	4.0	0	F
6314	1	1936	9	24	14	11	0.		37.60	122.88	D	4.0	0	F
7315	1	1937	12	5	1	36	0.		36.00	121.00	D	3.5	0	F
7010	1	1937	2	22	18	10	0.		36.17	121.53	C	4.0	0	F
7240	1	1937	9	18	13	29	0.		36.50	121.50	D	4.0	0	F
7288	1	1937	11	12	2	50	0.		37.00	122.00	D	4.0	0	
7316	1	1937	12	5	1	37	0.		36.00	121.00	D	4.0	0	
7438	1	1938	5	27	22	3	0.		36.20	120.00	D	3.5	0	
7575	1	1938	10	14	13	10	0.		37.00	120.00	D	3.5	0	
7427	1	1938	5	10	10	41	0.		36.20	121.30	D	4.0	0	F
7429	1	1938	5	13	19	34	0.		36.20	121.30	D	4.0	0	
7543	1	1938	9	16	6	11	0.		36.40	121.20	D	4.0	0	F
7576	1	1938	10	14	15	31	0.		36.58	121.40	D	4.0	0	
7974	1	1939	9	24	11	57	40.		36.40	121.00	D	3.5	0	
7898	1	1939	7	4	10	49	0.		36.40	121.00	C	4.0	0	F
7965	1	1939	9	20	2	45	29.		36.68	121.56	C	4.0	0	F
8274	1	1940	6	26	8	56	0.		36.08	120.32	C	3.5	0	
8338	1	1940	9	7	10	36	30.		36.50	121.50	D	3.5	0	
8339	1	1940	9	7	10	38	36.		36.50	121.50	D	3.5	0	
8113	1	1940	3	2	13	27	0.		36.80	121.45	B	4.0	0	F
8317	1	1940	8	13	22	7	29.		36.23	120.32	B	4.0	0	
8351	1	1940	9	19	8	20	18.		38.00	121.00	D	4.0	0	F
8352	1	1940	9	20	18	59	0.		38.00	121.00	D	4.0	0	F
8757	1	1941	12	8	0	29	42.		36.00	121.00	D	3.5	0	
8512	1	1941	4	14	16	16	54.		36.80	121.80	D	4.0	0	F
8672	1	1941	9	18	7	33	0.		37.38	121.68	A	4.0	0	F
9009	1	1942	8	8	22	30	27.		36.90	121.28	C	3.5	0	F
9058	1	1942	10	8	2	30	45.		36.87	120.65	C	3.5	0	
9018	1	1942	8	14	15	14	13.		37.98	121.88	A	3.6	0	F
8874	1	1942	3	6	2	1	12.		36.78	121.52	C	3.7	0	F
9109	1	1942	10	31	12	56	10.		36.57	121.30	B	3.7	0	
8803	1	1942	1	18	3	3	54.		36.67	121.17	B	3.8	0	F
9054	1	1942	10	4	17	49	54.		38.07	120.27	C	3.8	0	
8837	1	1942	2	4	9	8	24.		37.00	121.30	D	4.0	0	F
8901	1	1942	4	8	14	20	14.		36.60	121.30	A	4.0	0	F
8907	1	1942	4	11	8	40	57.		36.75	121.32	B	4.0	0	F
8799	1	1942	1	14	9	44	40.		36.65	121.22	B	4.2	0	F
8958	1	1942	6	5	12	33	25.		36.98	121.67	B	4.2	0	F

9065	1	1942	10	19	13	53	56	.	36.48	121.40	B	4.3	0	F
9184	1	1942	12	29	18	18	14	.	37.72	122.12	A	4.3	0	F
9332	1	1943	5	7	2	1	4	.	36.60	121.10	D	3.5	0	
9404	1	1943	7	27	3	4	5	.	37.57	120.75	C	3.5	0	
9458	1	1943	9	26	17	48	26	.	37.50	121.73	B	3.5	0	F
533	1	1943	12	8	3	43	49	.	37.10	121.07	C	3.5	0	
9265	1	1943	3	31	7	6	9	.	37.68	121.85	B	3.6	0	
9305	1	1943	4	21	9	1	22	.	36.63	121.08	C	3.6	0	
9523	1	1943	11	16	21	36	47	.	37.78	122.12	B	3.6	0	F
9353	1	1943	5	30	1	4	9	.	37.67	121.80	C	3.7	0	F
9249	1	1943	3	19	22	3	40	.	38.80	121.10	D	3.9	0	
9389	1	1943	7	5	16	30	29	.	36.38	121.83	C	3.9	0	
9286	1	1943	4	15	15	23	4	.	37.68	121.77	A	4.0	0	
9528	1	1943	11	30	21	57	18	.	36.30	120.50	D	4.0	0	
9287	1	1943	4	15	15	31	2	.	37.68	121.77	A	4.1	0	F
9312	1	1943	4	26	11	54	0	.	37.62	121.78	A	4.1	0	F
9258	1	1943	3	29	11	45	55	.	37.63	121.87	A	4.2	0	F
9308	1	1943	4	21	23	39	44	.	37.68	121.72	A	4.2	0	F
9842	1	1944	11	4	13	33	41	.	36.60	121.10	D	3.5	0	
9532	1	1944	2	2	11	5	38	.	36.67	120.90	C	3.6	0	
9849	1	1944	11	9	7	2	16	.	36.60	121.30	D	3.6	0	
9890	1	1944	12	15	20	8	54	.	36.57	121.45	C	3.6	0	
9838	1	1944	11	2	5	0	34	.	36.80	121.00	D	3.7	0	
9591	1	1944	2	21	13	0	11	.	36.17	120.93	C	3.8	0	
9604	1	1944	3	13	14	43	15	.	37.45	121.77	B	3.9	0	F
9605	1	1944	3	15	8	14	45	.	36.83	121.62	C	3.9	0	F
9641	1	1944	5	16	0	43	33	.	37.00	121.00	D	3.9	0	F
9655	1	1944	6	7	12	35	38	.	36.58	121.28	B	4.0	0	
9996	1	1945	4	13	14	39	31	.	36.60	121.10	C	3.5	0	
10128	1	1945	9	7	0	36	44	.	38.57	122.12	C	3.5	0	
147	1	1945	9	27	5	24	47	.	37.07	121.08	B	3.5	0	
10187	1	1945	11	21	22	56	10	.	38.42	122.78	B	3.5	0	F
10190	1	1945	11	25	22	40	49	.	37.25	121.57	C	3.5	0	
9954	1	1945	2	25	20	18	38	.	36.00	120.48	C	3.6	0	
10121	1	1945	8	29	19	40	7	.	38.48	121.93	C	3.6	0	
10050	1	1945	6	14	22	57	48	.	36.70	121.45	B	3.7	0	F
10110	1	1945	8	23	17	33	51	.	36.58	121.30	C	3.7	0	
10178	1	1945	11	12	1	12	8	.	37.63	121.82	B	3.7	0	
10170	1	1945	11	3	15	50	22	.	36.63	121.25	C	4.2	0	F
10176	1	1945	11	3	20	9	17	.	37.17	121.52	C	4.2	0	F
10418	1	1946	4	22	7	42	8	.	37.72	121.57	B	3.5	0	F
10641	1	1946	10	1	19	23	3	.	37.52	121.68	B	3.5	0	F
10230	1	1946	2	6	20	51	55	.	36.90	121.40	D	3.6	0	
10475	1	1946	5	25	12	1	30	.	36.57	121.18	A	3.6	0	
10423	1	1946	4	25	21	50	38	.	37.57	121.92	A	3.8	0	F
10253	1	1946	3	5	15	4	27	.	38.70	120.30	C	4.1	0	F
10580	1	1946	8	5	4	9	45	.	36.85	121.78	C	4.1	0	F
10237	1	1946	2	10	11	1	19	.	36.50	121.00	D	4.2	0	F
10930	1	1947	4	12	23	7	0	.	36.52	121.58	B	3.5	0	
11358	1	1947	12	23	2	2	0	.	37.82	121.85	A	3.5	0	F
10938	1	1947	4	14	16	35	44	.	36.52	121.58	B	3.6	0	F
10975	1	1947	5	7	15	41	0	.	36.82	121.20	C	3.6	0	
11342	1	1947	12	16	9	21	3	.	36.25	120.77	C	3.6	0	F
10746	1	1947	1	25	11	49	0	.	36.73	121.88	B	3.7	0	
11215	1	1947	9	25	12	27	51	.	36.87	122.18	B	3.7	0	F
11211	1	1947	9	20	18	1	52	.	36.87	121.87	B	3.8	0	F
299	1	1947	11	15	22	29	36	.	36.78	122.12	B	4.1	0	F
10800	1	1947	2	25	11	45	18	.	35.20	120.50	D	4.2	0	
11037	1	1947	7	7	4	40	0	.	36.77	121.42	A	4.3	0	F
11150	1	1947	8	10	21	56	0	.	36.88	121.42	A	4.4	0	F

11381	1	1948	1	9	21	56	5	.	36.70	121.28	C	3.5	0	
11592	1	1948	5	20	18	29	33	.	37.18	122.28	C	3.5	0	
11627	1	1948	6	11	11	52	9	.	36.60	121.20	D	3.5	0	
1680	1	1948	7	17	15	7	40	.	36.93	121.55	B	3.5	0	
11399	1	1948	1	31	2	54	54	.	37.97	121.95	B	3.6	0	F
11445	1	1948	2	22	1	9	2	.	36.93	121.65	A	3.6	0	F
11553	1	1948	4	13	21	25	10	.	38.17	122.53	C	3.6	0	F
11673	1	1948	7	11	19	58	43	.	37.28	121.77	B	3.6	0	
11717	1	1948	8	7	0	10	46	.	36.90	121.62	B	3.6	0	
11686	1	1948	7	20	6	11	41	.	37.45	121.82	B	3.7	0	F
11386	1	1948	1	14	4	57	1	.	37.87	121.70	A	3.8	0	F
11513	1	1948	3	26	22	36	0	.	36.85	121.57	C	4.0	0	F
11566	1	1948	4	27	16	41	8	.	36.77	121.27	A	4.0	0	
11626	1	1948	6	11	8	28	11	.	36.60	121.20	D	4.0	0	
11383	1	1948	1	11	5	37	28	.	36.43	121.48	B	4.3	0	F
11567	1	1948	4	27	20	22	25	.	36.77	121.27	A	4.4	0	F
12256	1	1949	5	28	17	58	35	.	36.53	121.22	B	3.5	0	
12372	1	1949	8	9	0	39	27	.	38.58	122.67	B	3.6	0	
12483	1	1949	10	17	4	38	6	.	37.00	121.22	A	3.6	0	
12486	1	1949	10	18	12	25	38	.	37.00	121.22	A	3.6	0	
11987	1	1949	1	24	0	10	32	.	36.63	121.33	C	3.7	0	F
12121	1	1949	4	23	9	18	9	.	36.38	121.37	C	3.7	0	
12481	1	1949	10	17	2	41	45	.	37.00	121.22	A	3.8	0	
12038	1	1949	2	25	2	28	2	.	36.90	120.70	D	3.9	0	F
12278	1	1949	6	16	3	47	34	.	36.75	121.67	B	3.9	0	F
12577	1	1949	11	10	5	16	35	.	36.63	121.13	A	3.9	0	F
12612	1	1949	11	30	8	31	54	.	38.62	122.13	B	4.0	0	F
12290	1	1949	6	22	16	8	46	.	37.33	121.68	B	4.1	0	F
12072	1	1949	3	14	6	10	15	.	37.02	121.48	B	4.4	0	F
12611	1	1949	11	30	8	31	45	.	38.00	124.00	C	4.4	0	
12949	1	1950	6	13	3	11	37	.	38.70	120.08	B	3.5	0	F
12833	1	1950	3	25	3	25	32	.	36.63	121.18	B	3.6	0	
12927	1	1950	6	2	17	25	10	.	36.93	121.65	C	3.6	0	F
13168	1	1950	9	30	21	26	33	.	36.90	121.38	B	4.1	0	
13300	1	1950	11	23	13	58	24	.	36.82	121.52	C	4.1	0	F
13737	1	1951	7	29	22	40	26	.	36.57	121.15	C	3.5	0	F
13757	1	1951	8	8	16	55	27	.	36.67	121.68	B	3.6	0	
13792	1	1951	8	25	0	12	12	.	36.63	121.22	C	3.6	0	F
13866	1	1951	10	23	15	47	21	.	36.92	121.53	C	3.6	0	F
13847	1	1951	10	3	14	45	14	.	36.78	121.30	C	3.7	0	F
13949	1	1951	12	20	4	13	6	.	36.00	120.05	C	3.7	0	
13591	1	1951	4	27	11	34	53	.	36.67	121.17	A	3.8	0	F
13695	1	1951	7	9	5	0	8	.	36.62	121.02	C	3.8	0	
13753	1	1951	8	6	9	54	28	.	36.62	121.22	B	3.8	0	F
13714	1	1951	7	24	3	3	34	.	37.92	122.27	A	3.9	0	F
13744	1	1951	8	2	5	9	25	.	36.35	121.27	B	3.9	0	F
13755	1	1951	8	6	17	21	45	.	36.62	121.22	B	3.9	0	F
13879	1	1951	10	30	21	8	46	.	36.90	121.42	A	3.9	0	F
13883	1	1951	11	1	8	8	20	.	36.90	121.42	C	3.9	0	F
13877	1	1951	10	30	19	55	14	.	36.90	121.42	A	4.0	0	F
13878	1	1951	10	30	19	59	18	.	36.90	121.42	B	4.2	0	F
14150	1	1952	5	22	18	7	39	.	38.67	120.27	C	3.5	0	
14540	1	1952	12	2	19	33	15	.	36.70	121.20	D	3.5	0	
13992	1	1952	1	31	21	33	12	.	36.40	121.40	C	3.6	0	
14433	1	1952	9	13	9	6	47	.	36.62	121.42	B	3.8	0	F
14470	1	1952	10	22	0	45	51	.	37.80	122.15	D	4.0	0	F
14458	1	1952	10	13	0	34	9	.	37.75	122.18	B	4.2	0	F
14737	1	1953	2	28	1	0	20	.	36.57	121.15	A	3.5	0	
14862	1	1953	4	29	5	26	53	.	36.00	121.15	C	3.5	0	F
15404	1	1953	12	16	0	5	26	.	36.92	121.65	B	3.5	0	F

15414	1	1953	12	17	6	54	47	.	36.92	121.67	B	3.5	0	F
14946	1	1953	5	23	21	16	22	.	37.30	121.62	A	3.6	0	F
15415	1	1953	12	17	5	39	41	.	36.92	121.67	B	3.6	0	F
15407	1	1953	12	16	10	54	22	.	36.55	121.40	C	3.7	0	F
15410	1	1953	12	17	4	50	30	.	36.92	121.67	B	3.7	0	F
15115	1	1953	7	25	16	13	58	.	37.12	121.77	B	3.8	0	F
15246	1	1953	9	22	7	36	58	.	36.40	121.20	D	3.8	0	
15292	1	1953	10	19	23	35	25	.	37.35	121.58	A	3.8	0	
15409	1	1953	12	16	23	9	39	.	36.92	121.67	B	3.8	0	F
15445	1	1953	12	28	1	32	42	.	36.90	121.62	C	3.8	0	F
14920	1	1953	5	15	3	43	17	.	36.60	121.03	C	3.9	0	
14760	1	1953	3	16	8	52	6	.	36.95	121.67	B	4.0	0	F
14951	1	1953	5	25	0	23	30	.	36.82	121.47	B	4.0	0	F
15412	1	1953	12	17	5	13	12	.	36.92	121.67	A	4.2	0	F
15572	1	1954	2	8	15	23	45	.	37.80	122.13	A	3.5	0	F
15764	1	1954	4	1	0	36	24	.	36.73	121.30	B	3.5	0	F
15628	1	1954	4	25	21	25	25	.	36.93	121.68	C	3.5	0	
15914	1	1954	6	4	11	53	38	.	36.45	121.13	C	3.5	0	
15722	1	1954	3	21	15	59	29	.	36.80	121.40	D	3.6	0	
15953	1	1954	6	22	17	8	0	.	36.57	121.42		3.6	0	
16375	1	1954	12	22	21	12	24	.	36.00	121.00	D	3.7	0	F
15618	1	1954	4	21	2	36	43	.	37.03	121.65	B	3.8	0	
15820	1	1954	4	22	18	44	10	.	36.90	121.68	B	3.8	0	
15949	1	1954	6	22	11	51	17	.	36.57	121.42		3.8	0	
16376	1	1954	12	22	21	12	28	.	36.00	120.60	D	3.8	0	
15946	1	1954	6	22	9	33	3	.	36.57	121.42	A	3.9	0	
16100	1	1954	8	12	12	50	6	.	36.90	121.65	B	3.9	0	F
16254	1	1954	11	1	6	42	40	.	37.07	120.58	C	4.0	0	F
15790	1	1954	4	10	22	16	53	.	37.18	121.80	A	4.1	0	F
16073	1	1954	7	29	8	51	36	.	37.42	121.33	B	4.2	0	F
15821	1	1954	4	22	18	50	13	.	36.90	121.68	B	4.3	0	F
15951	1	1954	6	22	12	50	4	.	36.57	121.42		4.3	0	F
15747	1	1954	3	27	15	43	31	.	36.52	121.17	C	4.4	0	F
15948	1	1954	6	22	11	49	29	.	36.57	121.42	A	4.4	0	F
17069	1	1955	12	11	20	10	38	.	36.27	120.72	C	3.5	0	
16658	1	1955	4	29	15	14	38	.	38.95	122.77	B	3.6	0	F
16999	1	1955	11	10	18	2	16	.	37.83	122.05	B	3.6	0	F
16474	1	1955	1	26	17	22	41	.	37.18	121.60	A	3.7	0	
16549	1	1955	3	2	6	4	43	.	36.87	121.65	B	3.7	0	F
16551	1	1955	3	2	20	2	53	.	36.00	120.93	B	3.7	0	
16979	1	1955	11	1	8	50	54	.	37.98	122.03	C	3.7	0	F
16718	1	1955	6	8	0	2	59	.	36.78	121.43	C	3.8	0	F
17081	1	1955	12	16	14	43	11	.	36.03	120.87	C	3.8	0	F
16674	1	1955	5	7	14	56	15	.	38.93	122.87	B	4.2	0	
16938	1	1955	10	22	7	4	13	.	36.22	120.33	C	4.2	0	F
17668	1	1956	7	18	23	3	7	.	38.65	122.73	A	3.5	0	F
17369	1	1956	3	15	10	16	11	.	36.62	121.33	C	3.6	0	
17811	1	1956	10	7	2	38	19	.	37.82	121.83	C	3.6	0	F
17297	1	1956	2	19	0	6	42	.	36.67	121.30	B	3.8	0	F
17303	1	1956	2	19	3	23	37	.	36.67	121.30	B	3.8	0	F
17361	1	1956	3	11	19	5	43	.	36.55	121.16	C	3.8	0	F
17838	1	1956	10	19	12	32	3	.	36.65	121.23	C	4.1	0	F
17295	1	1956	2	18	23	58	30	.	36.67	121.32	B	4.2	0	F
17429	1	1956	4	5	4	29	32	.	38.53	122.52	C	4.2	0	F
17881	1	1956	11	22	16	43	58	.	36.60	121.30	D	4.2	0	F
17428	1	1956	4	5	4	29	13	.	38.53	122.52	B	4.4	0	F
17892	1	1956	12	1	14	11	25	.	36.87	121.60	B	4.4	0	F
18056	1	1957	3	23	22	48	0	.	37.65	122.45	A	3.5	0	F
18154	1	1957	4	29	3	7	38	.	37.95	122.00	B	3.5	0	F
18302	1	1957	8	13	23	14	6	.	36.75	121.63	C	3.5	0	F

17978	1	1957	2	14	10	30	27.	36.00	120.60	C	3.6	0	
18039	1	1957	3	22	21	7	47.	37.68	122.48	A	3.6	0	F
18313	1	1957	8	21	7	38	34.	36.47	121.52	C	3.6	0	
18027	1	1957	3	22	18	48	23.	37.67	122.47	A	3.8	0	F
18040	1	1957	3	22	21	18	29.	37.65	122.48	A	3.8	0	F
18052	1	1957	3	23	12	54	32.	37.65	122.48	A	3.8	0	F
18329	1	1957	9	3	5	19	21.	37.05	121.50	C	3.9	0	F
18042	1	1957	3	23	0	26	55.	37.65	122.48	A	4.0	0	F
18411	1	1957	10	31	19	47	6.	37.35	122.22	A	4.1	0	F
18050	1	1957	3	23	8	13	48.	37.70	122.52	A	4.2	0	F
18041	1	1957	3	22	23	14	35.	37.65	122.45	A	4.4	0	F
18641	1	1958	4	21	6	40	26.	36.87	121.30	C	3.6	0	
18657	1	1958	5	4	15	21	58.	37.18	121.60	B	3.6	0	
18690	1	1958	5	27	23	9	32.	36.90	121.60	D	3.8	0	F
18622	1	1958	4	4	17	5	48.	36.67	121.33	C	3.9	0	F
18811	1	1958	8	8	13	43	15.	36.30	121.20	D	3.9	0	F
18980	1	1958	11	27	6	4	26.	36.37	121.15	C	3.9	0	F
18639	1	1958	4	20	21	6	58.	38.62	122.35	B	4.0	0	F
18700	1	1958	5	31	22	7	11.	37.97	122.00	A	4.1	0	F
18768	1	1958	7	9	5	23	40.	37.25	121.67	B	4.1	0	F
18937	1	1958	10	31	0	26	14.	37.48	121.78	A	4.2	0	F
18949	1	1958	11	7	21	33	24.	36.87	121.88	B	4.3	0	F
19154	1	1959	3	4	20	55	6.	37.00	121.62	B	3.5	0	F
19164	1	1959	3	10	0	18	40.	36.98	121.58	C	3.5	0	
19303	1	1959	5	9	17	26	3.	37.62	122.50	B	3.5	0	F
19105	1	1959	2	2	14	26	58.	36.53	121.10	B	3.6	0	
19155	1	1959	3	4	21	6	21.	37.00	121.60	B	3.6	0	F
19366	1	1959	6	8	10	23	38.	36.62	121.23	C	3.6	0	F
19617	1	1959	10	14	8	37	51.	36.87	121.48	C	3.6	0	F
19641	1	1959	10	25	9	59	52.	37.00	121.30	D	3.6	0	
19709	1	1959	12	2	9	44	43.	37.47	121.13	B	3.6	0	
19133	1	1959	2	25	4	13	3.	36.73	121.38	C	3.7	0	F
19369	1	1959	6	11	1	28	58.	37.33	121.65	A	3.7	0	F
19151	1	1959	3	4	8	1	43.	37.00	121.62	C	3.8	0	F
19552	1	1959	9	5	5	45	34.	36.50	121.70	D	3.8	0	
19149	1	1959	3	3	18	32	13.	36.98	121.60	B	4.0	0	F
19562	1	1959	9	11	18	5	3.	36.70	121.30	D	4.0	0	F
19201	1	1959	3	24	2	21	13.	37.27	121.72	C	4.1	0	F
19609	1	1959	10	11	2	3	9.	36.45	121.12	C	4.1	0	F
19731	1	1959	12	16	2	28	42.	38.58	122.37	C	4.1	0	F
19098	1	1959	1	29	16	41	23.	37.13	121.57	A	4.3	0	F
19145	1	1959	3	3	7	23	46.	37.00	121.60	B	4.4	0	F
19929	1	1960	4	18	2	16	21.	36.62	121.23	B	3.5	0	
20040	1	1960	6	24	18	13	12.	36.45	121.22	B	3.5	0	
20052	1	1960	6	28	12	40	44.	36.92	121.75	C	3.5	0	F
20249	1	1960	11	20	23	50	1.	36.82	121.45	C	3.5	0	F
19781	1	1960	1	20	3	47	51.	36.78	121.43	B	3.6	0	F
19918	1	1960	4	9	8	1	14.	36.50	121.13	B	3.6	0	
19819	1	1960	2	13	17	16	49.	36.85	121.53	C	3.7	0	F
20020	1	1960	6	11	17	39	48.	36.30	120.90	D	3.7	0	
20206	1	1960	10	23	3	43	7.	36.80	121.40	B	3.8	0	F
20275	1	1960	12	15	5	40	26.	38.03	121.83	B	3.9	0	F
20219	1	1960	11	3	6	50	24.	36.53	121.13	B	4.1	0	F
20331	1	1961	1	13	8	36	12.	36.92	121.75	C	3.5	0	
20448	1	1961	4	7	22	4	42.	36.55	121.38	A	3.5	0	
20482	1	1961	4	28	14	5	50.	36.62	121.42	B	3.5	0	F
20583	1	1961	5	26	12	58	35.	36.78	121.55	C	3.5	0	F
20632	1	1961	6	27	18	27	8.	37.82	122.28	B	3.5	0	F
20802	1	1961	10	19	8	8	47.	37.42	121.77	B	3.5	0	
20312	1	1961	1	3	23	0	21.	36.87	121.67	B	3.6	0	

20381	1	1961	2	16	20	12	58	.	37.83	122.22	A	3.6	0	F
20625	1	1961	6	25	13	15	26	.	36.48	121.35	C	3.6	0	F
20857	1	1961	11	12	4	20	11	.	36.97	121.67	B	3.7	0	
20860	1	1961	11	17	2	16	56	.	37.97	122.03	A	3.8	0	
20694	1	1961	6	18	1	30	37	.	37.93	122.00	B	3.9	0	F
0604	1	1961	7	22	18	1	55	.	36.40	121.20	C	4.0	0	F
20313	1	1961	1	4	0	30	17	.	36.87	121.67	B	4.1	0	F
20479	1	1961	4	28	1	2	52	.	36.60	121.37	C	4.2	0	F
21043	1	1962	4	2	16	41	56	.8	37.04	121.51	B	3.5	20	F
21052	1	1962	4	7	7	19	37	.5	37.28	121.92	B	3.5	20	F
21153	1	1962	7	4	8	26	38	.2	37.88	120.10	B	3.5	10	
21247	1	1962	9	26	10	19	30	.6	36.90	121.73	B	3.6	19	F
21324	1	1962	11	28	3	3	41	.8	36.73	121.02	B	3.6	19	
21326	1	1962	11	28	3	23	7	.9	36.72	121.02	B	3.6	20	
20942	1	1962	1	24	15	13	5	.5	37.83	122.25	B	3.7	11	F
21042	1	1962	4	2	3	6	3	.2	36.25	120.10	B	3.7	16	F
21258	1	1962	10	13	17	49	39	.5	36.35	120.42	B	3.7	17	
21373	1	1962	12	24	0	16	23	.4	36.85	121.78	B	3.7	17	F
21128	1	1962	6	7	4	20	58	.9	37.27	121.72	B	3.8	18	F
20960	1	1962	1	31	3	17	21	.3	36.55	121.22	B	3.9	15	F
20918	1	1962	1	1	17	20	59	.5	38.88	123.40	A	4.1	15	F
21027	1	1962	3	17	21	38	44	.5	37.82	121.88	B	4.1	20	F
21614	1	1963	5	30	20	26	39	.4	36.58	121.25		3.5	14	
21849	1	1963	11	18	7	31	38	.5	36.22	120.30	C	3.5	0	F
21427	1	1963	2	8	9	44	24	.1	36.63	121.48	B	3.6	19	
21666	1	1963	7	16	18	17	54	.6	36.86	121.61		3.6	16	F
21808	1	1963	11	29	20	49	36	.0	36.79	121.59		3.6	17	
21420	1	1963	2	2	13	58	20	.1	36.74	121.59	B	3.7	27	F
21560	1	1963	5	6	3	4	28	.8	36.64	121.34		3.7	19	F
21774	1	1963	9	21	4	32	45	.2	37.26	121.67		3.8	17	F
21620	1	1963	6	7	12	4	42	.2	37.97	122.04		3.9	16	F
1693	1	1963	7	31	6	45	53	.4	36.84	121.39		3.9	17	F
21898	1	1963	12	30	13	47	7	.8	38.89	122.51		3.9	20	F
21697	1	1963	8	4	17	35	3	.6	37.61	122.56		4.0	15	F
21737	1	1963	8	31	16	31	14	.2	36.76	121.58		4.2	19	F
21563	1	1963	5	7	7	7	48	.0	36.86	121.65		4.4	17	F
22245	1	1964	11	23	9	5	8	.0	37.04	121.72		3.5	0	
22261	1	1964	11	30	21	16	18	.2	36.87	121.70		3.7	14	
22151	1	1964	9	1	19	49	16	.5	36.87	121.67		3.8	16	F
22049	1	1964	5	13	12	18	37	.2	36.55	121.16		4.0	17	F
22226	1	1964	11	8	1	19	19	.0	36.00	120.00		4.0	15	
22400	1	1965	3	28	2	32	21	.0	36.20	120.40		3.5	0	
22420	1	1965	4	11	5	41	56	.6	36.51	121.15		3.5	11	
22557	1	1965	7	31	6	54	27	.5	36.55	121.20		3.5	14	
22630	1	1965	9	20	1	15	47	.5	37.79	122.18		3.5	15	F
22513	1	1965	6	28	11	15	11	.4	37.56	121.67		3.6	14	F
22538	1	1965	7	18	19	3	43	.4	36.79	121.56		3.6	15	F
22375	1	1965	2	22	17	57	11	.8	36.77	120.88		3.8	14	
22575	1	1965	6	15	23	6	52	.5	36.00	121.20		4.0	0	F
22624	1	1965	9	14	9	9	24	.2	36.63	121.36		4.0	17	F
22777	1	1966	1	21	4	10	36	.0	36.98	121.47		3.5	14	F
23103	1	1966	12	23	2	23	37	.	38.85	123.17		3.5	8	F
22814	1	1966	3	16	18	24	4	.1	36.78	121.55		3.6	8	F
22855	1	1966	6	6	7	23	13	.9	37.32	121.74		3.7	14	F
22834	1	1966	4	29	8	9	27	.2	36.61	121.25		3.8	10	F
22600	1	1966	2	10	14	21	8	.	37.80	121.70		3.9	8	F
2769	1	1966	1	17	2	3	20	.0	36.98	121.49		4.1	16	F
23061	1	1966	10	10	6	53	46	.	36.58	121.22		4.1	12	F
23062	1	1966	10	14	20	34	28	.9	36.98	121.75		4.2	12	F
23183	1	1967	5	25	8	19	58	.	38.90	124.00		3.5	14	

3261	1	1967	9	28	21	8	11	.6	37.23	121.61	3.5	12	F
[illegible]	1	1967	7	22	9	23	26	.6	36.53	121.16	3.8	16	F
[illegible]	1	1967	5	30	3	47	28	.4	37.27	121.67	3.9	14	F
23375	1	1968	3	25	16	25	55	.4	36.63	121.29	3.5	11	
23455	1	1968	6	20	7	50	27	.5	36.83	121.60	3.5	7	
23374	1	1968	3	25	11	32	7	.4	36.37	120.70	3.6	8	F
23356	1	1968	2	21	14	39	48	.0	37.19	121.56	3.8	10	F
23543	1	1968	12	11	21	37	37	.4	37.16	121.56	3.9	7	F
23371	1	1968	3	21	21	55	0	.3	37.02	121.75	4.3	7	F
23588	1	1969	3	13	3	23	25	.8	36.05	121.86	3.5	8	F
23686	1	1969	10	2	5	14	21	.	38.50	122.70	3.5	0	
23726	1	1969	12	7	11	54	41	.9	36.92	121.45	3.5	11	
23697	1	1969	10	6	14	28	7	.	38.45	122.71	3.9	0	F
23608	1	1969	5	8	22	10	53	.1	38.70	122.17	4.0	10	F
23716	1	1969	11	15	20	58	3	.6	36.75	121.41	4.2	9	
23720	1	1969	11	19	6	28	50	.	36.45	121.52	4.2	8	F
23692	1	1969	10	2	12	27	5	.5	38.49	122.68	4.3	0	F
23719	1	1969	11	17	20	49	19	.5	36.43	121.05	4.4	10	F
23892	1	1970	5	26	22	10	35	.2	37.80	121.94	3.5	8	F
23901	1	1970	5	29	2	55	49	.5	37.80	121.94	3.5	8	F
23962	1	1970	8	16	16	29	5	.2	36.63	121.30	3.5	9	F
23989	1	1970	9	23	4	51	27	.9	37.40	122.22	3.5	9	F
24022	1	1970	11	9	13	35	52	.1	36.96	121.61	3.5	9	F
23971	1	1970	8	30	13	16	50	.9	36.91	121.49	3.6	11	F
23972	1	1970	8	31	12	12	58	.7	38.11	121.95	3.6	9	F
23752	1	1970	1	3	2	51	58	.4	37.30	122.09	3.7	25	F
23960	1	1970	8	13	5	6	19	.8	36.17	121.70	3.7	11	
2395	1	1970	5	26	23	33	39	.9	37.80	121.95	3.8	8	F
23913	1	1970	6	12	3	30	55	.0	37.80	121.93	3.9	8	F
23757	1	1970	1	6	2	29	7	.5	36.53	121.15	4.0	19	F
23919	1	1970	6	12	16	3	32	.1	37.81	121.94	4.2	6	F
23912	1	1970	6	12	3	30	4	.0	37.60	121.93	4.3	8	F

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EARTHQUAKES NOT CONSIDERED

NUMPER	KODE	YEAR	MONTH	DAY	HOUR	MIN	SEC	SECTEN	LAT	LONG	QUAL	MAG	STNS	FELT
17603	1	1956	6	23	20	3	20 .		36.52	121.30	B	3.3	0	
17444	1	1956	4	10	14	5	8 .		37.83	121.88	C	1.2	0	
17446	1	1956	4	10	14	29	56 .		37.80	121.90	D	1.2	0	
17553	1	1956	5	25	0	1	9 .		38.00	122.43	C	1.4	0	
17600	1	1956	6	21	21	25	25 .		37.18	122.15	C	1.4	0	
17616	1	1956	7	2	1	44	52 .		37.20	122.12	C	1.5	0	
17449	1	1956	4	10	17	52	52 .		37.83	121.88	C	1.6	0	
17447	1	1956	4	10	16	25	14 .		37.83	121.88	C	1.7	0	
17472	1	1956	4	19	6	58	57 .		37.22	121.78	C	1.8	0	
17485	1	1956	4	25	13	17	51 .		37.88	122.03	B	1.8	0	
17443	1	1956	4	10	13	51	5 .		37.83	121.88	C	1.9	0	
17457	1	1956	4	12	15	43	56 .		37.15	122.22	C	1.9	0	
17475	1	1956	4	19	19	13	12 .		37.15	122.23	B	1.9	0	
17486	1	1956	4	25	23	57	54 .		38.32	122.38	B	1.9	0	
17468	1	1956	4	16	5	27	6 .		37.28	121.55	A	2.0	0	
17473	1	1956	4	19	14	3	42 .		37.22	121.65	C	2.0	0	
17511	1	1956	5	8	22	55	4 .		37.22	122.22	B	2.0	0	
17496	1	1956	4	30	3	5	12 .		37.25	121.88	A	2.1	0	
17510	1	1956	5	8	12	2	5 .		37.33	121.68	B	2.1	0	
17554	1	1956	5	25	12	4	1 .		37.97	121.97	A	2.1	0	
17483	1	1956	4	25	1	46	22 .		36.70	121.50	D	2.2	0	
17499	1	1956	5	3	0	58	13 .		37.25	121.70	D	2.2	0	
17522	1	1956	5	12	13	28	24 .		37.37	121.77	A	2.2	0	
17558	1	1956	5	26	19	32	23 .		37.95	122.55	B	2.2	0	
17574	1	1956	6	5	1	26	18 .		38.18	121.85	C	2.2	0	
17607	1	1956	6	24	7	23	14 .		37.28	121.67	A	2.2	0	
17552	1	1956	5	24	22	49	10 .		37.20	121.62	C	2.3	0	
17575	1	1956	6	5	16	28	9 .		37.53	121.68	B	2.3	0	
17588	1	1956	6	14	4	17	40 .		36.60	121.40	D	2.3	0	
17597	1	1956	6	19	18	32	57 .		38.28	122.52	B	2.3	0	
17620	1	1956	7	3	17	28	43 .		37.40	121.85	B	2.3	0	
17448	1	1956	4	10	17	44	41 .		37.83	121.88	C	2.4	0	
17495	1	1956	4	29	20	16	15 .		36.60	121.20	D	2.4	0	
17505	1	1956	5	5	21	59	9 .		37.37	121.78	A	2.4	0	
17514	1	1956	5	10	4	7	4 .		36.95	121.68	B	2.4	0	
17549	1	1956	5	23	16	54	0 .		36.92	121.58	C	2.4	0	
17497	1	1956	5	1	15	6	33 .		36.50	121.00	D	2.5	0	
17560	1	1956	5	28	18	0	41 .		37.82	121.83	A	2.5	0	
17595	1	1956	6	18	5	52	17 .		36.58	121.30	A	2.5	0	
17596	1	1956	6	18	22	30	8 .		36.60	121.40	D	2.5	0	
17504	1	1956	5	5	18	33	47 .		37.40	121.75	A	2.6	0	
17573	1	1956	6	4	13	5	8 .		37.65	121.67	A	2.6	0	
17589	1	1956	6	15	12	31	14 .		36.67	121.50	C	2.6	0	
17608	1	1956	6	24	12	47	48 .		36.97	121.77	B	2.6	0	
17490	1	1956	4	28	11	3	38 .		36.55	121.30	C	2.7	0	
17594	1	1956	6	17	11	58	56 .		36.92	121.72	B	2.7	0	
17609	1	1956	6	24	23	46	15 .		36.55	121.15	C	2.7	0	
17453	1	1956	4	11	13	14	45 .		38.47	122.48	B	2.8	0	
17591	1	1956	6	15	23	42	3 .		36.30	121.80	D	2.8	0	
17593	1	1956	6	17	2	12	34 .		37.18	121.60	B	2.8	0	
17450	1	1956	4	10	20	53	21 .		36.30	121.00	D	2.9	0	
17487	1	1956	4	27	22	28	59 .		37.53	121.70	B	2.9	0	F
17492	1	1956	4	29	4	19	35 .		36.60	121.30	D	2.9	0	
17494	1	1956	4	29	8	46	3 .		38.73	120.15	C	3.0	0	
17500	1	1956	5	3	3	30	30 .		38.43	122.53	C	3.0	0	

17454	1	1956	4	11	13	15	5 .	38.47	122.48	B	3.1	0	
17445	1	1956	4	10	14	9	20 .	37.83	121.88	A	3.2	0	F
17585	1	1956	6	11	0	48	37 .	36.00	120.97	C	3.2	0	
17452	1	1956	4	11	13	12	17 .	38.47	122.48	A	3.3	0	F

APPENDIX B

The Markov Dependent Bernoulli Trials model is developed in this appendix. Development of the model follows closely that presented by Parzen [18].

The Markov Dependent Bernoulli Trials model used to calculate the probability of earthquake occurrence is developed herein. It obeys the one-step memory concept. Given n trials of an experiment with two possible outcomes denoted s for success and f for failure one defines them to be Markov Dependent if the Markov property is satisfied. Consider an integer k having values from 1 to $n-1$ and $k+1$ events A_1, $A_2 \ldots A_{k+1}$ depending respectively on the first, second ...(k+1) trials. The trials are Markov Dependent if

$$P\left[A_{k+1}/A_k,\ A_{k-1},\ \ldots A_1\right] = P\left[Ak+1/Ak\right] \quad .$$

Thus, the probability of the event A_{k+1} depends only on the event A_k and is said to possess a "one-step memory."

In order to develop the model the following definitions[18] must be made.

$p(s,s)$ = probability of success on the (k+1)st trial, given a success on the k th trial.

$p(f,s)$ = probability of success at the (k+1)st trial, given a failure on the k th trial.

$p(f,f)$ = probability of failure on the (k+1)st trial, given a failure on the k th trial.

$p(s,f)$ = probability of failure on the (k+1)st trial, given a success on the k th trial.

These probabilities are independent of the trial number k. Hence, in the case of earthquakes they can be calculated from the historical record. The probabilities which are necessary to compute are the following:

$p_k(s,s)$ = conditional probability of success at the $(k+1)$st trial, given success at the first trial.

$p_k(s,f)$ = conditional probability of failure at the $(k+1)$st trial, given success at the first trial.

$p_k(f,f)$ = conditional probability of failure at the $(k+1)$st trial, given failure at the first trial.

$p_k(f,s)$ = conditional probability of success at the $(k+1)$st trial, given failure at the first trial.

These are the probabilities that one seeks to calculate. Not all four probabilities are independent. The following relations indicate that it is sufficient to calculate the probabilities $p_k(s,f)$ and $p_k(f,s)$.

$$p_k(s,f) = 1-p_k(s,s)$$

$$p_k(f,s) = 1-p_k(f,f) \quad .$$

Now consider having had a success on the first tiral and either a success or failure at the $(k-1)$st trial. Then, $p_k(s,s)$ can be written as

$$p_k(s,s) = p_{k-1}(s,s)\ p(s,s) + p_{k-1}(s,f)\ p(f,s)$$

and

$$p_k(f,f) = p_{k-1}(f,f)\ p(f,f) + p_{k-1}(f,s)\ p(s,f)$$

for

$$k = 2,3\ldots n \quad .$$

Rearranging and using the preceding equations

$$p_k(s,s) = p_{k-1}(s,s)\ [p(s,s) + p(f,f)-1] + [1 - p(f,f)]$$

$$p_k(f,f) = p_{k-1}(f,f)\ [p(s,s) + p(f,f)-1] + [1 - p(s,s)] \quad .$$

Thus, what is obtained here are two difference equations. If

$$a = p(s,s) + p(f,f)-1$$

$$b = 1-p(f,f)$$

$$c = 1-p(s,s)$$

with the condition that

$$|p(s,s) + p(f,f) - 1| < 1$$

the the difference equations can be solved. The solution for the first equation

$$p_k(s,s) = a\ p_{k-1}(s,s) + b$$

is

$$p_k(s,s) = \left[p_1(s,s) - \frac{b}{1-a}\right] a^{n-1} + \frac{b}{1-a} \quad .$$

The solution to the second equation

$$p_k(f,f) = a\, p_{k-1}(f,f) + c$$

is

$$p_k(f,f) = \left[p_1(f,f) - \frac{c}{1-a}\right] a^{n-1} + \frac{c}{1-a} \quad .$$

These equations can be written out as

$$p_k(s,s) = \left[p_1(s,s) - \frac{1-p(f,f)}{2-p(s,s)-p(f,f)}\right] \times \left[p(s,s)+p(f,f)-1\right]^{k-1} + \left[\frac{1-p(f,f)}{2-p(s,s)-p(f,f)}\right]$$

$$p_k(f,f) = \left[p_1(f,f) - \frac{1-p(s,s)}{2-p(s,s)-p(f,f)}\right] \times \left[p(s,s)+p(f,f)-1\right]^{k-1} + \left[\frac{1-p(s,s)}{2-p(s,s)-p(f,f)}\right] \quad .$$

However, note that

$$p_1(s,s) = p(s,s)$$

$$p_1(f,f) = p(f,f) \quad .$$

Therefore,

$$p_k(s,s) = \frac{1-p(s,s)}{2-p(s,s)-p(f,f)} \left[p(s,s)+p(f,f)-1\right]^k + \left[\frac{1-p(f,f)}{2-p(s,s)-p(f,f)}\right]$$

$$p_k(f,f) = \frac{1-p(f,f)}{2-p(s,s)-p(f,f)} \left[p(s,s)+p(f,f)-1\right]^k + \left[\frac{1-p(s,s)}{2-p(s,s)-p(f,f)}\right]$$

for

$$k = 1,2\ldots n \quad .$$

APPENDIX C

The following computer generates the matrix of probabilities for the model Markov Dependent Bernoulli Trials.

The program is written in Fortran Five for the IBM 360-67. It begins by reading the historical probabilities p(s,s) and p(f,f). These are the only quantities necessary to perform the calculation. In the program these historical probabilities are designated pss and pff respectively.

The program generates probabilities for a period of 30 years. The output is $p_k(s,s)$, $p_k(f,f)$, $p_k(s,f)$ and $p_k(f,s)$ which in the program are designated pkss, pkff, pksf and pkfs, respectively.

There are four outputs corresponding to large, moderate, small and very small earthquake designations. The outputs follow the computer program.

```
$WATFIV
      100 READ(5,801)  PSS,PFF
      801 FORMAT (2F10.7)
          REAL PKSS,PKSF,PKFS,PKFF
**WARNING**  ST-9
          WRITE(6,601)
      601 FORMAT('1',40X,'LARGE EARTHQUAKES'//)
          WRITE(6,602)
      602 FORMAT(' ',20X,'PSS',14X,'PFF'/)
          WRITE(6,603) PSS,PFF
      603 FORMAT(' ',15X,F10.6,7X,F10.6//)
          WRITE(6,604)
      604 FORMAT(' ',20X,'K',15X,'PKSS',10X,'PKSF',10X,'PKFS',10X,'PKFF'/)
          K=1
          AGF=(2.0-PSS-PFF)
          ABC=(1.0-PSS)/AGF
          CDE=(1.0-PFF)/AGF
          EFG=(PSS+PFF-1.0)
      300 CONTINUE
          PKSS=((ABC)*((EFG)**K))+CDE
          PKFF=((CDE)*((EFG)**K))+ABC
          PKSF=-((ABC)*((EFG)**K))+ABC
          PKFS=-((CDE)*((EFG)**K))+CDE
          WRITE(6,605) K,PKSS,PKSF,PKFS,PKFF
      605 FORMAT(' ',16X,I5,11X,F10.6, 4X,F10.6, 4X,F10.6, 4X,F10.6)
          K=K+1
          AK=K
          IF ((AK-31.0) .LT. 0.)  GO TO 300
          STOP
          END
```

LARGE EARTHQUAKES

PSS	PFF
0.000000	0.969700

K	PKSS	PKSF	PKFS	PKFF
1	0.000000	1.000000	0.030300	0.969701
2	0.030300	0.969701	0.029382	0.970619
3	0.029382	0.970619	0.029410	0.970591
4	0.029410	0.970591	0.029409	0.970592
5	0.029409	0.970592	0.029409	0.970592
6	0.029409	0.970592	0.029409	0.970592
7	0.029409	0.970592	0.029409	0.970592
8	0.029409	0.970592	0.029409	0.970592
9	0.029409	0.970592	0.029409	0.970592
10	0.029409	0.970592	0.029409	0.970592
11	0.029409	0.970592	0.029409	0.970592
12	0.029409	0.970592	0.029409	0.970592
13	0.029409	0.970592	0.029409	0.970592
14	0.029409	0.970592	0.029409	0.970592
15	0.029409	0.970592	0.029409	0.970592
16	0.029409	0.970592	0.029409	0.970592
17	0.029409	0.970592	0.029409	0.970592
18	0.029409	0.970592	0.029409	0.970592
19	0.029409	0.970592	0.029409	0.970592
20	0.029409	0.970592	0.029409	0.970592
21	0.029409	0.970592	0.029409	0.970592
22	0.029409	0.970592	0.029409	0.970592
23	0.029409	0.970592	0.029409	0.970592
24	0.029409	0.970592	0.029409	0.970592
25	0.029409	0.970592	0.029409	0.970592
26	0.029409	0.970592	0.029409	0.970592
27	0.029409	0.970592	0.029409	0.970592
28	0.029409	0.970592	0.029409	0.970592
29	0.029409	0.970592	0.029409	0.970592
30	0.029409	0.970592	0.029409	0.970592

MODERATE EARTHQUAKES

PSS	PFF
0.000000	0.924240

K	PKSS	PKSF	PKFS	PKFF
1	0.000000	1.000000	0.075760	0.924240
2	0.075760	0.924240	0.070020	0.929980
3	0.070020	0.929980	0.070455	0.929545
4	0.070455	0.929545	0.070422	0.929578
5	0.070422	0.929578	0.070425	0.929575
6	0.070425	0.929575	0.070425	0.929575
7	0.070425	0.929575	0.070425	0.929575
8	0.070425	0.929575	0.070425	0.929575
9	0.070425	0.929575	0.070425	0.929575
10	0.070425	0.929575	0.070425	0.929575
11	0.070425	0.929575	0.070425	0.929575
12	0.070425	0.929575	0.070425	0.929575
13	0.070425	0.929575	0.070425	0.929575
14	0.070425	0.929575	0.070425	0.929575
15	0.070425	0.929575	0.070425	0.929575
16	0.070425	0.929575	0.070425	0.929575
17	0.070425	0.929575	0.070425	0.929575
18	0.070425	0.929575	0.070425	0.929575
19	0.070425	0.929575	0.070425	0.929575
20	0.070425	0.929575	0.070425	0.929575
21	0.070425	0.929575	0.070425	0.929575
22	0.070425	0.929575	0.070425	0.929575
23	0.070425	0.929575	0.070425	0.929575
24	0.070425	0.929575	0.070425	0.929575
25	0.070425	0.929575	0.070425	0.929575
26	0.070425	0.929575	0.070425	0.929575
27	0.070425	0.929575	0.070425	0.929575
28	0.070425	0.929575	0.070425	0.929575
29	0.070425	0.929575	0.070425	0.929575
30	0.070425	0.929575	0.070425	0.929575

SMALL EARTHQUAKES

PSS	PFF
0.800000	0.250000

K	PKSS	PKSF	PKFS	PKFF
1	0.800000	0.200000	0.750001	0.249999
2	0.790000	0.210000	0.787500	0.212500
3	0.789500	0.210500	0.789375	0.210625
4	0.789475	0.210525	0.789469	0.210531
5	0.789474	0.210526	0.789474	0.210527
6	0.789474	0.210526	0.789474	0.210526
7	0.789474	0.210526	0.789474	0.210526
8	0.789474	0.210526	0.789474	0.210526
9	0.789474	0.210526	0.789474	0.210526
10	0.789474	0.210526	0.789474	0.210526
11	0.789474	0.210526	0.789474	0.210526
12	0.789474	0.210526	0.789474	0.210526
13	0.789474	0.210526	0.789474	0.210526
14	0.789474	0.210526	0.789474	0.210526
15	0.789474	0.210526	0.789474	0.210526
16	0.789474	0.210526	0.789474	0.210526
17	0.789474	0.210526	0.789474	0.210526
18	0.789474	0.210526	0.789474	0.210526
19	0.789474	0.210526	0.789474	0.210526
20	0.789474	0.210526	0.789474	0.210526
21	0.789474	0.210526	0.789474	0.210526
22	0.789474	0.210526	0.789474	0.210526
23	0.789474	0.210526	0.789474	0.210526
24	0.789474	0.210526	0.789474	0.210526
25	0.789474	0.210526	0.789474	0.210526
26	0.789474	0.210526	0.789474	0.210526
27	0.789474	0.210526	0.789474	0.210526
28	0.789474	0.210526	0.789474	0.210526
29	0.789474	0.210526	0.789474	0.210526
[illegible]	0.789474	0.210526	0.789474	0.210526

VERY SMALL EARTHQUAKES

PSS	PFF
0.974360	0.000000

K	PKSS	PKSF	PKFS	PKFF
1	0.974360	0.025640	1.000000	0.000000
2	0.975018	0.024983	0.974360	0.025640
3	0.975001	0.024999	0.975018	0.024983
4	0.975001	0.024999	0.975001	0.024999
5	0.975001	0.024999	0.975001	0.024999
6	0.975001	0.024999	0.975001	0.024999
7	0.975001	0.024999	0.975001	0.024999
8	0.975001	0.024999	0.975001	0.024999
9	0.975001	0.024999	0.975001	0.024999
10	0.975001	0.024999	0.975001	0.024999
11	0.975001	0.024999	0.975001	0.024999
12	0.975001	0.024999	0.975001	0.024999
13	0.975001	0.024999	0.975001	0.024999
14	0.975001	0.024999	0.975001	0.024999
15	0.975001	0.024999	0.975001	0.024999
16	0.975001	0.024999	0.975001	0.024999
17	0.975001	0.024999	0.975001	0.024999
18	0.975001	0.024999	0.975001	0.024999
19	0.975001	0.024999	0.975001	0.024999
20	0.975001	0.024999	0.975001	0.024999
21	0.975001	0.024999	0.975001	0.024999
22	0.975001	0.024999	0.975001	0.024999
23	0.975001	0.024999	0.975001	0.024999
24	0.975001	0.024999	0.975001	0.024999
25	0.975001	0.024999	0.975001	0.024999
26	0.975001	0.024999	0.975001	0.024999
27	0.975001	0.024999	0.975001	0.024999
28	0.975001	0.024999	0.975001	0.024999
29	0.975001	0.024999	0.975001	0.024999
30	0.975001	0.024999	0.975001	0.024999

APPENDIX D

The derivation of the governing equation for the decision model appears in reference [17]. The derivation is repeated here for ready reference.

As before, the quantity $C_i(t)$ is defined as the total expected cost accumulated in the time t, where t is the time remaining in the life of the structure, if the system began in state i. The expected cost $C_i(t)$ is considered as an income or an expense. In the former state, it is given a positive sign and in the latter a negative sign. The entire process is assumed to terminate at the time $t=0$, when the useful or design life of the structure has been exhausted. At such a time the boundary conditions, if any, can be imposed.

Consider a small interval of time δt. At a time $(t+\delta t)$ remaining in the life of a structure the expected cost is $C_i(t+\delta t)$. The quantity can be related to the expected cost $C_i(t)$ at time t.

The transitions among the various states are governed by the transition probability matrix $[A]$. This matrix is a continuous matrix. It has the distinguishing property that

$$a_{ii} = -\sum_{\substack{i \neq j \\ j=1}}^{n} a_{ij} \qquad \text{(D-1)}$$

$$A = \begin{bmatrix} a_{11} & a_{12} & \cdots & a_{1n} \\ a_{21} & a_{22} & \cdots & a_{2n} \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ a_{m1} & a_{m2} & \cdots & a_{mn} \end{bmatrix} . \qquad \text{(D-2)}$$

During the interval of time δt the structure either remained in state i or made a transition to state j. If the structure remained in state i, there are two components of the expected cost to be considered. There is the cost per unit time $c_{ii} \cdot \delta t$ and the expected cost in the time t remaining in the life of the structure which is $C_i(t)$. The associated probability is

$$(1 - \sum_{\substack{i \neq j \\ j=1}}^{n} a_{ij} \cdot \delta t) \quad .$$

If the structure makes a transition to state j then there is the associated cost $c_{ij} \cdot \delta t$ and the expected cost $C_j(t)$ in the remaining time t. The associated probability is $a_{ij} \cdot \delta t$ summed over all states $i \neq j$.

For a decision model to be realistic the dependence on time of a sum of money to be received or given out must be considered. It is assumed here that discounting is done continuously at a rate β. Therefore, a unit sum of money received at the end of a time interval δt will have a value $(1-\beta \cdot \delta t)$ at the beginning of the interval.

With the above development the following equation can be written.

$$C_i(t+\delta t) = (1-\beta \cdot \delta t) \left\{ (1 - \sum_{\substack{j \neq i \\ j=1}}^{n} a_{ij} \cdot \delta t) (c_{ii} \cdot \delta t + C_i(t)) + \sum_{\substack{j \neq i \\ j=1}}^{n} a_{ij} \, \delta t \, (c_{ij} + C_j(t)) \right\} \quad . \tag{D-3}$$

If equation (D-3) is rearranged and passage to the limit is made the following equation is obtained.

$$\frac{dC_i(t)}{dt} + \beta C_i(t) = c_{ii} + \sum_{\substack{j \neq i \\ j=1}}^{n} \cdot a_{ij}\;{}_{ij} + \sum_{j=1}^{n} a_{ij}\, c_j(t) \qquad \text{(D-4)}$$

$$i = 1,2,\ldots n \quad .$$

For a given model it is advantageous to define

$$q_i = c_{ii} + \sum_{j \neq i} a_{ij}\, c_{ij} \quad . \qquad \text{(D-5)}$$

This quantity represents a fixed sum. Consequently, the governing equation for the decision model can be written

$$\frac{dC_i(t)}{dt} + \beta\, C_i(t) = q_i + \sum_{j=1}^{n} a_{ij}\, C_j(t) \qquad \text{(D-6)}$$

$$i = 1,2,\ldots n \quad .$$

For a system with n states there will be n coupled equations. Solution of the n differential equations gives the governing expressions for the structure or system. If the number of equations is small, LaPlace transformation provides a rapid means of solution. However, when n is large the solution of the differential equations becomes time consuming. Therefore, in the case of n large, it is necessary to modify the differential equation (D-6). By putting

$\frac{dC_i(t)}{dt}$ equal to zero a set of n algebraic equations is obtained whose solution is much quicker to obtain. The solution of the algebraic equations provides an asymptotic solution which is accurate only if one is concerned with what will happen away from the time $t=0$; the terminal point.

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