UNITED STATES DEPARTMENT OF THE INTERIOR

GEOLOGICAL SURVEY

PROCEEDINGS OF

CONFERENCE XIII

EVALUATION OF REGIONAL SEISMIC HAZARDS AND RISK

SPONSORED BY THE U.S. GEOLOGICAL SURVEY

25-27 August, 1980

OPEN-FILE REPORT 81-437

MENLO PARK, CALIFORNIA

1981

CONFERENCES TO DATE

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Conference II	Experimental Studies of Rock Friction with Application to Earthquake Prediction Not Open-Filed
Conference III	Fault Mechanics and Its Relation to Earthquake Prediction Open-File No. 78-380
Conference IV	Use of Volunteers in the Earthquake Hazards Reduction Program Open-File No. 78-336
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Conference XIII	Evaluation of Regional Seismic Hazards and Risk Open-File No. 81-437

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Editor and Chairman of the Organizing Committee

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Open-File Report 81-437

Compiled by
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Menlo Park, California

1981

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north in the Illinois-Kentucky fluorspar district and the Wabash Valley fault zone? Why are large northwest-trending faults such as the Ste. Genevieve fault zone inactive? What are the driving forces of past and present seismicity in the New Madrid region?

- What are the constraints on using estimates of seismicity in New Mexico based on a historical record that only extends to the mid-nineteenth century to extrapolate to longer periods of time required in seismic hazard evaluations? How do you reconcile discrepancies between the geologic evidence for major tectonic movements in the geologic past and that provided by current and historic patterns of seismicity in a region?

Speakers in the second session addressed the subject of "models for evaluating the earthquake ground-shaking hazard." Oral presentations on this subject were made by Bob Kennedy, David Perkins, Otto Nuttli, and Susan Dubois. Technical issues identified by these speakers and in subsequent discussions included:

- Can seismologists and engineers resolve the technical problems currently associated with the first group's use of the parameters instrumental peak acceleration and instrumental peak velocity and the second group's need for the parameters design ground acceleration (also called effective peak acceleration) and design ground velocity in regional maps of ground motion? Can the effect of duration of shaking be incorporated in regional maps of ground motion?

- In the preparation of regional ground-shaking hazard maps, what areas meet the requirements that entitle them to be identified as seismic source zones? What criteria should be used for identifying seismic source zones? Can future seismicity be described adequately? How should future earthquakes be modelled? What level of probability is appropriate for constructing probabilistic ground-shaking hazard maps?

- In assessing the earthquake ground-shaking hazard for a region, can one accurately assign a magnitude to the largest earthquake which can be expected to occur on a particular fault or in a particular seismic source zone in a given period of time?

- How adequate is the information derived or inferred from a historic event (e.g., the 1887 Sonora, Mexico earthquake) for predicting the potential earthquake hazards and risk in Arizona and New Mexico?

The third session focused on "Examples of earthquake hazard studies." The presentations made by Ted Algermissen, Bob Bucknam, W. G. Milne, and Albert Rogers discussed the following technical issues:

- Can estimates of earthquake recurrence rates derived from studies of fault scarps be combined usefully with recurrence rates derived from studies of historic seismicity date, in spite of the differences in time represented by the two types of data, to calculate levels of ground motion expected at sites having exposure times of a few tens to a few hundreds of years?

- Can seismotectonic data be incorporated effectively into regional and national probabilistic ground-shaking hazard maps of the United States?

- What are the critical parameters needed for constructing useful seismic zoning maps in Canada? How can modifications to them be introduced effectively?

- Can ground response data acquired in a large geographic region such as Los Angeles be used as a basis for microzoning? What constraints should be placed on using ground-motion data acquired at low levels of dynamic shear strain for representing the ground-shaking hazards at high levels of dynamic shear strain? Can the local ground response of a region be satisfactorily predicted either empirically or deterministically?

The fourth session developed the theme of "seismic risk," emphasizing the distinction between hazards and risk and describing current research. Oral presentations were made by Ted Algermisen, Karl Steinbrugge, Roger Scholl, and John Wiggins. They identified a number of technical issues, including:

- Are economic and life loss from earthquakes in the United States likely to show substantial increases in the future? On the basis of empirical data from earthquakes, over how wide a range do average annual losses to dwellings vary in California?

- What are the essential elements of a general yet rigorous capability for predicting damage?

- How sensitive are loss comparisons to various damage algorithms? What level of uncertainty is associated with various ground motion parameters (e.g., maximum magnitude of certain seismic source zones, the seismic attenuation function for various geographic regions) and exposure model parameters (e.g., qualities and classes of structures that exist in each county throughout the nation, not only now, but through the year 2000) and how sensitive are loss comparisons to them?

The fifth session, which concluded the conference, developed the theme, "applications of seismic hazards and risk studies." Oral presentations were made by Libby Lafferty, Roland Sharpe, and Del Ward. The issues they identified included the following:

- Who will take the responsibility and pay the price for educating people to understand and deal with seismic hazards? What "life-saving" and "life sustaining" information is needed by people? What is the best way to reach the general population and to motivate them to prepare before a major seismic event occurs?

- Is the present procedure for incorporating research on earthquake hazards into building codes (e.g., the Uniform Building Code) adequate?

- Are currently available data describing Utah's earthquake potential adequate for utilization at the local level; that is, for making assessments of seismic risk for particular types of community development and populations? Can the available methodologies be utilized to develop a comprehensive methodology for assessing seismic risk in Utah? Can assessments of risk for particular types of buildings and populations in Utah be formulated so that explicit policy recommendations can be made? Are the evaluations of risk sufficiently reliable so as to justify the establishment of public policy for mitigation in Utah?

The participants in the conference concluded that a great deal of useful research has been performed in the national Earthquake Hazards Reduction Program by USGS and non-USGS scientists and engineers and that the state-of-knowledge concerning the evaluation of seismic hazards and risk has been advanced substantially. Many of the technical issues raised during the conference are less controversial now because of new information and insights gained during the first three years of the expanded research program conducted under the Earthquake Hazards Reduction Act. Utilization of research results by many groups of users has also improved during this period and further improvement in utilization appears likely.

Additional research is still required to resolve more completely the many complex technical issues summarized above and described in the papers contained in the proceedings. Improved certainty of research results on the evaluation of regional seismic hazards and risk is required before full utilization can be made by state and local governments who deal with people frequently having a different perception of the hazard and its risk to them than that perceived by scientists or engineers.

Each of the papers contained in the proceedings contain throughtful recommendations for improving the state-of-knowledge. Two papers, in particular, focussed on this particular theme. The first was presented by Lynn Sykes in the Geologic Keynote Address. He identified geographic areas throughout the world which may be considered as counterparts or analogues of seismic zones in the United States. He concluded that much can be learned about prediction, tectonic settings, earthquake hazards, and earthquake risk for sites in the United States by studying their tectonic analogues in other countries. The second paper was presented by John Blume in the Engineering Keynote Address. He suggested 20 specific research topics that, in his opinion, will significantly advance the state-of-the-art in earthquake-resistant design. The papers by Sykes and Blume are presented in the front of the proceedings.

ACKNOWLEDGEMENTS

A special note of appreciation is extended to each of the participants who gave oral presentations at the conference. Their presentations provided the stimuli for a vigorous and healthy exchange of views.

Roger Scholl and John Schlue served with me on the Conference Organizing Committee and contributed greatly in many ways. John Schlue served as the local host in Santa Fe and made everything work efficiently.

Mary Weddle, U.S. Geological Survey, provided strong support in organizing and implementing the Conference. Her help is greatly appreciated.

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WORLD-WIDE FEATURES WITH TECTONIC SETTINGS SIMILAR TO THOSE OF MAJOR EARTHQUAKE ZONES OF THE UNITED STATES

by

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ABSTRACT

Most of the zones of seismic activity in the United States have counterparts and analogues in other areas of the world. It is the thesis of this paper that much can be learned about the prediction, tectonic setting, earthquake hazards and tsunami risk for sites in the United States by studying tectonic analogues in other countries. Certain aspects of zones in the U.S. that are not clearly developed (i.e., pronounced overburdern masking basement feature, lack of a long historic record, etc.) become impediments or road-blocks to understanding, particularly if efforts are concentrated solely on specific features in the U.S. Critical keys to the understanding of major strike-slip faults like the San Andreas may come from work on similar features in say Turkey, Guatemala, New Zealand, Venezuela, the Philippines, Japan, Alaska, Iran, Pakistan, western China, and the U.S.S.R. A comparison of some of these fault systems indicates that parts of them are in various stages of the earthquake cycle. By studying critical earthquake-generating features in other countries, it may be possible to "catch" fore-running or precursory features of large shocks and to apply that experience prior to the occurrence of the next large earthquake on say the San Andreas fault.

Intra-plate earthquakes and the state of stress in Australia, Canada, northern Europe, the U.K., parts of Africa, and peninsular India show many similarities to those associated with the central and eastern U.S. Many shocks in those areas seem to occur along old fault systems that have moved many times throughout geologic history in response to various plate-tectonics events. The configuration of major tectonic elements and the reactivation of fault systems in the southeastern U.S., the site of the Charleston, earthquake of 1886, are similar to those of west Africa near Accra, Chana, and the Benue Trough of Nigeria. The tectonic setting of the New Madrid zone in the central U.S. is similar to that of the seismic zone that extends into the interior of Australia near Adelaide. Several zones of intra-plate shocks are similar to those of the eastern and central U.S. in that they are characterized by very large felt areas for a given level of energy release.

Probably a great deal can be learned from studies in other countries about the repeat times of large earthquakes along given segments of strike-slip and convergent plate boundaries. Sykes and Quittmeyer have completed a study of observed repeat times of large shocks along simple plate boundaries of the strike-slip and subduction type. The average repeat time is a function of the long-term rate of plate movement and the geometry of the rupture zone. Variations in repeat time at a given place appear to be associated with the length of the rupture zone and the amount of seismic slip associated with the last large shock along that zone.

Since the historic record of earthquakes along the Alaska-Aleutian arc is so short, information from convergent zones near Japan, New Zealand, India, Pakistan, the U.S.S.R., the Lesser Antilles, Mexico, Central and South America and other similar areas can be applied to earthquake-related problems for Alaska and the Aleutians. Similarly, the unusual style of plate motion to the north of Puerto Rico and the Virgin Islands--thrust faulting on nearly horizontal planes with the slip vector nearly parallel to the plate boundary--is similar to that in the westernmost Aleutians and in the Andaman Islands.

WHAT IS NEEDED TO SIGNIFICANTLY ADVANCE THE STATE OF THE ART IN EARTHQUAKE-RESISTANT DESIGN

by

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INTRODUCTION

What things are necessary to produce a significantly higher level of development in the field of earthquake-resistant design? One reply to this provocative question might be: more earthquakes and the time and funds to study their effects and to transform what we learn into design practice. However, this would not only constitute a simplistic, rather trivial reply; it would be less than correct. Much more than data points is needed, and much information that is available now is used inconsistently or not at all.

Investigators are likely to learn something new from almost any damaging earthquake in a populated region, and it is not uncommon for individuals to be so forcibly impressed by something that occurs during an earthquake that they become convinced of a fact that they should have accepted long before. However, most of what happens -- and what does not happen, which is just as important -- should by now have come to be expected by qualified experts in the field. Anomalies are merely happenings that are not yet understood. For example, there are logical reasons for all that happened and did not happen in the 1971 San Fernando earthquake and in the Imperial Valley event in 1979. The problem for experts is to find a means for quickly translating the design implications of what is continually being learned into design practice that provides risks that are acceptable to a well-informed public.

BUILDING CODES

Let us first consider some facts about the building codes for earthquake-resistant design:

(1) Codes are necessary. Without them, most structures would be designed with little or no regard for earthquake safety.

(2) The codes are not intended, nor should they in most cases attempt, to prevent <u>all</u> possible damage from earthquake motion; that would be poor economics. In spite of this, building owners and the public -- and all too many architects and engineers -- consider "meeting the code" to be the limit of their responsibility. Most laymen expect that <u>no</u> damage will occur if the code is met.

(3) It is possible to meet all code requirements and still produce a poor building with a poor risk. The layout and the geometry of the structure and its elements are very important factors in this regard.

(4) Even the best and most up-to-date codes lag behind knowledge, perhaps by 10 years or more. An example is found in the number of years it took for the concept of ductile concrete to be incorporated into (some) codes, a period during which such California structures as the Olive View Hospital, the Imperial County Services Building, and countless other such structures were being designed and built. The ATC code was drafted many years ago, but apparently many more years will have to pass before it attains general use. Moreover, not all experts are in agreement about all of its concepts.

(5) Codes, almost by definition, are compromises finally agreed to by large groups of people not all of whom are expert in all aspects of the complex problem. Whether or not the compromises are actually adopted depends upong public representatives, who are generally laymen in technical matters.

(6) Since seismic codes first began in the United States, there has been a divergence in opinions about them, represented by those who believe codes should be short and to the point, with the basic guidelines for expected performance and the details of execution left to competent professionals, and those who believe that codes should be specific, detailed like cookbooks, and voluminous. The first group makes references to the medical profession, whose members are taught basic medicine but are given wide latitude in its practice to make case-by-case professional judgments. (They enjoyed an even wider latitude before legal suits for medical malpractice became fashionable.) That group also points out that an entire library of code material could not prescribe a procedure for every possible circumstance. Even if it could, who would read it? Who would keep it up to date?

(7) The code problem is further exacerbated by the fact that essentially every designer or engineer who reads or uses a seismic code considers himself -- and represents himself -- to be an earthquake engineer or an earthquake expert, whereas in fact he may have little or no knowledge of or sensitivity to the complex nature of the problem. The public accepts this; the law permits it. Lacking the knowledge to challenge the self-styled earthquake professional, the public is understandably confused.

- What is needed regarding codes? Building codes must be continued, but influence should be exerted to make them more realistic (which should not be construed to necessarily entail an increase in design coefficients), more up to date, shorter, and more performance oriented. They should allow more professional latitude but at the same time provide for better control over the means for determining whether a designer is qualified to make professional judgments. They should emphasize the need to follow <u>all</u> of the applicable laws of physics and mechanics, regardless of code specifics. For buildings important to the general welfare, there should be two earthquake designs: one for the initial, elastic state and another for the inelastic, survival state, anticipating the rare, but possible, great earthquake. A preliminary paper that discusses such a double-design dynamic code (Blume, 1973) has been used privately by some individual engineers as a supplement to code designs but has apparently inspired little enthusiasm from contemporary code writers.

THE REACTION SYNDROME

In 1933, most of the public school buildings in Long Beach, California, were demolished by an earthquake within an hour or two after the school children went home. As a result, the California Field Act was passed to control the design and construction of public school buildings through the high school level. The legislation has been highly beneficial. In 1971, the San Fernando earthquake demolished hospitals and elevated freeway structures. As a result, the Veterans' Administration checked its hospitals all over the country, and California adopted state requirements for hospitals. Elevated freeway structures were reviewed; as a consequence, details were in many cases altered, and new standards were adopted.

No reasonable person could object to these improvements in public safety. At the same time, it must be recognized that they cam about through a combination of factors that are less than reliable: the reaction of the public, treatment by the news media, politics, etc. The public is pragmatic, believing in what it can see, not in what it has been warned to expect. Had they been given the funds and time, authentic earthquake experts, even those of 10 or 20 years ago, could have predicted the failures that would occur from shaking at the level experienced during the events described above. Such predictions would not have been limited to schools, hospitals, and elevated freeways. They would have included dams, which are now receiving a long-overdue review, and nuclear power plants (which are currently being designed to base shears 10 to 15 times those of high-rise buildings and are permitted little if any inelastic response), and various building types.

What will be our next legislative reaction? Perhaps it will be concerned with high, slender, multistory box-type buildings with thin slabs and no columns, beams, or girders but with a great deal of glass. Or perhaps it will deal with what one might term "vagrant architecture" -- buildings with no visible means of support at the first story.

- What is needed is a more rational philosophy than that expressed by the attitude "wait until it falls down and then pass legislation." We need not wait for the alarm bell to ring. Advanced, experienced, really qualified experts can provide needed warnings now, before an earthquake. Even though this might not improve high-risk structures that have already been built, it would curtail or prevent their proliferation under the umbrella of meeting a code.

THE DOUBLE STANDARD

We have a double standard that says so-called high-risk facilities such as nuclear power plants must be designed to a set of standards completely different from those applied to other structures, such as multistory office buildings. Nuclear plants in California, for example, are generally designed for many times the lateral forces applied to schools, hospitals, and even dams. Moreover, the procedures employed for nuclear structures are much more advanced than those found in any building code. Instead of working from maps and codes, specialists make very detailed investigations and analyses concerning potential sources of earthquakes, the geophysical aspects of sites, soil-structure interaction, structure design, and all of the equipment and piping. Risk evaluations are made. Redundancy, as well as great strength, is provided. Unfortunately, the compounding of safety factors one upon the other is substituted for appropriate probabilistic analyses that make use of all the parameters with their means and real dispersion characteristics. Hazard and risks cannot be properly evaluated by "enveloping" or taking the worst possible case for each parameter. The end product may be a design for the one chance in millions or billions that something will go wrong. If the structure is designed to provide protection against a so-called hazard such as, for example, a crack in a wall that is itself a redundant item, money is indeed being wasted.

Another example of the excessively conservative approach is that of a nuclear power plant for which three separate risk analyses were recently conducted (Blume, 1977a) using seismic data provided by earth scientists. One procedure involved only the known earthquake history of the region; another treated the State of California as a tectonic plate boundary; the third considered interpretations of geologic data concerning fault slips and seismic history going back as far as 20,000,000 years. The results from the three procedures (actually as many as six or seven, allowing for parametric studies) were surprisingly consistent (Blume and Kiremidjian, 1979). Investigation of the seismic criteria that had been provided for analysis of the plant revealed the use of an average return period of about 100,000 years just to reach yield level, at which there would be no damage but beyond which the great reserves of the inelastic range would come into play.

Compare the foregoing example to design practice for buildings and other structures in every major city in the United States. Clearly, high-risk facilities, which are situated in areas of low population density, are designed to standards that are higher in the extreme than those that are applied to structures such as office buildings, apartments, and hotels, which are almost always densely occupied. The difference is much greater than is indicated by the nature of nuclear materials to be necessary. Are the high-risk structures overdesigned, or are the rest of the buildings underdesigned? Very likely it is a little of each.

- What is needed eventually is application of the highest level of expertise not only to high-risk facilities but to entire metropolitan areas. This program would have to be carried out on a regional basis to make it economically feasible. With the participation of the public, the allowable risk for each unit -- the nuclear power plant, the liquefied natural gas facility, the hospital, the office building, the condominium, the residence -- would be established to provide the desired balance of risk, consequences, and cost.

MAGNITUDE AND SOURCE PARAMETERS

Even Dr. Richter would be likely to agree that we are too magnitude conscious. There are now many ways to describe magnitude, and often there are major differences of scientific opinion on what the magnitude of a particular event actually was. The most recent technique is to vary magnitude with directionality, as was done for the Santa Barbara – Goleta event (Miller and Felszeghy, 1978). If one studies the many structures that have been exposed over the years to nearby large-magnitude earthquakes, it becomes fairly clear that the amplitude of ground shaking during very large magnitude events does not increase much, if at all, over that during events of, say, M 6-1/2 or M 7. The shaking will last longer and of course will extend over much greater areas during the great events, but there seems to be some sort of limit, or saturation level, for local amplitude. The work of Hanks and Johnson (1976) on this subject is of particular interest. Perhaps an M 8.3 earthquake is actually a series of M 6-1/2 to M 7 events that are very closely spaced in the time domain.

- We need a better understanding of all of the source and propagation parameters of earthquakes. We need to know the physics of the problems so that better designs, as well as better risk analyses, can be made. Seismic moment, stress drop, fault rupture dimensions, fault slip rise time, all seem to be important. An objective of current investigation is to develop a procedure for obtaining structural response directly from one or two source parameters. To realize that goal in the near future, we need to know more about the probability distributions of various source parameters as well as their interrelationships.

FOCUSING

The natural phenomenon of focusing is receiving a great deal of current attention.

- We need clarification of the false impression that tremendous beams of seismic energy can spread out in all directions, to reconcile it with facts which seem to indicate that the range of focusing is limited and that there is a small sector angle relative to a fault. The fault geometry is also an important factor.

TOPOGRAPHY AND SEGMENTS

Mounds of rock have natural periods of vibration, as do man-made structures. Certain kinds of strong motion records no doubt show dynamic amplification and are therefore misleading insofar as the basic free-field motion is concerned. The Pacoima Dam ridge record is an example. It is not unreasonable to assume that underground topography or fractured segments of bedrock between faults may account for some of the rather strange records that have been obtained, at Stone Canyon and at Imperial Valley, for example.

- We need more rock records, and we need more study of topographic rock features, those below the ground surface as well as those above it. We also need more study of the "feather-edge" conditions of soil over rock as they relate to ground motion.

PEAK ACCELERATION

All too often, peak acceleration is misguidedly used as a direct measure of earthquake damage potential. It is convenient to discuss an earthquake that has just occurred in terms of its location, magnitude, and peak acceleration. But, as time goes on, magnitude and acceleration may no longer be so relevant. There is a vast gap between peak instrumental acceleration and the base shear coefficients used to design buildings. Although the two are not the same, even theoretically (except for a few simple, rigid types of structures), they are often confused. They can be reconciled only if all the many factors related to the earthquake are considered (Blume, 1979). Response spectra made from the time histories of recorded acceleration are much more meaningful than peak acceleration values insofar as structural response is concerned. But one could reduce the recorded accelerations by as much as 30% on an absolute basis over the entire record and produce only about a 5% decrease in the peak spectral values (Blume, 1979). From these observations, it follows that the spikes observed on records have little or no structural design significance.

- We need to continue to record acceleration and to treat it as a valuable tool but to recognize that it is not a reliable index of damage potential. Newton's second law is still valid, but the earthquake problem is much more complex than that law alone would imply.

- The record of acceleration is useful because it has frequent zero crossings and provides a good representation of the ground motion. Velocity and displacement are also important, especially for long-period structures, but for basic recording they may tend to lack fidelity and to filter what might be significant high-frequency motion. The acceleration record can be corrected; it can be integrated, and Fourier and response spectra can be made from it.

- Measurements should be made under realistic conditions applicable to design. For example, it is not at all clear that vertical motion recorded on a pad of concrete or on a floor slab is the vertical motion that "drives" structures.

ATTENUATION

Gutenberg and Richter were the first to formulate an attenuation law; today we find dozens of attenuation laws, based upon analyses of various combinations of event records. Their proliferation has created no small amount of confusion and, in some cases, misuse.* Recent studies (Blume, 1977b) propose site-acceleration-magnitude relationships -- the SAM IV and SAM V equations -- that include allowances for site impedance, magnitude, saturation effects, and probabilistic variations. The relationships can be refined further as more data become available. The problem with all of these attenuation laws is in defining the data population. What types of earthquakes are of concern? Should one combine strike slip and thrust faults, for example? How much data from one widely recorded event can be used without biasing the population? What should the lower-limit cutoff be -- 0.11g, 0.10g, or the instrument's triggering level? The regression mean and the standard deviations vary with the lower cutoff value although in many cases the mean may go up while the standard deviation goes down (and vice versa) so that, if one is concerned about a value at some confidence level above the mean, the difference between the results of the high-cutoff and low-cutoff procedures may not be great. The most realistic analysis will make use of <u>all</u> recorded values and <u>both</u> of the horizontal components of many earthquakes similar to the one being investigated.

- We need to devote more attention to the data populations we use in attenuation studies; to avoid biased populations; and to establish standards for data selection. The procedure of using epicentral (or hypocentral) distances in the development of the attenuation curve and then associating the curve with normal fault distances in design analysis should be improved as more data become available. Both horizontal components should be considered, not just the higher of the two.

"THE ONLY TEST"

There are those, including some engineers, who appear to believe that the only valid test of the earthquake-resistive capacity of a particular type of structure is an actual earthquake. Even if this were true, we could not afford to wait until every type of structure is shaken by an earthquake. Consider these facts: earthquakes may occur infrequently in a particular area; what is more important, earthquake motions are random variables with wide ranges of possible amplitudes, periodicity, duration, angles of incidence and azimuth, and other characteristics. How could one such event be the real test of a building? The next earthquake to shake it could be entirely different.

- We need to place more reliance in competent advanced analyses and testing, both static and dynamic, not

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*The writer's early attenuation law was apparently the second to appear, but it was so often misquoted or incorrectly applied that he has advised its retirement in favor of his more recent formulations (Blume, 1977b).

omitting correlation with new earthquake experience that is acquired. At the same time, analyses must be realistic, avoiding convenient but possibly unrealistic assumptions about key parameters.

THE STRUCTURES

- We need to give much more attention to the actual values of the parameters of structures, as opposed to their code values, if we are to advance into more realistic, dynamic design procedures of the kind needed for nuclear power plants. We should consider in more detail those structures that have not been damaged in an earthquake, or that have been damaged very little, instead of concentrating solely on dramatic examples. We learn from building failures, but we should also learn from buildings that have not failed. What was the reason for their survival?

- We need to produce designs with simple, direct stress paths that either are very strong or are ductile and energy absorbing and that preferably are redundant as well. We need to avoid the use of complex layouts, geometry that has column or shear wall offsets, setbacks, assymmetry in plan, or abrupt transitions of structural elements in size or location. Above all, whether or not these complexities can be avoided, all of the laws of mechanics and dynamics must be observed. Most codes call for a rational system for delivering each pound of inertial force to the ground. This requirement, instead of appearing as the fine print of codes, should be printed in boldface type.

- We need to emphasize that the lateral forces shown in building codes are fictitious for major earthquakes and may be much greater if the structures remain essentially elastic. Seismic design must therefore provide great strength (which few owners would pay for) or else must provide for ductile response in the inelastic range.

SOIL-STRUCTURE INTERACTION

The interaction of structures with the soils upon which they are founded is a favorite subject of investigation for engineers and geophysicists today, although it is not a new one. The writer worked on the problem some decades ago -- mathematically, in field work analysis, and on the shaking table -- and is investigating particular aspects of it in current work. There can be little doubt that free-field motion is affected by a major structure and that the motion of the structure is affected by the soils below it. The soil and the structure constitute a dynamic system, often of a nonlinear type. Moreover, waves of different types (P, SH, SV, and surface) arrive from different angles of incidence and of azimuth -- they are not all vertical, or all horizontal, or all 45° but rather are usually a very complex assortment, especially those that come from particular sources and travel along particular propagation paths. Scattering is a feature that has been largely neglected, together with feedback and the question of the importance of each of the six possible degrees of freedom. The influence of structure-soil-structure interaction is also of importance.

- We need to learn more about the subject of soil-structure interaction and to put it into proper perspective. Much work is currently being done in this area, but much more is needed.

ENERGY AND WORK DONE

Using the concept of the real energy capacity of buildings as the basis for a design procedure to supplement the requirements for meeting applicable codes is not a new idea: it has been proposed by the writer on many occasions over the past two and one-half decades. The concept requires that the great bursts of seismic wave energy (as determined from spectral velocities at the periods of concern) be resisted by a combination of strain energy and the work done by the available elements of a building. Allowances are then made for any deterioration of damaged elements, with consequent period lengthening, and the design may be revised to reduce the damage level or to prevent collapse under continued motion. The reserve energy technique (RET), which has been published in various places (Blume, 1958, 1960, 1961), is such a procedure. The complete time history need not come into play so long as the structure is not stressed beyond its yield level; the low-level energy is converted to heat in hysteretic cycling. The earthquake problem is not one of fatigue failure because earthquakes do not cause enough cycling at high stress levels to produce fatigue failures. Therefore, the response spectrum is generally an adequate basis for analyses.

A simple dynamic code that employs the RET has also been suggested (Blume, 1973). It requires a building to be checked twice: once in its virgin state, and once again when it is likely to be approaching collapse. Although the RET has been used often, both in concept and as a procedure, in various forms and often under various names, it has received neither general recognition nor rebuttal. It is a simple technique that may be performed by hand, without computer aid; it is also easily programmed. Perhaps the procedure has at times been discounted for that very reason: because it is so simple, lacking in impressive rigor. However, setting aside the question of its other merits, use of the RET can be extremely valuable in providing the designer with a sensitivity to the structure that he would not otherwise have.

- We need to strive for a more general recognition, translated into practice, that the real earthquake problem is that of taking excessive kinetic energy and converting it to other forms of energy and other ways of doing work, all in a manner that will let buildings survive with an acceptable level of distress. In other words, we must design for energy as well as for force. The technique of using energy reserves and the response spectrum technique are no doubt the two most basic tools available for attacking the earthquake design problem. Dividing a response spectrum by the ductility factor for inelastic response is not adequate for real buildings, nor does it give accuracy in all cases; moreover, it does not give the designer the control he should have.

DISCIPLINES

When the earth shakes, it has no interest in whether one is a seismologist, a geologist, a structural engineer, a civil engineer, an architect, a sociologist, a building official, a professor, or a layperson. Although depth in individual fields of study is essential for progress, collaboration among members of all disciplines, and proper integration of the results of their investigations, is equally important for finding answers to the earthquake problem.

- In the study of earthquakes, we need generalists as well as specialists, and the complete cooperation of all disciplines for the public welfare is essential. The latter is one of the major objectives of the Earthquake Engineering Research Institute, an organization that welcomes representatives of all fields of study, indeed any person concerned about the earthquake problem. We need to lend our full support to such organizations, and we need to inform ourselves about their activities and keep abreast of what is being learned.

TECHNOLOGY TRANSFER

The flood of technical literature and technical reports has reached almost overwhelming proportions. Peer review and publication were once considered essential if the results of investigation were expected to gain academic acceptance, if not general acceptance. This may still be the case in theory, but in practice there are several problems: it is the rare individual who has time to read all that is printed in his field; much of the subject matter is becoming so complex that many practitioners, especially those working in design, do not have the time, the inclination, or perhaps the specialized background to study the material in proper depth; there is considerable repetition or reinventing of the wheel (some of which is useful), the original or prior work being either unknown to the current worker or, if known, presented without reference. For these reasons, publication, per se, does not carry the significance that it did in prior decades. The rule "publish or perish" should be reevaluated, placing greater emphasis on technical quality and innovation.

- Better procedures are needed for translating research results and other findings into design practice. Part of that need can be met by keeping the codes current, but a designer should have many other useful technical documents at his disposal as well.

PUBLIC COMMUNICATION

In recent years, the general public has finally become aware of earthquakes. But awareness is not enough; those of us who are knowledgeable in the field must take the responsibility of helping to keep the public properly informed. There is a delicate balance between scaring people needlessly, or unduly, and advising them of risks and potential hazards. The media have a large responsibility in this matter as well. This is illustrated in a personal example: Shortly after the recent Mammoth Lake earthquakes, some friends telephoned from Europe because they had heard that part of California had dropped into the Pacific Ocean. Something had obviously become exaggerated somewhere along the line in the news reporting. If our efforts are to be useful, they must be accepted -- and to some degree understood -- by the public.

We face a statistical problem also in that the public, observing the nominal loss of life in this country thus far from earthquake motion, is generally unimpressed with the hazard. It must be made clear that the earthquakes in San Francisco in 1906, at Santa Barbara in 1925, at Long Beach in 1933, and at San Fernando in 1971 occurred at very fortunate times of the day and that if they had occurred at less fortunate times the statistics would have been very different. Moreover, these areas are now more heavily populated than before. Finally it must be pointed out that the new structures that have been built in response to the general increase in population have in many cases been constructed on more risky sites. Probabilistic risk studies, properly explained and compared with other risks, are very useful in communicating these facts to the public.

- We need to inform the public properly and professionally about the earthquake problem. The media need to know where to obtain reliable information quickly. Much effort has been put into establishing sources of such information, but some amazing misconceptions are still perpetuated in reports of earthquakes. The public should be advised not only about safety precautions but about earthquake problems in general. It is the public that adopts building codes and pays the taxes used for most of the research. Above all, people who may be the occupants of buildings during earthquakes must be made aware that buildings are not designed to remain entirely free of damage. The codes say that, in fine print, but does the public realize it?

CONCLUSION

Although this paper discusses many things the writer believes are needed to significantly advance the state of the art in earthquake-resistant design, there are, clearly, other needs that might be mentioned. Moreover, it is not to be expected, especially in the often controversial field of earthquake engineering, that every reader will be in complete agreement with the assessment of the earthquake problem and the steps toward solving it that are presented here. However, there should be no difficulty in achieving consensus on these points: man's knowledge of earthquakes and of earthquake engineering has increased greatly during recent decades, and good design is now possible;

one of the things needed most is a means to ensure that poor designs for earthquake protection cannot exist under the umbrella of building codes.

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EARTHQUAKE MECHANICS AND NEAR-FIELD GROUND MOTIONS

by

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INTRODUCTION

The characteristics of site-specific ground motion must be well understood to provide suitable criteria for evaluating the earthquake resilience of structures. Ground motion criteria are derived from predictions of what would happen if hypothesized earthquakes were to occur in the future. Such predictions are necessarily plagued by uncertainty. Recordings of past earthquakes indicate that ground motion is a complex function of the earthquake source and the intervening earth properties. For critical structures, such as nuclear power plants, unduly conservative criteria are often established to accommodate these uncertainties.

During the recent five years, computer models have been developed, tested and refined for simulating earthquake processes over the frequency band of principal interest -- 0. to 20. Hz. This bold undertaking has proven useful for explaining features of past earthquake recordings and for extrapolating these data to different site conditions. In the process new information is being revealed about the nature of earthquake rupture, the influence of earth structure on recorded motions, and the characteristics of siesmic waves that impinge on structures. This paper briefly describes the modeling procedures, the particular earthquakes that have been modeled, and some of the more important features of earthquake rupture that have been revealed in these studies. Then more generic aspects of earthquake ground motions are discussed.

MODELING PROCEDURE

The principal objective of the modeling procedure is to predict site-specific ground motion by simulating those physical processes that most significantly influence the predicted motions. Earthquake rupture and wave propagation involve irregularities that are beyond our ability to model deterministically at this time. Random processes have been introduced in the modeling procedure to approximate such irregularities. While this procedure falls short of simulating certain details of particular recordings (e.g. wiggle-for-wiggle reproductions), the procedure does provide a means for estimating a range of plausible consequences, particularly a range of spectral ordinances.

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1. Consultant for Del Mar Technical Associates

Technical details and evolutions of the modeling procedure are discussed in the references listed at the end of this presentation. Basically, the modeling is performed in three steps:

(1) Green's functions are calculated for the particular earth structure. That is, surface motions are computed for several hundred point sources distributed over a closely spaced grid of epicenteral distances and focal depths. These earth response calculations include all wave types present in the vertically stratified, viscoelastic representation of the earth over the frequency range 0. to 20. Hz.

(2) Fault slip is characterized in terms of fault type (strike-slip, dip-slip, etc.), rupture velocity, dynamic stress drop (slip velocity at the onset of rupture at each point on the fault), static stress drop (fault offset), and duration of slip at each point. Additionally, random processes are included to approximate perturbations or irregularities in the actual earthquake rupture. A single earthquake simulation is repeated several times to determine the range of effects introduced by the random processes.

(3) Ground motions for a distributed rupture are synthesized by convolving in time and space the characterization of fault slip (from step 2) with the earth response functions (from step 1). The spatial information is assigned at this stage of the calculations, namely the hypocentral location, rupture extent, and site distance and orientation with respect to the rupture.

Several of the parameters are assigned values generically. The gross rupture velocity, for example, is set to 90% of the local shear-wave velocity for each layer undergoing rupture. The duration of slip at each point is taken as the travel time of shear waves across the narrowest dimension of fault rupture. Furthermore, the P- and S-wave quality factors, Q_α and Q_β, are empirically related to seismic velocities:

$$Q_\beta = 30\beta^{1.25}$$

and

$$Q_\alpha = 3/4(\alpha/\beta)^2 Q_\beta$$

where α and β are the P- and S-wave velocities, respectively, in units of km/sec (see DELTA report of 1978).

Other parameters are assigned values based on the particular earthquake being modeled. These include rupture dimensions, hypocentral location, fault type (slip orientation), and fault offset. The slip velocity at the onset of rupture is determined directly from near-field recordings of ground motion. Because the initial slip velocity strongly influences computed ground motions for frequencies above about 2 Hz, it is important to anticipate this parameter.

MODELING PAST EARTHQUAKES

Strong motion recordings from six past earthquakes have been modeled: 1933 Long Beach ($M_S \sim 6.3$), 1940 Imperial Valley ($M_S \sim 7.0$), 1966 Parkfield ($M_S \sim 6.4$), 1971 San Fernando ($M_S \sim 6.5$), 1976 Brawley ($M_S \sim 4.9$), and 1979 Imperial Valley ($M_S \sim 6.9$). Certain parameters were adjusted to cause reasonable agreement with the recorded data. In particular, the fault dimensions, offset, hypocenter, and earth properties were set based principally on results of other researchers. Then the single parameter, initial slip velocity, was adjusted to produce comparable, or slightly more, high frequency ground motions than was observed. A value of 800 cm/sec was found to be suitable for all the earthquakes that were modeled, apparently independent of magnitude and static stress drop. Since the initial slip velocity is related to material strength, the uninversality of initial slip velocity suggests a high degree of uniformity in material strength from one earthquake to the next. It should be noted that the initial slip velocity of 800 cm/sec, which corresponds to a dynamic stress drop on the order of one kilobar, could change if other parameters in the model were changed.

Coherent rupture overpredicts the effects of focusing for frequencies above about 3 Hz. For example, using strictly coherent rupture the model produces more than five times higher peak acceleration at Parkfield Station 2 (directly in the line of rupture) than at Parkfield Station 5 (positioned about 5 km off the line of rupture). Analytical solutions yield comparably large ratios of peak acceleration. In contrast, the peak accelerations recorded for these two stations differ by only about 10%. The effects of focusing are more apparent for lower frequency motions. Notably, the peak velocities recorded for these two stations differ by a factor of three as predicted by the model.

Apparently, rupture irregularities subdue the effects of focusing for frequencies above about 3 Hz. Random perturbations have been applied to the strike, dip, rake, spreading direction, and onset time for fault rupture over 1-km cells. Excessive high frequency focusing is still produced by coherent rupture within the 1-km cells. In recent studies to simulate recorded data for the 1979 Imperial Valley earthquake, additional rupture irregularities were introduced on a scale of 50 meters by randomly perturbing the time for rupture initiation in each 50-meter subcell. This procedure effectively subdues focusing at high frequencies and produces synthetic ground motions that closely resemble recorded motions.

MECHANISTIC EFFECTS ON GROUND MOTIONS

The modeling procedure simulates earthquake processes in sufficient detail as to provide useful interpretations of near-field recordings of past earthquakes. Furthermore, conditions can be changed in the model to assess how such changes might influence the recorded motions. The combination of model studies and recorded data provides the basis for inferring relationships between ground motions and earthquake source and intervening properties of the earth. We note the following:

1) The production of high frequency energy per unit area of rupture appears to be remarkably independent of earthquake magnitude, fault mechanism and static stress drop.

2) The effects of rupture focusing are highly subdued for frequencies greater than about 3 Hz due to local irregularities in the rupture.

3) The upper limit for peak acceleration within a few kilometers of the rupture is probably insensitive of earthquake magnitude for magnitudes greater than about 5. The expected peak acceleration within a few kilometers of the rupture becomes insensitive of earthquake magnitude for for magnitudes greater than about 6.5.

4) The preponderance of seismic energy for large earthquakes originates along the rupture surface at depths generally greater than 3 km. Near-field motions are remarkably insensitive to the presence or absence of rupture within 0.5 km of the earth's surface.

5) Earthquake motions are significantly influenced by earth properties ranging from within a few meters of the surface to depths greater than 10 km. Lateral changes in material properties can also influence surface motions. The trend is for wave amplification in regions of relatively low shear-wave velocity. However, the amplification of high-frequency shear waves is generally compensated by relatively high material attenuation so that peak accelerations on siols are about comparable to peak acceleration in rock within 20 km of the source.

CONCLUSIONS

Despite the many complexities associated with earthquakes, progress is being made toward understanding and quantifing the processes most directly related to earthquake hazards. The current state of the art is such that, with considerable effort, detailed modeling studies can be performed to estimate site specific ground motions from postulated future earthquakes. It would be most difficult at this time, however, to provide generic guidelines for establishing ground motion criteria.

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LATE-QUATERNARY FAULTING AS A GUIDE TO REGIONAL VARIATIONS IN LONG-TERM RATES OF SEISMIC ACTIVITY

by

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ABSTRACT

A major element in the current goal of revising the national probabilistic ground motion map produced in 1976 by Algermissen and Perkins is improved definition of the seismic source zones used in the calculations for the map. This paper describes some of the information used in developing a seismotectonic framework for the delineation of source zones in the Great Basin region of the western United States and their extension to defining zones in much of the western United States.

We have used regional mapping of late-Quaternary surface faulting to characterize large seismic source regions that are distinctive on the basis of predominant ages of most recent movements on the faults within the region and the frequency of movement in late-Quaternary time. Development of a general geomorphic dating method to provide approximate ages for fault scarps has provided a means of assigning ages to most fault scarps rather than only at isolated sites where radiometric ages may be available.

We have focused our detailed mapping on western Utah, systematically scanning 1:60,000-scale aerial photos for all possible fault-related offsets of geomorphic surfaces developed on surficial materials. Features were then studied in the field to eliminate those not resulting from surface faulting. Those confirmed as fault scarps and suitable for quantitative geomorphology were studied by profiling according to the procedures described in Bucknam and Anderson (1979).

The measurements of the scarp have provided data both on the ages of the scarps and on the surface displacement at the scarp. An important assumption in our study is that fault scarps provide a useful estimate of the number of earthquakes of a given magnitude range that have occurred in a given span of time. Combined with surface displacement data from historic earthquakes, our studies indicate that within the Great Basin the fault scarps that we have mapped represent a nearly complete record of earthquakes in the magnitude range 7-7 1/2 that have occurred there during Holocene time. To the extent that the record is incomplete, rates of seismicity determined from Holocene fault scarp data would be expected to underestimate the frequency of large earthquakes.

In order to generalize the patterns of faulting, we have taken a largely heuristic approach in defining seismic source regions, modified locally where suggested by our understanding of geologic structure and history. Characteristics of the faulting within mapped areas allow delimitation of large regions that are distinctive on the basis of ages of most recent movement on the faults within the region and the frequency of movement on the faults in late-Quaternary time. Regions defined in this way do not carry direct implications as to the potential of any given fault within the region to undergo movement (in other words, generate an earthquake). Nor, as mentioned earlier, is it implied that all faults capable of generating earthquakes within each region are shown.

The source areas defined in this manner are large regions within which the long-term average seismic activity has certain distinctive characteristics. The new source regions in the Great Basin and their associated seismic characteristics form the basis for evaluating, in a probabilistic manner, the level of peak accelerations expected there at sites underlain by rock.

The fault scarp data from the various seismic source regions that we have defined permit calculation of the rates of occurrence of magnitude 7 and greater earthquakes (events). The rates are expressed as the number of earthquakes of magnitude 7 and greater per 10,000 years (length of Holocene epoch) per 10,000 km^2 (an area equivalent to about 1° of latitude and longitude). Values range from 0.7 events/10^4 yrs/10^4 km^2 to 3.4 events/10^4 yrs/10^4 km^2. The highest value was determined by R. E. Wallace (1978) in an area in north-central Nevada. Combining rates and areas for each of the source regions gives an average recurrence interval of 240 years/event for the entire Great Basin. This value does not reflect the contribution of fault systems of possibly higher than average rates of activity such as the Wasatch fault in Utah or the historically very active Nevada Seismic Zone.

On a national scale, development of seismotectonic data for defining seismic source zones has been done through a series of four regional workshops convened during the past year by the USGS under the direction of F. A. McKeown. The goal of each workshop has been to prepare a preliminary map of geologically determined seismic source zones based on the advice of panels of knowledgeable experts from the USGS, universities, and consulting firms in a forum which considers the problems associated with the preparation of probabilistic ground motion maps. The factor used most consistently by these panels in establishing the seismotectonic aspects of zones in much of the western United States is the age of last faulting. Therefore, the maps of source zones developed at those meetings tend to show areas within which faults of Holocene versus late Pleistocene versus Quaternary age are distributed and other areas where no faults of those ages are known. However, other factors such as tectonic province boundaries, boundaries of basin and mountain blocks, volcanic fields, volcanic alinements, possible buried magma chambers, basement configuration, and physiography (including drainage control) were given consideration. Comparisons with the data from the Great Basin were made to assign very general activity rates to each zone along with an estimate of the maximum magnitude expected for each zone.

Combining historic data on seismicity with the geologically determined source zones raises some problems in evaluating expected ground motion for exposure times on the order of a few tens to a few hundreds of years. Examples occur in the case where historically high rates of seismicity occur in areas for which there is geologic evidence of a much lower average rate during the past few thousand years and in the opposing case where there is geologic evidence of a higher average rate of activity than indicated by historic seismicity.

PRELIMINARY RESULTS OF MICROEARTHQUAKE STUDIES IN THE CENTRAL MISSISSIPPI VALLEY

by

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INTRODUCTION

The earthquake problem in eastern North America is interesting as well as frustrating. Large earthquakes have occurred in the past, such as the 1811-1812 New Madrid and the 1929 Grand Banks earthquakes, but a working model for their occurrence has yet to be discovered. There seem to be patterns of earthquake occurrence, but at the same time damaging earthquakes also occur away from these patterns.

A statement of the problem is expressed as a series of questions:

a) Why do earthquakes occur in eastern North America, especially in the middle of a tectonic plate?

b) Given the occurrence of these earthquakes, how large could the earthquake be?

c) What are the source zones, if any, and where <u>cannot</u> a large earthquake occur?

Similar questions can be asked about any specific earthquake zone, even the central Mississippi given recently acquired microearthquake data.

It is the object of this report to present a summary of the present knowledge of earthquakes in the central Mississippi Valley as well as to address additional research that is required.

HISTORICAL DATA

Nuttli (1979) compiled a catalog of over 1100 historical earthquakes in the central United States prior to 1975. Figure 1 shows the 116 earthquakes in the nineteenth century that occurred in the central Mississippi Valley. The earthquakes in southeast Missouri predominate. Significant earthquakes also occurred in south central Illinois. Figure 2 shows the location of 265 earthquakes which occurred in the first half of the twentieth century while Figure 3 presents data from 164 earthquakes for the third quarter of this century. A very interesting observation is the migration of activity southward from Cairo, Illinois during the twentieth century. Southeastern Illinois seems somewhat spatially stable. Finally, Figure 4 shows the 1190 earthquakes located by a regional microearthquake array for the six year period between July 1, 1974 and June 30, 1980. Very definite seismicity patterns are obvious near New Madrid. In addition an interesting north-south pattern west of the Missouri bootheel, a north-south pattern in southeastern Illinois and a southeast trending pattern near Cape Girardeau, Missouri are evident.

MAGNITUDE PATTERNS

One must be careful about the specification of patterns, though, because they may be due to varying magnitude detection thresholds of the seismic array. For example, Figure 4 apparently indicates a very dense cluster of seismicity near New Madrid, MO and Ridgely, TN with a "thinning out" southwestward into Arkansas.

Figures 5, 6, 7 and 8 are plots of all earthquakes located in a 6 year period with $m_b \geq 3.0$, $m_b \geq 2.5$, $m_b \geq 2.0$ and $m_b \geq 1.5$, respectively in a 4^o by 4^o search area. Figures 9, 10, 11 and 12 are plots of all earthquakes located in the 6 year period in a 1.5^o by 1.5^o search area for $m_b \geq 3.0$, 2.5, 2.0 and 1.5, respectively. No patterns are obvious in the $m_b \geq 3.0$ plots. However, the high seismicity trend from New Madrid into Arkansas is well defined by the $m_b \geq$ 2.5 data. Besides this, the pattern seems quite uniform in space. This spatial uniformity is also apparent in the $m_b \geq 2.0$ data. However, the $m_b \geq 1.5$ plots indicate considerable clustering. Because of the spatial uniformity at higher magnitude cutoffs, a magnitude dependent detection capability is indicated. This indicates that the seismic activity near New Madrid, MO and Ridgely, TN is not necessarily that much different from other areas within the zone. The north-south trend in Arkansas and southern Missouri at 91^oW also suffers from detection capability but may be a legitimate pattern, or zone of seismicity. A north-south zone in southeastern Illinois at 88.5^oW may also represent a definite source zone.

VERTICAL DEPTH PROFILES

Given the distinctly linear patterns of seismicity near New Madrid, a projection of the seismicity onto vertical plane could serve to define the fault plane orientation. As a first attempt, the six year microearthquake data base was searched for all free depth hypocenter solutions between January 1, 1976 through December 31, 1979. The epicenters are plotted in Figure 13 as well as a rectangular search area for a study of the 100 km long seismicity trend in northeastern Arkansas. The orientations of two planes of projection, WX and YZ, are indicated. Figure 14 shows the vertical projections. The majority of well located solutions occur at depths between 5 and 15 km. Profile YZ indicates that the seismicity pattern is very narrow and extends down to at least 15 km and that the fault zone is almost vertical.

Two other zones of interest are examined in Figures 15 and 16. The

Ridgely, TN search area and the corresponding projections, OP and QR, indicate a northwest striking seismicity zone, which is not that evident from the epicenter plots. The New Madrid, MO cluster does not much other than the fact that the seismicity has an east-west trend.

The previous plots were contaminated somewhat by the fact that the earthquake data was extracted from the data base without qualification. In an attempt to refine the depth profiles, a search was made of all earthquakes which occurred between April 1, 1977 and June 30, 1979 which were located by 4 or more stations and which had data in a distance range adequate to constrain focal depth. This selected data set was then relocated using joint hypocenter techniques. Amazingly, there was little difference in the hypocenter locations before or after the JHD relocation.

Figure 19 shows the relocated epicenters as well as the search areas near New Madrid, MO and Ridgely, TN. Profile OP now shows a very narrow pattern for the Ridgely trend, and indicates a steeply dipping pattern of seismicity to the southwest. It also shows that many of the earthquakes used really were not well constrained in depth as evidenced by the migration of focal depths to the minimum depth of 1 km permitted in the inversion. With respect to the vertical profiles near New Madrid, a very tight pattern was obtained by allowing the plane of projection to strike N 20^{o}E. Thus a southerly dipping fault plane striking N 110^{o}E is inferred.

REVIEW OF FOCAL MECHANISMS

Canas and Herrmann (1978) presented focal mechanisms for earthquakes in the Central Mississippi Valley obtained by composite focal mechanism techniques as well as surface wave studies of larger earthquakes. Unambiguous composite focal mechanisms were obtainable for only a few of the linear patterns of seismicity evident in Figure 12. Along the Arkansas trend, Figure 17, the P-wave first motion data could be fit by focal mechanisms with significant components of right lateral strike-slip motion with a strike in the direction of the seismicity trend. The Ridgely, TN area, Figure 19, indicated a northwest striking reverse fault with one nodal plane dipping steeply to the northeast and the other dipping at an angle of about 30^{o} to the southwest.

Surface wave solutions were available for three points along the Arkansas trend, indicating significant components of right lateral strike slip motion along a northeast trending nodal plane, and in two of the solutions, reverse faulting with the nodal plane dipping about 60^{o} to the northwest. Two surface wave focal mechanisms near the New Madrid trend of Figure 19 indicated predominantly left lateral strike-slip motion on an east west trending nodal plane.

There is some discrepancy between the focal mechanism inferred fault plane orientation and the fault plane geometry inferred from the hypocenter distribution. In particular, the Arkansas trend indicates a very steeply dipping plane to the southeast while the surface wave data infers a fault plane dipping about 60^{o} to the northwest. This is also a problem on the Ridgely trend. The focal mechanism and hypocenter dips along the New Madrid trend are in agreement, but the inferred strikes differ by 30°.

DISCUSSION

We know much more today about central Mississippi Valley earthquakes that we knew six years ago. Major questions remain to be answered. As before, we are hampered in seismological studies by relatively low occurrence rates as well as high levels of cultural noise, especially in the active agricultural areas of southern Illinois and the Mississippi Embayment. The major problems that require more data are:

a) What are the northern and southern extents of a possible 1811-1812 earthquake sequence?

b) What are the strong ground motions that can be generated by such earthquakes?

c) How can we be specific about the northern and southern extent of possible major earthquakes if we do not know why the earthquakes are occurring at all? The central Mississippi Valley may be a reactivated ancient zone of weakness, but what are the precise characteristics of this zone.

These are difficult questions to be answered, but the data base is slowly being improved upon which judgment can be based.

Some research areas and tasks are suggested, though.

a) Continue monitoring of microearthquake activity with an enhanced seismic network. The new seismicity patterns in Arkansas and southeastern Missouri require monitoring.

b) Carefully reanalyze the existing 6 year data base using joint hypocenter relocation techniques to pin down the geometrical patterns of the seismicity. Given this, reconstruct composite focal mechanisms.

c) Obtain detailed Q, P and S velocity models for the central Mississippi Valley. The installation of a dense accelerograph network in the area by the USGS will yield important strong motion data, but an accurate earth model is required for interpretation of that data base.

The earthquakes in the central Mississippi Valley are contributing a great deal of new data which must continue to be analyzed with the view of applying these results to the broader problem of earthquake hazard in eastern North America.

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Nuttli, O.W. (1979). Seismicity of the central United States: Reviews in Engineering Geology IV, Geological Society of America, 67-93.

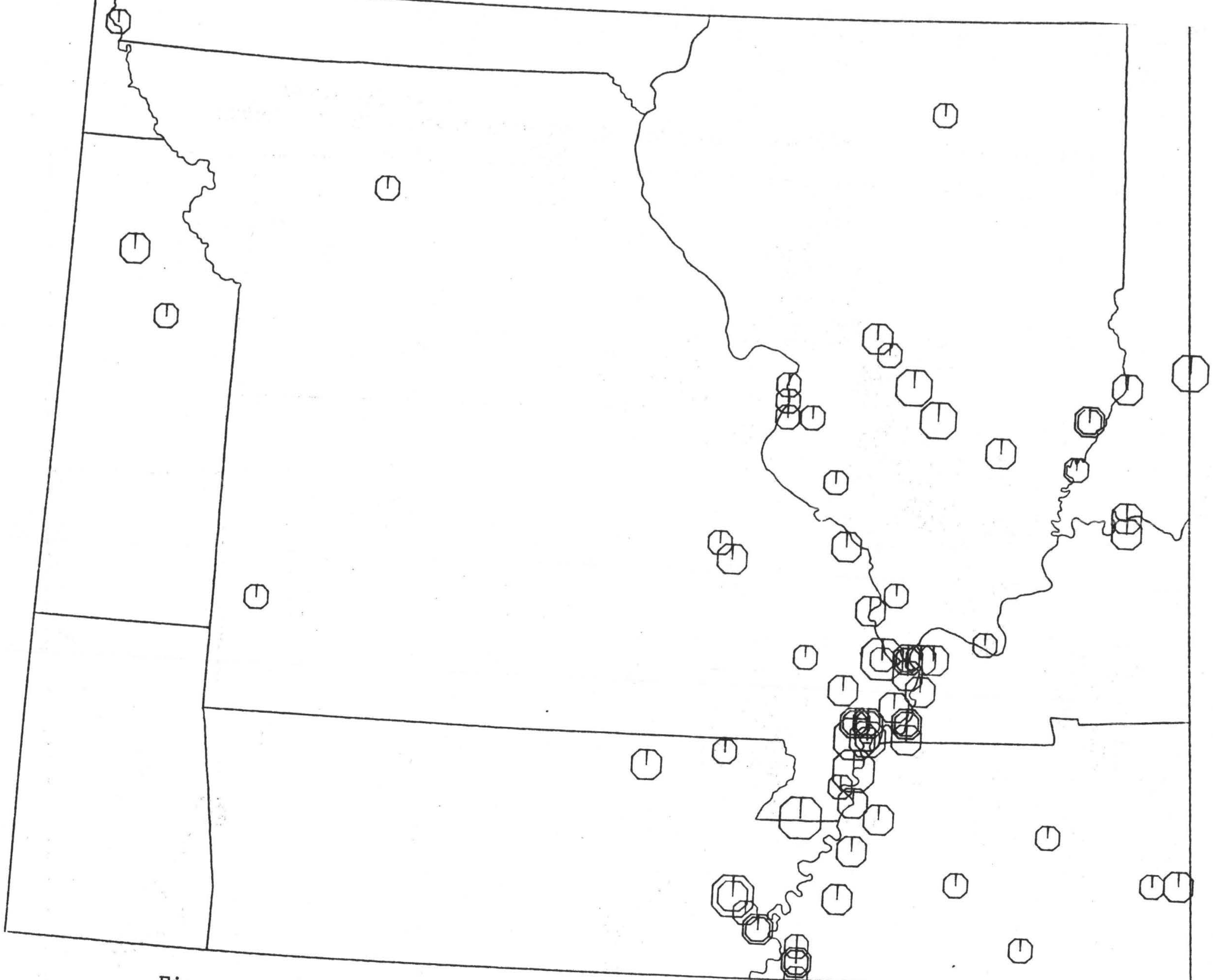


Figure 1. Epicenters from the Nuttli (1979) earthquake catalog for the years 1800-1899. Symbol size is proportional to magnitude.

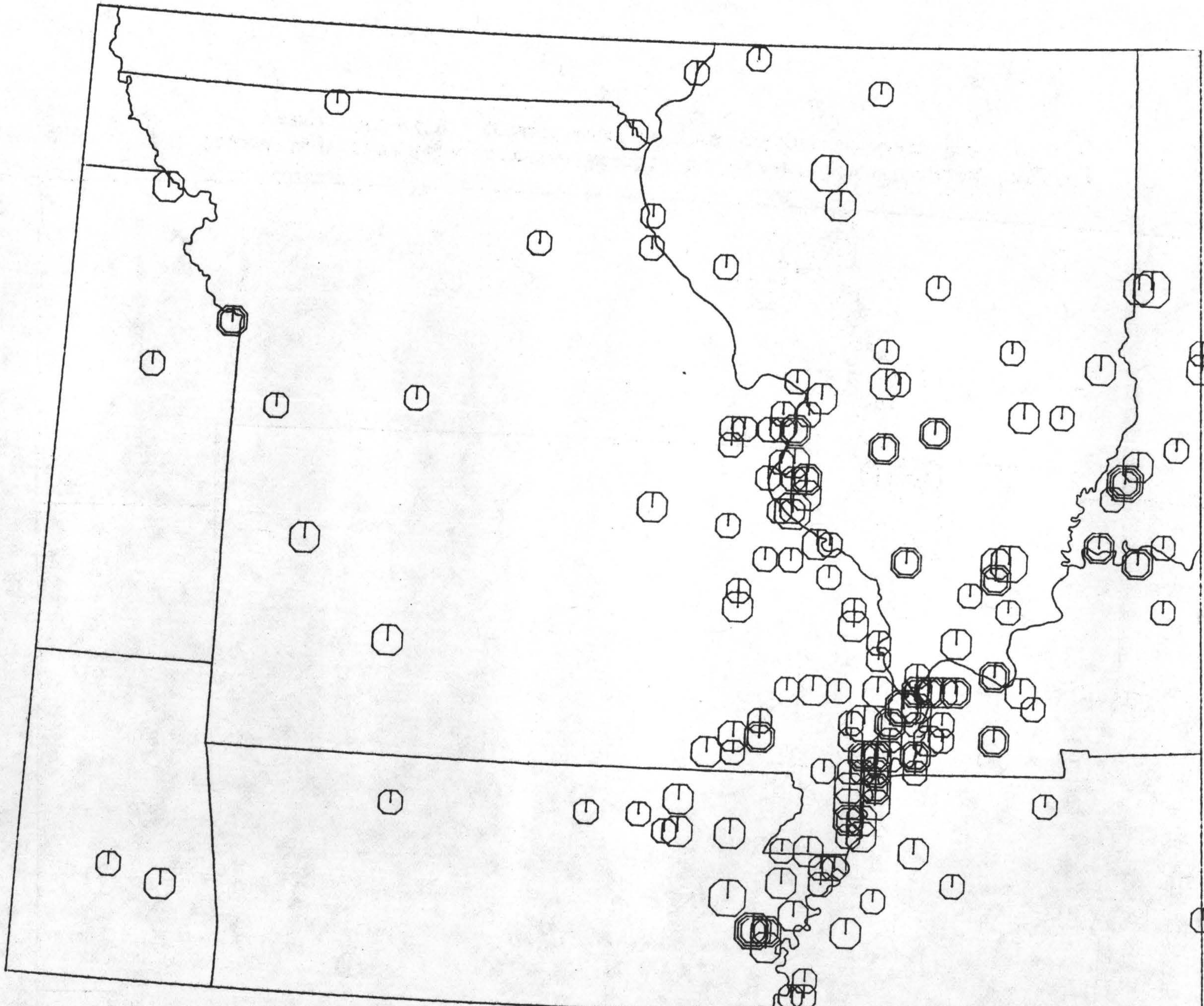


Figure 2. Epicenters from the Nuttli (1979) earthquake catalog for the years 1900-1949.

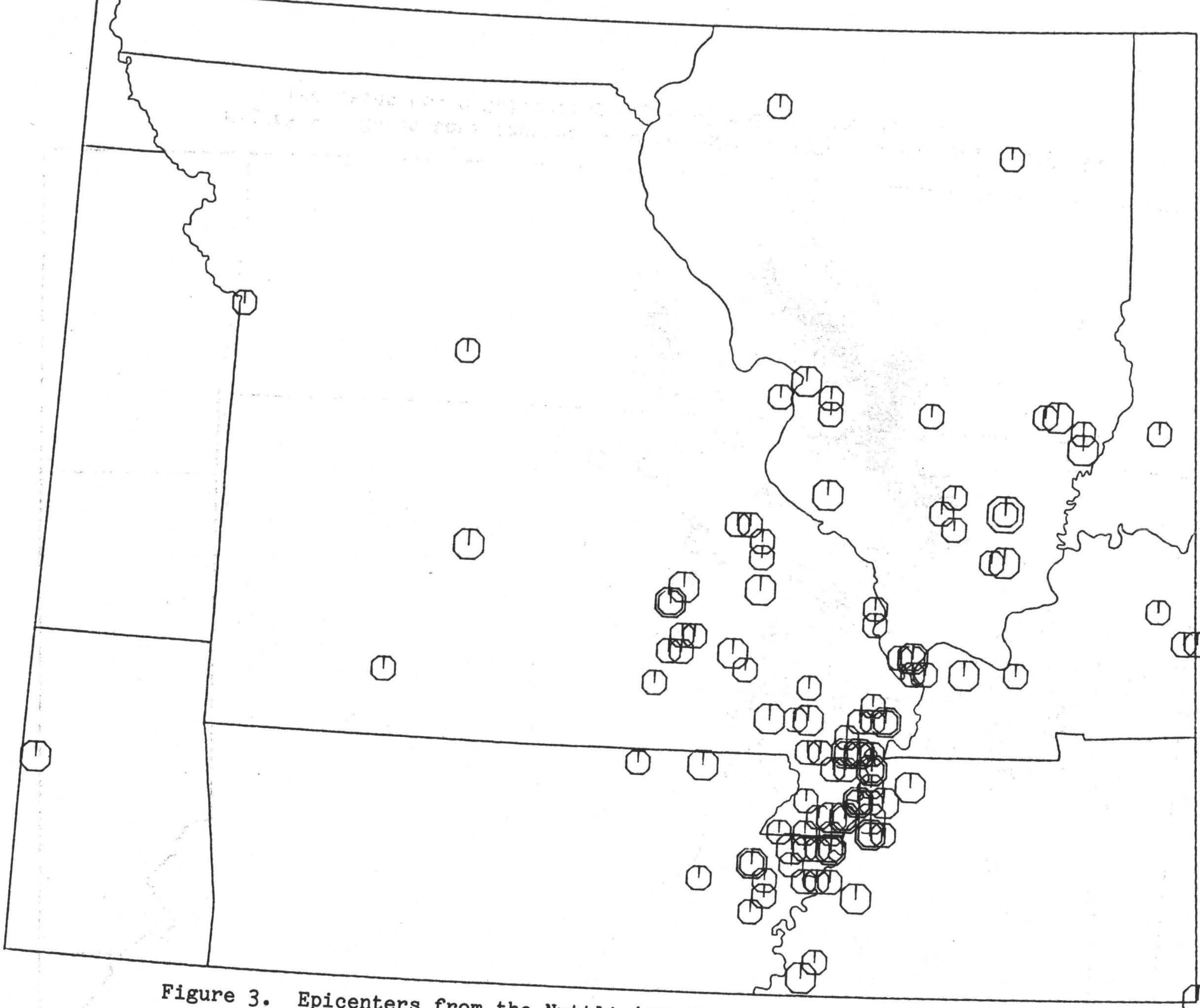
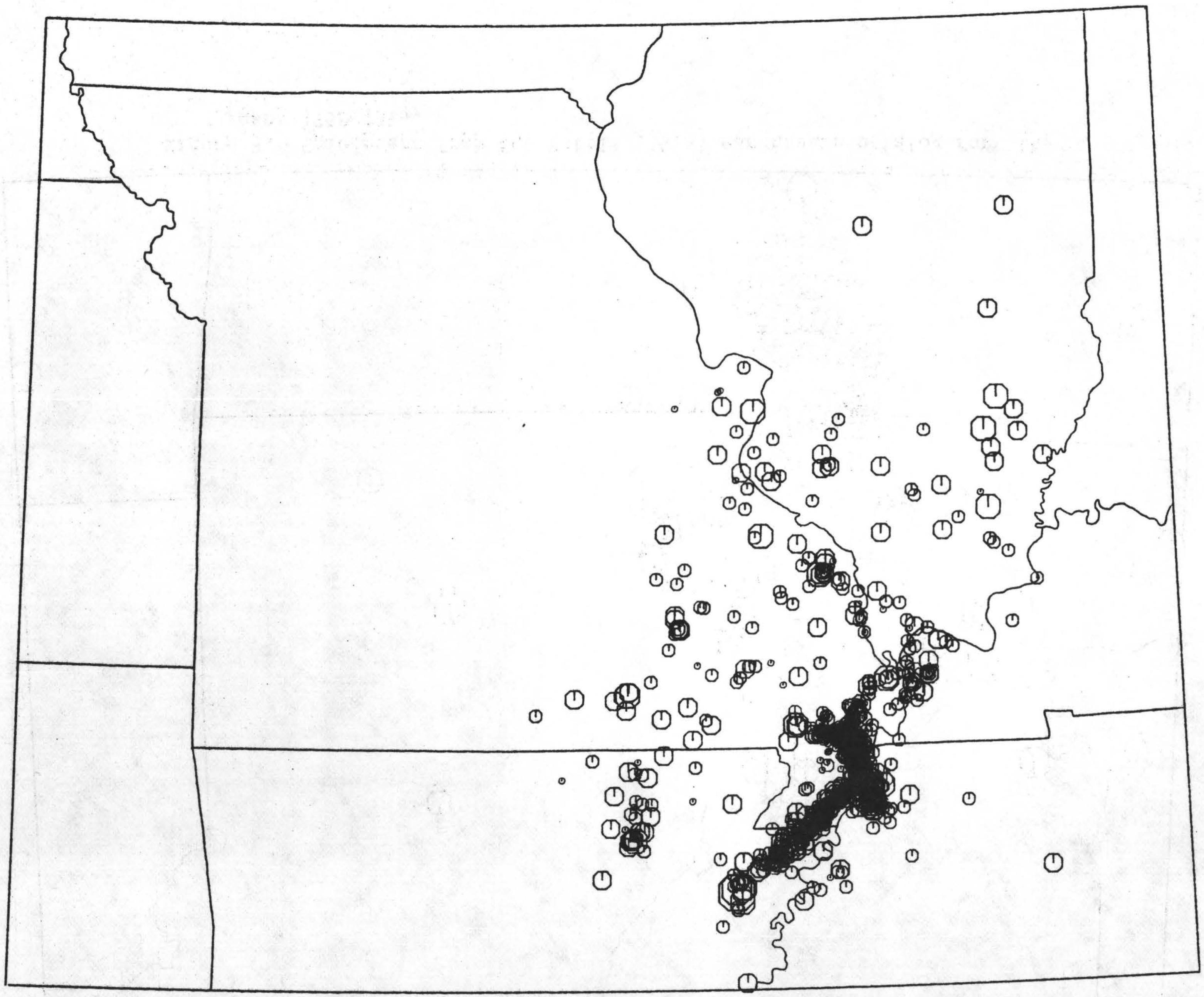


Figure 3. Epicenters from the Nuttli (1979) earthquake catalog for the years 1950-1974.



Figur3 4. Epicenters located between July 1, 1974 - June 30, 1980 by the Saint Louis University regional seismic array.

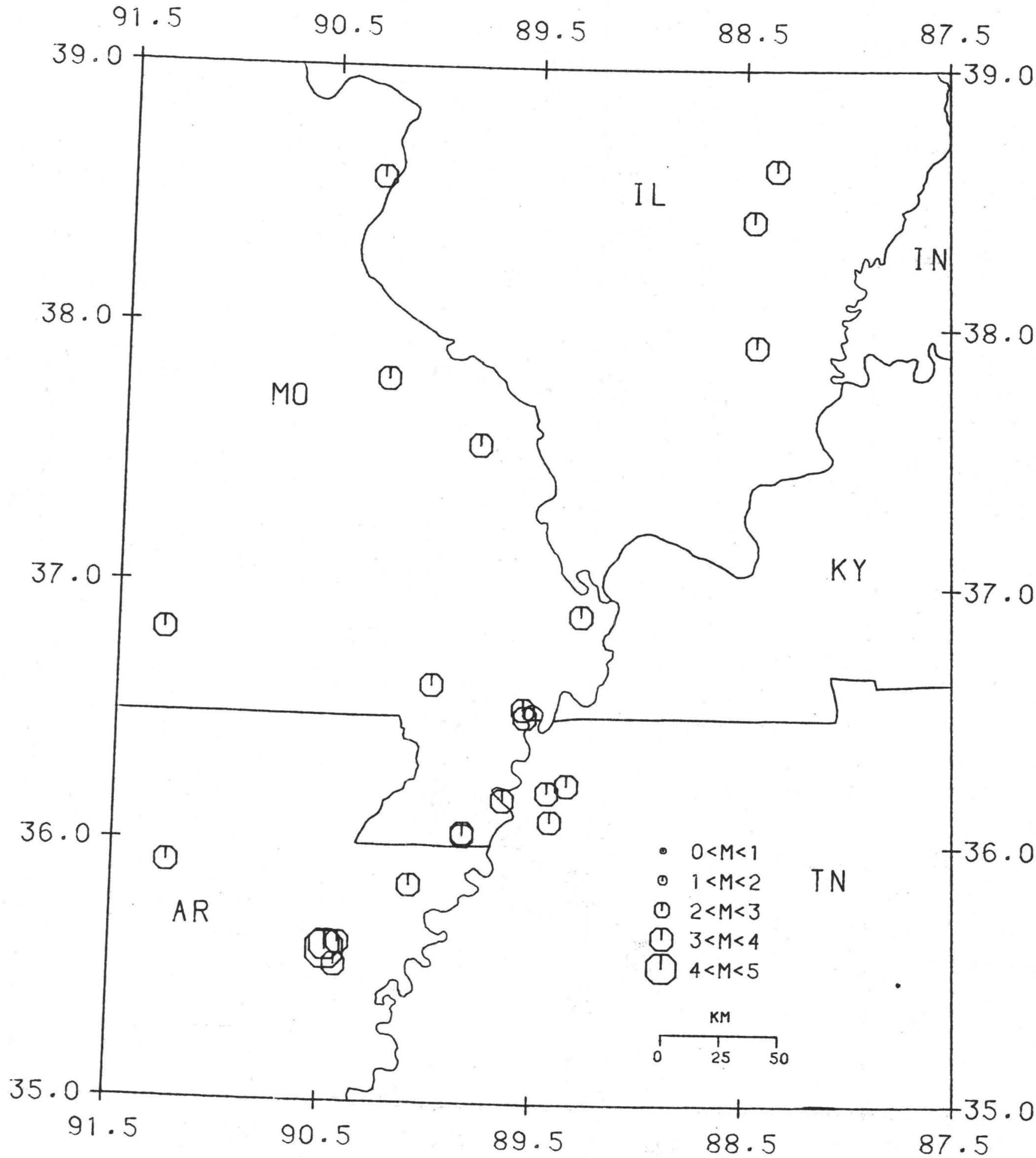


Figure 5. Plot of all earthquakes located in 6 years with $m_b \geq 3.0$.

REPORTING PERIOD 01 JUL 1974 TO 30 JUN 1980
LEGEND . △ STATION ⊙ EPICENTER

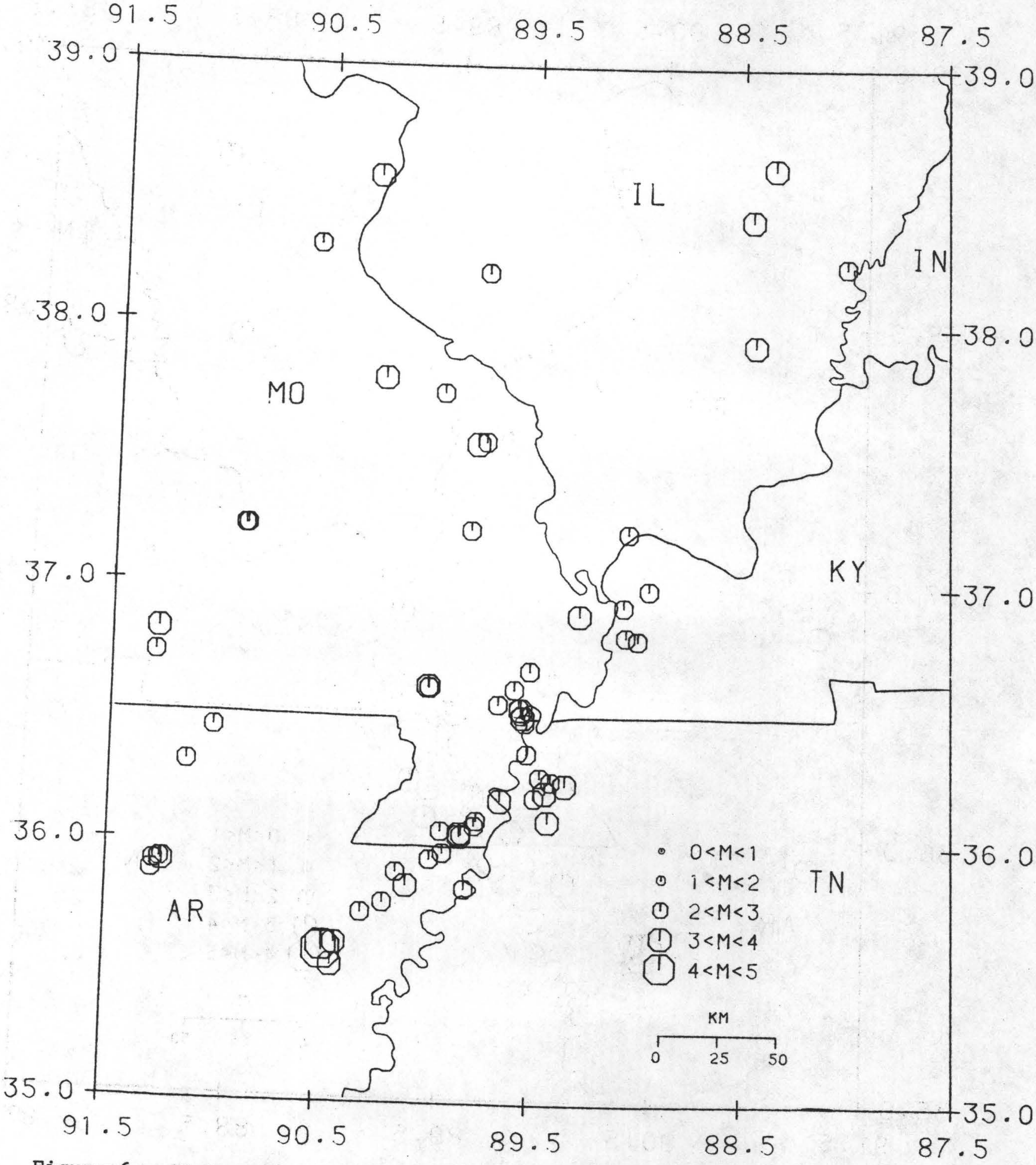


Figure 6. Plot of all earthquakes located in 6 years with $m_b \geq 2.5$.

REPORTING PERIOD 01 JUL 1974 to 30 JUN 1980
LEGEND . △ STATION ⊙ EPICENTER

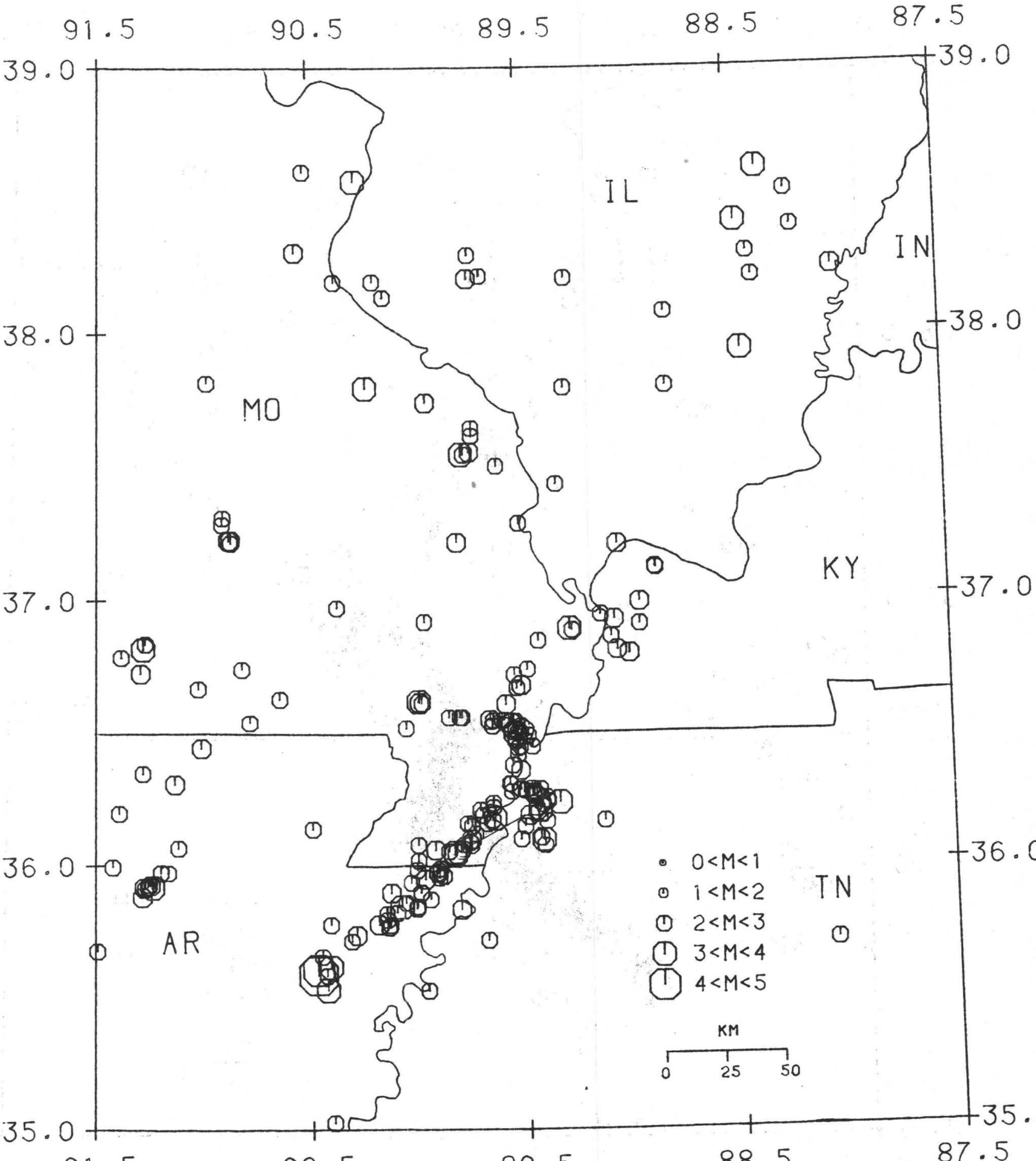


Figure 7. Plot of all earthquakes located in 6 years with $m_b \geq 2.0$.

REPORTING PERIOD 01 JUL 1974 TO 30 JUN 1980
LEGEND . △ STATION ⊙ EPICENTER

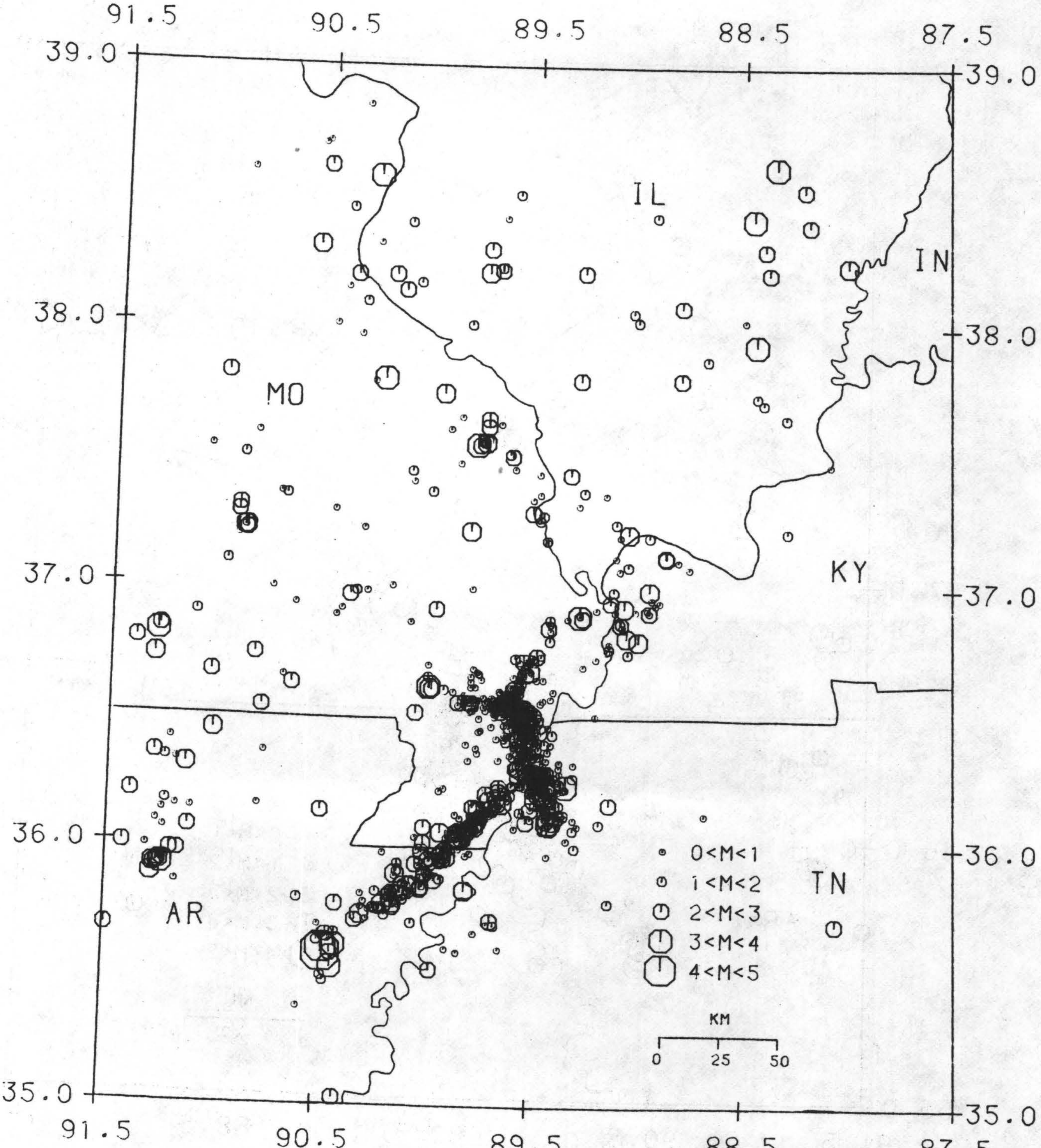


Figure 8. Plot of all earthquakes located in 6 years with $m_b \geq 1.5$.

REPORTING PERIOD 01 JUL 1974 TO 30 JUN 1980
LEGEND . ▲ STATION ⊙ EPICENTER

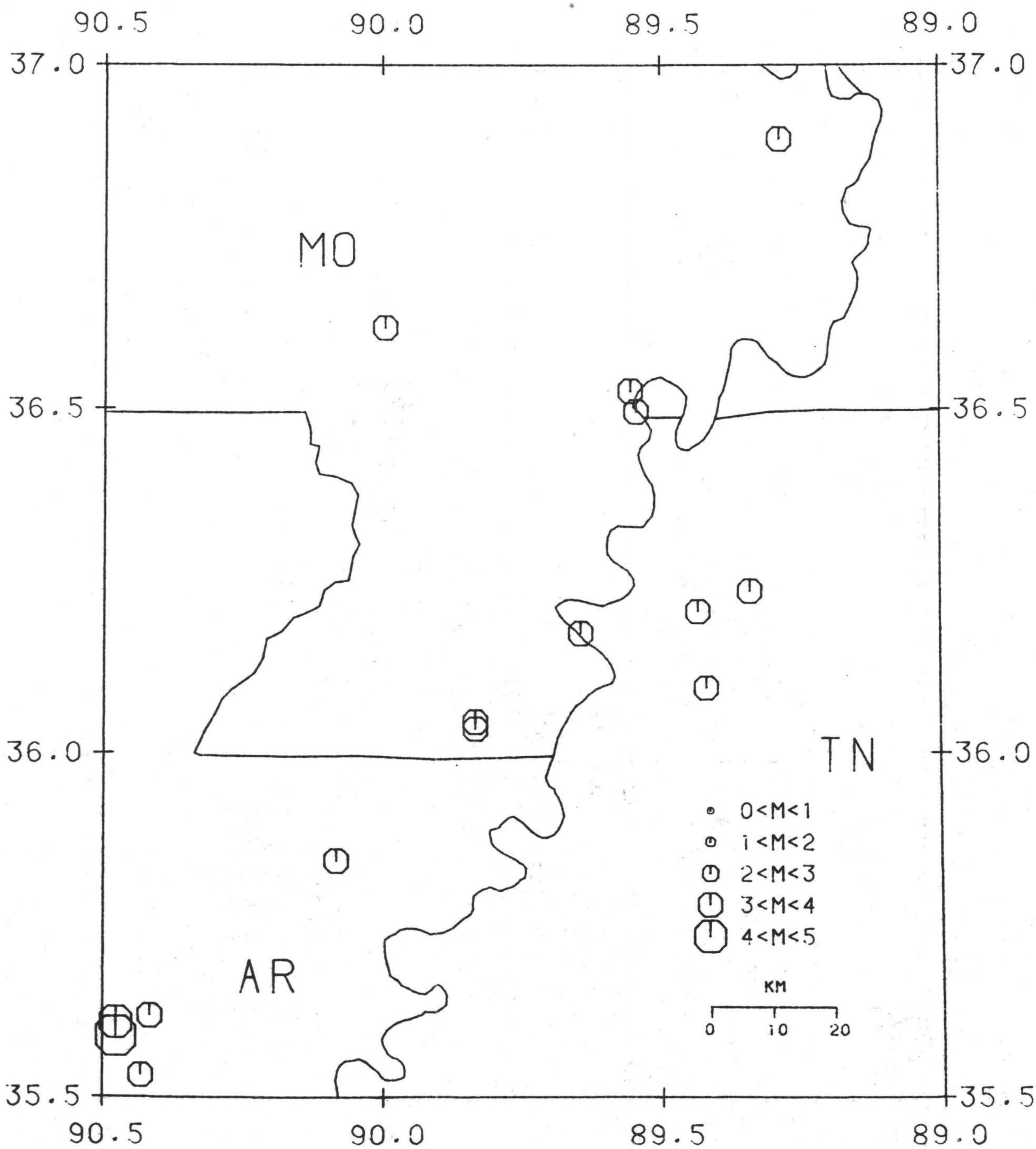


Figure 9. Plot of all earthquakes located in 6 years with $m_b \geq 3.0$.

REPORTING PERIOD 01 JUL 1974 TO 30 JUN 1980

LEGEND . △ STATION ○ EPICENTER

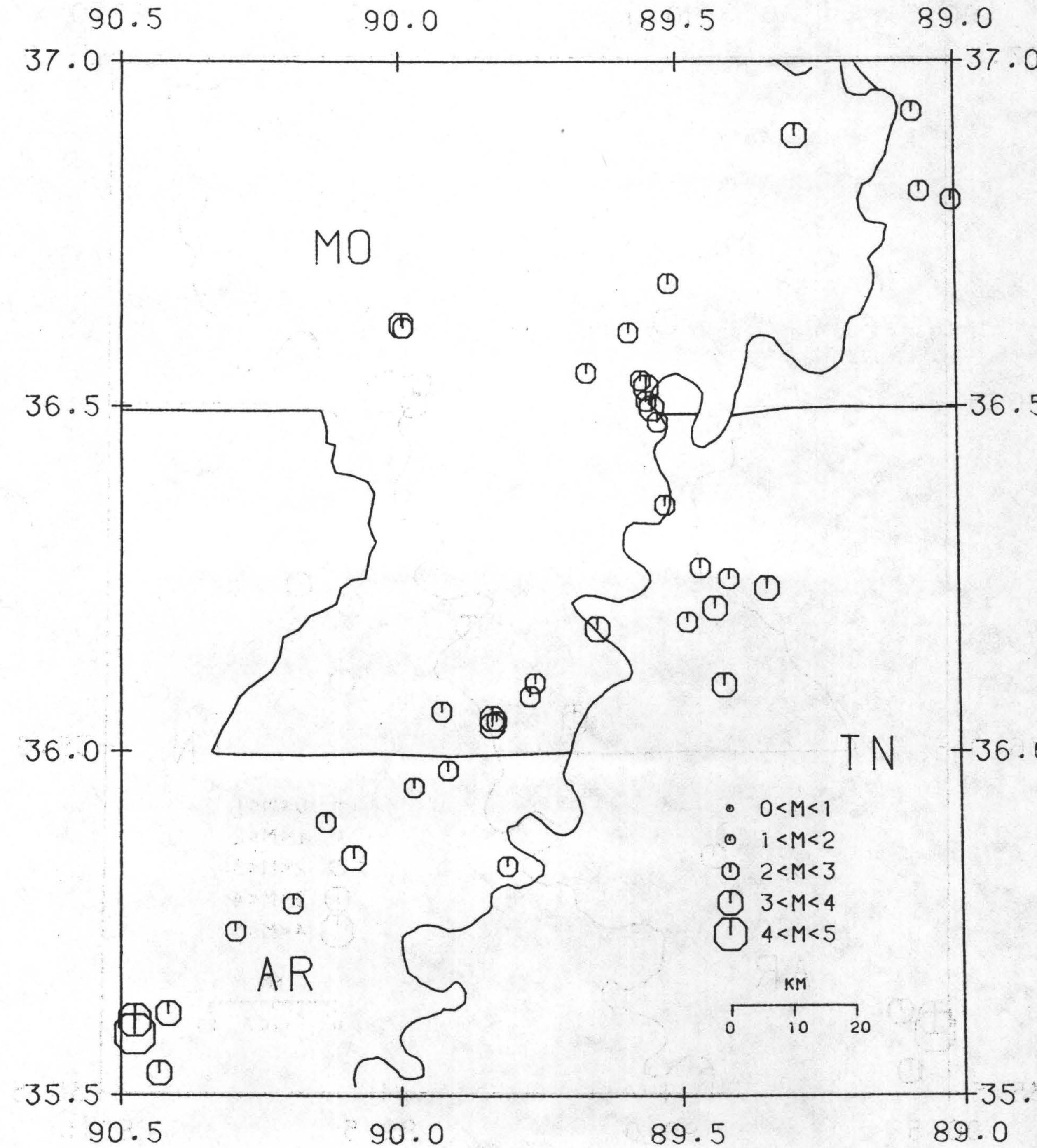


Figure 10. Plot of all earthquakes located in 6 years with $m_b \geq 2.5$.

REPORTING PERIOD 01 JUL 1974 to 30 JUN 1980
LEGEND . ▲ STATION ⊙ EPICENTER

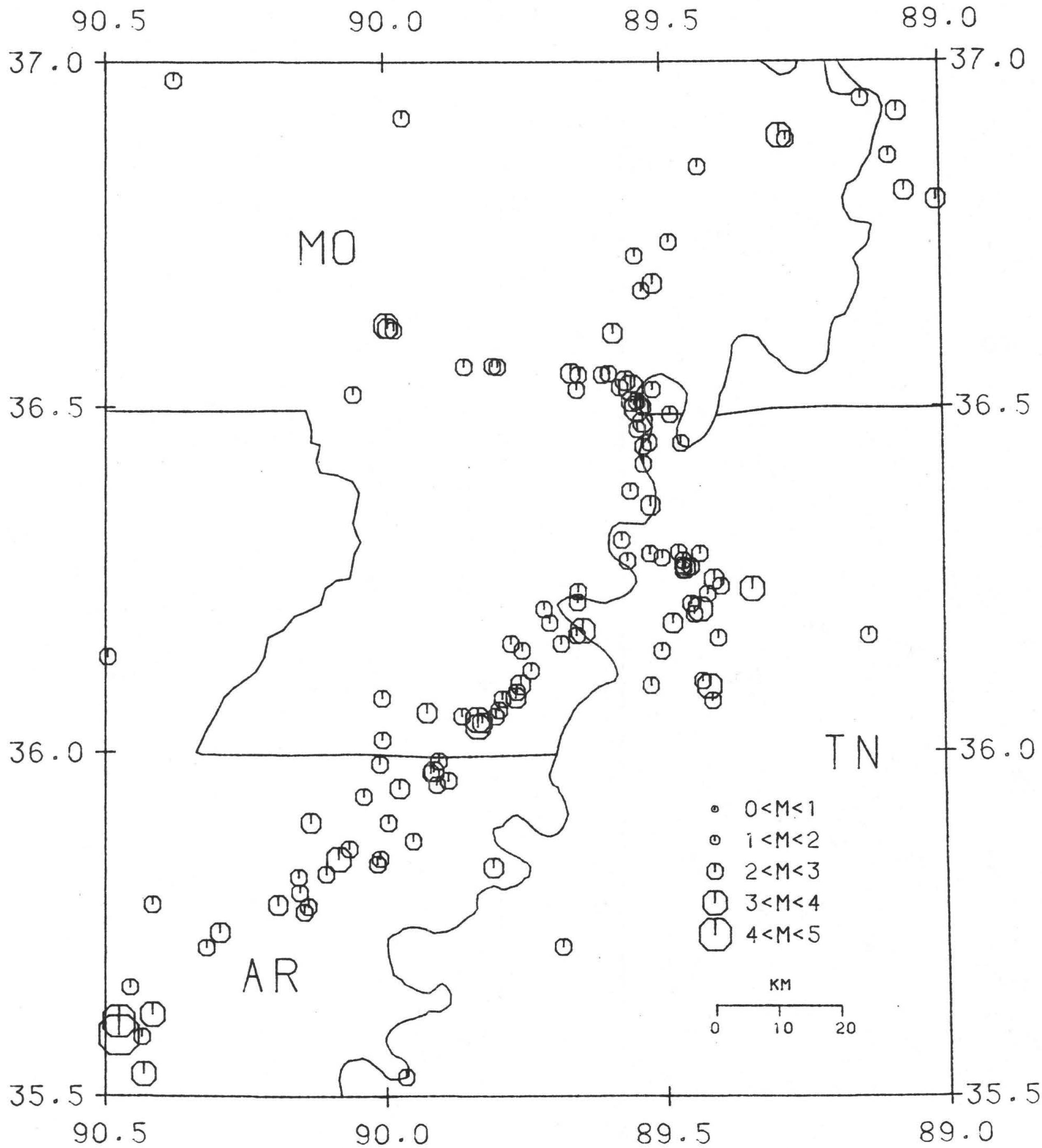


Figure 11. Plot of all earthquakes located in 6 years with $m_b \geq 2.0$.

REPORTING PERIOD 01 JUL 1974 TO 30 JUN 1980
LEGEND . △ STATION ⊙ EPICENTER

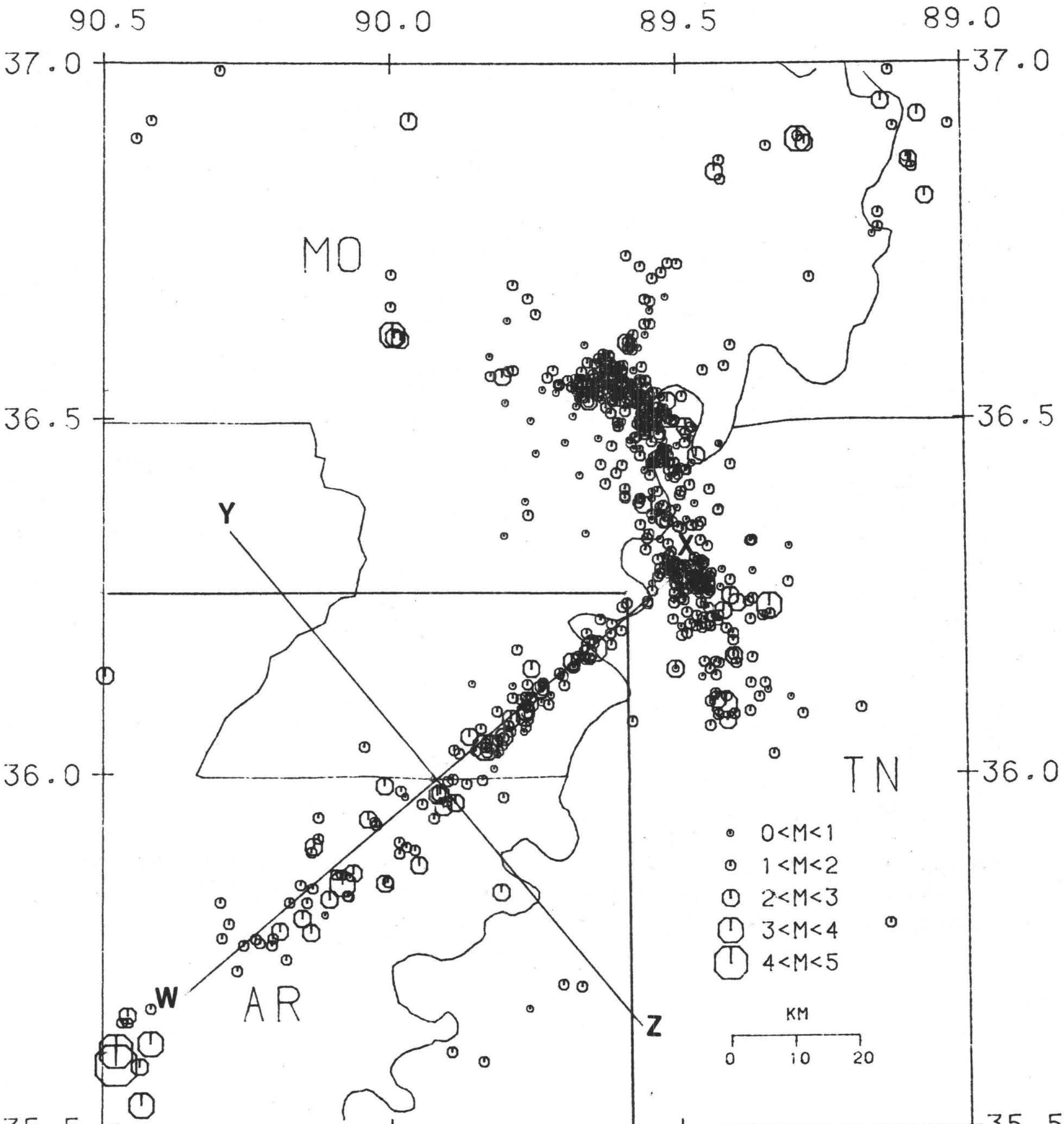


Figure 13. Epicenters located between January 1, 1976 through December 31, 1979. The search area for a study of the Arkansas trend is indicated as well as the orientation of planes of projection.

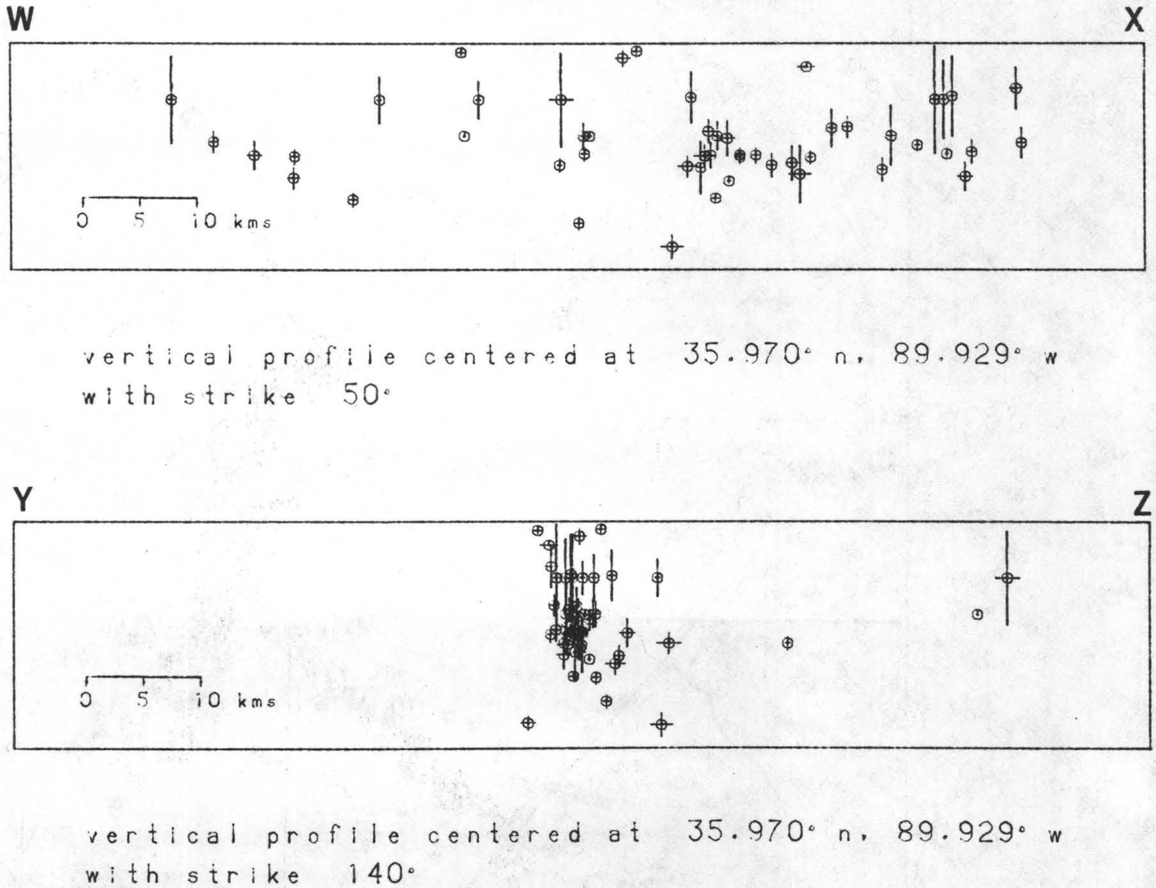


Figure 14. Vertical projections of hypocenters for the Arkansas trend. Error bars are the 95% confidence limits of the HYPO71 solutions. There is no vertical exaggeration in the plots.

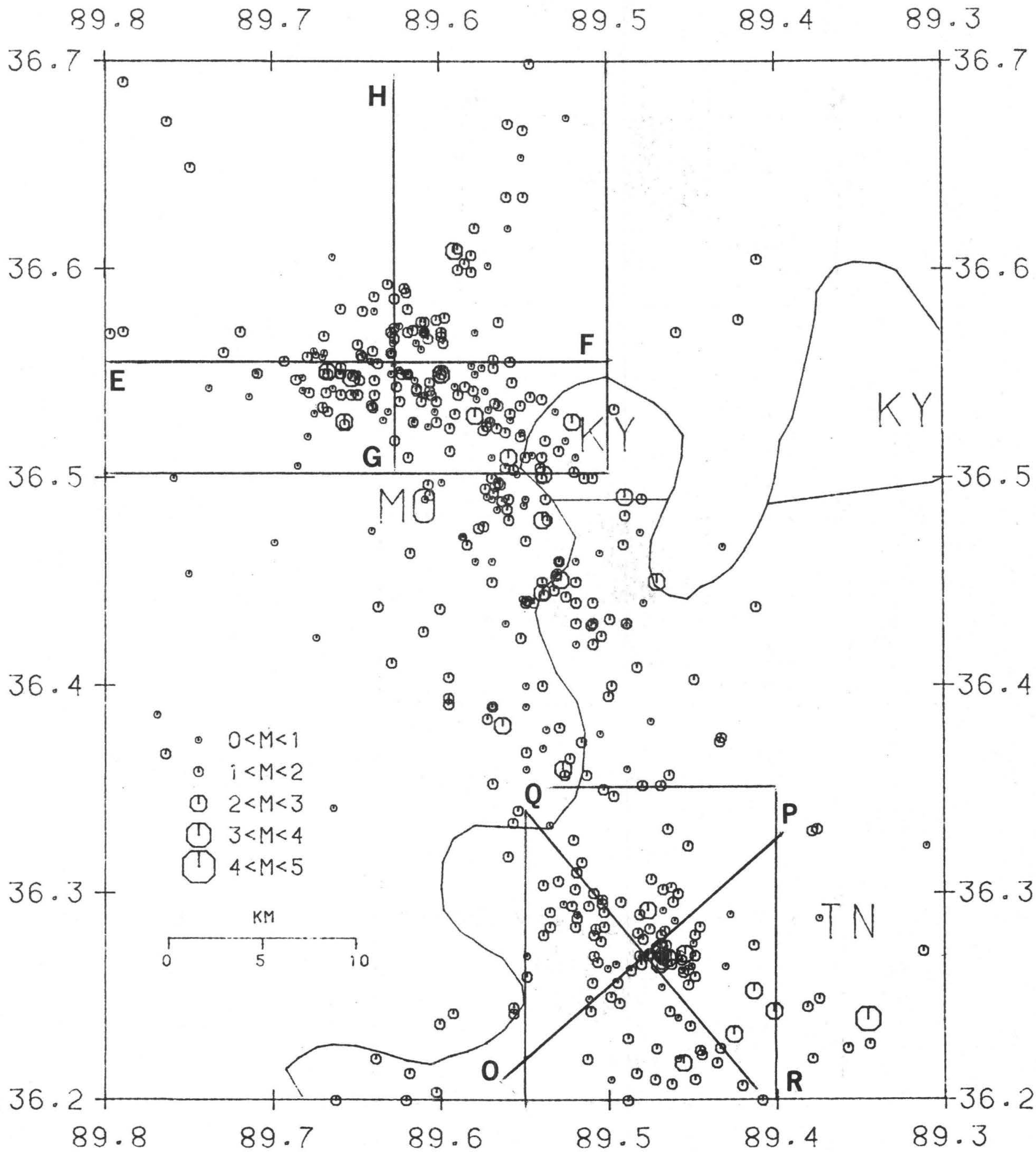
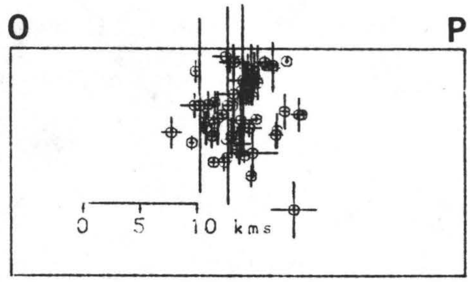
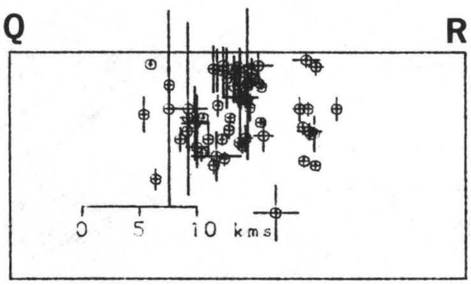


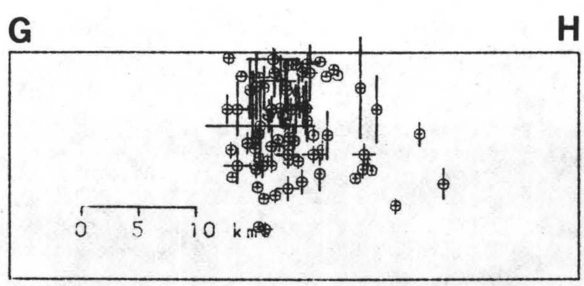
Figure 15. Epicenters located between January 1, 1976 and December 31, 1979. Search areas and profiles are indicated near New Madrid, MO, EF and GH, and Ridgely, TN, OP and QR.



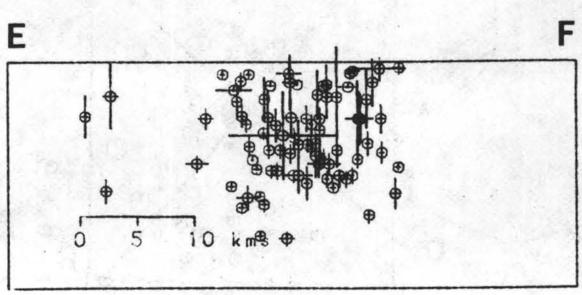
vertical profile centered at 36.270° n, 89.480° w
with strike 50°



vertical profile centered at 36.270° n, 89.480° w
with strike 140°



vertical profile centered at 36.555° n, 89.626° w
with strike 0°



vertical profile centered at 36.555° n, 89.626° w
with strike 90°

Figure 16. Vertical hypocenter profiles corresponding to the profiles indicated in Figure 15.

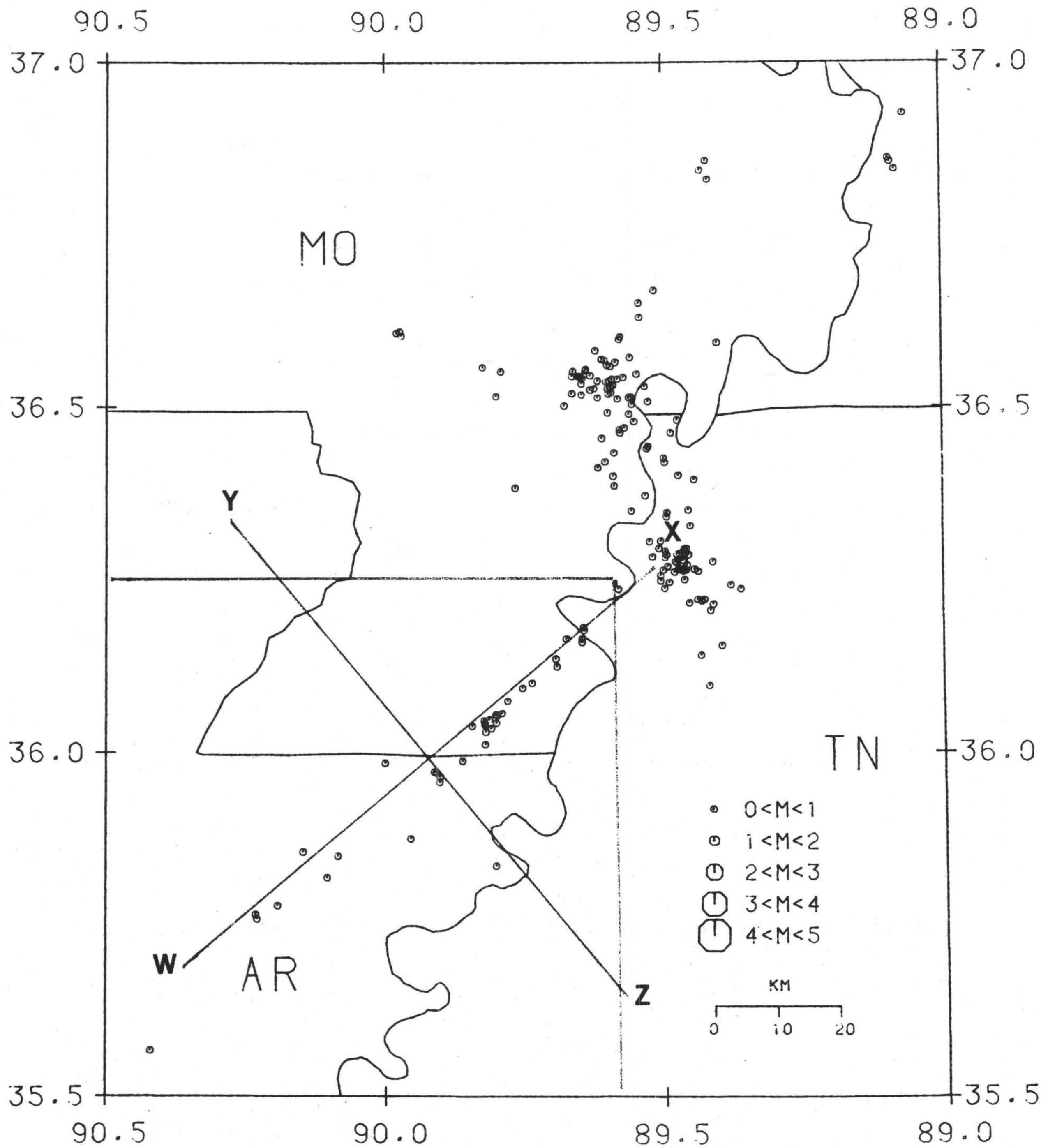


Figure 17. Plot of 177 relocated earthquakes for the time period April 1, 1977 through June 30, 1979. The search zone for the Arkansas trend is indicated.

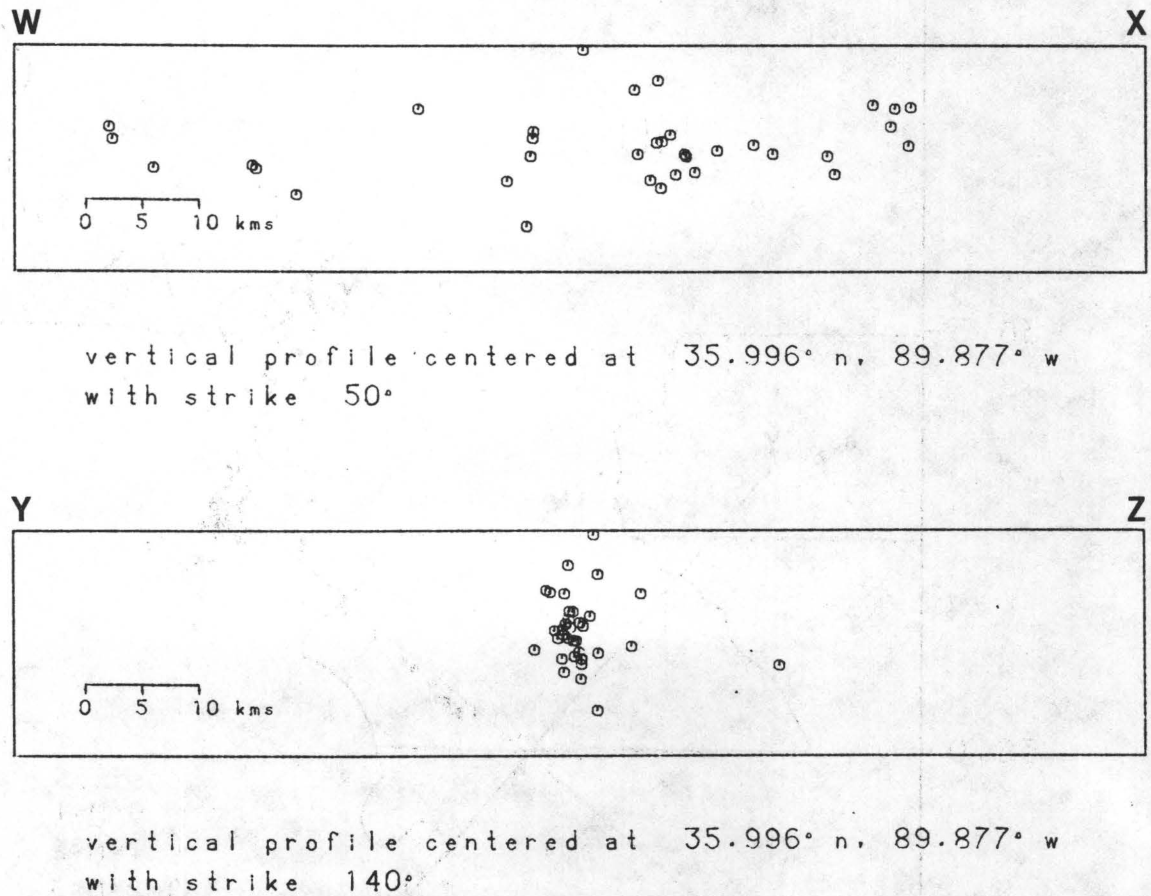


Figure 18. Vertical depth profiles of relocated hypocenters.

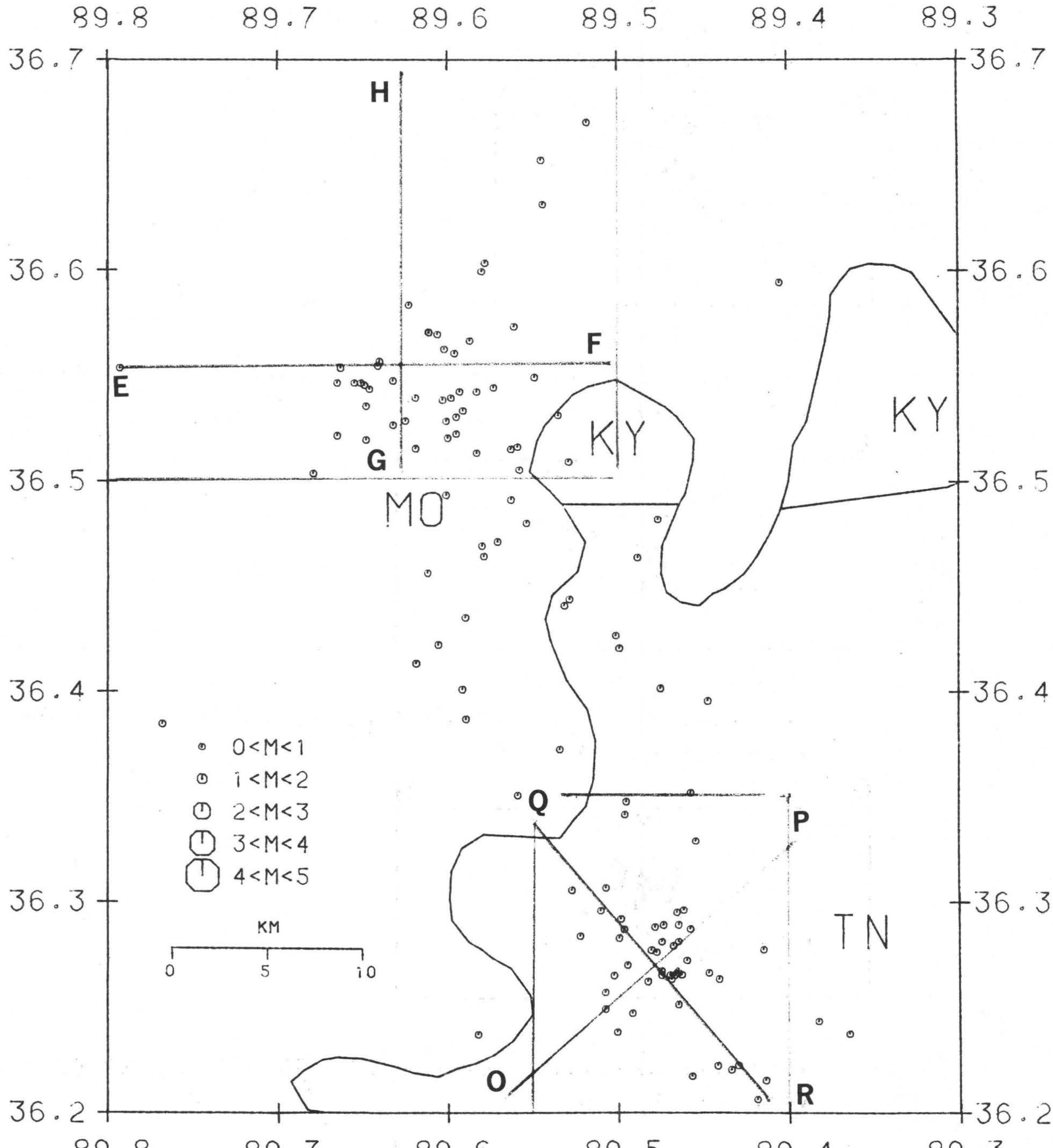
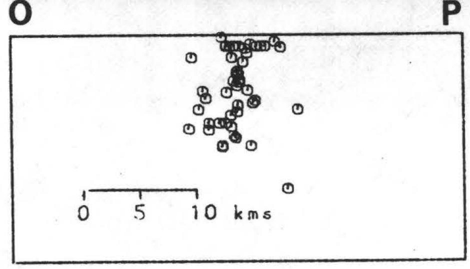
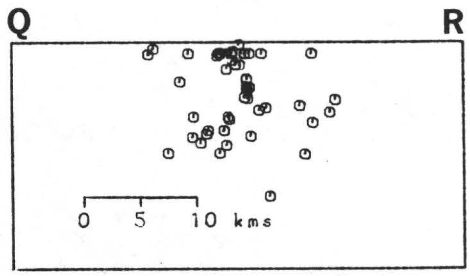


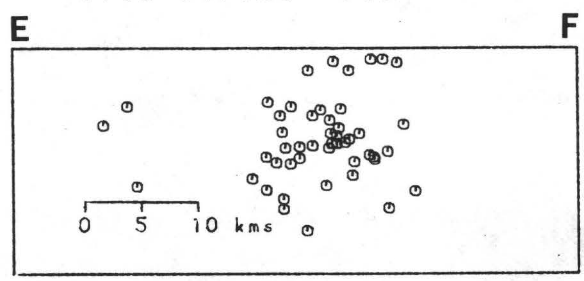
Figure 19. Relocated epicenters and search areas near New Madrid, MO and Ridgely, TN.



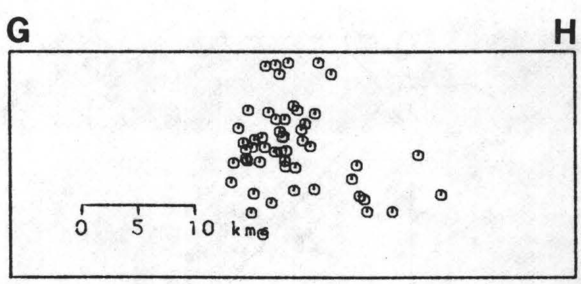
vertical profile centered at 36.272° n, 89.474° w
with strike 50°



vertical profile centered at 36.272° n, 89.474° w
with strike 140°



vertical profile centered at 36.553° n, 89.637° w
with strike 90°



vertical profile centered at 36.553° n, 89.637° w
with strike 0°

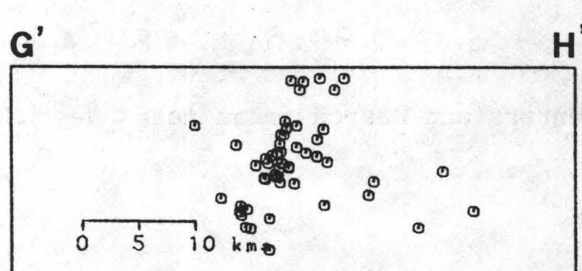


Figure 20. Vertical profiles of hypocenters within the search zones of Figure 19. The profile G'H' strikes N20°E.

Seismotectonics of the New Madrid Region

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Abstract

The process of seismic risk assessment must be based on information about the underlying cause of seismicity. Otherwise, it can only be assumed that the characteristics of future seismicity will be similar to those of past seismicity. A true understanding of seismicity requires that its relationship with geologic structure and regional tectonic processes be established. Determining such relationships for the areas of most important seismicity is a principal goal of the Earthquake Hazards Reduction Program. Recent results of the studies of the New Madrid seismicity zone illustrate the types of information that can be obtained. Synthesis of gravity, magnetic, seismicity, seismic reflection, geomorphic, and drill hole geologic data with basic geologic knowledge and concepts yields a model that can account for the main features and modes of deformation in the New Madrid region.

Introduction

Earthquake activity in the U.S. is widely distributed. Although the most intense seismic zones lie in Alaska, California, and other western States, important seismicity also occurs east of the Rocky Mountains. Because the eastern U.S. has areas of dense population, relatively older buildings, and most of the nation's nuclear reactors, to name but a few reasons, the assessment of seismic risk in Eastern United States is of critical importance.

In the western U.S., much of the seismicity can be explained as a manifestation of movement between the Pacific and North American plates. Alaskan seismicity results mostly from plate collision, seismicity along the San Andreas fault system results from the plates scraping past one another, and activity in the Great Basin from lateral spreading of the crust. Although the real situation is considerably more complex than these simple models would suggest, at least they provide a framework in which seismic risk can be evaluated. Thus it is possible in the West to identify seismic gaps and determine rates of fault movement. Importantly, the models provide a basis for assessing earthquake potential and fault activity.

No such conceptual models exist for evaluating potential seismicity of the eastern U.S., however, which means that the basis for seismic risk assessment there is much weaker. In fact, there is even considerable uncertainty about the causes of eastern U.S. earthquakes. Of course, it is accepted that sudden movement along a fault that occurs when stress exceeds strength is the phenomenon that excites seismic waves. But the origin of the stress field and the nature of fault movement is a subject of debate. Ideas about the cause of eastern U.S. earthquakes are reviewed by Sykes (1978).

Progress in understanding the origin of eastern seismicity will certainly require improvement of knowledge about the relationship between seismicity and geologic structure. Because earthquakes occur at focal depths up to 25 km, the crust is the primary target of study. Information about the upper mantle is highly relevant in determining the forces responsible for the tectonic deformation.

The area in the eastern U.S. that experienced the largest earthquakes in history is the Mississippi River valley in the region where the states of Arkansas, Missouri, Kentucky and Tennessee meet. Three earthquakes with magnitudes (m_b) from 7 to 7 1/2 struck that area in the winter of 1811-12 (Nuttli, 1973). Extensive disruption of the land resulted and the thinly populated area was devastated including the town of New Madrid, Missouri, the main settlement in the area. The shocks were felt over a larger area than any other U.S. earthquake.

The cause of the New Madrid earthquakes has remained largely a mystery until the last decade. Under the Earthquake Hazards Reduction Program, research on the New Madrid seismic zone was substantially expanded in the early 1970s. Important new results have been found that have established a geologic basis for evaluating seismic risk in the area. The studies in the New Madrid region have been multidisciplinary in scope, and have involved a combination of university, industry, and government personnel. Altogether, the New Madrid studies provide a good example of how the varied interests and capabilities of the earth sciences can be brought to bear on the problem ofseismic risk assessment. A brief review of the main results from the work illustrates the type of information that can be obtained toward this goal.

Tectonic Setting

Prior to about 1970 and until recent expansion of multidisciplinary earth science studies in the New Madrid region, the tectonic setting of the New Madrid seismic zone was poorly known. The zone is located in the northern end of the Mississippi Embayment, a south-plunging broad syncline filled with Cenozoic and Upper Cretaceous sediments, that lies between the Ozark Dome and the Nashville Dome. Both of the domes pre-date the formation of the syncline and during late Paleozoic time were connected by the Pascola Arch. Because of the northeast alignment of earthquakes from about Memphis, Tenn., to Vincennes, Ind., and the many northeast-trending faults in the Wabash Valley and Fluorspar district of southern Illinois and western Kentucky (fig. 1), Heyl and Brock (1961) defined the New Madrid fault zone as a major tectonic feature related to the earthquakes. The continuity of faults, however, between the Fluorspar District and the embayment is, today, uncertain even though much new data has been acquired.

Major changes in the concepts of the tectonic setting of the New Madrid seismic zone started being made with the application of plate tectonics to parts of the United States and elsewhere by Burke and Dewey (1972) and the defining of the Reelfoot Rift by Ervin and McGinnis (1975). Since 1975, much evidence has been acquired that indicates that the New Madrid seismic zone is associated with a late Precambrian-early Cambrian rift, parts of which have been reactivated several times since the Cambrian (fig. 1). The reactivation included faulting on a much smaller scale than during initial rift formation and intrusion of small bodies of igneous rocks, all or most of which appear to be alkalic and ranging from felsic to mafic compositions. Some of the recent specific information that supports the association of earthqukes with the rift and reactivation of parts of it is discussed in some detail in sections of this report that describe the results of recent seismicity, gravity, aeromagnetic, seismic reflection profiling, and geomorphic studies.

Seismicity

A major achievement toward a better understanding of the cause of New Madrid earthquakes has been the improvement in hypocenter determinations. This has come about through the installation of a moderately-dense seismograph network by St. Louis University (Stauder and others, 1976). The epicenter pattern for the New Madrid region, which was formerly diffuse, now shows clear lineations as a result of the greater accuracy of locations.

The main lineation in the epicenter pattern (fig. 2) strikes northeasterly for about 100 km from near Marked Tree, Ark., to near Caruthersville, Mo. A shorter lineation of epicenters trends north-northwest from near Dyersburg, Tenn., to near Lilbourne, Mo. Another lineation in the seismicity pattern extends northeast from near Lilbourn toward Charleston, Mo., where a strong earthquake occurred in 1895.

Fault-plane solutions (Herrmann and Canas, 1978; Herrmann, 1979) suggest that movement on the two northeast-striking seismic zones is predominantly right-lateral strike-slip. Fault movement on the north-northwest-striking zone apparently is in the reverse sense. Consideration of the seismicity pattern and the fault-plane solutions together suggests that the stresses causing deformation of the region are compressive and oriented approximately east-west. This direction is consistent with that determined elsewhere in the mid-continent region (Zoback and Zoback, 1980).

Gravity and Magnetics

The national gravity map compiled by Woollard (1958) shows generally positive values in the area of the Mississippi Embayment. This gravity high was interpreted by Ervin and McGinnis (1975) to indicate the presence of a broad, arched structure under the embayment with a pillow of material in the lower crust having a velocity of 7.4 km/s. Such a velocity is indicative of mafic intrusive bodies. Evidence for the existence of a layer with this velocity was found along the northwestern margin of the embayment in a seismic refraction study (McCamy and Meyer, 1966). Ervin and McGinnis concluded that the embayment is underlain by a rift that formed in the late Precambrian.

Further evidence for the rift-like nature of the embayment comes from magnetic data (Hildenbrand and others, 1977; Hildenbrand and others, 1979). The data reveal an 80-km wide, northeast-striking graben with a structural relief of 1.6 to 2.6 km; this graben strikes more easterly than the axis of the embayment. The seismicity zone from Marked Tree, Ark., to Caruthersville, Mo., is along the axis of the graben, and other seismicity lies mostly in the graben or near its boundaries. The stratigraphic continuity of Upper Cambrian and Ordovician rocks across the area of the graben suggests a pre-Late Cambrian origin for the graben (E. E. Glick, oral comm., 1979).

Seismic Reflection Profiling

To learn more about the fault systems associated with the graben and responsible for the modern seismicity, 280 km of multi-channel, common-depth-point seismic reflection profiles were run across the linear zones of seismicity and in other areas of suspected faulting (Zoback and others, 1980). Two profiles that cross the 100-km-long seismicity trend that strikes northeastward from Marked Tree (fig. 2) show strong reflections from beneath the erosional Paleozoic surface, and show strong shallower reflections that correlate with post-Paleozoic reflections on other profiles (fig. 3). A fault zone that has a vertical displacement of about 1 km and a strike of about N. 45° E. is interpreted from offset of the reflections below the Paleozoic surface. Reflections in the Upper Cretaceous and Tertiary sedimentary rocks show many smaller faults across the fault zone that have a cumulative vertical displacement of about 60 m over a distance of 9 km. The smaller faults have the same sense of movement as the deeper faulting; a strike-slip component cannot be precluded from the profile data and is suggested by fault-plane solutions of recent earthquakes to be the modern predominant component of movement. The profiles and stratigraphic studies reveal that major faulting took place after deposition of Upper Cambrian and younger rocks. A second stage of faulting, having less cumulative vertical displacement, took place in post-middle Eocene time and probably includes some Holocene movement. Two ofthe largest 1811-12 New Madrid earthquakes (Nuttli, 1973) are believed to have been associated with the northeasterly-trending seismic zone that is coincident with the fault zone.

Profiles run in northwestern Tennessee and southeastern Missouri show that the Dyersburg to Lilbourn seismic zone is characterized by numerous faults, highly fractured Paleozoic rock, and areas of localized subsurface uplift. Most of the faults trend northeast from Ridgely, Tenn., to Reelfoot Lake, Tenn., along the axis of the buried graben (fig. 2). The faults offset sediments at least as young as Eocene age; an increase of offset with age suggests recurrent movement since late Paleozoic time. The largest fault in this area (Cottonwood Grove) has a vertical displacement (reverse) of about 80 m. Localized uplifts appear to be associated with intrusive masses that appear to be laccoliths or sills emplaced as recently as Tertiary time.

Geomorphic Evidence of Quaternary Deformation

Geomorphologic studies have been conducted in the New Madrid region to determine the nature of Quaternary tectonism and to establish its relationship to modern seismicity. The studies have provided data on locations and types of faults, on earthquake recurrence, and on subtle warping of the earth's surface. They are especially useful in an area such as the Mississippi Embayment where deformed bedrock is buried under about 600 m of unconsolidated sediments and where Cenozoic structural relief is relatively small.

Though the total amount of Cenozoic structural relief is small, the rate of Holocene deformation, measured by geomorphic techniques, is relatively high. If the observed deformation continues for several tens of thousands of years at the rates determined for the Holocene, the resulting structural relief will be anomalously large for a cratonic area.

This section of the paper discusses evidence for Cenozoic faulting and tectonic upwarping by examining data obtained by exploratory trenching, morphometric and topographic analysis, air-photo analysis, and field mapping. In addition, river flood plains and terraces have been investigated for anomalous tilts, gradients, and morphologic patterns.

The known occurrences of faults in the northern Mississippi embayment, Kentucky-Illinois fluorspar district, and Ozark Mountains areas has been plotted on a regional seismotectonic map by Heyl and McKeown (1978) (fig. 1). Few of the faults on Heyl's and McKeown's map displace Tertiary strata and fewer still Quaternary strata. Most of the Cenozoic faults that have been mapped occur in western Kentucky and were identified during the U.S. Geological Survey's Kentucky mapping program (fig. 1). Investigations undertaken since the publication of Heyl's and McKeown's map in 1978 have resulted in the discovery of several additional Cenozoic faults. These faults are located on Crowleys Ridge in northeast Arkansas and southeast Missouri (D. P. Russ, 1980, unpub. data; R. A. Ward, 1980, personal comm., and Amos and Blankenship, 1980), near Cape Girardeau, Mo. (D. P. Russ, 1980, unpub. data), and in the Mississippi River valley near New Madrid (Zoback and others, 1980) and are shown in figure 4.

In order to determine if any of the faults in the area of modern seismicity moved during the Quaternary, they were investigated by air-photo analysis, field mapping, and, in some cases, exploratory trenching. The investigations concentrated on Reelfoot fault and Cottonwood Grove fault, both of which are dip-slip faults located in northwestern Tennessee (fig. 2 and fig. 4). These faults were initially identified on the seismic reflection profiles described above.

Reelfoot fault has a northerly strike and lies along the east flank of the Lake County uplift (fig. 5). The seismic reflection profiles show that the fault displaces the Paleozoic surface, located about 600 m beneath the ground surface, by about 50 m with the east side down (Zoback, 1979). Mapping of sediments in the walls of an exploratory trench situated on Reelfoot scarp along the east border of the Lake County uplift (fig. 5) revealed numerous faults, including a 1-m-wide zone of east-dipping normal faults located at the base of Reelfoot scarp (Russ and others, 1978, Russ, 1979). The normal faults displace the Holocene alluvium by more than 3 m and may be the surface expression of Reelfoot fault. Radiocarbon age dates and faulting and sand-blow relationships derived from the trench reveal that there have been at least three earthquakes in the area in the last 2,000 years that were strong enough to liquefy sediments and generate faulting. An average recurrence interval of 600 years can therefore be postulated for large earthquakes in the New Madrid region (Russ, 1979).

Cottonwood Grove fault trends to the northeast from near the town of Cottonwood Grove, Tennessee, to the southern shore of Reelfoot Lake (fig. 2 and fig. 4). As seen on the seismic reflection profiles, the fault displaces strata as young as middle Eocene age in a reverse sense. The fault can be traced on the profiles to within 150 m of the ground surface; here, the offset is 65 m on the profiles. In September, 1980, an exploratory trench was dug across the surface projection of the fault in the small town of Cottonwood Grove to determine whether or not the offset is present at the ground surface. Easily mappable strata of alluvium were evident in the trench walls providing good control for the detection of possible vertical faulting. The sediments, however, showed no evidence of faulting, indicating that either the Cottonwood Grove fault has not ruptured since the time of deposition of the sediments, that the projection of the fault to the surface is incorrect, or that possible recent displacement on the fault did not propagate to the surface. Absence of surface breakage along faults associated with earthquakes in the New Madrid region appears to be the normal case. There is, for instance, no evidence of surface rupture along the main seismic trend from near Marked Tree, Arkansas to Caruthersville, Missouri, which should have been the trend of rupturing had it occurred during the large earthquakes of 1811-12 (Nuttli, 1973; Zoback and others, 1980). The apparent lack of surface rupturing is enigmatic, but could be due to the depth of large earthquake hypocenters if they were more than about 20 km deep. Another reason for lack of surface rupture could be that fault slip was absorbed within the unconsolidated sediments of the Mississippi Embayment.

The Lake County uplift warps the Mississippi River flood plain by as much as 10 m in the area of greatest seismicity between Ridgely, Tennessee and New Madrid, Missouri, and is the most significant surficial structure yet to be identified in the region (fig. 5 and fig. 6). On the seismic reflection profiles, the Paleozoic surface and Upper Cretaceous and Tertiary strata are warped up about 50 m. Historical reports and faulting and sand-blow relationships indicate that the Lake County uplift formed primarily in association with earthquake activity. The uplift is an irregular, segmented structure that is subdivided into Tiptonville dome and Ridgely Ridge. Using a reconstructed-contour technique (D. P. Russ, unpub.), an isobase map of the uplift has been prepared (fig. 6). The map clearly shows the segmented nature of the deformation.

The Tiptonville dome is a north-trending bulge that has the greatest structural relief of the uplift. Reelfoot scarp, a monoclinal fold more than 180 m wide, marks the eastern border of the dome. The scarp also forms the western shore of Reelfoot Lake, a body of water that was enlarged and deepened by tectonic subsidence and sedimentary compaction during the 1811-12 New Madrid earthquakes. The remaining borders of Tiptonville dome are difficult to accurately delineate primarily because of modification by Mississippi River erosion and overbank deposition. Radiocarbon age dating and stratigraphic relationships indicate that most of Tiptonville dome formed within the last 2,000 years. Longitudinal topographic profiles constructed along natural levees and the lowland flood plain of the Mississippi River reveal that the northwestern part of the dome was uplifted about 2 m during the 1811-12 earthquakes (fig. 7). This area corresponds to one of the locations where waterfalls reportedly formed across the Mississippi River during the earthquake of February 7, 1812. In this area also, river boats that normally travelled on tributaries immediately west of the Mississippi River were no longer able to do so following the earthquake, presumably because uplift caused the tributary beds to shallow (Broadhead, 1902). A hydrographic profile of the Mississippi River surface constructed in 1962 and adjusted over a recent 20-year base (U.S. Army Corps of Engineers, 1976), is convex-upward suggesting that uplift may be currently occurring along the northwestern part of Tiptonville dome.

The Mississippi River flood plain has a relief of about 6 m (fig. 6 and fig. 7) on Ridgely Ridge 3 km south of Tiptonville dome. The ridge overlies a zone of faults that trends northeast from about Ridgely, Tennessee, to Reelfoot Lake. Geomorphic evidence suggests that the ridge is older than the Tiptonville dome, but is less than 6,000 years old (D. P. Russ, unpub. data). The relationship of Holocene tectonic movement of the ridge to seismicity is problematic. Whereas the surficial uplift and underlying faults detected on the reflection profiles trend to the northeast, recently compiled composite fault-plane solutions of micro-earthquakes suggest that northwest-trending faults are currently active (Nicholson and Singh, 1978; D. R. O'Connell and others, 1980, written communication).

Fault-plane solutions and modern seismicity trends suggest a possible origin of the Lake County uplift. The two linear northeast-trending seismic zones (fig. 1) have been interpreted to represent primarily right-lateral strike-slip faults (Herrmann, 1979; Herrmann and Canas, 1978). These two zones are oriented to one another in a left-stepping en echelon manner, and the Lake County uplift is situated between them in the area of intense seismicity that runs from near Ridgely to New Madrid (fig. 1). Displacement on the two fault zones would cause compression in the region between them, thereby producing vertical strain resulting in uplift (fig. 8). Several focal mechanisms in the area of uplift indicate reverse faulting and thus support this interpretation (Nicholson and Singh, 1978; D. R. O'Connell and others, 1980, written communication). A similarly created uplift has been reported by Clark (1972) and Sharp and Clark (1972) in the Ocotillo Badlands of southern California. Here, the uplift lies between left-stepping segments of the right lateral strike-slip Coyote Creek fault that ruptured in three places during the 1968 Borrego Mountain earthquake.

Conclusions

Despite the new information gained by the geophysical and geomorphological investigations, important questions remain on most aspects of New Madrid seismotectonics. It is not known, for example, whether the faults associated with the graben are genetically related to faults farther north in the Illinois-Kentucky fluorspar district and the Wabash Valley fault zone. The inactivity of large northwest-trending faults such as the Ste. Genevieve is enigmatic. Both northeast- and northwest-striking faults are conjugates to the modern east-west stress field and theoretically should have an equal opportunity to be active. The relationship of the earthquakes to the buried graben, the nearby Ozark Mountains, and the Pascola arch need to be better understood.

Little is known about the driving forces of the earthquakes. It appears, however, that the fault systems responsible for the seismicity of the New Madrid region originally formed under an extensional stress regime during an episode of intracontinental rifting that occurred during the late Precambrian or Cambrian. In contrast, in the area of greatest seismicity, modern seismicity and faulting that is younger than 50 million years is caused by compressive stress oriented east-west. Thus, the earthquakes are caused by sudden slip on fault zones of ancient origin that are oriented in the current stress field in such a way as to be particularly susceptible to movement.

Despite the uncertainties about the cause of New Madrid seismicity, there appears to be an adequate geologic basis for assessing seismic risk. It must be recognized though, that seismic risk assessment is a continuing process that should be sufficiently flexible to incorporate new information, yet at the same time sufficiently definitive to yield useful engineering design parameters.

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Figure Captions

Figure 1.--Seismotectonic map of the northern Mississippi Embayment. Modified after Heyl and McKeown, 1978.

Figure 2.--Map showing earthquake epicenters (dots), the locations of seismic reflection profiles (thick lines with numbers), principal faults inferred from the profiles (thin solid lines) and boundraries of rift (hachured lines).

Figure 3.--Part of seismic reflection profile D-1 and a corresponding line drawing. Profile crosses 100-km-long seismicity trend that runs northeastward from Marked Tree, Arkansas. Note that the depth scale is nonlinear.

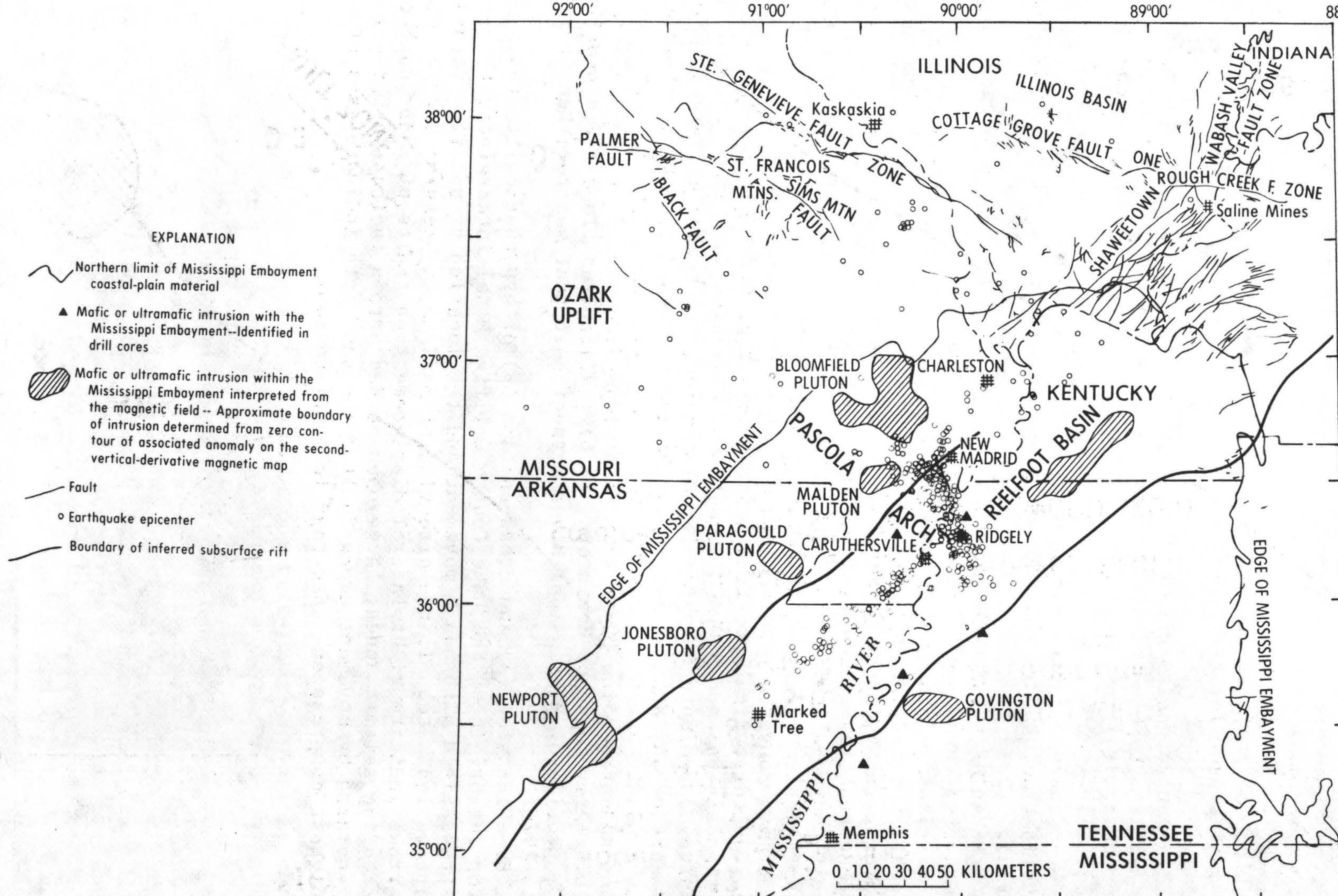
Figure 4.--Cenozoic faults identified in the Mississippi Embayment since the publication of Heyl and McKeown's seismotectonic map (Heyl and McKeown, 1978).

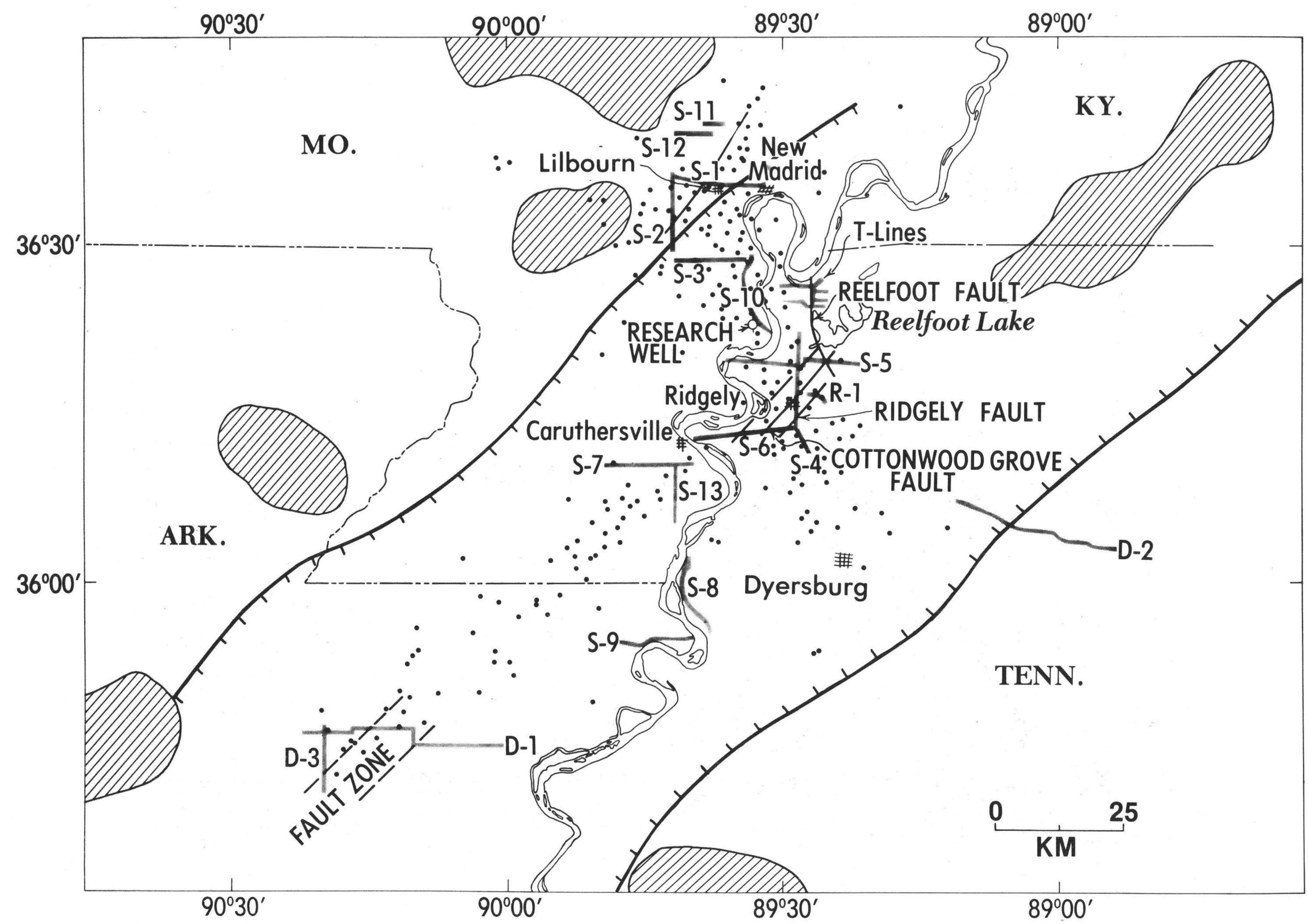
Figure 5.--Map of the New Madrid region showing location of geomorphic features, towns, and limit of the Lake County uplift.

Figure 6.--Isobase map showing amount and pattern of deformation of Lake County uplift. Bold lines indicate contours of equal uplift; solid where determined by direct measurement; dashed where calculated by reconstruction process. Thin lines indicate idealized preuplift meander-belt contours; solid where determined by direct measurement; long and short dashed where determined by reconstruction process. Values are in feet; to convert to meters divide values by 3.281.

Figure 7.--Longitudinal profiles along Mississippi River between channel mileposts 845 and 930. Mileposts and elevation data from U.S. Geological Survey 7 1/2-minute topographic quadrangles. Locations of islands shown on figure 6. A, natural-levee profile and projected elevations of adjacent Lake County uplift; low water reference plane from data collected in 1962 and adjusted using 20-yr average minimum discharge data from 1954 to 1975 (U.S. Army Corps of Engineers, 1976). B, lowland flood-plain profile. C, profile of Mississippi River meander belt that was reworked from 1820 to 1970.

Figure 8.--Plan view (A) and cross-section (B) of hypothetical tectonic model for the origin of the Lake County uplift. Model interrelates the modern stress field with fault movement and the location of uplift. Barbed arrows indicates 1, the direction of maximum compressive stress; single arrows indicate right-lateral displacement on faults of major northeast-trending seismic zones; large open-headed arrows indicate compression generated by movement along faults; and saw teeth indicate area of highest compression. See text for explanation.


EXPLANATION
Northern limit of Mississippi Embayment coastal-plain material
Mafic or ultramafic intrusion with the Mississippi Embayment--Identified in drill cores
Mafic or ultramafic intrusion within the Mississippi Embayment interpreted from the magnetic field -- Approximate boundary of intrusion determined from zero contour of associated anomaly on the second-vertical-derivative magnetic map
Fault
Earthquake epicenter
Boundary of inferred subsurface rift
92°00'
91°00'
90°00'
89°00'
88°00'
38°00'
37°00'
36°00'
35°00'
ILLINOIS
ILLINOIS BASIN
INDIANA
WABASH VALLEY FAULT ZONE
STE. GENEVIEVE FAULT ZONE
Kaskaskia
COTTAGE GROVE FAULT ZONE
ROUGH CREEK F. ZONE
SHAWEETOWN
Saline Mines
PALMER FAULT
ST. FRANCOIS MTNS.
SIMS MTN FAULT
BLACK FAULT
OZARK UPLIFT
BLOOMFIELD PLUTON
CHARLESTON
KENTUCKY
PASCOLA ARCH
NEW MADRID
REELFOOT BASIN
MISSOURI
ARKANSAS
MALDEN PLUTON
PARAGOULD PLUTON
CARUTHERSVILLE
RIDGELY
EDGE OF MISSISSIPPI EMBAYMENT
JONESBORO PLUTON
NEWPORT PLUTON
MISSISSIPPI RIVER
Marked Tree
COVINGTON PLUTON
Memphis
TENNESSEE
MISSISSIPPI
0 10 20 30 40 50 KILOMETERS



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36°30′
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S-10
S-5
R-1
S-6
S-4
S-7
S-13
S-8
S-9
D-1
D-2
D-3
Lilbourn
New Madrid
T-Lines
REELFOOT FAULT
Reelfoot Lake
RESEARCH WELL
Ridgely
RIDGELY FAULT
Caruthersville
COTTONWOOD GROVE FAULT
Dyersburg
FAULT ZONE
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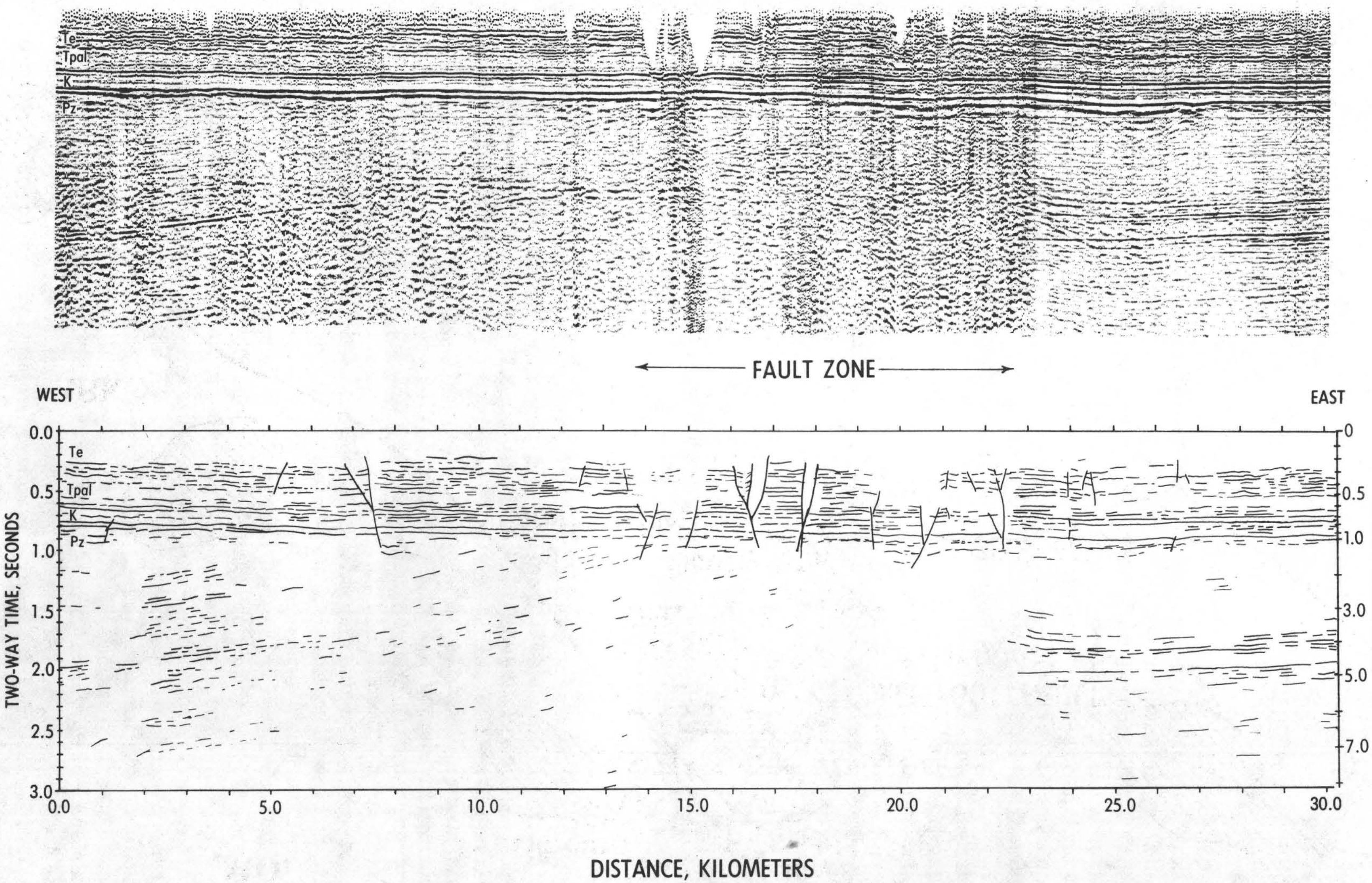
KM



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WEST
EAST
TWO-WAY TIME, SECONDS
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0.5
1.0
1.5
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APPROXIMATE DEPTH, KILOMETERS
0
0.5
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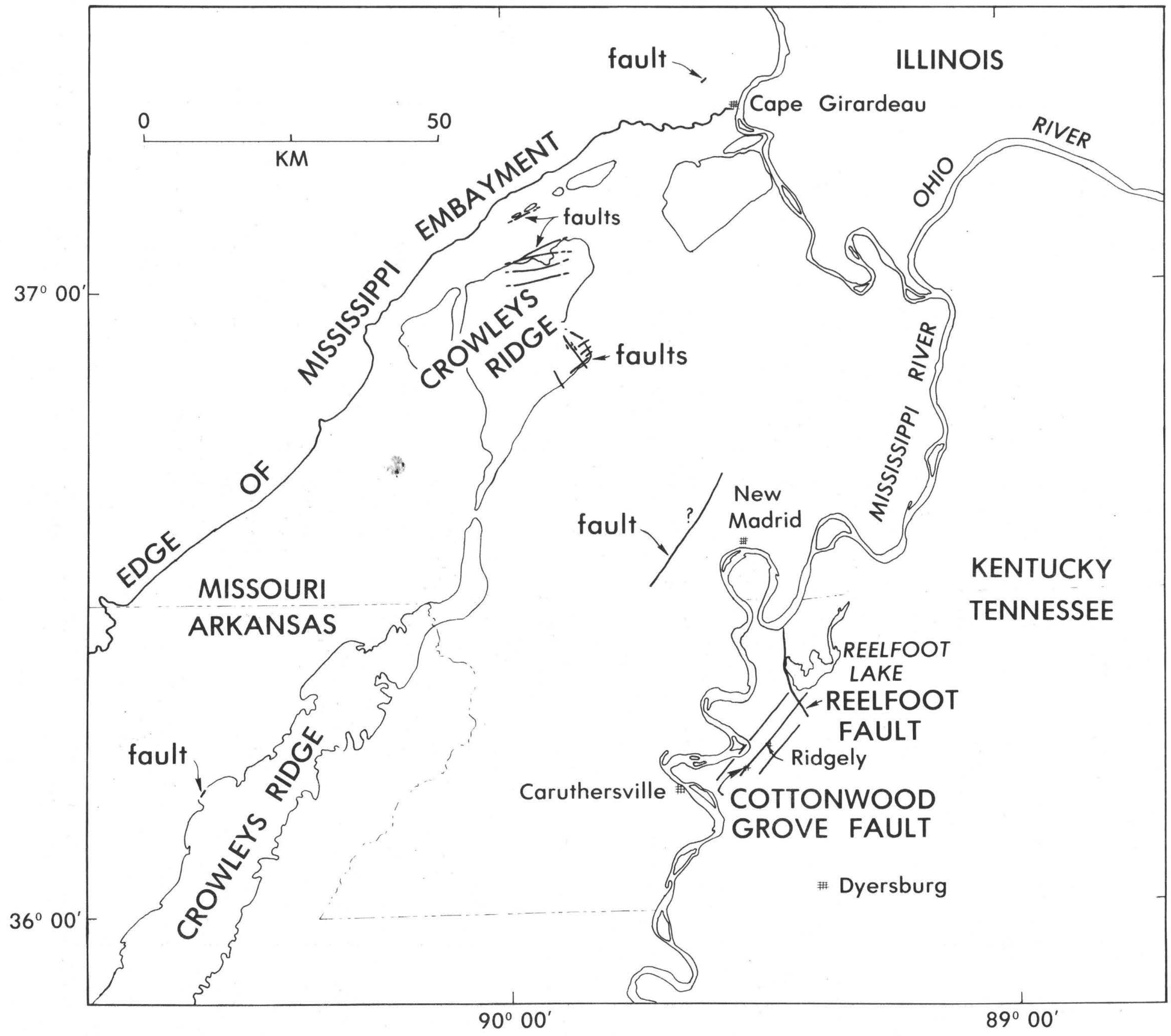

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CROWLEYS RIDGE
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New Madrid
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TENNESSEE
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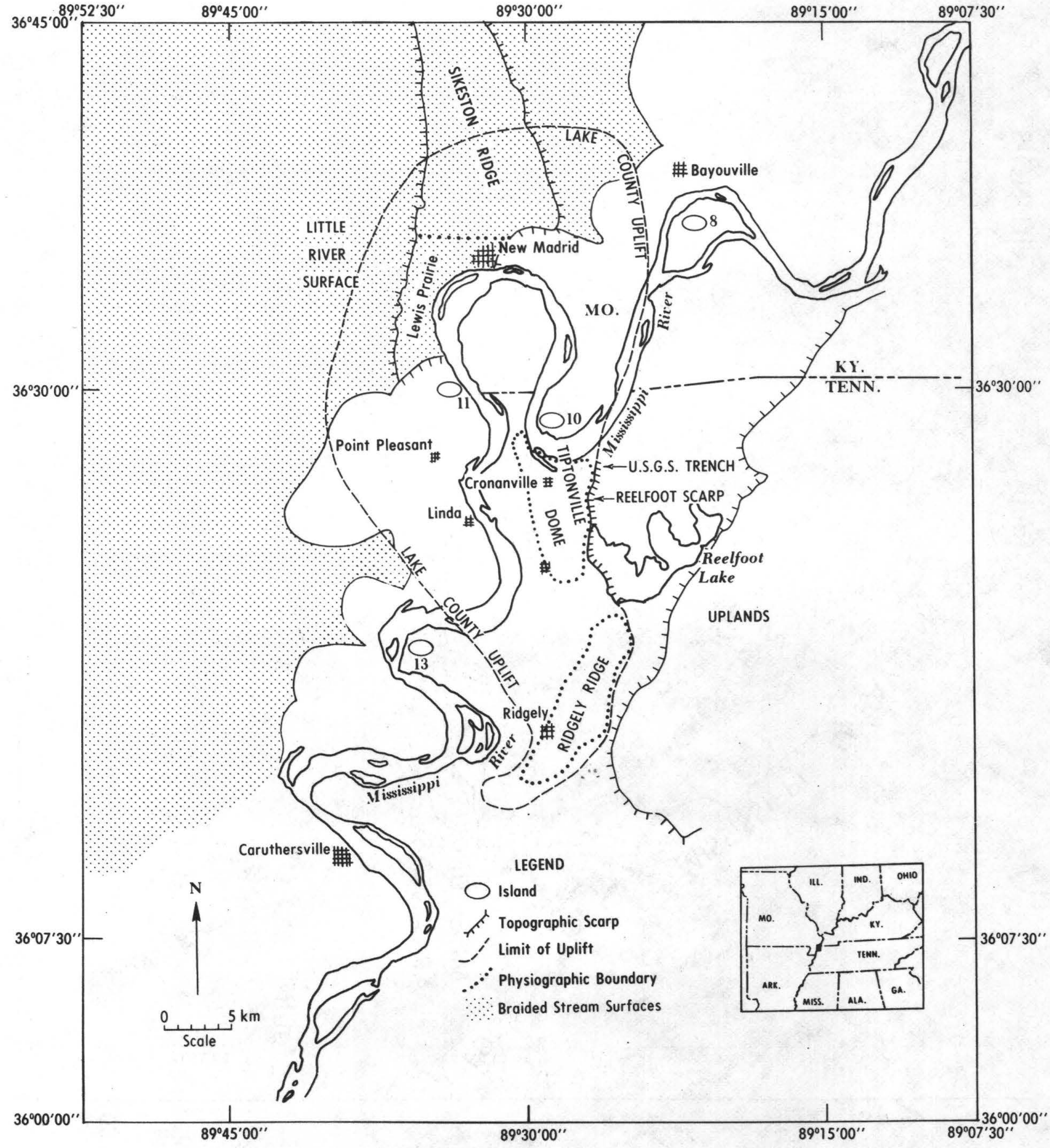
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RIDGE
LAKE
COUNTY
UPLIFT
Bayouville
8
LITTLE
RIVER
SURFACE
New Madrid
Lewis Prairie
MO.
River
KY.
TENN.
11
10
Mississippi
Point Pleasant
TIPTONVILLE
DOME
U.S.G.S. TRENCH
Cronanville
REELFOOT SCARP
Linda
Reelfoot
Lake
UPLANDS
13
RIDGELY RIDGE
Ridgely
River
Mississippi
Caruthersville
LEGEND
Island
Topographic Scarp
Limit of Uplift
Physiographic Boundary
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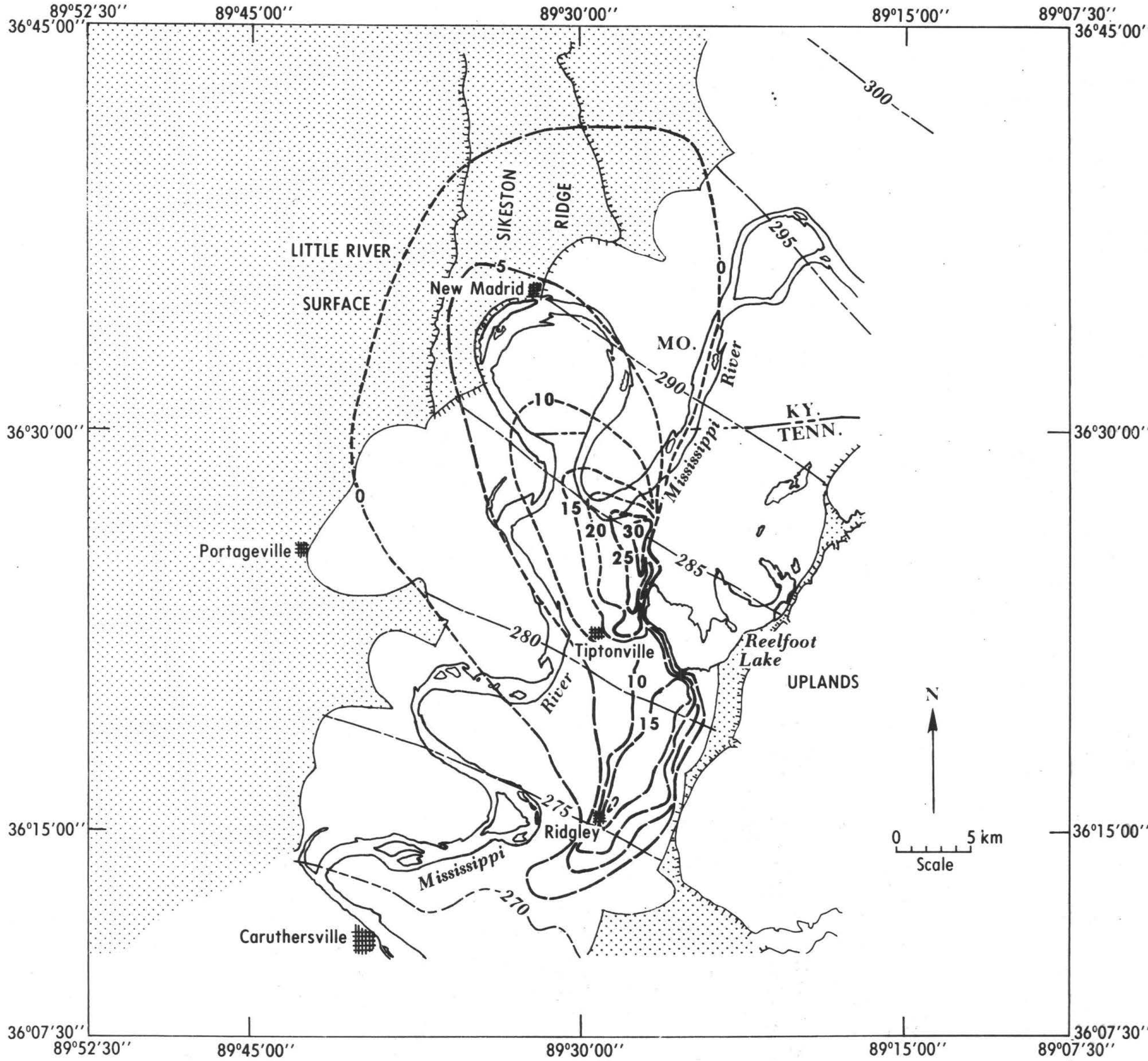

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SIKESTON
RIDGE
LITTLE RIVER
SURFACE
New Madrid
MO.
KY.
TENN.
Mississippi
River
Portageville
Tiptonville
Reelfoot
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UPLANDS
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5 km
Scale
300
295
290
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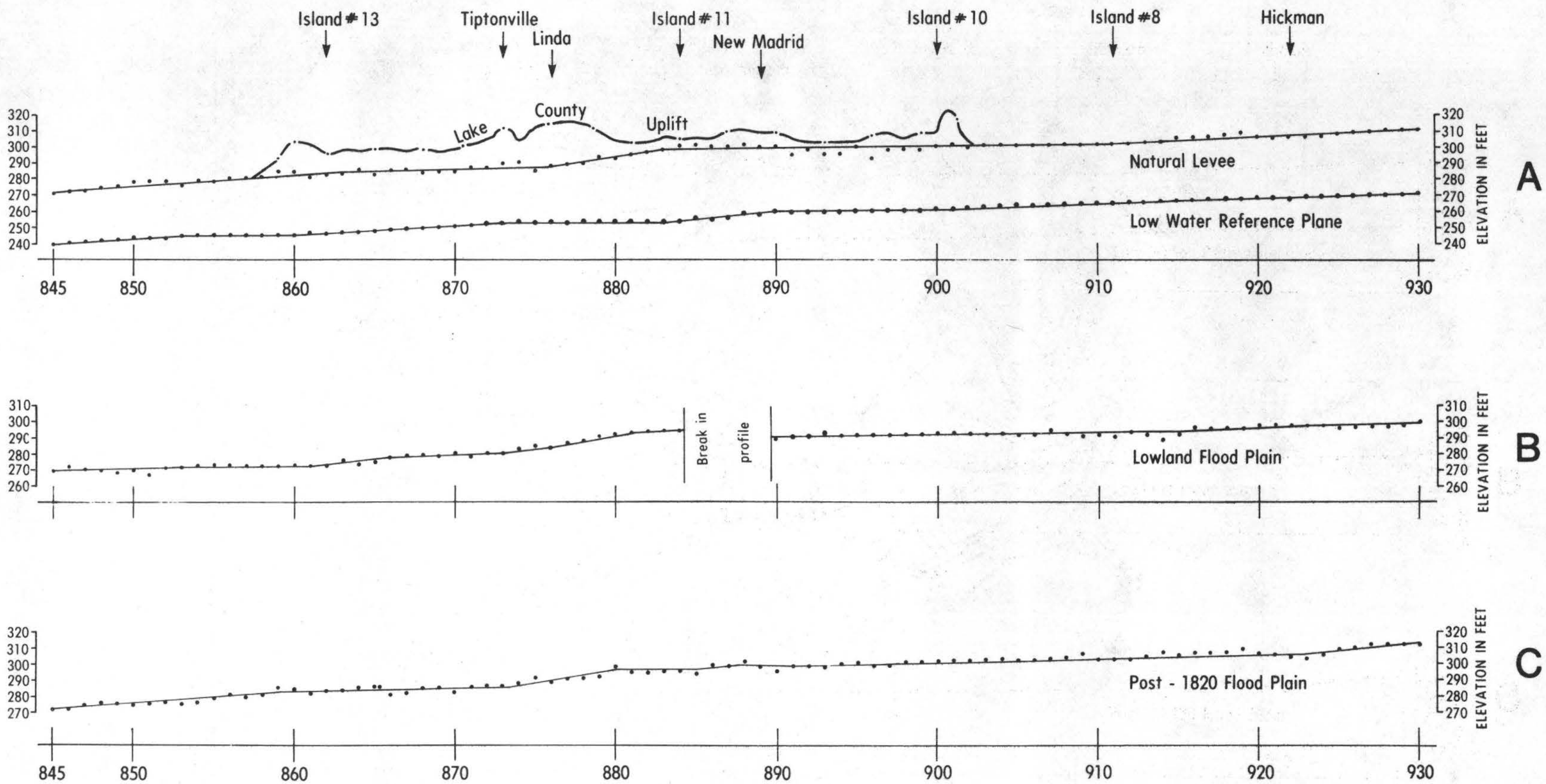

Island #13
Tiptonville
Linda
Island #11
New Madrid
Island #10
Island #8
Hickman
Lake County Uplift
Natural Levee
Low Water Reference Plane
A
Break in profile
Lowland Flood Plain
B
Post - 1820 Flood Plain
C
ELEVATION IN FEET
320
310
300
290
280
270
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250
240
845
850
860
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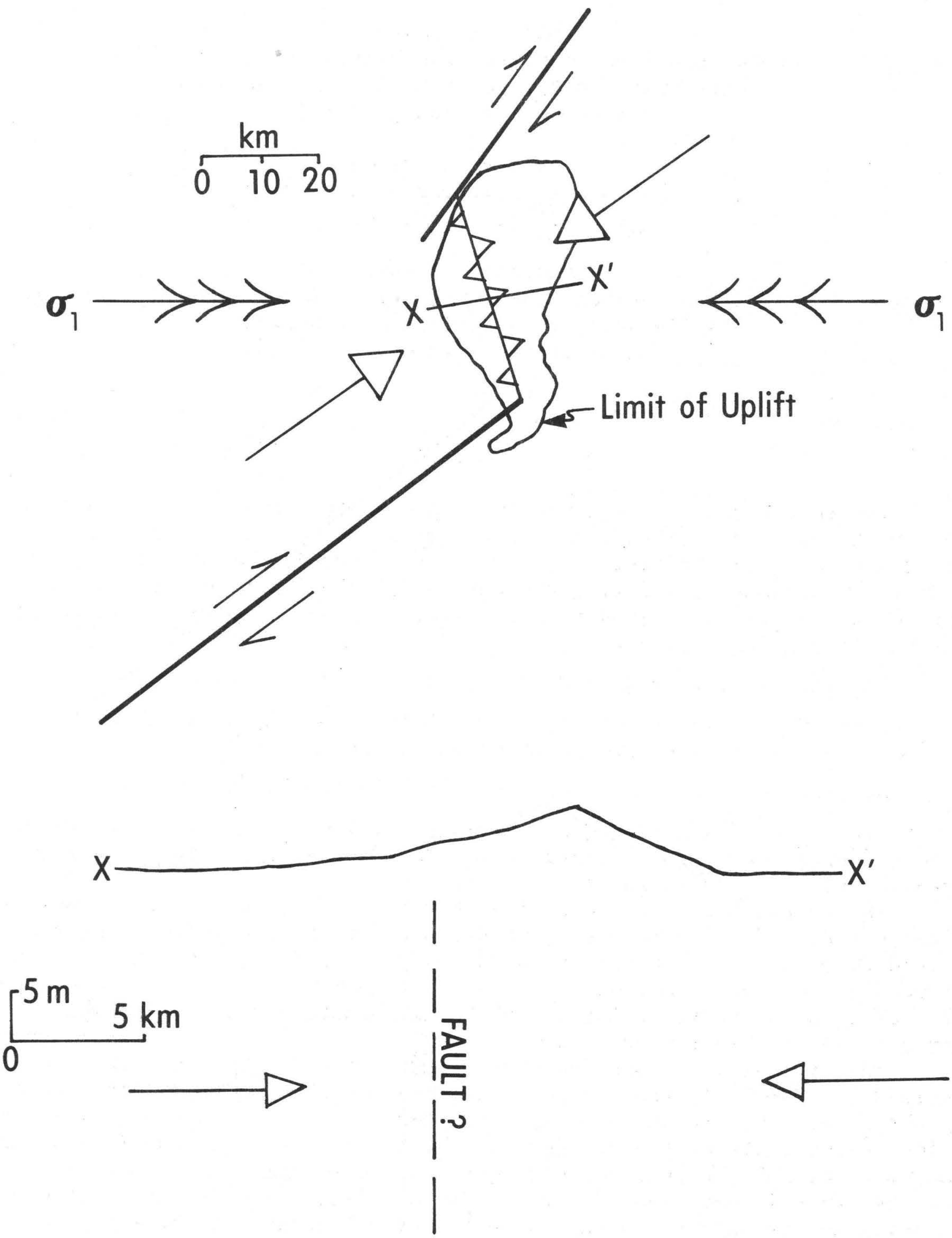
880
890
900
910
920
930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X'
Limit of Uplift
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5 m
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0
FAULT ?

Seismicity of New Mexico

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Abstract

The analysis of the seismicity of New Mexico is based primarily on (1) 53 shocks with maximum reported intensities of V or greater during the period 1849-1961 and (2) 224 earthquakes with local magnitudes of 1.5 or greater located instrumentally during the period 1962-1977. Magnitude-earthquake frequency relations derived from the instrumental data (97 shocks exceeding a threshold magnitude of 2.2) are:

$\log_{10}\Sigma N = 4.15 - 0.97\ M_L$ (Total Area) for the state,

$\log_{10}\Sigma N = 2.70 - 0.62\ M_L$ ($100{,}000\ km^2$) for the Colorado Plateau and High Plains provinces combined, and

$\log_{10}\Sigma N = 4.81 - 1.35\ M_L$ ($100{,}000\ km^2$) for the Rio Grande rift.

These relations indicate a generally low level of seismicity for New Mexico and different modes but comparable levels of activity in the stable and unstable tectonic provinces of the state.

The earthquakes for the period 1849-1961 are concentrated in the Rio Grande rift and a few of these are estimated to have magnitudes greater than would be predicted from the 1962-1977 earthquake data. The two known occurrences of Holocene faulting in the Rio Grande rift indicate periods within the past 10,000 years during which the earthquake activity was much higher than it is at the present time.

Collectively, all observations suggest that New Mexico's seismicity is episodic. Currently the level is low, and probably controlled by local geologic conditions within each physiographic province rather than by a statewide regional stress field. Some of the diverse geologic conditions that may account for the observed earthquake activity are (1) movement of magma in the crust in sections of the Rio Grande rift, (2) hydrocarbon recovery, primary and secondary, in the southeast High Plains, and (3) exceptionally steep gradients on the Precambrian surface along the margins of the San Juan Basin in the Colorado Plateau.

Introduction

Groups at New Mexico Institute of Mining and Technology (NMT), Los Alamos Scientific Laboratory (LASL) and U.S.G.S. Albuquerque Seismological Laboratory (ASL) have been engaged in instrumental studies of the seismicity of New Mexico for several years. A paper describing these studies and other data on the seismicity of New Mexico for the period 1849 through 1977 is in press (Sanford, Olsen, and Jaksha, in press). Presented in that paper are tabulations and maps of (1) felt shocks with maximum intensities of V or greater for the period 1849 through 1951 and (2) instrumentally located earthquakes with $M_L \geq 1.5$ for the period 1962 through 1977. Described are the procedures used for locating earthquakes and calculating magnitudes. Magnitude-earthquake frequency relations based on the instrumental data are presented for the state as a whole and for some of its physiographic provinces. The relations in conjunction with historical activity in New Mexico, the seismicity of southern California, and known Holocene faulting are used to draw conclusions about the short- and long-term seismicity of New Mexico. Local geologic conditions that may be responsible for earthquake activity observed in all physiographic provinces of New Mexico are discussed.

This report is an expanded summary of the paper described above. The major omissions are tabulations of the earthquake origin times, epicenters and strengths, and detailed descriptions of data analysis procedures.

Previous Studies

The existence of moderate seismicity in New Mexico was documented by a number of early investigators (Reid, 1911; Northrop, 1945 and 1947; and Richter, 1959). Early studies were based totally on reports of felt shocks, some fairly strong, dating from the latter half of the nineteenth century. Perhaps because of the relatively low population of New Mexico and an absence of strong shocks after the early part of this century, instrumental studies of seismic activity did not begin until 1960. A number of papers on instrumental studies have been published, those related most directly to the seismicity of the state being Sanford (1965), Sanford and Cash (1969), Toppozada and Sanford (1972), Sanford and others (1972), Northrop and Sanford (1972), Sanford and Toppozada (1974), Hoffman (1975), vonHake (1975), and Sanford, Olsen, and Jaksha (1979). Northrop (1976) published a paper on New Mexico's seismicity which emphasized a large amount of non-instrumental data he had accumulated on the state's earthquakes since the early 1930's.

Earthquake Data

1849-1961

Information on the location and strengths of earthquakes in New Mexico prior to 1962 is based almost entirely on reports of "felt" earthquakes. Although settlement by the Spanish began in the early 17th century, little is known of seismic activity in the state prior to its becoming part of the U.S. in 1848. No doubt reports of earthquakes exist in Spanish and Mexican archives, but such information is difficult to extract and to our knowledge has not been attempted.

The earliest report of earthquakes after U.S. occupation is the description of a swarm of shocks in the Rio Grande rift at Socorro by a U.S. Army surgeon (Hammond, 1966). The swarm, which contained 22 felt shocks, commenced on December 11, 1849 and lasted until February 8, 1950. No shock in this swarm was reported felt at distances greater than 25 km which indicates maximum intensities were probably less than or equal to IV (Modified Mercalli). Similar sequences of shocks located away from population centers along the Rio Grande valley or elsewhere in the state could easily have gone unreported before the start of instrumental studies.

For the period 1849-1961, Northrop (1961, 1976) cites evidence, primarily from old newspaper files, for over 600 felt earthquakes in New Mexico. About 95 percent of these shocks occurred along a 150-km section of the Rio Grande rift from Albuquerque to Socorro; the majority in the 75 km from Belen to Socorro. Shown in figure 1 is the location of the 53 felt earthquakes whose maximum reported intensities were V (MM) or greater. The primary source of data for this figure is Coffman and vonHake (1973). Note that for this second data set, a larger fraction, about 25 percent, of the earthquake activity is outside the Rio Grande rift.

The distribution of seismic activity shown in figure 1 could be influenced considerably by population density. For most of the 1849 through 1961 period, New Mexico's population was concentrated within the Rio Grande rift system. Earthquakes with a maximum intensity of V (MM) could have gone unreported in all areas of the state including low population segments of the rift.

The three strongest earthquakes in the 1849 through 1961 period occurred near Socorro in 1906. Because these shocks were felt to distances of 200 to 300 km, it is unlikely that they could have gone unreported had they occurred anywhere in the state.

A characteristic of the strong 1906 Socorro shocks as well as many other known earthquakes in the rift from Albuquerque to Socorro is that they are associated with earthquake swarms. Listed in table 1 are parameters for known earthquake swarms in the Rio Grande rift during the period 1849 through 1961. By far the strongest and longest earthquake swarm was the 1906-07 swarm at Socorro which appears comparable to the Matsushiro swarm which some believe may have been caused by magmatic intrusion at shallow depth (Stuart and Johnston, 1975). Although the evidence is not absolutely conclusive, the distribution of isoseismals for the 1906-07 swarm (Reid, 1911) suggests hypocenters beneath the Socorro Mountain horst block, a relatively young north-south structural feature in the central part of the Rio Grande rift (Chapin and Seager, 1975). The December 1935 swarm centered near Belen also appears to have originated near the axis of the rift. At Los Lunas, 18 km north of Belen, the shocks of the 1935 swarm were much weaker than at Belen, an unlikely observation if the epicenters were on the margins of the rift which are located about 30 km to the east and west of these two communities. Recent instrumental studies (Sanford, Olsen, and Jaksha, 1979) between Albuquerque and Socorro show considerable seismic activity within the rift but little associated with the well-defined boundary faults.

As has been noted for many years, earthquake swarms are observed in the vicinity of active volcanoes and in regions that have had volcanic activity in geologically recent times (Richter, 1958). Late Pliocene and Quaternary basalt flows, from north of Albuquerque to south of Socorro, are generally confined to the central part of the rift (Kelley and Kudo, 1978; Bachman and Mehnert, 1978). This observation in conjunction with the location of earthquake swarms may indicate that magma is continuing to be injected into the central part of the rift. A number of geophysical studies in the Socorro area support this hypothesis (Sanford, Alptekin, and Toppozada, 1973; Reilinger and Oliver, 1976; Sanford and others, 1977; Chapin and others,

TABLE 1 - CHARACTERISTICS OF EARTHQUAKE SWARMS IN THE RIO GRANDE RIFT, 1849 THROUGH 1961.*

Date	Duration in weeks	Location of nearest pupulation center	Number of reported shocks	Maximum intensity (M.M.) of strongest earthquake(s)	Reference	Remarks
Dec 11, 1849 to Feb 8, 1950	8	Socorro	22	IV	Hammond (1966)	Extent of felt region suggests a location beneath Socorro Mountain, an intragraben horst block
Sep, 1893	12	Los Lunas	Daily?	VII	Woollard (1968, Coffman and vonHake (1973), Northrop (1976)	One listing (Woollard, 1968), indicates daily shocks at Sabinal (35 km south of Los Lunas) with maximum intensities > V for 3 months
Jan 19, 1904 to Mar 8, 1904	8	Socorro	34	V	Bagg (1904), Woollard (1968)	Newspaper accounts indicate that shocks on Sept. 10, 1904 at Socorro were not a continuation of this swarm
Jul 2, 1906 to Jan, 1907	28	Socorro	Daily	VIII(2)	Reid (1911)	Distribution of isoseismals suggests hypocenters beneath Socorro Mountain, an intragraben horst block
Dec 12, 1935 to Dec 30, 1935	3	Belen	>24	V-VI	Neumann (1937), Coffman and vonHake (1973)	At Los Lunas (18 km north of Belen) shocks were much weaker than at Belen. This suggests epicenters near the central part of the rift rather than the margins

*From Sanford, Olsen, and Jaksha, in press.

1978; Rinehart, Sanford and Ward, 1979; Brown and others, 1979; Brown and others, 1980; Reilinger and others, 1980).

1962-1977

Beginning in 1962 the number of seismograph stations in New Mexico, Arizona and west Texas became adequate to permit location of a relatively large number of earthquakes throughout New Mexico and bordering areas. For the period 1962 through 1972, 211 earthquakes were located by New Mexico Institute of Mining and Technology (NMT). About 30 percent of these shocks were also located by the National Earthquake Information Service (U.S. Geological Survey) and the governmental agencies preceding it (U.S. Coast and Geodetic Survey and National Oceanic and Atmospheric Administration).

In September 1973, the number of located shocks in the northern half of New Mexico jumped when Los Alamos Scientific Laboratory (LASL) installed an array of continuously recording stations. In 1976, an increase in the number of shocks located in central New Mexico occurred when the USGS-Albuquerque Seismological Laboratory (ASL) installed a permanent array of stations in and around the Albuquerque-Belen basin.

Merging of data obtained by the three organizations required adjustments in magnitudes inasmuch as the procedure used by LASL and ASL to estimate magnitude differed from that used by NMT. All magnitudes adopted were obtained by the NMT procedure or a correction of -0.5 was applied to LASL and ASL magnitudes; the latter was based on an average difference observed between LASL/ASL and NMT magnitudes. The NMT procedure for calculating magnitude consisted of (1) calculation of a Richter local magnitude, (2) correction for more efficient propagation of seismic waves in New Mexico than California ($-0.0014 \cdot \Delta$(km)) and (3) normalization of magnitudes to the station at Albuquerque. The distance correction is consistent with recent measurements of crustal Q which indicate a mean value for New Mexico about twice as great as California (Singh and Herrmann, 1979).

Shown in figure 2 are the epicenters for 224 shocks, with local magnitudes greater than or equal to 1.5, that were located instrumentally during the period 1962 through 1977. In figure 2, weak earthquakes appear more frequently in the central part of the state north of 33.5°N than elsewhere. This is primarily a consequence of the geographic distribution of stations which were most numerous in north-central New Mexico during the study period. The effects of this station bias can be removed by eliminating shocks whose magnitudes are less than a threshold value of 2.2. The threshold magnitude is defined such that the event count is essentially complete for the state above this value. Removal of shocks with magnitudes less than 2.2 produces a far more uniform distribution of activity throughout the state than appears to be the case in figure 2.

About 95 percent of the epicenters in figure 2 are believed to be within 20 km of the true locations. This precision is not adequate to associate earthquakes with specific known faults. Another reason for being careful about assigning earthquakes to specific faults is the area of fault surface associated with the majority of earthquakes shown in figure 2. Ninety-eight percent of the shocks have local magnitudes of less than 3.5. An earthquake of magnitude 3.5 can be generated by displacement on fault surfaces ranging in area from 0.05 to 3.0 km^2 (Thatcher and Hanks, 1973). Thus many of the New Mexico earthquakes could have occurred on minor and unknown faults.

For lack of close stations, little is known about the depths of focus for the shocks shown in figure 2. In the Socorro area of the Rio Grande rift, detailed studies of microearthquakes (nearly all with $M_L < 1.5$) indicate no seismic activity below a depth of 13 km (Sanford, Olsen and Jaksha, 1979). Similar studies of very small earthquakes by ASL and LASL in the Rio Grande rift north of Socorro indicate most activity is occurring in the upper crust at depths of less than 20 km. Detailed microearthquake surveys have not been made in other physiographic provinces within the state.

Earthquake Statistics 1962-1977

Magnitude-Earthquake Frequency

The distribution of numbers of earthquakes relative to magnitude can be quantified by using the linear relation

$$\log_{10}\Sigma N = a - bM_L \quad , \qquad (1)$$

where $\log_{10}\Sigma N$ = logarithm of the cumulative number of detected shocks exceeding M_L,

M_L = local magnitude,

and a,b = constants which depend on the observed seismicity.

Richter (1958) and others have established the validity of this linear relationship for many seismic areas in the world. The only constraint is that the linear fit be based on the observed earthquake data that falls above the established threshold magnitude. It is probable that the relation becomes non-linear at magnitudes approaching the strongest earthquake that a given region can sustain. However, the instrumental data on New Mexico earthquakes is for such a short period that no events anywhere near the largest possible earthquakes for this area are included in the data set.

On the basis of data gathered from 1962 through 1977, the relation between cumulative number of earthquakes (ΣN) and magnitude for New Mexico earthquakes is

$$\log_{10}\Sigma N = 4.15\ (\pm 0.06 \text{ s.d.}) - 0.97\ (\pm 0.02 \text{ s.d.})\ M_L \qquad (2)$$

As indicated by the small values of the standard deviations (s.d.), the linear fit, which is based on 97 shocks with $M_L \geq 2.2$, is good. Even for the high magnitudes, equation (2) matches observations closely, e.g. the largest quake for the 16-year period according to equation (2) should have had an M_L equal to 4.28, whereas the strongest observed quake had a calculated magnitude of 4.29 (January 23, 1966; 20:10:59; near Dulce, N.M.).

Equation (2) indicates a relatively low level of seismicity for New Mexico which can be demonstrated rather dramatically by comparing the magnitude-earthquake frequency relation for the state with one for southern California. The latter relation is based on 29 years of data over an area of 296,100 km^2 in the southern part of California (Allen and others, 1965). For comparative purposes, the relations given below for New Mexico (NM) and southern California (SC) have been normalized to 25 years and 100,000 km^2:

$$\log_{10}\Sigma N = 3.84 - 0.97\ M_L \quad \text{(NM)}, \qquad (3)$$

$$\log_{10}\Sigma N = 6.15 - 0.86\ M_L \quad \text{(SC)}. \qquad (4)$$

The difference in seismicity indicated by these equations is very large. For example, the largest quake in a 25-year period is 7.2 for SC and 4.0 for NM. The number of shocks exceeding magnitude 4.0 in SC during the 25-year period is 512.

If we assume the 1962-1977 level of seismicity is representative of earthquake activity for the past 100 years, the relation between cumulative number of shocks and magnitude is

$$\log_{10}\Sigma N = 4.95 - 0.97\ M_L \qquad \text{(100 years).} \qquad (5)$$

Equation (5) indicates that the strongest earthquake in the past 100 years should have had a local magnitude of 5.1. The relationship can be tested by estimating the strength of the strongest earthquake in the state since the late 1800's.

The strongest earthquake, which occurred near Socorro, November 15, 1906, was felt over an area of 245,000 km^2. Several investigators have developed empirical relations between the area of perceptibility and magnitude for different physiographic provinces (Slemmons, Jones and Gimlett, 1965; Wiegel, 1970; Toppozada, 1975). The relations for the Rocky Mountain or Basin and Range provinces appear to be most applicable for the Socorro earthquake and they yield magnitudes of 4.9 and 6.5, respectively. Crustal Q values measured by Singh and Herrmann (1979) suggest that the true relation between the area of perceptibility and magnitude for New Mexico earthquakes will lie between those for the Rocky Mountain and Basin and Range provinces. Thus a reasonable estimate for the magnitude of the 1906 shock could be 5.7, a value substantially greater than that predicted by equation (5). Furthermore, the magnitudes of two other shocks in the 1906 Socorro swarm are likely to have exceeded magnitude 5.1 (Sanford, Olsen and Jaksha, 1979, table 1).

From the available data, it appears that the intensity and distribution of seismic activity in the 1962-1977 period was different than that for the previous 123 year period. For the 1962-1977 period, earthquakes occurred in all physiographic provinces of New Mexico at about the same level of intensity (figure 2), whereas for the 1849-1961 period, about 75 percent of the shocks seem to have occurred in the Rio Grande rift (figure 1). As shown above, the magnitudes of strong shocks in the earlier part of this century indicate a higher level of seismicity for that period than would be estimated from an extrapolation of the observations made during the 1962-1977 period. An obvious explanation for the differences is that there has been a real temporal change in the intensity and distribution of seismic activity in New Mexico. An alternate, but less likely, possibility is that magnitudes of the strong shocks in 1906 are being overestimated and that many moderately strong shocks in the Colorado Plateau and High Plains went unreported in the century preceding instrumental studies.

Temporal Variations

A major temporal change in the seismic activity of New Mexico occurred during the 16 year observational period. Plotted in figure 3 are the cumulative percent of earthquakes versus time; the data used are for the 97 shocks whose local magnitudes exceeded or equalled the threshold value of 2.2. For the period 1962 through 1970, the number of shocks averaged about 4 each year, whereas for the period 1971 through 1977, the average was about 9 shocks each year.

The change in rate of activity ($M_L \geq 2.2$) was accompanied by a change in _b_ values, from 1.10 for the period 1962-1970 to 0.88 for the 1971 through 1977 period (see upper part of figure 3). The small data sets (47 and 50) for these two periods, as well as other periods in figure 3, prohibit positive conclusions on the temporal behavior of _b_ other than a probable decrease in value with time.

Spatial Variations

As mentioned earlier, the earthquake activity in the High Plains and Colorado Plateau provinces for the 1962-1977 period was comparable to that occurring in the Rio Grande rift. However, the data suggest a difference in the manner in which the activity is occurring in these provinces.

The relation between cumulative number of shocks and magnitude for the Colorado Plateau and High Plains combined, but exclusive of shocks along the Jemez Lineament, is

$$\log_{10}\Sigma N = 2.70 - 0.62\ (\pm 0.025\ \text{s.d.})\ M_L \qquad (n=28)(\text{CP-HP}), \qquad (6)$$

whereas for the Rio Grande rift the relation is

$$\log_{10}\Sigma N = 4.81 - 1.35\ (\pm 0.07\ \text{s.d.})\ M_L \qquad (n=50)(\text{RGR}). \qquad (7)$$

For comparative purposes, both magnitude-earthquake frequency relations have been normalized to areas of 100,000 km^2. Equations (6) and (7) indicate that earthquakes are more numerous in the Rio Grande rift but that stronger earthquakes are more prevalent in the High Plains-Colorado Plateau. For example, the cumulative number of shocks with magnitude greater than or equal to zero is 64,600 for the RGR but only 500 for the CP-HP over the time period being considered. On the other hand, the strongest quake for the RGR, local magnitude equal to 3.6, falls short of the strongest quake for the CP-HP by one magnitude unit.

The data sets, upon which equations (6) and (7) are based, might be too small to be certain of a real difference between these areas. However, there can be no question that the level of seismicity for the stable tectonic provinces, the Colorado Plateau and the High Plains, is comparable to that in the Rio Grande rift at the present time.

The latter observation coupled with the generally low level of seismic activity throughout the state suggests the absence at this time of a regional extensional stress field throughout the entire state. If such a stress field presently existed, the province most affected would probably be the Rio Grande rift because geologic and geophysical evidence indicates it is the major crustal flaw in the region (Chapin and Seager, 1975).

The absence of a regional extensional stress field might not be a long-term condition. Extrapolation of the magnitude-earthquake frequency relation for the RGR (equation (7)) to 10,000 years gives

$$\log_{10}\Sigma N = 7.61 - 1.35\ M_L \qquad (8)$$

This equation indicates that on the basis of the 1962-1977 seismicity, the magnitude of the strongest earthquake in a 10,000 year period in the Rio Grande is only 5.6.

Machette (1980) has found evidence for major offsets of Holocene deposits at two locations within the Rio Grande rift, along the La Jencia fault on the eastern margin of the Magdalena Mountains (~20 km west of Socorro) and along the Cox Ranch fault on the eastern margin of the Organ Mountains (~65 km north of El Paso). Other faults along the Rio Grande rift have reported but as yet undocumented Holocene movements. Formation of identifiable fault scarps requires major earthquakes, on the order of magnitude 7 or greater. It is apparent from this geologic record that there have been periods within the past 10,000 years when the earthquake activity was much higher than it is at the present time. Episodic seismicity has been observed in many areas of the world on time scales ranging from tens to thousands of years (Richter, 1958).

Relation of Seismicity to Geology

As discussed in the previous section, the evidence available suggests that the low level of seismic activity observed in all physiographic provinces of New Mexico is controlled by local geologic conditions rather than a state-wide regional stress field. The local geologic conditions that may be responsible for earthquake activity in the three major physiographic provinces of the state are quite diverse. Within the Rio Grande rift concentrations of earthquake activity at Socorro and north of Los Alamos may be related to movement of magma in the crust. In the Socorro area, the epicenters are roughly centered on a 1700 km^2 mid-crustal magma body that has been detected using S-phase reflections on microearthquake seismograms (Sanford, Alptekin, and Toppozada, 1973; Sanford and others, 1977; Rinehart, Sanford, and Ward, 1979) and P-phase reflections in deep crustal profiling (Brown and others, 1979; Brown and others, 1980). An analysis of first-order level-line surveys in the Socorro area (Reilinger and Oliver, 1976; Reilinger and others, 1980) has revealed surface uplift that is spatially coincident with the extensive mid-crustal magma body. In the Los Alamos area, the activity occurs in a region of crustal subsidence, also discovered through the analysis of level-line data (Reilinger and York, 1979). These observations suggest that earthquakes in the Socorro area are the result of inflation of a magma body whereas those north of Los Alamos are caused by deflation of one (Sanford, Olsen, and Jaksha, 1979).

On the High Plains of New Mexico, earthquakes occur most frequently in the southeast corner of the state and near the eastern border approximately 200 km south of the Colorado border. Epicenters in southeastern New Mexico are on the western edge of a large region of seismic activity that extends southward and eastward into Texas (Sanford and Toppozada, 1974; Rogers and Malkiel, 1979; Sanford and others, in press). Earthquakes throughout this region could be induced by hydrocarbon recovery practices although conclusive proof is lacking. In the north-central High Plains of New Mexico, the earthquakes occur within and on the flanks of the Tucumcari Basin and along the transition between the Sierra Grande Arch and the Amarillo Uplift. Earthquakes have occurred along the latter structure through west Texas and into southern Oklahoma (Shurbet, 1969).

On the Colorado Plateau, the activity along the eastern and northwestern borders of the San Juan Basin takes place in regions where gradients on the Precambrian surface are known to be very high. The activity along the southwestern margin of the San Juan Basin may be occurring along buried faults whose existence is suggested by stratigraphic changes in the transition zone between the Zuni Uplift and the basin (C. Smith, personal communication). The line of earthquake epicenters which crosses the southern part of the Colorado Plateau and extends northeastward across the Rio Grande rift and into the northern High Plains of New Mexico appears to be associated with the Jemez Lineament, a major crustal flaw defined by Pliocene and Pleistocene volcanic centers.

Summary and Conclusions

The seismicity of New Mexico during the 1962-1977 period was unexpectedly low. The magnitude-earthquake frequency relation based on the observations during the 16-year period indicates that the present level of seismicity is below that observed in the previous 100-year period and well below that anticipated from known occurrences of Holocene faulting in the Rio Grande rift.

Therefore, an increase in earthquake activity some time in the future, particularly in the Rio Grande rift, is a reasonable expectation.

There is no correlation between the distribution of seismic activity in 1962-1977 and geologic evidence of recent tectonic movement. Nearly all known (Quaternary faulting is located within the rift system (Seager and Morgan, 1979). The basins with the thickest accumulations of Tertiary and Quaternary sediments are within the rift system as well. On the other hand, seismic activity is presently as intense on the High Plains and Colorado Plateau as it is in the Rio Grande rift. This observation in conjunction with the overall low level of seismicity suggests that a state-wide regional stress field does not exist at this time. The earthquake activity presently observed is a general background seismicity generated by local stress conditions within each physiographic province.

Acknowledgments. Much of the information presented in this paper could not have been developed without collaboration with other institutions. We are particularly indebted to D. H. Shurbet, Texas Tech University and David Dumas, University of Texas - Austin for readings from their seismic stations.

At New Mexico Institute of Mining and Technology, a number of graduate and undergraduate students have made contributions to the instrumental studies of earthquakes. Recent work by assistants Terry Wallace, Scott Sandford, Joel Sheldon, Tim Wallace and Roger Ward was particularly helpful in preparing this report.

At Los Alamos Scientific Laboratory we would like to acknowledge the contributions and advice of Dan Cash, John Stewart, Norma McFarland, Joyce Wolff and Deborah Wechsler in the compilation of LASL epicenters. Fred Homuth and Ted Handel were responsible for much of the design and installation of the LASL regional network.

The authors are pleased to acknowledge the contribution of Jerry Locke (USGS - Retired) to this study.

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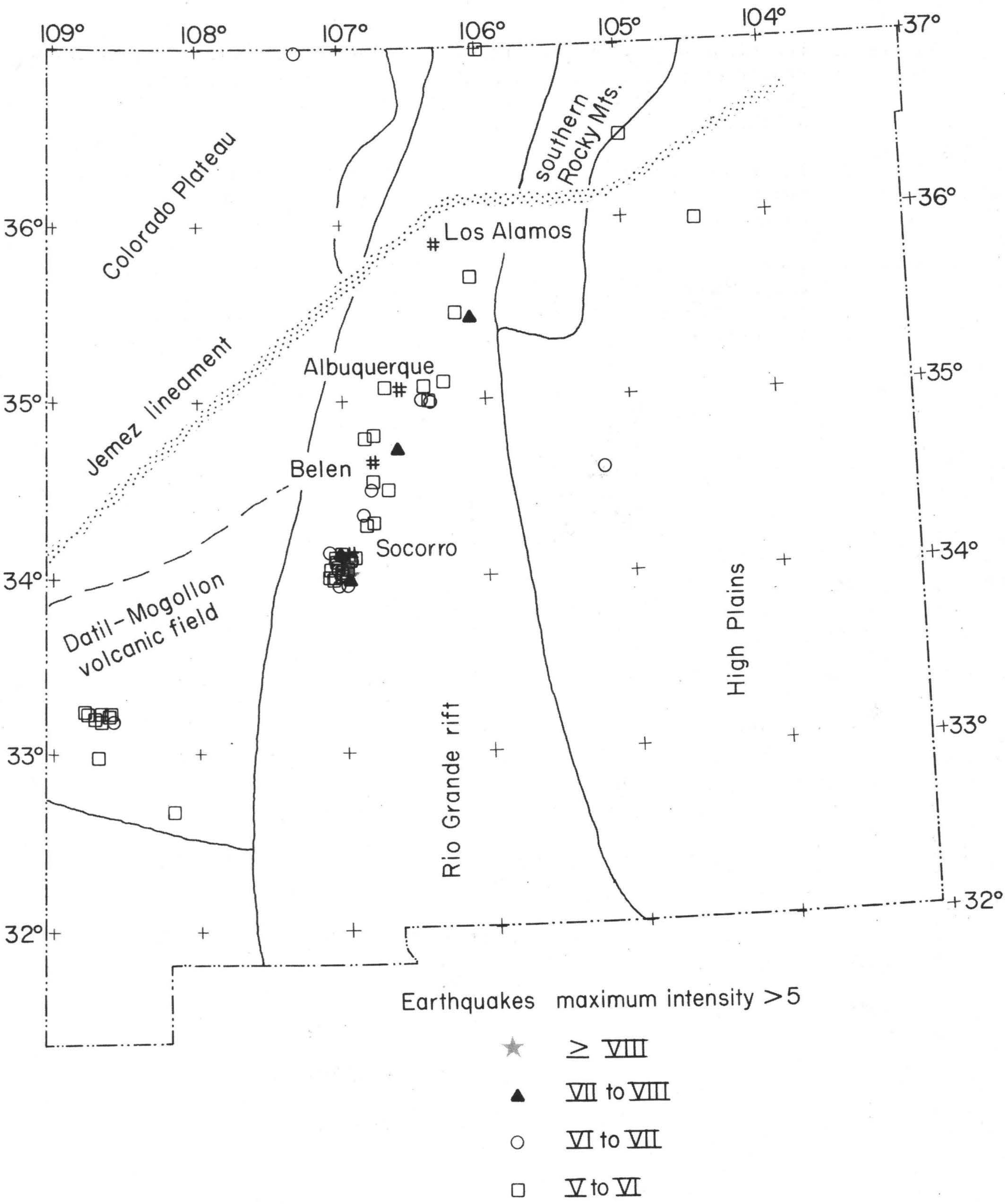
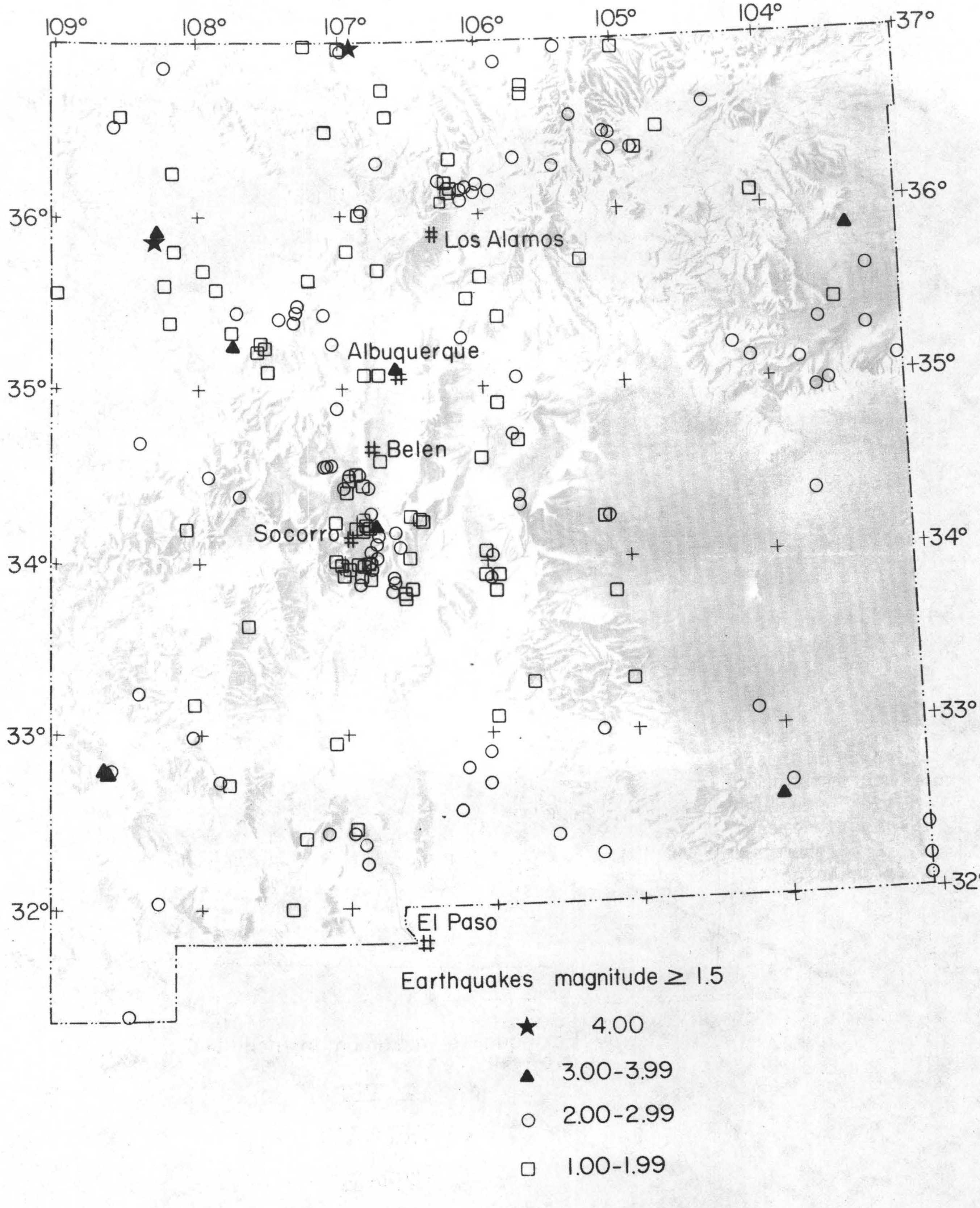
Figure 1. Locations of earthquakes reported prior to 1962 with maximum intensities of V or greater. Also shown on the map are the major physiographic provinces in New Mexico. (From Sanford, Olsen, and Jaksha, in press.)

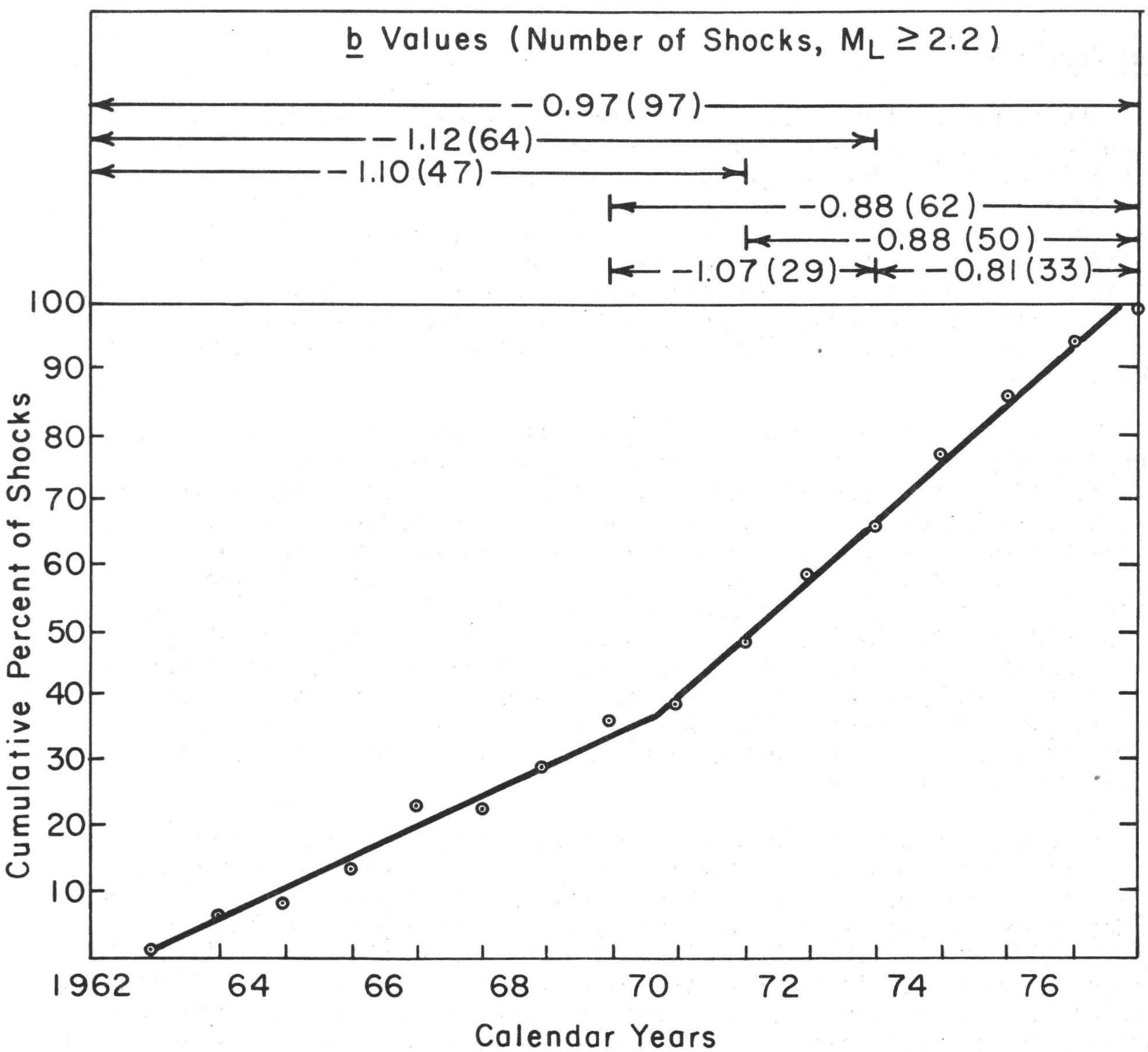
Figure 2. Instrumental epicenters for earthquakes ($M_L \geq 1.5$) recorded during the period 1962 through 1977. (From Sanford, Olsen, and Jaksha, in press.)

Figure 3. Cumulative percent of shocks (with $M_L \geq 2.2$) versus time in calendar years. Also shown are <u>b</u> values for different intervals of time from 1962 through 1977. (From Sanford, Olsen, and Jaksha, in press.)


109°
108°
107°
106°
105°
104°
37°
36°
35°
34°
33°
32°
Colorado Plateau
Jemez lineament
Datil-Mogollon volcanic field
southern Rocky Mts.
Los Alamos
Albuquerque
Belen
Socorro
Rio Grande rift
High Plains
Earthquakes maximum intensity >5
≥ VIII
VII to VIII
VI to VII
V to VI



109°
108°
107°
106°
105°
104°
37°
36°
35°
34°
33°
32°
Los Alamos
Albuquerque
Belen
Socorro
El Paso
Earthquakes magnitude ≥ 1.5
4.00
3.00-3.99
2.00-2.99
1.00-1.99



b Values (Number of Shocks, $M_L \geq 2.2$)
-0.97(97)
-1.12(64)
-1.10(47)
-0.88(62)
-0.88(50)
-1.07(29)
-0.81(33)
100
90
80
70
60
50
40
30
20
10
Cumulative Percent of Shocks
1962
64
66
68
70
72
74
76
Calendar Years

GROUND MOTION PARAMETERS USEFUL IN STRUCTURAL DESIGN

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INTRODUCTION

The ground motion input for the seismic evaluation and design of critical structures (nuclear power plants, offshore platforms, major pipeline projects, etc.) is generally defined in terms of a design response spectrum for which the structure is expected to remain elastic. For less critical facilities, a design response spectrum may not be directly used. Even so, the lateral force coefficient for which such structures are designed can be related back to a design response spectrum. The following discussions are made in terms of a design response spectrum but are equally applicable for structures designed for a lateral force coefficient (building code approach) based upon a design response spectrum.

The design response spectrum is generally a broad banded spectrum with broad frequency content. It expresses the peak linear response of a whole series of single-degree-of-freedom oscillators at a specified damping level. Either site-independent or site-dependent response spectra are specified. A site-independent spectrum uses a broad standard spectrum shape which is considered applicable to a wide range of local geologic and seismological conditions while a site-dependent spectrum tends to be less broad banded and is geared more to the local site conditions. The site-independent spectrum is anchored to one parameter of the ground motion while the site-dependent spectrum is anchored to one or more ground motion parameters and local site conditions. The concept of large regional mapping of ground motion parameters is more consistent with the use of site-independent spectrum which will be emphasized herein.

Figure 1 presents a representative site-independent response spectra which has been commonly used for nuclear power plants in the United States. This spectra (as well as most other site-independent spectra) is anchored to a design ground acceleration with the entire spectra being defined in terms of this one ground motion parameter. Newmark (1973) states that in the high frequency region of interest (approximately 2 to 10 Hz) for stiff structures, the design spectra are most accurately anchored to the design ground acceleration. On the other hand, for more flexible structures (approximately 0.5 to 2 Hz frequency) the design spectra are more accurately anchored to the design ground velocity. Furthermore, the ground velocity is less sensitive to local geologic and seismological conditions than is the ground acceleration. Thus, Hall, Mohraz, and Newmark (1976) have recommended the design response spectra be constructed from the design ground velocity with the design ground acceleration and displacement values being inferred from this design ground velocity based upon local site conditions.

Thus, the minimum ground motion parameters which should be regionally mapped consist of either the design ground acceleration, or the design ground velocity, or both. This author has a minor preference for the use of either the design ground velocity or both parameters. However, most research has been conducted in conjunction with defining a design ground acceleration parameter and for this reason it may be preferable to emphasize this parameter. The remainder of this paper emphasizes approaches for defining the design ground acceleration because this is the parameter for which the research is available. The ideas presented should also be applicable for the design ground velocity.

Design Ground Acceleration Versus Instrumental Peak Acceleration

Seismologists have tended to concentrate on defining ground motion in terms of the Instrumental Peak Acceleration, A_{IP}, which represents the absolute peak acceleration recorded during the entire earthquake motion by a reliable strong-motion instrument situated at the free ground surface (i.e., not significantly influenced by soil-structure interaction or local topographic conditions). This parameter represents a relatively easily determined quantitative value not strongly influenced by subjective judgements. Unfortunately, as illustrated by many studies (e.g., see Hoffman, 1974; Page and others, 1972; Housner, 1975, 1979; Housner and Jennings, 1977: Newmark, 1975; Blume, 1979; Nuttli, 1979), A_{IP} is a poor measure of the damaging potential of earthquake ground motions. It has been noted, particularly in connection with near-source ground motions due to low- to moderate-magnitude earthquakes, that structures have performed much better during earthquakes than would be predicted considering the instrumental peak acceleration to which the structures were subjected. Examples of this behavior may be seen from the 1966 Parkfield earthquake, the 1971 Pacoima Dam earthquake record, the 1972 Ancona earthquakes, and the 1972 Melendy Ranch Barn earthquake record. These earthquake records had instrumental peak accelerations of between 0.5 and 1.2 g and yet only minor damage occurred in the vicinity of the recording sites. In these cases and others, the differences in measured ground motion, design levels, and observed behavior is so great that it cannot be reconciled with typical safety factors associated with elastic seismic analyses used for design.

The problem with A_{IP} is twofold. First, a limited number of high frequency spikes of high acceleration but very short duration have little effect on the elastic response spectra within the region of primary interest (0.5 to 20 Hz). Secondly, the elastic response spectra describe elastic response while structure damage is related to structures being strained into the inelastic range in which the duration of motion or the number of cycles of straining substantially influence the damage. The first problem in which A_{IP} is not a good parameter to use for defining an elastic response spectrum is discussed in this section. The second problem in which an elastic response spectrum computed from an instrumental time-history is not a good basis for a design response spectrum is discussed in the next section. Both problems can be corrected through the use of a Design Ground Acceleration, A_D, as a ground motion parameter to which the design response spectrum is anchored. Unfortunately, A_D (often called effective peak acceleration) is more difficult to quantitatively define. It is defined herein as that acceleration at which the design response spectrum is anchored at zero-period (or infinite frequency).

Even within the higher frequency range (2 to 20 Hz) the elastic response spectrum values are primarily influenced by the energy contained within a number of cycles of ground motion and are little influenced by a few spikes of very high acceleration. Blume (1979) has shown that clipping the highest 30% off the measured acceleration-time history (using only 70% of the record, in an absolute sense, closest to the zero line) produced only about a 5% reduction in the elastic response spectrum. Similar results have been shown by Schnabel and Seed (1973) and Ploessel and Slosson (1974). Newmark (1976) has shown that the elastic response spectrum from the 1.25g Pacoima Dam record can be conservatively enveloped within the frequency range of interest by a broad-banded design spectrum anchored to a design ground acceleration of 0.75g. These findings have led to a number of recommendations for defining A_D, including the use of sustained or repeatable peak acceleration (Nuttli, 1979), the use of an equivalent cyclic motion (Whitman, 1978), and the use of filtered time histories in which high frequency spikes are removed by passing the measured time history through an 8 to 9 Hz cutoff frequency filter (Page and others, 1972; Ploessel and Slosson, 1974). Based upon a review of these recommendations, this author would like to suggest the following as a candidate procedure for defining the design ground acceleration, A_D:

$$A_D = 1.25 * A_{3F} \qquad (1)$$

where A_{3F} is the 3-rd highest peak acceleration from the filtered time-history record. The filter chosen by Page (1972) which is centered at 8.5 Hz with a value of 1.0 at 8.0 Hz and 0.0 at 9.0 Hz appears to be a reasonable filter approach. It has been shown (Kennedy, and others, 1980) that broad-banded design spectra anchored to this acceleration tend to envelop the elastic response spectra. This definition is illustrated using the 1.25g Pacoima Dam record. Figure 2 presents the unfiltered and filtered Pacoima Dam record. The 3-rd highest peak, A_{3F}, from the filtered record is 0.62g. The A_D from Equation (1) is 0.78g which agrees with Newmark's (1976) recommendations for this record. On the other hand, for the 1940 north-south El Centro record in which there were several lower frequency near-peak excursions the design ground acceleration, A_D, would be essentially equal to the instrumental peak acceleration of 0.35g by this definition.

As noted earlier, the elastic response spectrum values are primarily influenced by the energy fed into a structure by a number of cycles of ground motion. Arias (1970) and Housner (1975) have suggested that E(T) given by:

$$E(T) = \int_{t_o}^{t_o+T_D} a^2(t)dt \qquad (2)$$

can serve as a measure of the total energy fed into the structure between time t_o and time $t_o + T_D$. The Arias Intensity is proportional to E(T). In Equation (2), a(t) represents the instrumental acceleration at time t, and T_D is the duration of strong motion. The average rate of energy input (earthquake power) is then given by:

$$P = E(T)/T_D \qquad (3)$$

Alternately, Mortgat (1979), and McCann and Shah (1979) have suggested the root-mean-square acceleration, rms, as the ground motion parameter of interest. This rms acceleration is given by:

$$rms = \sqrt{P} \tag{4}$$

Both the power, P, and rms acceleration are heavily influenced by the procedure used to select the duration of strong motion, T_D. Often the duration of strong motion has been selected as the time between the first and last excursion of the absolute acceleration above a selected percentage of the peak acceleration (such as 10 or 20 percent) or the time between the first and last crossing of a particular acceleration level (such as 0.05g). Such definitions give anomalous results for duration, power, and rms acceleration for a record such as the 1940 El Centro record which appears to consist of three distinct zones of strong motion during the time history. It has been found (Kennedy, and others, 1980) that the cumulative time the ground motion exceeds a selected percentage (such as 10 percent) of the peak acceleration provides a more consistent estimate of the strong motion duration, power, and rms acceleration for a number of records.

Use of the rms acceleration as the basis for the design acceleration, A_D, has many attractions. It is an objective and easily computed quantity. As shown by Mortgat (1979), it enables a design acceleration to be selected at any desired probability of exceedance during the time history. A design acceleration defined in this fashion can be used to define the elastic response spectrum with a given probability of exceedance. The design acceleration is related to the rms acceleration by:

$$A_D = K_p * rms \tag{5}$$

where K_p is a function of the acceptable exceedance probability for each individual peak of the time history. Considering the design acceleration as that which is expected to occur once on the average over the duration of strong motion for a stationary random Gaussian motion, Vanmarcke and Lai (1980) have determined K_p to be:

$$K_p = \sqrt{2 \ln (2T_D/T_o)} \tag{6}$$

except K_p is not less than $\sqrt{2}$, where T_o is the predominant period of the ground motion which can be taken to be between 0.2 and 0.4 seconds for most records. Equations (5) and (6) appear to work well for defining a design acceleration to which elastic response spectra can be anchored.

The usage of Equations (2) through (6) can be illustrated using the 0.7g 1972 Melendy Ranch recording (Figure 3) and the 0.18g 1952 Taft recording (Figure 4). Both records contain relatively similar total energy content despite the nearly fourfold greater instrumental peak acceleration for the Melendy Ranch record. The Melendy Ranch record has a much shorter strong motion duration of about 1.5 seconds versus about 16 seconds for the Taft record. With these durations, the design accelerations given by Equations (5) and (6) are 0.34g for the Melendy Ranch record and 0.14g for the Taft record.

The design acceleration ranges from 50 percent of the instrumental peak acceleration for Melendy Ranch to 70 percent for Taft which illustrates the effect of the short duration for Melendy Ranch.

For several earthquake records Table 1 compares instrumental peak accelerations, and design accelerations given by Equations (1) or (5) and (6) (T_0 = 0.3 seconds). Also presented is the strong motion duration. In each case, the design acceleration from Equations (1) or (5) and (6) is judged to be a consistent basis for anchoring the design response spectrum for the purpose of computing elastic response in the 1 to 10 Hz frequency range. One can note the influence of duration on the ratio of A_D to A_{IP} for elastic response.

The Importance of Duration

The design ground acceleration, A_D, can be defined by either Equation (1) or (5) and (6) if the purpose is to anchor an elastic response spectrum for computing peak elastic structural response. However, neither these design accelerations, nor an elastic response spectrum obtained from an instrumental time history serves as a good measure of damage to structures. Each ignores the effect of duration on damage and underestimates the effect of the number of cycles of near-peak excursions.

There are energy absorbing mechanisms during seismic response of structures such that a limited number of cycles of even very high acceleration ground motion might not produce any noticeable effects on a structure. Such energy absorbing mechanisms include concrete cracking, bond slip of reinforcement bars, friction at bolted connections and other locations, and other mechanisms. These energy absorbing mechanisms cause nonlinear behavior of sufficient amount to considerably reduce required design force levels from those calculated assuming totally elastic behavior. For each cycle of earthquake motion, energy absorption has a small deteriorating or degrading effect on the structure; for sufficient numbers of cycles these degrading effects would eventually accumulate to produce noticeable structural damage. For example, when a reinforced concrete shear wall is subjected to sufficient transverse shear forces during an earthquake, the concrete will crack even though the steel continues to behave elastically. This would be acceptable behavior even for a critical facility such as a nuclear power plant. Such a member in shear would exhibit softer unloading stiffness and degrading stiffness during reloading because the concrete cracks do not heal during unloading and the concrete begins to deteriorate. This behavior is illustrated in Figure 5. For a limited number of cycles of seismic response such that the energy of the seismic excitation was less than the energy absorption capacity of the structure, such a structure as that described above would shake down to pseudo elastic behavior possibly at a reduced stiffness and possibly with some permanent set but the structure would be stable and safe and would not have experienced significant damage. This behavior is illustrated schematically in Figure 6. On the other hand, for a strong earthquake in which the number of cycles of seismic response is such that the energy of the seismic excitation exceeds the energy absorption capacity of the structure, such a structure as that described above would reach displacement amplitudes corresponding to significant structural damage and possibly total collapse. This behavior is illustrated schematically in Figure 7.

Short and others (1980a, 1980b) have studied the effect of a high acceleration, short duration record such as the Melendy Ranch record (Figure 3) on a nuclear power plant structure designed for a long duration, much lower acceleration record like the Taft record (Figure 4). The shear wall type structure was designed to ultimate strength for a broad-banded design spectrum anchored to 0.2g. The structure was subjected to the Melendy Ranch record. Concrete elements were defined to have highly degrading stiffness characteristics similar to those shown in Figure 5. The Melendy Ranch record shows maximum 5% damped spectral acceleration in excess of 1.5g in the 5 to 6 Hz frequency range and the structure was designed to have a fundamental frequency within this range. The nonlinear response of this structure was found to be highly stable with a single inelastic excursion followed by pseudo elastic behavior with a slightly degraded stiffness as shown in Figure 6. Thus, a highly degrading structure designed for a design response spectrum anchored to a design ground acceleration, A_D, of 0.2g shows perfectly satisfactory behavior when subjected to the Melendy Ranch record. Thus, this record should be taken to have a design acceleration value of 0.2g or less as opposed to the 0.34 to 0.45g defined in the previous section for calculating elastic response. This example illustrates the importance of duration and number of near-peak excursions.

Ignoring duration and considering only the elastic response spectra or the design acceleration defined by Equations (1) or (5) and (6) would lead one to conclude that the Melendy Ranch record was more severe than the Taft record, and the the 1966 Parkfield record was more severe than the 1940 El Centro record. Both conclusions would be incorrect and illustrate the inadequacy of the elastic response spectrum to define the damage capability of an earthquake. The problem is that the elastic response spectrum values are related primarily to the power of the earthquake (Equation 3), or the rms acceleration (Equation 4), or the design acceleration (Equations 1 or 5 and 6), while the damage capability is probably more related to the total energy fed into structures (Equation 2). Housner (1975) has proposed that this dilemma be solved by a two-parameter definition of the ground shaking in which one parameter could be any of the parameters relating to the power of the earthquake such as the design acceleration from Equations (1) or (5) and (6). The other parameter should be strong motion duration, T_D.

It would be desirable to have a single design ground acceleration parameter to which a design response spectrum which incorporated the influence of duration on damage could be anchored. Such a parameter would have to be related back to the total energy fed into a structure (Equation 2). Since the design acceleration, A_D, (Equation 5) is proportional to the square root of the power and the energy is simply the product of the power times duration (Equation 3), the design acceleration could be made a function of energy rather than power through the use of a "standard" duration of 20 seconds. Thus,

$$A_D' = 3.5 * rms * (T_D/20)^{1/2} \tag{7}$$

where the coefficient 3.5 has been empirically determined so that for the higher magnitude, more distant records (Taft, El Centro, and Olympia) the damage potential design ground acceleration, A_D', and elastic response design ground acceleration, A_D, would be nearly equal. With this definition, A_D' is proportional to the square root of the energy while A_D is proportional to the square root of the power. Thus, A_D could be used to anchor an elastic response spectrum if peak elastic response must be computed, while A_D' could be used to anchor a design response spectrum for a consistent damage potential. Based upon Equation (7) and the values previously given for the Melendy Ranch and Taft records, it is found that:

Melendy Ranch: $A_D' = 0.15g$

Taft: $A_D' = 0.15g$

This computed design acceleration is considered to be much more representative of the damage potential for the Melendy Ranch record than is the instrumental peak acceleration, A_{IP}, of 0.7g or the design acceleration, A_D, from Equations (5) and (6) of 0.34g.

The damage potential design acceleration, A_D', is given for other records in Table 1 where it can be compared with the elastic design acceleration, A_D, and instrumental peak acceleration, A_{IP}.

Conclusion

For the purposes of defining an elastic response spectrum both the design ground acceleration and the design ground velocity are useful ground motion parameters with a slight preference being expressed toward the use of the design ground velocity if only one parameter can be chosen. However, most studies have concentrated upon defining a design ground acceleration.

The instrumental peak acceleration is a highly inadequate and sometimes grossly conservative parameter for defining the design ground acceleration. If one is predominantly concerned with computing elastic structural response then it appears that either Equation (1) or (5) and (6) could be used to adequately define a design ground acceleration. These equations provide design ground acceleration values which correlate poorly with damage potential unless they are used in conjunction with a strong motion duration parameter, T_D. A single design ground acceleration parameter which appears to correlate better with damage is given by Equation (7) because it incorporates the effect of duration.

Although unstudied, it is believed that procedures similar to those recommended for defining the design ground acceleration could also be used to define the design ground velocity.

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TABLE 1. INSTRUMENTAL PEAK VERSUS DESIGN ACCELERATIONS

Earthquake Records		Instrumental Peak Accel. A_{IP} (g)		Elastic Response Design Accel. A_D (g)		Strong Duration T_D (Sec)	Damage Potential Design Accel. A'_D (g)
		Uncorrected	Corrected	$1.25*A_{3F}$	K_p*RMS		
Melendy Ranch	N61E	.50	.48	0.40	0.36	1.2	0.15
	N29W	.70	.52	0.45	0.34	1.5	0.15
Parkfield, Cholame	N65E	.51	.49	0.50	0.41	4.4	0.26
Temblor	S25W	.41	.35	0.26	0.26	4.2	0.13
Pacoima Dam	S74W	1.25	1.07	0.78	0.78	5.7	0.54
Hollywood Storage	S00W	.19	.17	0.15	0.15	9.3	0.12
PE Lot	N90E	.22	.21	0.21	0.19	9.0	0.16
El Centro	NS	.37	.35	0.30	0.28	13.1	0.26
Olympia	N86E	.31	.28	0.21	0.21	13.1	0.20
Taft	S69E	.20	.18	0.14	0.14	16.1	0.15

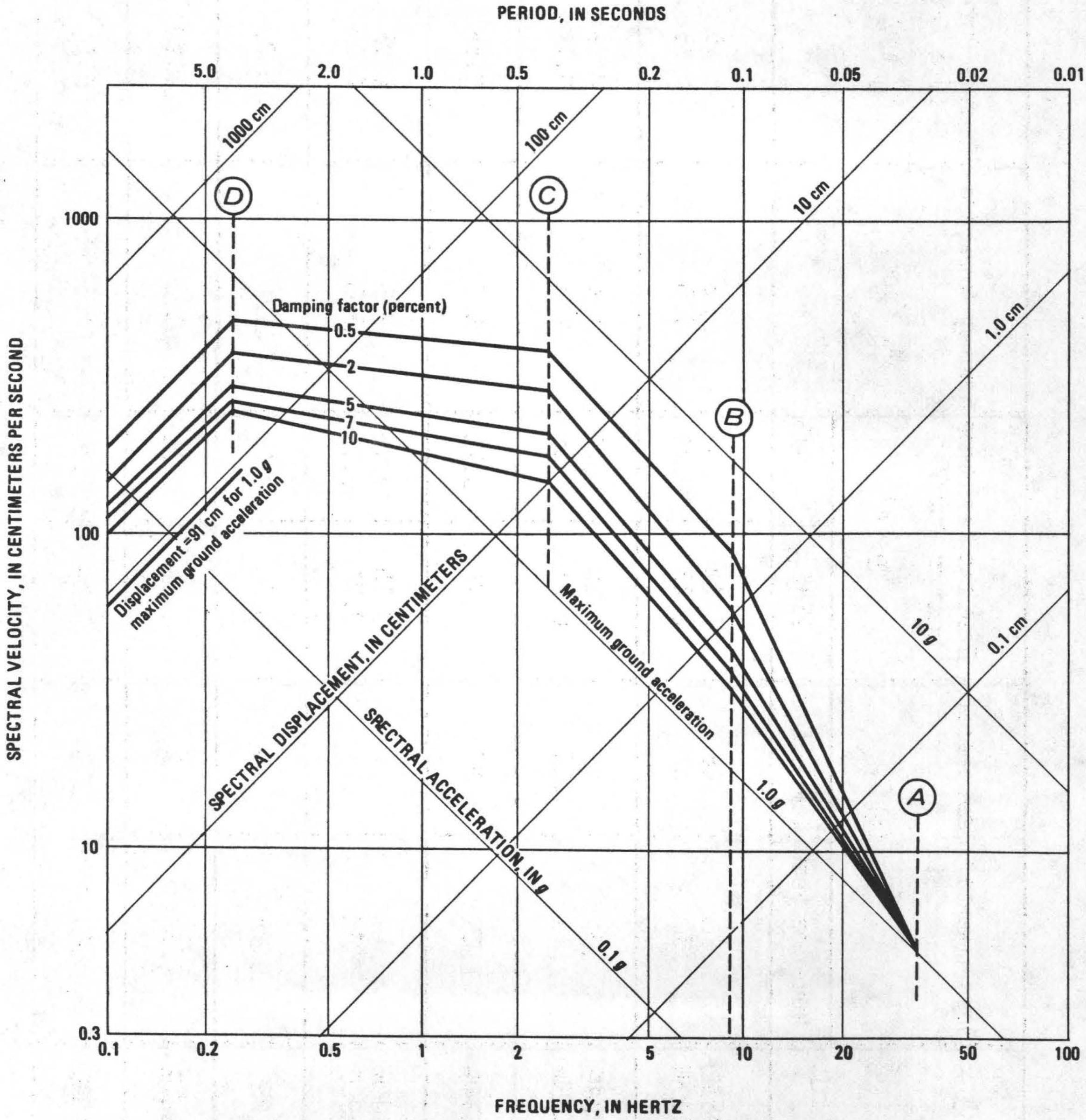


FIGURE 1. SITE-INDEPENDENT HORIZONTAL RESPONSE SPECTRA SCALED TO 1.0g

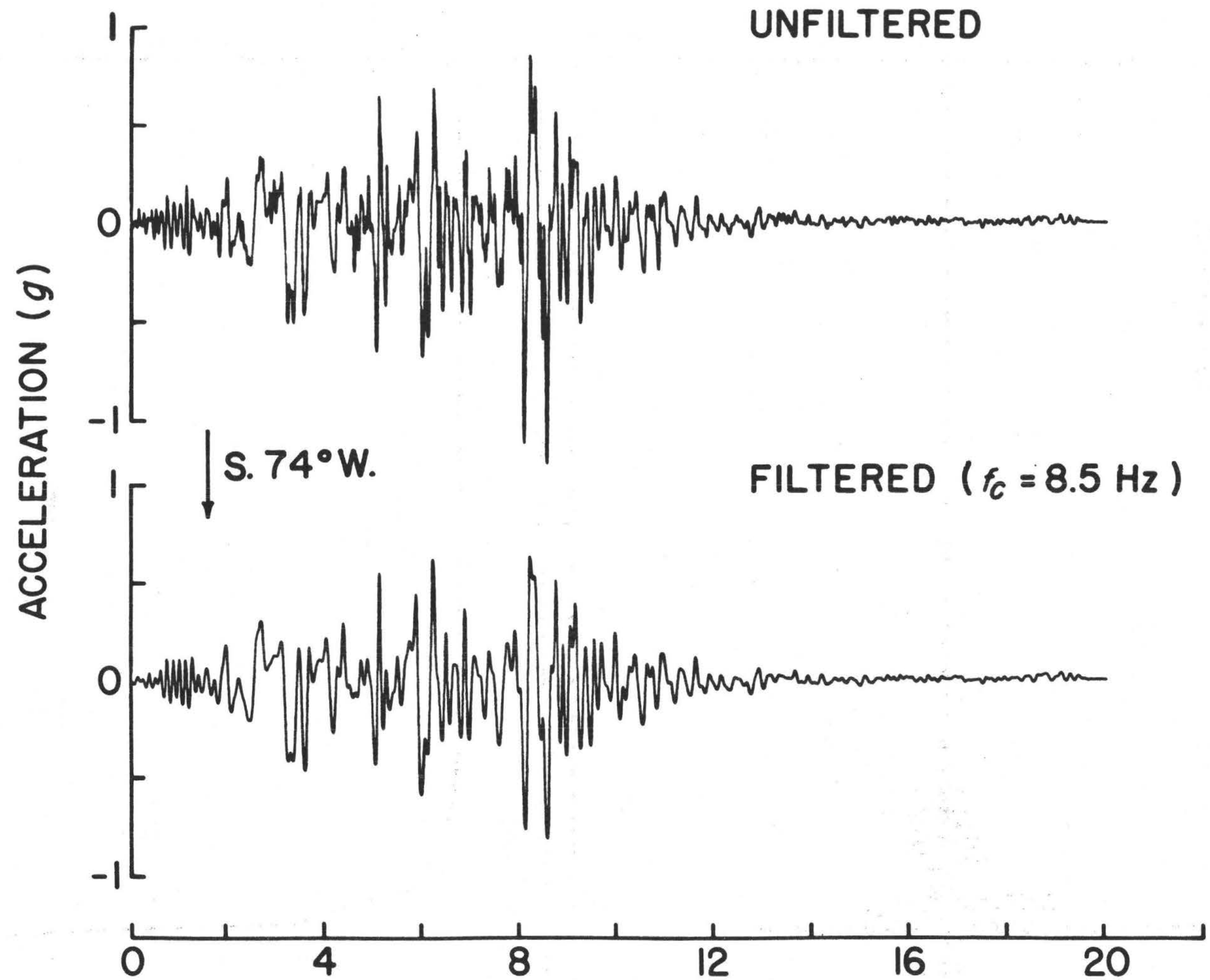


FIGURE 2. UNFILTERED AND FILTERED ACCELEROGRAMS OF THE 1971 SAN FERNANDO EARTHQUAKE FROM THE S. 74° W. ACCELEROGRAPH COMPONENT AT PACOIMA DAM. RESPONSE OF FILTER IS 1.0 AT FREQUENCIES LESS THAN 8 HZ AND 0.0 AT FREQUENCIES GREATER THAN 9 HZ WITH A HALF-WAVE COSINE TAPER FROM 8 TO 9 HZ. (PAGE AND OTHERS, 1972).

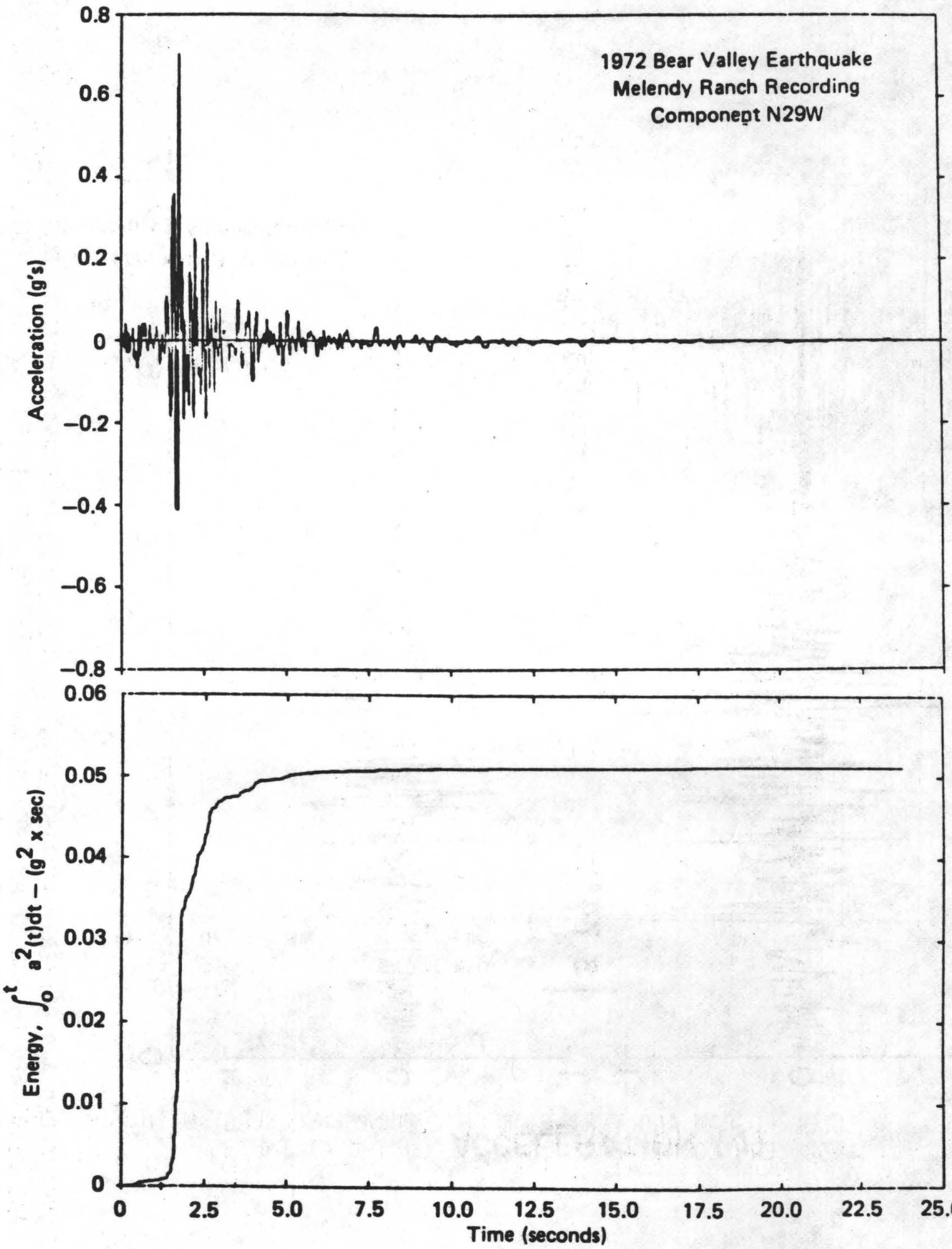


FIGURE 3. ACCELEROGRAM AND VARIATION OF CUMULATIVE ENERGY WITH TIME FOR THE 1972 BEAR VALLEY EARTHQUAKE RECORDING AT MELENDY RANCH

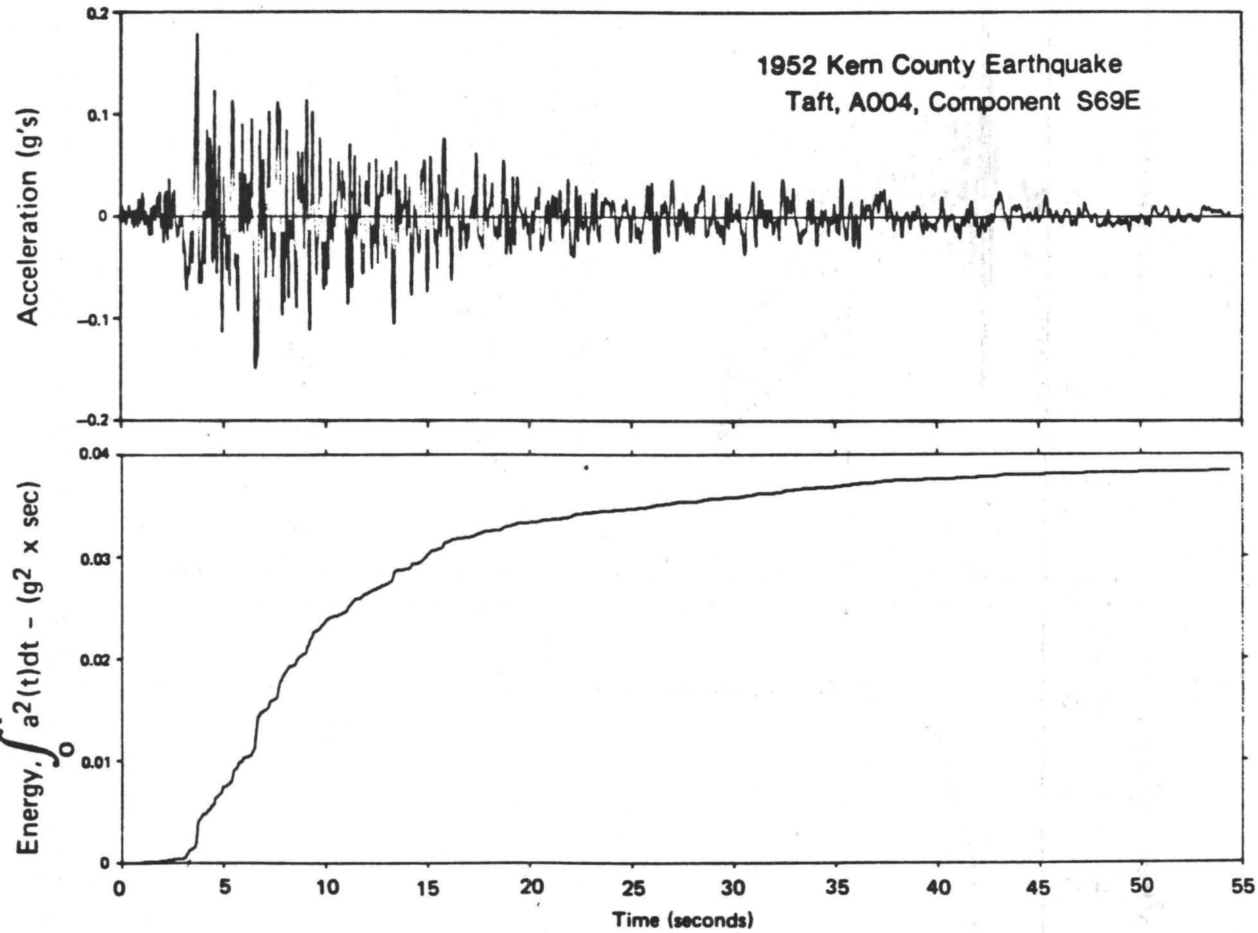


FIGURE 4. ACCELEROGRAM AND VARIATION OF CUMULATIVE ENERGY WITH TIME FOR THE 1952 KERN COUNTY EARTHQUAKE RECORDING AT TAFT

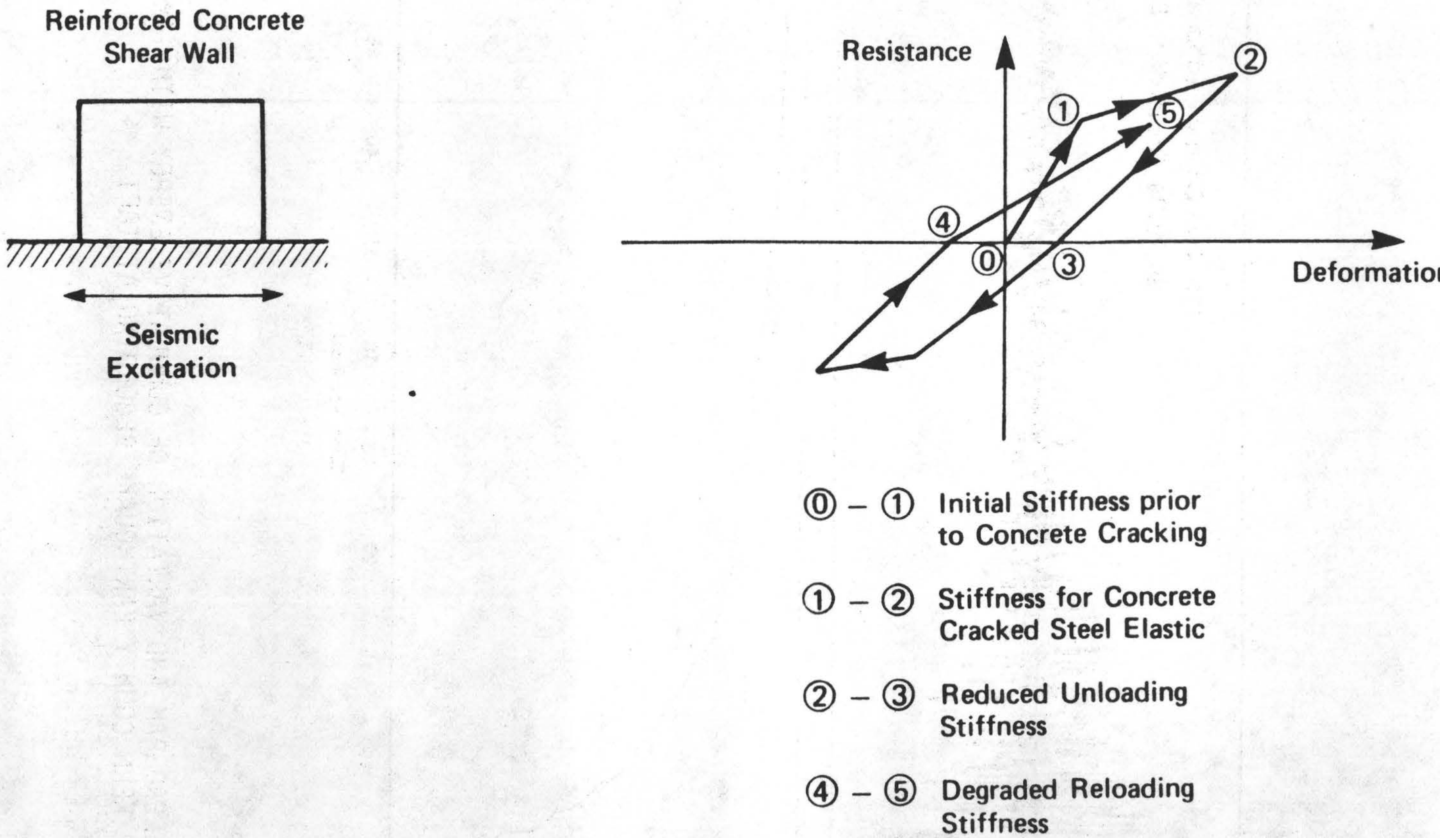
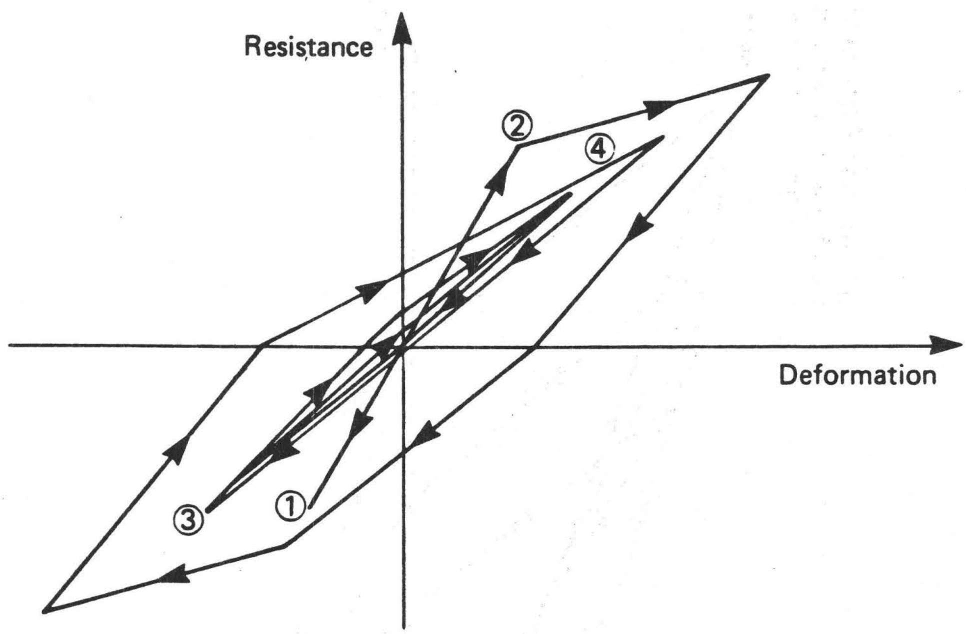


FIGURE 5. IDEALIZED CYCLIC FORCE-DISPLACEMENT BEHAVIOR OF REINFORCED CONCRETE SHEAR WALL



① – ② Initial Uncracked Concrete Stiffness

③ – ④ Pseudo-Elastic Stiffness

FIGURE 6. SEISMIC SHEAR FORCE-DISPLACEMENT RELATION FOR STRUCTURAL MEMBER WHICH SHAKES DOWN TO PSEUDO ELASTIC BEHAVIOR

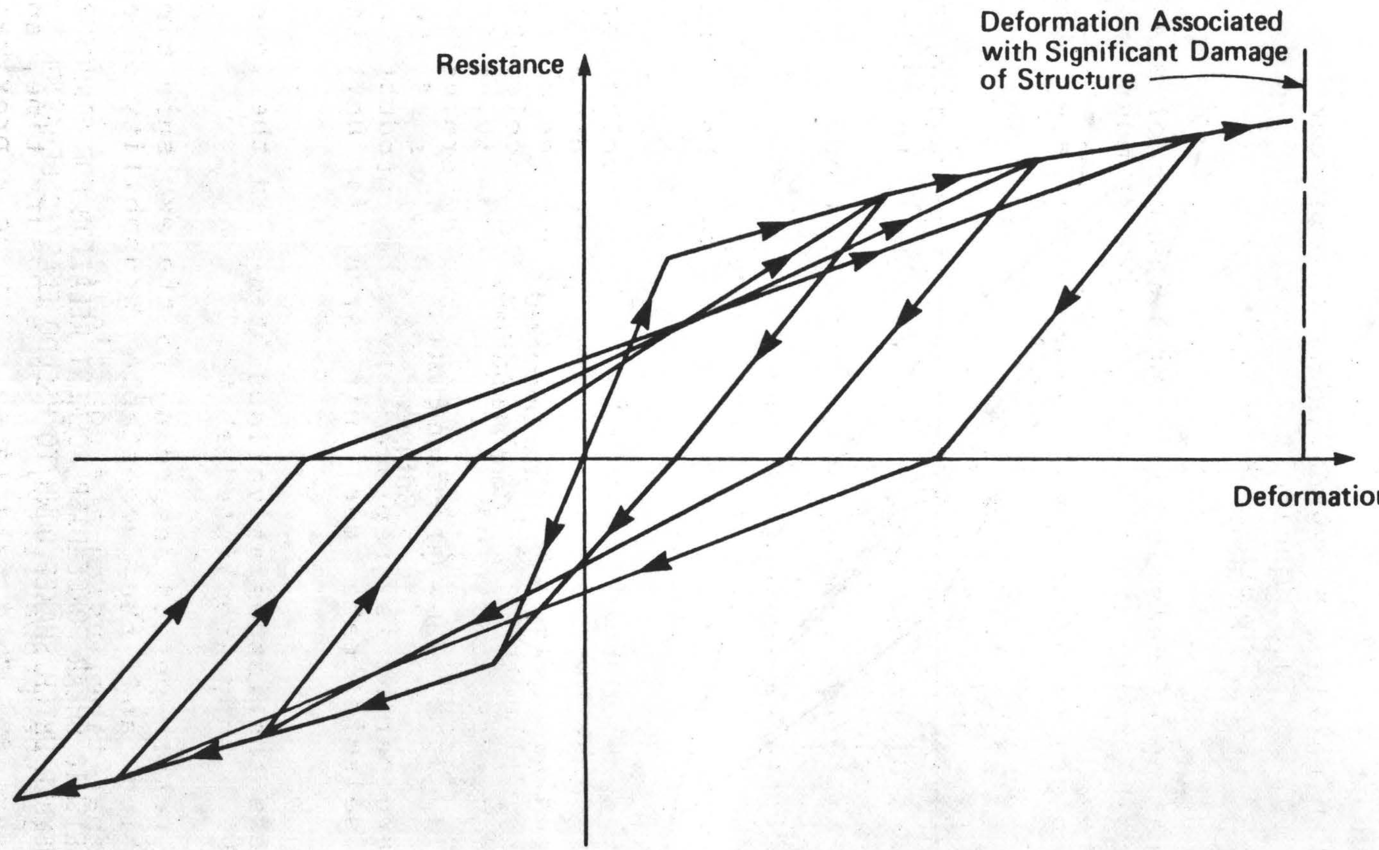


FIGURE 7. SEISMIC SHEAR FORCE-DISPLACEMENT RELATION FOR STRUCTURE MEMBER WHICH PROGRESSIVELY CYCLES TO DEFORMATION CORRESPONDING TO SIGNIFICANT DAMAGE

RULES OF THUMB RELATING TO ISSUES IN PROBABILISTIC GROUND MOTION MAPPING

by

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In the preparation of seismic risk maps a number of stages can be identified at which decisions are required. We must decide

1. What zones are to be used for sources and are they to be based on
   a) historical seismicity
   b) geology and tectonics
   c) historical seismicity generalized by geology-tectonics.
2. What values of parameters are to be chosen to describe the future seismicity?
3. How are the future earthquakes to be modelled?
   a) area point sources
   b) area finite rupture sources
   c) known faults using rupture sources
4. What levels of probability are to be chosen for representation on the map?

Questions at each of these stages may often involve issues which are decided as much by style or artistic preference as by right and wrong technique. In any case the consequences of the choices available are illuminated by examining log acceleration vs log return period curves or parameterized curves deriving from them. Probabilistic ground motions can be determined for most hypothetical situations using normalized versions of these curves in which return period is replaced by the product of return period and seismic rate per unit area (or unit length, for faults).

We will address the issues at various stages in the inverse order of that in which they are listed above.

4. Choose a return period according to the exposure time of the application and the acceptable probability of exceedance during that time, using the equation,

$$r = 1 - \exp(-T/RP)$$

where r is the probability, T the exposure time, and RP the return period. This relationship can be approximated by

$$r = T/RP$$

when T/RP is 0.1 or less.

Because the log accelerations vs log return period curves have lower and lower slopes for longer and longer return periods, if more than one map is to be presented, choose the return periods to differ by a factor or by a series of increasing factors. Otherwise there will be insufficient contrast in acceleration values between the maps. Because the slopes are less than one, choose factors of 3, 4, or more in order to produce doubling of accelerations between maps. Thus

50 - 100 - 150 is a poor series of return periods,
50 - 100 - 200 is a better choice, but
50 - 200 - 1000 is a still better choice.

3. For earthquakes above magnitude 6.5 to 7.5 it is usually better to model them as finite rupturing sources for sites in the near field of the fault. For sites greater than 75 km from the fault and where there is a local source zone, it is generally sufficient to model the earthquakes as point sources on or near the fault. In a region where there are no known faults to be modelled, but where large magnitude faults are to be expected, rupturing sources should be provided, either by a special algorithm or by putting equally spaced dummy faults in the source zone (four may be enough) and averaging over the probabilistic ground motion values in the interior of the zone.

   The probabilistic ground motion values from rupture source models may differ by as much as 15 percent for different formulas used for relating average rupture length and magnitude. McGuire has shown that this difference in results from using different rupture-length vs magnitude relationships can be greatly reduced if one models the statistical variability in the relationship. Surprisingly, Bender has discovered that statistical variability in the magnitude-fault length relationship can be adequately modelled using a single relationship at a fraction of the standard deviation greater than the zero-variability (mean) relationship.

2. Unfortunately, although it is almost always impossible to choose the maximum magnitude from the statistics of the historical seismicity, maximum magnitude is generally the most important of the parameters, in terms of its impact on the map. This is particularly true in regions of low seismicity, where maximum observed magnitudes are low. For low maximum magnitudes, around 4 or 5, increasing the maximum magnitude by one unit will double the ground motion at a given return period. For point source models the factor increase of acceleration decreases with increasing maximum magnitude, and for magnitudes greater than 7.0, the increase is not very important for high b values. However, for low b-values, which are to be expected in active zones, and especially for finite rupture models, regardless of b-value, increasing maximum magnitude always produces significant increases in mapped ground motion.

   A-values and b-values have a somewhat lesser effect on probabilistic ground motion. Doubling a zone's seismic rate will considerably less than double the ground motion at a given return period. For point sources the factor is about 1.4. Changing a b-value by 0.1 (for intensity b-value) is roughly equivalent to changing the return period for a given acceleration by about a factor of 2 or 3. When fitting historical data, a technique which produces a high b-value usually produces a low a-value and vice versa. Hence, a- and b-values, if jointly determined from historical seismicity, generally have a relatively low effect on mapped ground motion. Therefore one might expect it generally better to jointly determine a- and b-values from historical data, rather than to assign b-values and fit a-values from the historical data.

However, for earthquake samples numbering fewer than 100, there are increasingly greater biases in the a- and b-values, when determined by the usual "best" techniques: weighted least squares or Page-Aki maximum likelihood. This is true because these methods are strongly affected by the grouping of data in large magnitude ranges (as is common for historical seismicity because of the predominance of epicentral intensities) and the assumption that the mean magnitude of the range is the center of that magnitude range. The bias can be partly compensated for by adjusting to the proper mean with a trial b-value. Karnik's maximum likelihood technique is not sensitive to this mean bias error.

Accordingly, when fitting historical seismicity, zones must be kept large enough to collect an earthquake sample of suitable size, or zones must be combined, fitting performed, and then the fit seismicity back-allocated to the constituent zones. Although it is usually possible to correct for incompleteness in the historical record for the various magnitude intervals, it is not usually possible to do this for small zones, so the back-allocation must be based on judgement, choosing between the results of various consistent techniques: sum of observed earthquakes, number observed in a historically complete category, equivalent intensity VI's, various fits to log frequency graphs for constant b-value, seismic energy flux, etc (they all give good answers for very large data sets) which best reflect the character of the local incompleteness. (In general, energy flux gives too much emphasis on the largest events, and is unsuitable for a back-allocation technique.)

1. The balance between zoning on historical seismicity and zoning on geology should depend on how the geological information affects the probabilistic ground motion. But the value of ground motion depends upon the normalized return period. Hence, the relevance of the geological information depends upon the seismicity associated with it. Pre-late Quaternary faulting by itself does not usually produce a seismic rate which will produce probabilistic

ground motions which will not be dominated by historic seismicity. Hence, when there is significant historical seismicity, pre-late Quaternary faulting should not be used for zoning except to suggest a tectonic direction in which to extend historical seismicity.

If late Quaternary faulting is to have its implied seismicity distributed uniformly over a source zone, its seismicity will usually be less than that derived from one historical intensity VI event in that zone. If, instead of being distributed over a zone, a late Q fault is to be represented as its own zone with a background areal zone of historic seismicity, it will produce contours on the resulting map only if the background zone has a rate per 10,000 square kilometers less than 30 times the rate per 100 km on the fault. This little rule in fact varies with the acceleration level expected from the background zone, and a simple graphic procedure can be demonstrated to provide universal application.

It is sometimes the case that a hot spot of concentrated historical seismicity is observed. It is debated whether to provide it with its own zone or to merge its seismicity into its containing zone. Providing a "fuzzy" boundary for this zone is ineffective in decreasing the strong contouring around the small zone or the strong contrast between its interior probabilistic acceleration value and that of the containing zone. Instead, a simple and effective technique is to provide a probability number for the likelihood that the zone exists. The results for the case that it exists are combined with the results for the case that it doesn't exist. In this manner, practically any desired contrast can be obtained for the proper choice of probability. However, often the choice is constrained by subjective limits of credibility, and the result is a greatly decreased probabilitic ground motion in the vicinity of the hot spot, for a slight increase in the containing zone.

ON THE PROBLEM OF THE MAXIMUM MAGNITUDE OF EARTHQUAKES

by

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In a sense the problem of the maximum-magnitude earthquake is a semantic one. If one is willing to consider all of geologic time, the maximum-magnitude earthquake for any particular region probably will have to be taken as the largest earthquake which has occurred anywhere in the world. But, for purposes of seismic hazard assessment, this is not a very realistic approach. Rather we have to answer the question: What is the magnitude of the largest earthquake that is likely to occur in a reasonable amount of time? The two troublesome words in this question are "likely" and "reasonable".

The definition of "reasonable" will depend upon the type of structure which is at risk. For an ordinary single-family dwelling it will be no more than 100 years, and conceivably less. Even for most commercial buildings a 100-year interval might be considered appropriate. Obviously for long-lived structures, such as dams and nuclear power plants, the time the facility will be at risk is much greater. Numbers such as 1000, 2000 and 10000 years have been proposed. The word "likely" cannot be separated from "reasonable". The choice of a reasonable time will affect the magnitude of the maximum likely event, if one sets "likely" to be the 1% probability of occurrence, 5%, or some other number. The value of the maximum-magnitude earthquake which we obtain by this procedure thus will depend on our choice of a "reasonable" time period and a "likely" probability of occurrence. This seems to be begging the question, for in seismic hazard analysis one of the input parameters is the magnitude of the maximum earthquake.

We might address the problem in a different way. In a relatively long period of time, say 10000 years, can we assign a magnitude to the largest earthquake which can be expected to occur on a particular fault or in a particular seismic source zone? If we have a very active seismic region, where earthquakes of $M_S = 8.6$ occur on the average every 100 years, we can say that the maximum-magnitude earthquake for that region is $M_S = 8.6$. Here we rely on experience that there is an upper limit to magnitudes for earthquakes anywhere in the world which depends on the greatest strain that can be stored in a particular volume of rock before failure occurs. Thus we put a physical, deterministic limit on maximum magnitude, rather than a probabilistic one. Experience tells us this is about $M_S = 8.6$. If there is evidence, either from recorded seismic waves, from isoseismal maps, or from geologic field studies (such as displacements greater than 3 meters), of a great earthquake in the past, then we have to consider the maximum-magnitude earthquake for that source region to be $M_S = 8.6$.

Most seismic source zones will not show evidence of an $M_S = 8.6$ earthquake. Therefore we must ask if a particular source zone is capable of having an $M_S = 8.6$ earthquake, given enough time, or if there is something fundamental about the region itself which will not allow such a large earthquake to occur. One approach, an empirical one, is to look at the largest length or area of a particular fault which can be displaced by a single earthquake, and to correlate this with earthquake magnitude. A problem with such an approach is that we don't know what percentage of a long fault is capable of being ruptured by a single earthquake. For example, how much of the San Andreas can break in a given earthquake? Is the maximum-magnitude earthquake the same for all parts of the San Andreas fault?

The problem is even more difficult when we consider regions such as eastern North America, where with only a few exceptions the geologic structure which causes the earthquake is not known. For such regions the commonly employed procedure to estimate the maximum-magnitude earthquake uses the historic record of seismicity. One could say that the largest magnitude earthquake in the historic past will be the maximum-magnitude earthquake. This approach is difficult to justify, for it only sets a lower limit. A variation of this approach is to assume that the maximum earthquake will have a magnitude one unit larger than the largest observed. Besides the obvious question of why one unit, rather than one-half or two or some other number, this method will be overly conservative for regions which have experienced the maximum-magnitude event in historic times, and underconservative for other regions.

Is there any solution to these problems? If one wants an unqualified estimate of maximum-magnitude earthquake, the answer is no. However, the historic seismic record does contain useful information, and it should not be overlooked or dismissed. One premise we might make is that regions which have numerous minor and moderate earthquakes are more likely to have a major earthquake than regions which have little seismicity. That is, the "a" value in the recurrence relation

$$\log N = a - bM$$

is in some way related to the maximum-magnitude earthquake. If we assume that the length and/or area of the causative fault (even if we do not know the fault) places a physical limit on the magnitude of the largest earthquake that can occur, then the line represented by the recurrence equation either will have to be cut off abruptly at that magnitude, or it will have to bend rapidly so as to be parallel to the axis of ordinates at that magnitude. We still are faced with the practical problem of determining that magnitude.

In an attempt to come up with some resolution of the problem, the assumption was made that it is likely that at least one of the approximately twenty seismic source zones in the eastern United States must have experienced its maximum-magnitude earthquake in historic times. If a magnitude-recurrence curve is constructed for each source zone with the largest earthquake omitted, the annual probability of occurrence of the largest recorded earthquake can be determined. Figure 1 shows such a curve for the

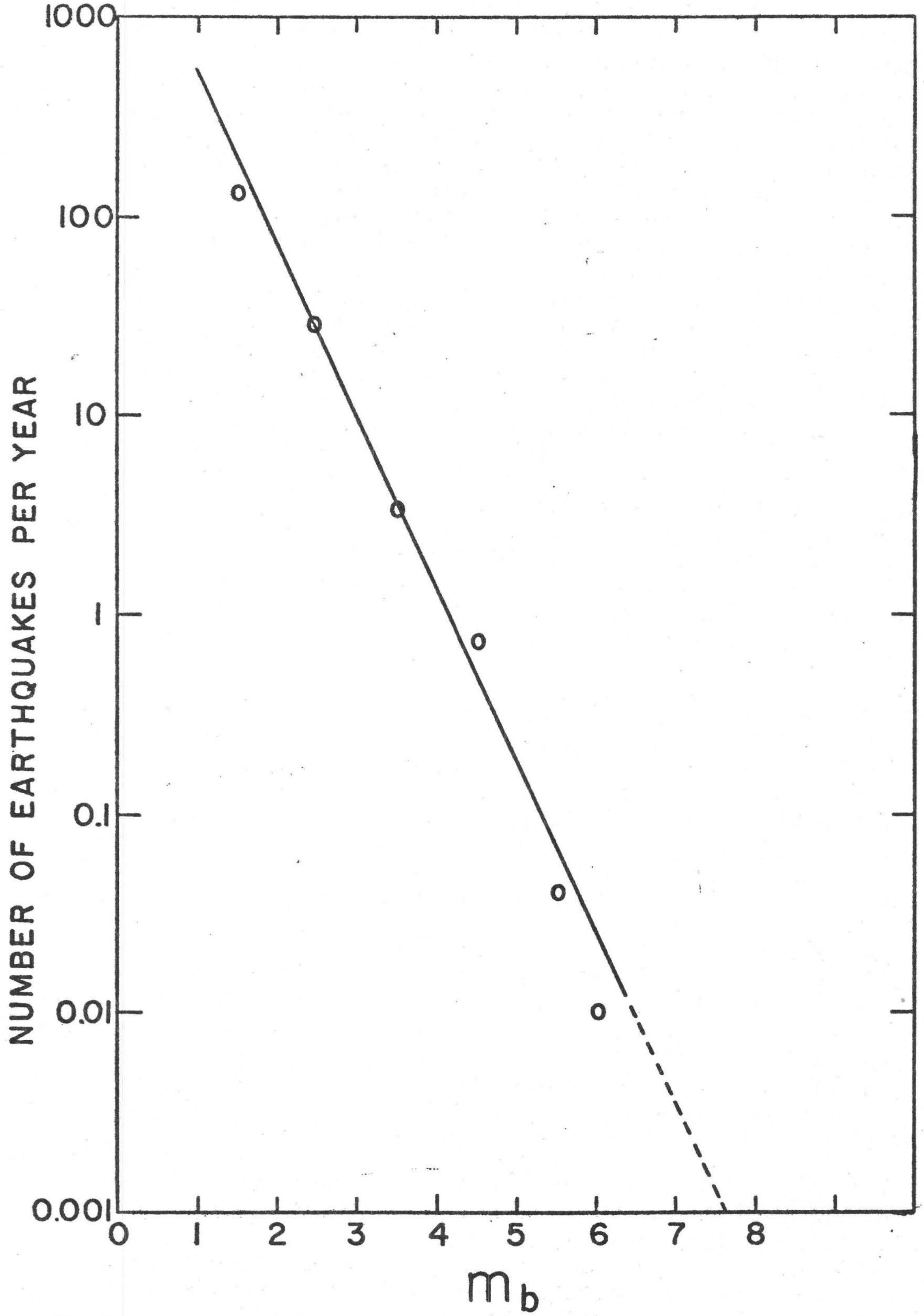

Figure 1. Cumulative magnitude-recurrence curve for the New Madrid seismic source zone (see Figure 5 for extent of zone), excluding the three principal earthquakes of the 1811-1812 sequence and all their aftershocks. It is interesting to note that the number and magnitude of those aftershocks in a 3-month period are greater than that of all earthquakes between the Rocky Mountains and the Appalachians since 1812.

New Madrid seismic zone. In it the 1811-1812 events are not included, and the curve is extrapolated to a return period of 1000 years. The magnitude of the largest of the 1811-1812 earthquakes was $m_b = 7.35$. The m_b scale saturates at about 7.3, so that no earthquake can have a larger value. Thus the maximum-magnitude earthquake for the New Madrid zone already has occurred. From Figure 1 we see that the recurrence time for an $m_b = 7.35$ earthquake is 600 years. Quite independently, by studying sediments in a trench in northwestern Tennessee, Russ (1979) concluded that there was evidence of three major earthquakes in the past 2000 years or less, and that the recurrence interval of such large earthquakes was approximately 600 years.

Another area of the eastern United States where a large earthquake has occurred in historic times, and which obviously is not as active as coastal California, is Charleston, S.C. The 1886 earthquake had an m_b of 6.6 to 6.9 (Nuttli et al, 1979). From the data in Tarr (1977) a recurrence curve can be constructed, as in Figure 2. In constructing this curve the 1886 earthquake and its aftershocks are not considered. Extrapolation of the curve to an annual probability of occurrence of 0.001 (1000-year recurrence period) gives an m_b of 6.85, approximately that of the 1886 earthquake. If it is assumed that the 1886 earthquake is the maximum-magnitude event for the Charleston region, then the extrapolation of the recurrence curve (with the largest magnitude earthquake deleted from the set) to a 1000-year return period gives the maximum-magnitude event. Thus it is assumed that the recurrence curve for the area would bend abruptly at $m_b = 6.85$, and parallel the axis of ordinates at about $m_b = 6.9$. This implies that the 2000 or 10000 year earthquake also would have an m_b of 6.9, and furthermore that the size of the earthquake-generating structure, the rate of strain accumulation and the amount of friction across the fault surface places a limit on the largest earthquake which can occur there which is smaller than the greatest earthquake that has occurred anywhere in the world.

Let us adopt this generalization as a working hypothesis and see what it implies about maximum-magnitude earthquakes in some other areas of the United States. But before we do this we must take into account the size of the earthquake source area. As the source area increases, the number of earthquakes occurring within it will increase. Unless this is taken into account, the extrapolation to a 1000-year return period will yield unrealistically large maximum-magnitude earthquakes for large source areas. That is, the number of earthquakes plotted in the recurrence relation must be equalized by the source area, but the size of the area of equalization is unknown. In our working hypothesis we will use values of both 30000 km^2 (radius of approximately 100 km) and 100000 km^2 for this area.

Figure 3 shows the source areas defined by Hileman et al (1973) for southern California and northern Mexico. They give magnitude-recurrence curves for each of these areas, based on earthquakes occurring from 1932 through 1972. They use M_L, local magnitude, as a measure of the size of the earthquake. Nuttli (1979) concluded that for western United States earthquakes M_L is approximately 0.4 units greater than m_b. If the m_b scale saturates at 7.3, the M_L scale should saturate at about 7.7.

Table 1 gives the source region, its area, the calculated maximum-magnitude earthquake when the source area is equalized to 30000 km^2, and the calculated maximum-magnitude earthquake when the source area is equalized to 100000 km^2 for the southern California data. Note that the southern California area

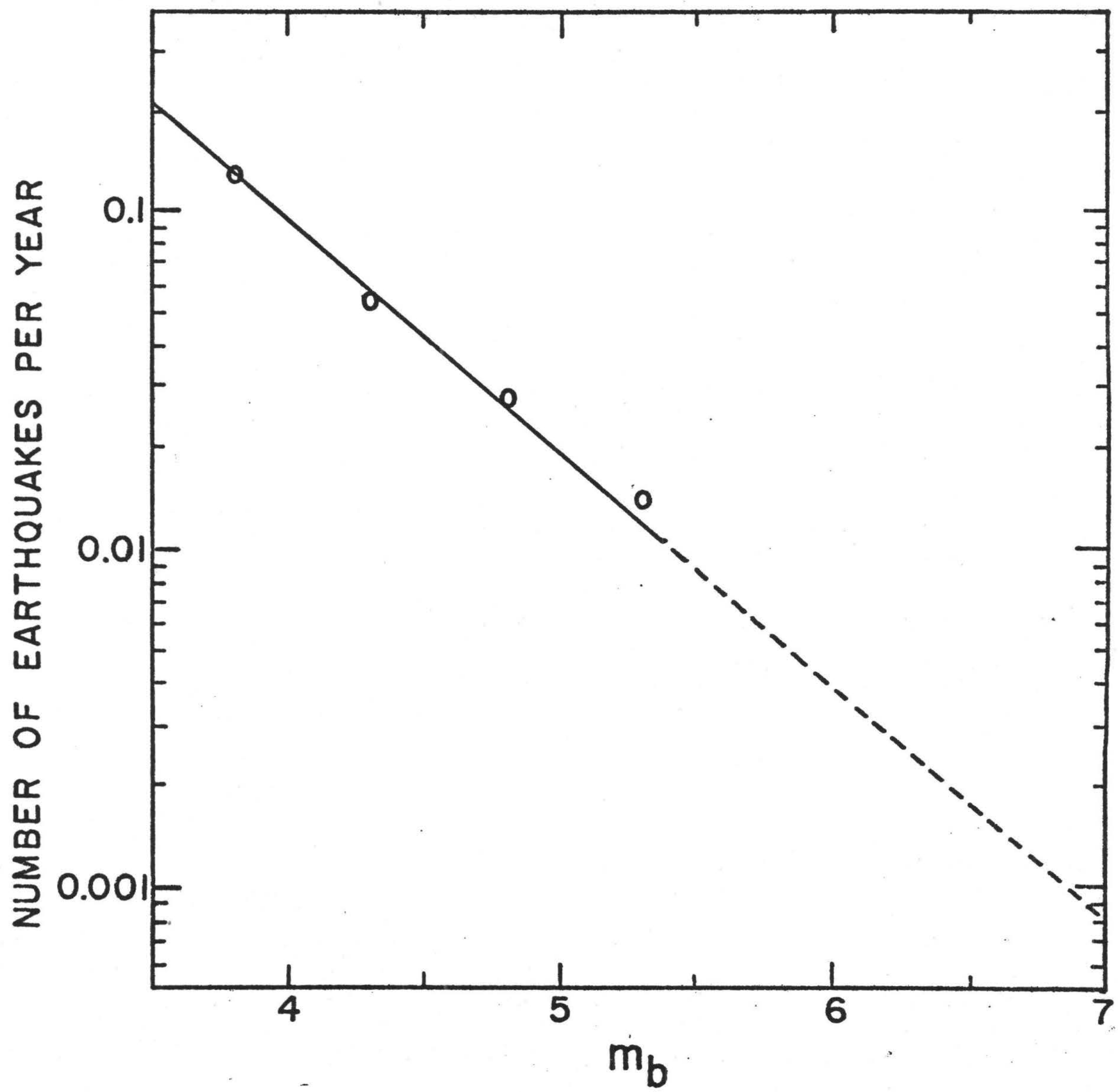

Figure 2. Cumulative magnitude-recurrence curve for the Charleston, S.C. source zone, excluding the 1886 earthquake and its aftershocks. The magnitude of the 1886 earthquake was m_b = 6.6 to 6.9, equal to that obtained when the curve is extrapolated to a 1000-year return period. The data used to plot the curve are taken from Tarr (1977).

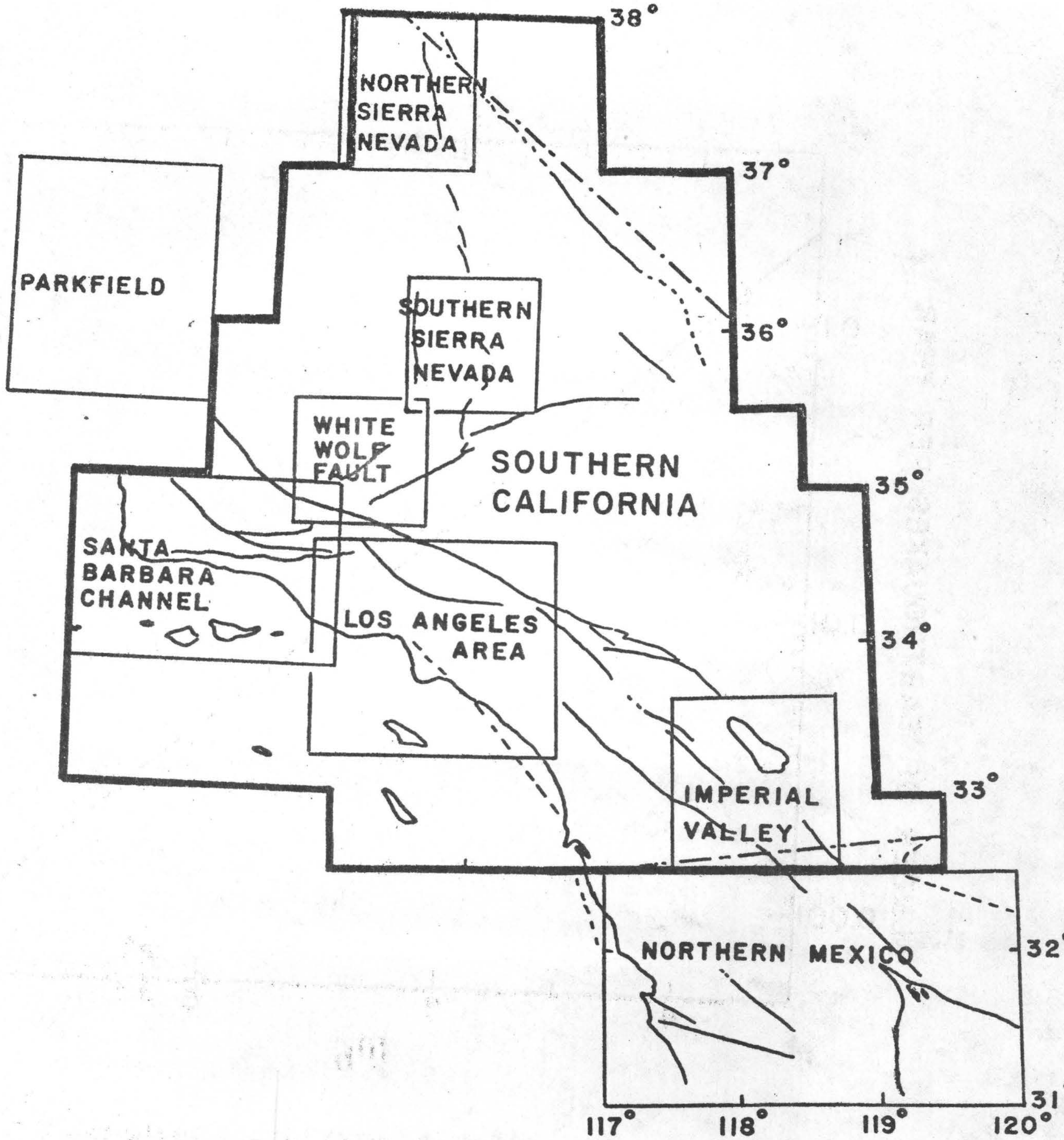

Figure 3. Seismic source zones for southern California and northern Mexico, as defined by Hileman et al (1973).

TABLE 1

ESTIMATES OF MAXIMUM-MAGNITUDE EARTHQUAKES FOR SOUTHERN CALIFORNIA AND NORTHERN MEXICO

Region	Land Area (km^2)	$M_{L,max}$ for 30000 km^2 area of equalization*	$M_{L,max}$ for 100000 km^2 area of equalization*
Southern California area	238,600	7.7	7.7 (200)
Los Angeles area	26,622	7.7 (500)	7.7 (500)
White Wolf fault area	8,400	7.2	7.2
Santa Barbara Channel	10,200	7.1	7.1
No. Sierra Nevada	10,600	7.6	7.6
So. Sierra Nevada	8,450	7.7 (1000)	7.7 (1000)
Imperial Valley region	15,102	7.7 (200)	7.7 (200)
Parkfield area	15,000	7.7 (400)	7.7 (400)
No. Mexico	47,200	7.7 (150)	7.7 (90)

* The number in parentheses indicates the recurrence period for the maximum possible earthquake, namely one of M_L = 7.7.

includes the Los Angeles, White Wold, Santa Barbara, northern Sierra Nevada, southern Sierra Nevada and Imperial Valley source regions, so that its maximum-magnitude earthquake should be as large or larger than that of any of those smaller areas. From the table it can be seen that a number of those areas are capable of producing the largest possible earthquake (M_L = 7.7). They differ, though, in the recurrence time of such an earthquake. The least recurrence time, implying the most active region in the southern California area, is 200 years for the Imperial Valley region. If the recurrence curve for the entire southern California area is equalized to 30,000 km^2, the M_L = 7.7 event has a recurrence period of 1000 years, larger than that of several of the subareas and thus not acceptable. But if the area of equalization is 100,000 km^2, the recurrence time for an M_L = 7.7 event is 200 years, the same as for the Imperial Valley and less than for all other subareas. The 100,000 km^2 area of equalization, thus, leads to an acceptable value for the maximum-magnitude earthquake for the entire southern California area. Nuttli and Herrmann (1978) also found that they needed a 100,000 km^2 area of equalization when they attempted to estimate the maximum-magnitude earthquake for the central Mississippi Valley, which is a large area that includes the New Madrid fault zone.

From Table 1 it can be seen that the northern Mexico area is the most active of the regions listed in the table. The recurrence time for an M_L = 7.7 earthquake is 150 or 90 years, depending on whether an equalization area of 30,000 or 100,000 km^2 is used.

Bolt and Miller (1971) gave magnitude-recurrence curves for three areas of northern and central California, based on data for the years 1962-1969. These are: 1) all earthquakes in central and northern California, including the Owens Valley, 2) the Cape Mendocino area, including the Gorda Escarpment and the Gorda Basin, and 3) the Coast Range region. Table 2 gives the estimates of the maximum-magnitude earthquake for the regions. As all three have areas less than 30,000 km^2, there is no need to equalize the source areas to 30,000 or 100,000 km^2.

Figure 4 shows two areas of Utah for which Smith and Arabasz (1979) gave recurrence curves based on data for the years 1962-1978. Table 3 gives the maximum-magnitude estimates for these regions. The data indicate that both regions, and particularly region III, are very seismically active areas.

Nuttli and Herrmann (1978) have delineated nine earthquake source regions in the central United States. Of these the most active, by far, is the New Madrid zone, whose maximum-magnitude earthquake already has been discussed. Figure 5 shows the location of the source zones. The residual zone is everything outside the eight outlined zones, and may be considered background seismicity. Table 4 gives the estimate of maximum-magnitude earthquakes for each region. Of those areas only the New Madrid appears capable of generating a truly major earthquake, although most are capable of moderate to major earthquakes.

East of the Appalachians the earthquake source zones are not so readily delineated, so it is difficult to assign maximum-magnitude earthquakes to that part of the country.

TABLE 2

ESTIMATES OF MAXIMUM-MAGNITUDE EARTHQUAKES FOR NORTHERN AND CENTRAL CALIFORNIA

Region	Land Area (km^2)	$M_{L,max}$*
Northern and central California	18,450	7.7 (350)
Cape Mendocino	3,470	7.7 (700)
Coast Ranges of central California	5,650	6.7

TABLE 3

ESTIMATES OF MAXIMUM-MAGNITUDE EARTHQUAKES FOR PORTIONS OF UTAH

Region	Land Area (km^2)	$M_{L,max}$ for 30000 km^2 area of equalization*	$M_{L,max}$ for 100000 km^2 area of equalization*
Region II' (includes Wasatch Valley)	31,200	7.3	7.3
Region III (southwest Utah)	49,950	7.7 (1000)	7.7 (600)

* The number in parentheses indicates the recurrence period for the maximum possible earthquake, namely one of $M_L = 7.7$.

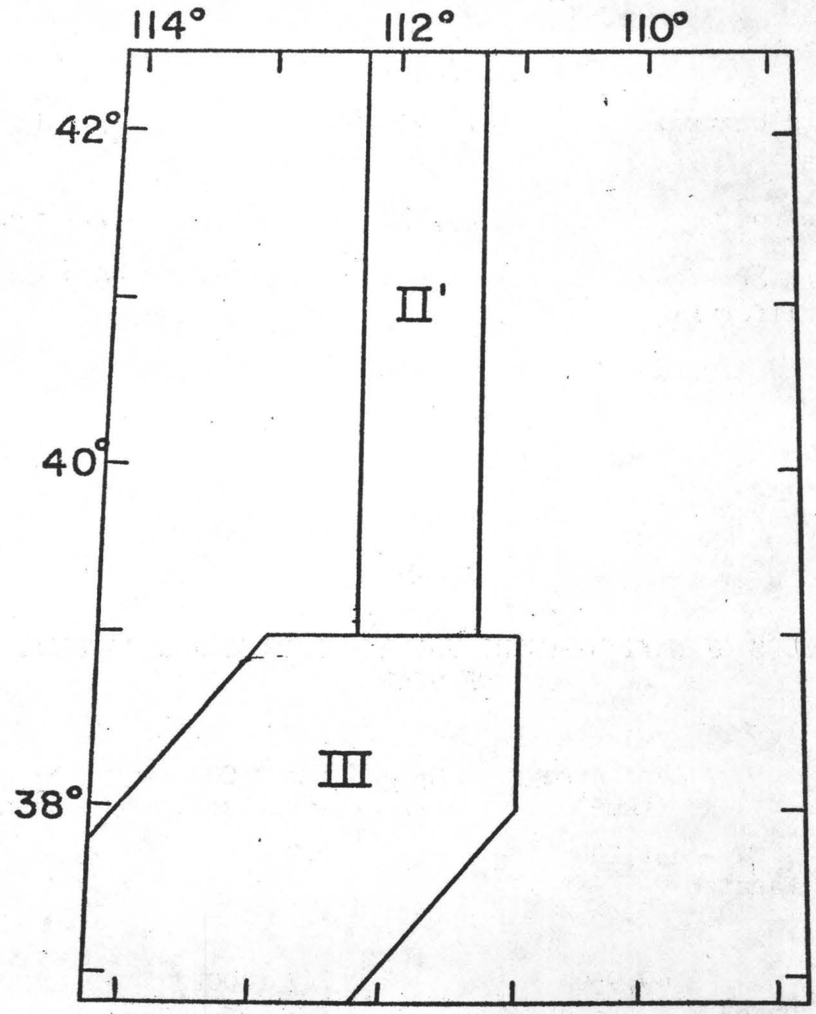

Figure 4. Seismic source zones for Utah, as defined by Smith and Arabasz (1979).

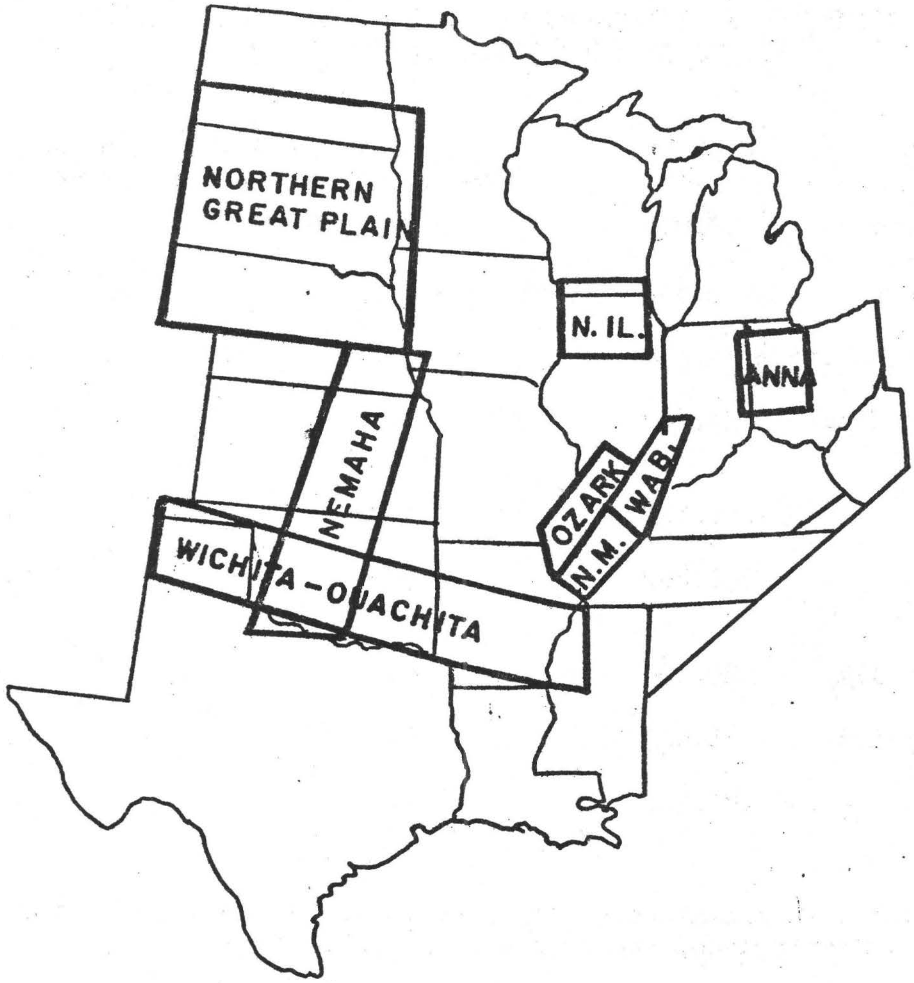

Figure 5. Seismic source zones for the central United States, as defined by Nuttli and Herrmann (1978).

TABLE 4

ESTIMATES OF MAXIMUM-MAGNITUDE EARTHQUAKES FOR SOURCE ZONES OF THE CENTRAL UNITED STATES

Region	Land Area (km^2)	$m_{b,max}$ for 30000 km^2 area of equalization*	$m_{b,max}$ for 100000 km^2 area of equalization*
New Madrid	15,000	7.35 (600)	7.35 (600)
Anna, Ohio	37,600	6.0	6.2
Northern Illinois	55,100	5.7	6.0
Northern Great Plains	426,700	5.2	5.7
Nemaha	206,100	5.7	6.2
Wichita-Ouachita	261,800	5.5	6.0
Wabash Valley	39,800	6.2	6.4
Ozark Uplift	36,600	6.4	6.5
Residual	6,185,000	4.5	5.0

* The number in parentheses indicates the recurrence period for the maximum possible earthquake, namely one of $m_b = 7.35$.

Throughout this paper either local magnitude, M_L, or body-wave magnitude, m_b, were used. Customarily for large earthquakes the surface-wave magnitude, M_S, or seismic moment, M_o, are used. For comparison purposes, an M_L of 7.7 or an m_b of 7.3 correspond roughly to an M_S of 8.6 and an M_o of 10^{28} dyne-cm or greater.

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The 1887 Earthquake in Sonora: Analysis of Regional Ground Shaking and Ground Failure

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Introduction

On May 3, 1887 a severe earthquake shook an area of about 1.6 million square km in northern Mexico and the southwestern United States. This earthquake ranks among the largest and most devastating seismic events in the western North America (exclusive of California), because of the 51 deaths, widespread damage to property and the associated surface faulting. A magnitude of 7¼ is estimated for this event from seismic moment.

Figure 1. Bavispe, Mx. (25 km SE of epicenter) Church destroyed by May 3, 1887 earthquake. The walls, two feet thick adobe, had stood for 200 years. Forty-two people were killed and 29, injured when the tile roof collapsed. (Photo courtesy of the Arizona Historical Society, Tucson, C.S. Fly Collection)

The 1887 earthquake can be used as a basic model for predicting intensities and damage from a large earthquake. Regional seismicity patterns and the presence of several Quaternary faults in Arizona, western New Mexico and the 1887 epicentral area indicate that a magnitude 7+earthquake should be considered as the likely maximum magnitude for this region. Since 1887, much urban development, increased population and water table changes in the seismically-affected area should be analyzed in order to better comprehend the risk of damage from repetition of a seismic event similar in size to the 1887 earthquake.

Research efforts during the past two years have focused upon collection and interpretation of historical reports concerning damage and other observed effects of the 1887 earthquake (DuBois and Smith, in press). Data from the investigation can be applied to the following questions:

Specific:
1. Estimated magnitude of the event.
2. Felt area size and shape.
3. Shaking intensity attenuation patterns.
4. Location of secondary hazards (i.e. rockfalls, liquefaction, fire).

General:
1. Problems inherent in commonly-used intensity scales.
    a. Assignment of intensities to ground fissures and rockfalls.
    b. High intensity - differentiating IX, X, XI and XII.
2. Application of the 1887 event, in particular, to the prediction of potential hazards to population in Arizona, New Mexico and Sonora, Mexico.

<u>Estimated Magnitude</u>

Seismic moment (Mo) is perhaps the most accurate measure of physical size of large earthquakes. It is not in common use, however, because it is more difficult to determine than magnitude. The moment can be calculated from observed parameters of the fault scarp using equation (1) taken from Brune (1968)

$$M_o = \mu S \langle D \rangle \qquad \text{(Eq. 1)}$$

where μ is the shear modulus, S is the fault area and $\langle D \rangle$ is the average displacement. A common value of μ for crustal rocks is 3.0×10^{11} dynes/cm^2. The area can be calculated from the fault length of 50 km and the down-dip fault width of 16 km determined from microearthquake studies (Natali and Sbar, 1980). The average displacement is 3 m based on field observations (Bull, personal communication, 1980). Use of these values yields Mo = 7.2×10^{26} dyne-cm.

Hanks and Kanamori (1979) derived an empirical relationship between magnitude and moment as shown in equation (2).

$$M = 2/3 \log M_o - 10.7 \qquad \text{(Eq. 2)}$$

For the 1887 earthquake data M would be either a surface wave magnitude (M_s) or local California M_L magnitude. Thus, M_s for the 1887 earthquake is about 7¼.

Analysis of Intensity

Hundreds of primary accounts were obtained from newspapers, scientific journals, Mormon diaries, pioneer journals, military archives and weather station reports, as well as from special collections of photographs, manuscripts and personal correspondence. Modified Mercalli intensity values were assigned to approximately 200 localities and isoseismal maps (Figs. 2 and 3) were then generated. Shaking intensities appeared to be higher on basin fill than bedrock. Therefore, high intensity ($\geq$ VII) "fingers" on the isoseismal maps tend to correspond with valley orientations. The influence of topography upon intensities was one parameter considered while drawing isoseismals.

Some difficulties were encountered in using the Modified Mercalli scale to determine intensity ratings for specific site reports. In the absence of detailed accounts of building damage, type of building material and construction, foundation conditions, etc., ratings of damage to structures were sometimes arbitrarily assigned from MM VIII to X. Likewise, reports of cracks or fissures from which water emanated were rated MM IX to XI. Differentiating IX, X and XI effects was problematic. Also, the Modified Mercalli scale was limited in assessing shaking intensity in areas where rockfalls and ground cracks occurred. Damage from liquefaction or landsliding may in fact have been restricted to narrow portions of saturated valleys and steep bedrock slopes respectively. In the process of regional intensity contouring, high MM values assigned to ground failure areas may bias the overall pattern of ground shaking intensity.

Felt Area Size and Shape

The estimated felt area for this earthquake is about 1.6 million km^2. Error in this value can be expected, due to the lack of data in the vicinity of MM I-III isoseismals. Population bias is especially evident southwest of the epicenter because of the Gulf of California.

The gross isoseismal pattern (Fig. 2) is elliptical and elongated in a southeasterly direction, a direction that corresponds with the NW-SE orientation of regional structures. Truncation of isoseismal contour patterns to the northwest may result from the structural boundary of the Colorado Plateau forming a partial barrier to wave propagation. The intensity IX and greater isoseismals shown in more detail in Figure 3 reflect the local north-south structural trends and the strike of the 1887 fault. Apparently, foundation material (soil vs. bedrock) and water table characteristics were significant factors in intensity attenuation locally. In southeast Arizona high intensity lobes follow valley orientations where unconsolidated deposits (mainly sand and gravel) often exceed a kilometer in thickness, and where the water table was within a few meters of the surface near stream channels.

Attenuation

A preliminary plot of the intensity of each locality vs. distance is shown in Figure 4. The scatter observed is typical, and is most likely due to differences in geologic setting at the sites and the inherent lack of

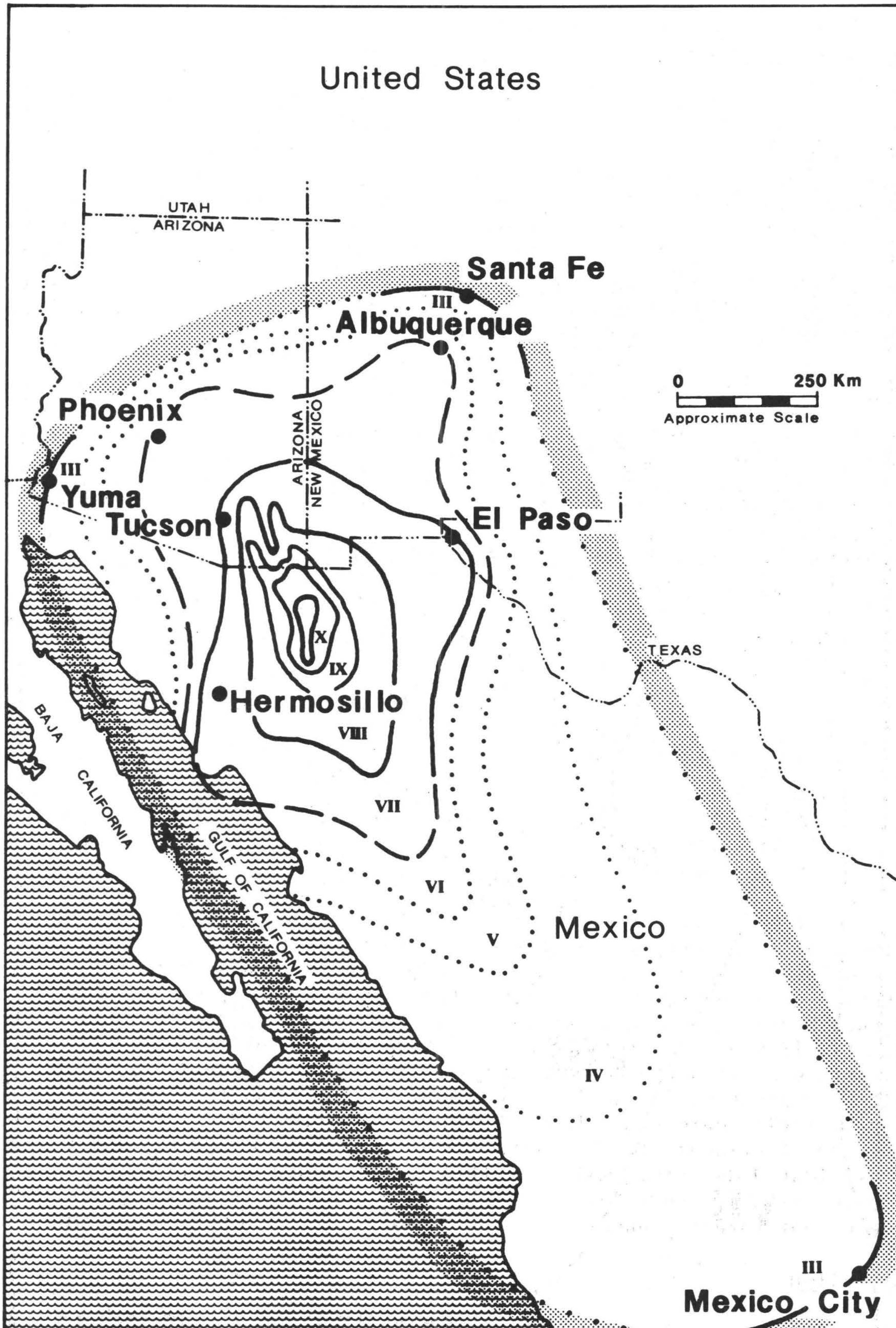

Figure 2. Isoseismal map of the 1887 felt area. Roman numerals depict Modified Mercalli intensity. Shaded zone indicates estimated limits of felt area. Dotted and dashed isoseismal contours show poorer control and reliability than the solid lines.

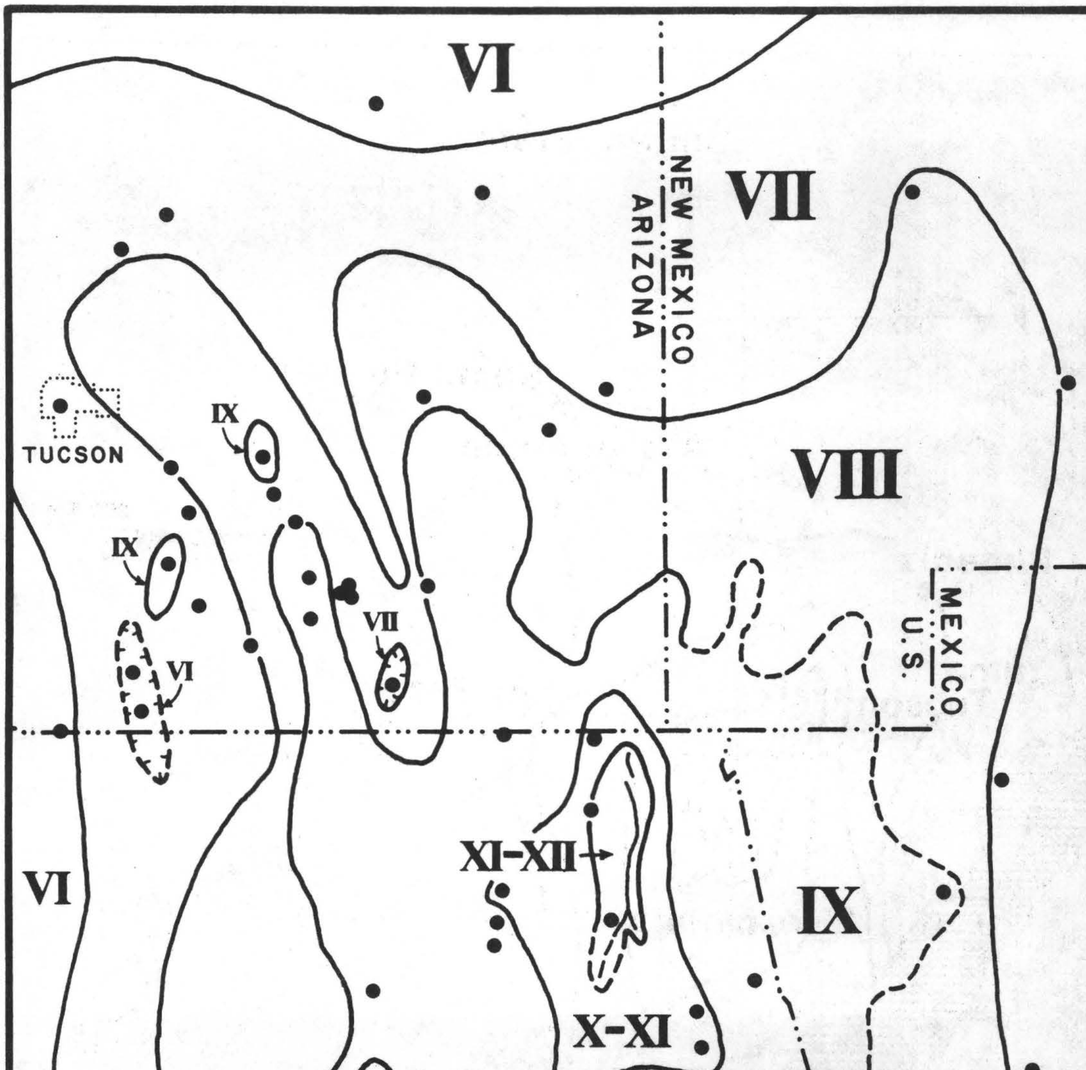

Figure 3. Isoseismal map of the epicentral region, May 3, 1887. The fault is shown in the highest intensity contour. Intensity data locations, which are depicted by dots, are generalized to represent local vicinities for which reports were found.

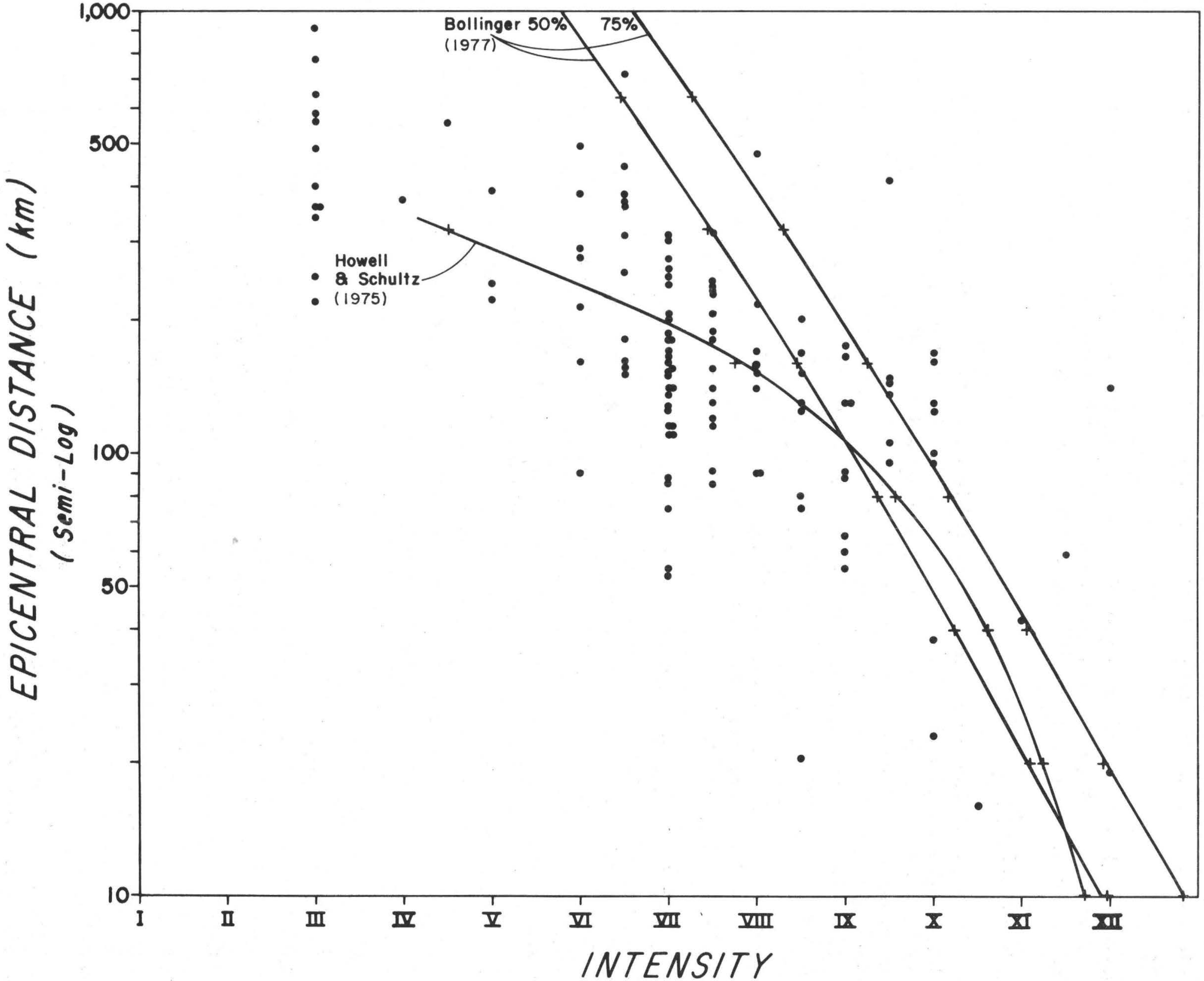

Figure 4. Intensity vs. distance data for 1887 earthquake. Attenuation curves are shown for comparison purposes. Bollinger (1977) used data from the Charleston, S.C. earthquake (1886). The other curve (Howell and Schultz, 1975) was developed from equation 9 for the San Andreas region.

precision in assigning intensity ratings. Regression curves determined by Bollinger (1977) for 50% and 75% fractiles for the 1886 Charleston earthquake and a curve from Howell and Schultz (1975) for the San Andreas province are plotted for comparison. The latter is a reasonable fit to the data.

A more detailed analysis of these data is warranted and will be pursued in the future. It is evident from Figure 2 that azimuthal variations in intensity exist. Analysis can also be done in terms of site conditions.

Secondary Effects

The influence of weathering and water table characteristics, lithology, geologic structure, and topography on seismically-induced hazards must also be considered in assessing seismic risk. Actual ground shaking intensity was often obscured by secondary effects of ground failure by either earth fissuring due to liquefaction or rockfalls and landslides. In order to assess the distribution and varying significance of these induced hazards, we mapped separately the reported hydrologic alterations, surface fissures and mass movements resulting from the earthquake (Figs. 5 and 6). Occurrences of fire (ignited by falling boulders) and flooding (from water escaping through earth fissures and new springs) represent additional hazards indirectly associated with the earthquake shock.

Widespread liquefaction, ground rupture, channel subsidence, and earthquake fountains were reported throughout the epicentral region - in San Bernardino, Fronteras, Bavispe, Yaqui and Sonora valleys (Fig. 5). Some of these effects were also reported in San Simon, Sulphur Springs, San Pedro and San Bernardino valleys of Arizona (DuBois and Smith, in press). Liquefaction appeared confined to areas near stream channels, where the water table was at depths less than 4-5 m. Flooding from fissures was observed at Batepito, Sonora and near Abbott's Ranch in Sulphur Springs valley, Arizona. Many of the reported fissures were 1-2 meters wide and continuous for many kilometers, in Arizona as well as Sonora. Since most of the population was concentrated in the valleys and travel routes mainly followed the stream channels, much of the ground failure resulting from the earthquake was probably observed directly and reported.

Secondary effects such as rockfalls and landslides were more likely to be observed from a distance. Many reports describe dust clouds, smoke and fire in surrounding mountains directly after the earthquake. Fresh debris at the base of the hills, boulder paths and brush fires were attributed to rockfalls triggered by the shaking. A few eyewitnesses were actually in the mountains with livestock at the time. They reported similar effects: rumbling and crashing of rocks as they fell, blinding dust and fires set off by sparks from bouncing boulders. The region affected by induced mass movement (Fig. 6) was larger than that of hydrologic alterations and earth fissuring.

Some difficulty was encountered in rating these reports on the Modified Mercalli intensity scale. For example, "incaving along river banks," according to the scale, is rated MM VII. However, the criteria listed for MM VIII and IX do not include progressively more intense mass movement

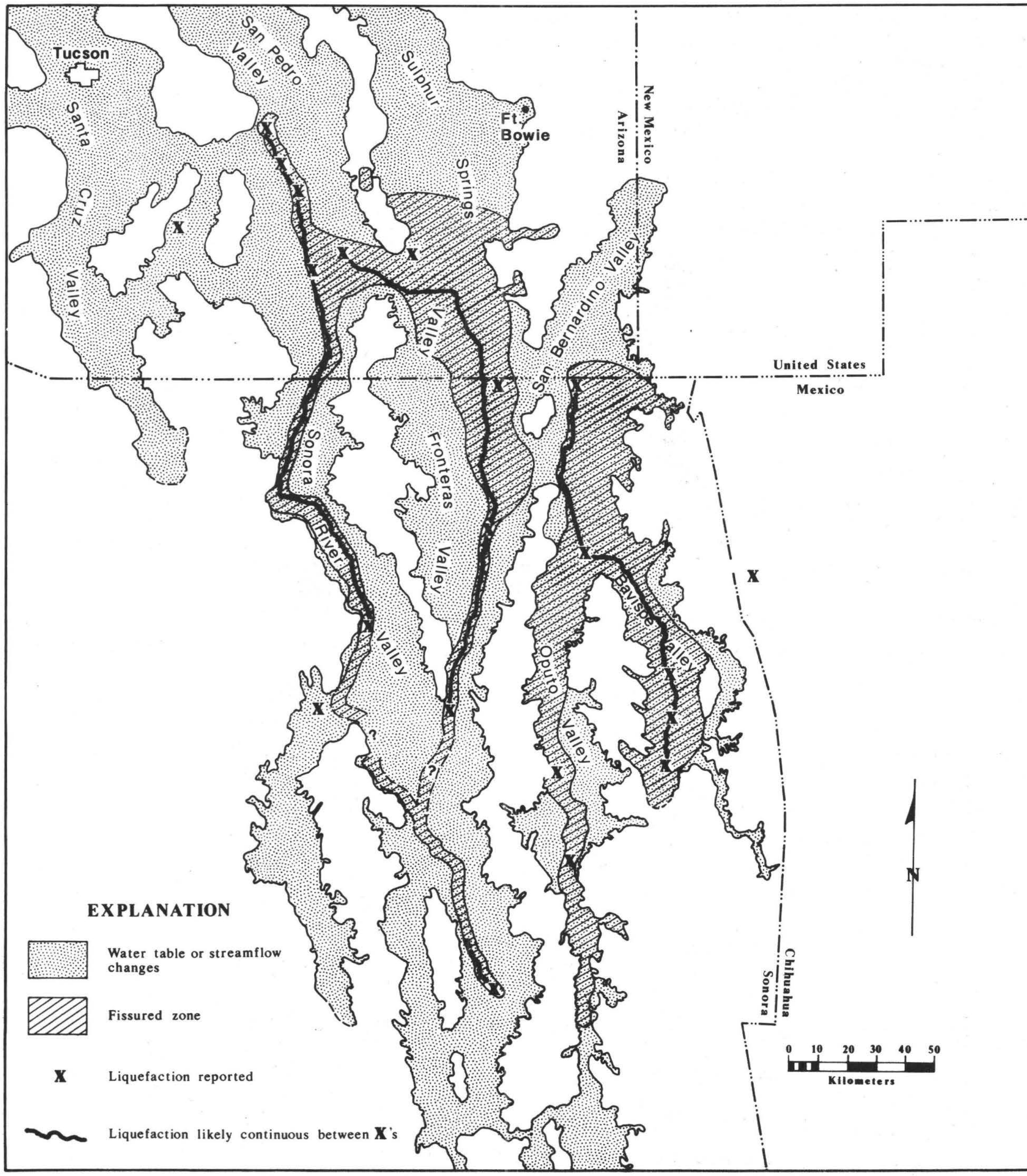

Figure 5. Liquefaction, ground fissures and hydrologic alterations associated with the May 3, 1887 earthquake in Sonora, Mx. (from DuBois and Smith, in press)

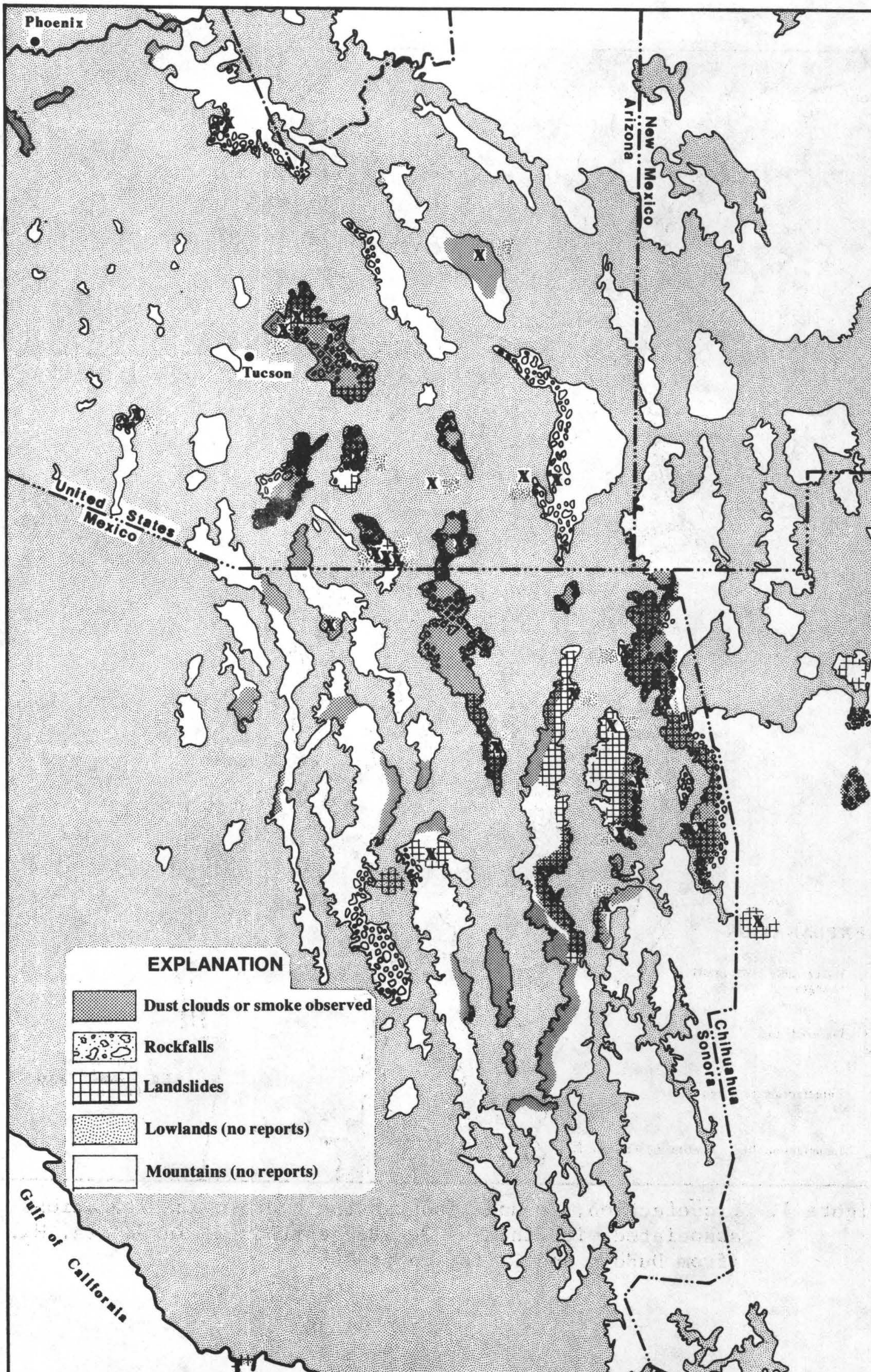

Figure 6. Mass movement induced by the 1887 earthquake in Sonora. "X's" indicate specific locations named in rockfall or landslide reports (from DuBois and Smith, in press).

effects. "Considerable landslides" rate MM X, "disturbances in ground many and widespread" indicates MM XI, and "landslides, falls of rock ... slumping ... numerous and extensive" rate MM XII. Differentiation between MM VIII and IX, X, XI and XII, based upon these effects, is similarly vague on the Modified Mercalli scale and thus promotes highly subjective ratings by individual investigators. Brazee (1978) developed a more detailed intensity scale which lists landslides and rockfalls under VII or greater effects. Although his scale includes cracking of wet ground as intensity VII and riverbed subsidence at VIII, actual liquefaction and widespread ground cracks indicate IX or higher intensity. For the purposes of isoseismal preparation, modifications presented by Brazee (1978) were followed whenever the Modified Mercalli scale did not include observed effects from the 1887 earthquake. Many investigators (Youd, Keefer, Harp, Steinbrugge, Wilson, pers. comm) have suggested that secondary geologic effects, such as liquefaction and mass movement, often occur at lower intensities (generally MM VI) when responses of buildings and objects only have been used as intensity criteria. A more standardized, and preferably a quantitative scale for geologic effects, needs to be developed so that the meaning of high intensity values is clarified. For example, it is difficult to determine from isoseismal maps if high ground acceleration occurred in high intensity regions or if poor foundation material, i.e. saturated sands, was more significant in causing damage. The distinction is important in building design.

Discussion

The Holocene fault scarp generated by the 1887 earthquake is of comparable length and displacement to known Pleistocene fault scarps in southeastern and northwestern Arizona (Soule, 1978; Bull and Menges, pers. comm.). Thus, it seems reasonable to treat the 1887 shock as the maximum possible earthquake for the seismically-active zones in Arizona, exclusive of Yuma. The Yuma area is very near the San Andreas system and thus should be treated separately for earthquake risk.

The isoseismal map for the 1887 earthquake (Fig. 2) provides a model for estimating the impact of another earthquake of this size on Arizona. Since the occurrence of that shock, the population (now over two million) and urban development in Arizona have increased dramatically. A scenario of probable damage associated with an earthquake similar to the 1887 event follows. Initially, consider a repeat of the 1887 earthquake today. First, note that the most intense effects of the 1887 earthquake were observed in the valleys. In southern Basin and Range country, nearly all cities, roads and utility lines lie in the valleys. Liquefaction and ground fissuring might be prevalent in many towns within about 100 km of the epicenter. Differential basin subsidence might also be triggered in Tucson, for example, because of previous extensive groundwater withdrawal. Urban development is occurring in bedrock areas at the base of the 3,000 m Santa Catalina Mountains. Rockfalls would be expected to cause significant damage there and in other similar areas of southern Arizona. In terms of shaking damage to buildings, private residences might be expected to suffer extensive damage during a large earthquake. In seismic zone 2 (Algermissen, 1969) which covers most of Arizona, seismic design standards are required for all stuuctures except for single family housing. Most private dwellings are constructed almost

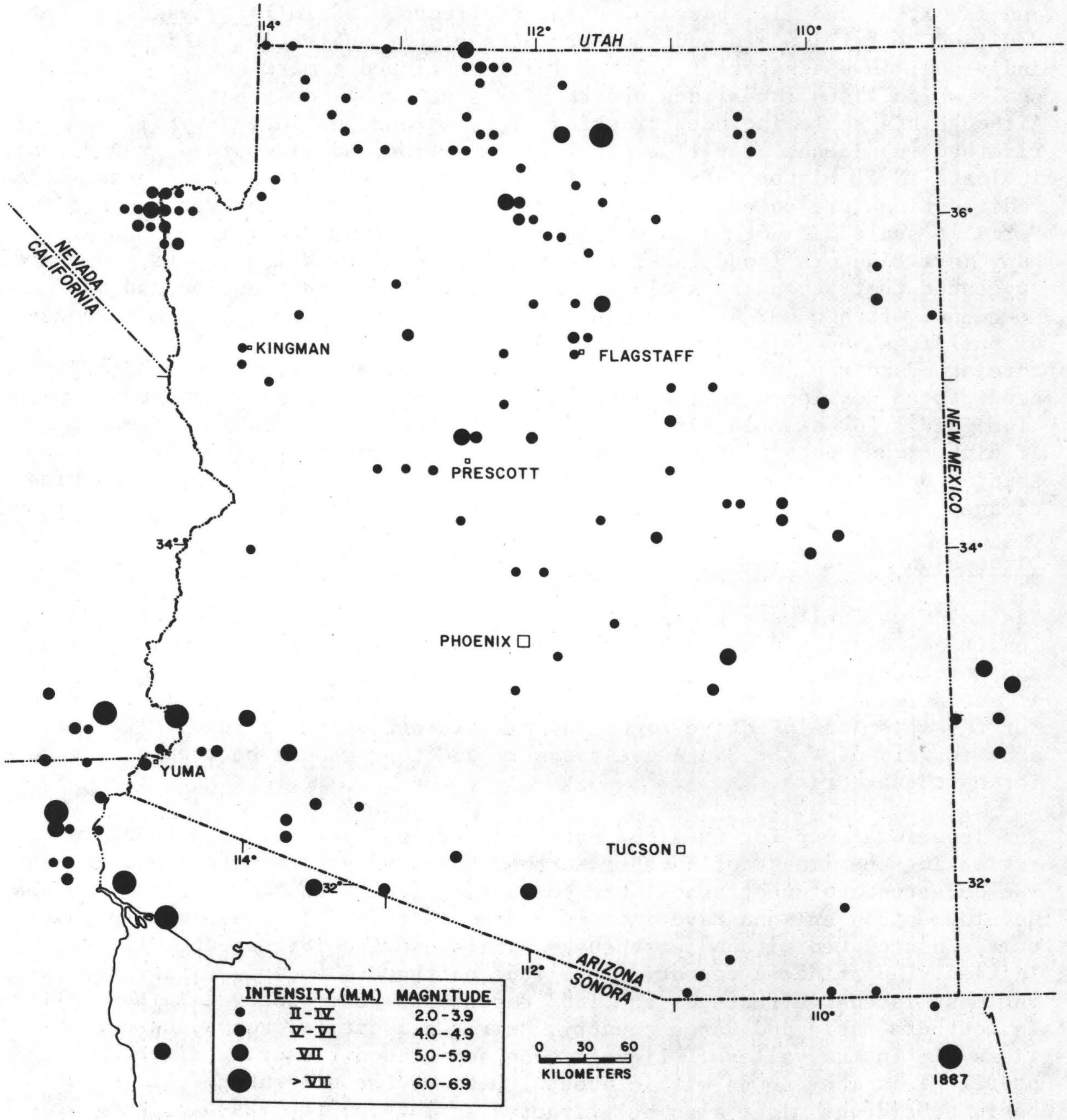

Figure 7. Preliminary map of historical earthquake epicenters in Arizona. Note: several of the epicenters SE of Yuma may have been mislocated by California networks.

entirely of unreinforced masonry. Therefore, homes in Tucson (which felt MM VII effects in 1887), and in other cities in southeastern Arizona, would likely suffer damage.

This scenario is fortunately not one of overwhelming disaster. A more critical situation would develop if an earthquake occurred in or near one of the major population centers of Arizona. Phoenix, Tucson and Flagstaff lie within a broad northwest-southeast trending belt of seismicity across the state (Fugro, 1975; Sumner, 1976). A magnitude 7 earthquake would be totally devastating to any of them. Near Phoenix there are a number of large dams, the construction of which predate development of seismic design standards. Failure of one of these dams would cause widespread flooding throughout the Phoenix metropolitan area. Even a magnitude 6 would have a great impact near the cities because most agencies are not prepared to handle earthquake effects. The Arizona Division of Emergency Services, however, is considering the impact of a major earthquake on the state and may be able to provide aid should such an event occur.

Much information on seismic hazards is necessary before responsible agencies can make rational decisions regarding seismic safety in building codes and emergency preparedness. More accurate earthquake locations, estimates of recurrence rates in various parts of the state, and updated seismic zonation are all needed for Arizona. Applicable data are being gathered by means of short-term microearthquake studies, analysis of historical earthquakes and geomorphic analysis of fault scarps. These studies should be augmented by longer term seismic recording in critical areas with a denser network of seismic stations than presently exists (for example, there are no seismic stations between Tucson and the 1887 epicenter and none between Tucson and Prescott). Specific studies should be undertaken to develop an attenuation model for the different geologic provinces in and around Arizona. Attenuation is an important parameter in the estimation of site ground motion. Strong motion instruments should be installed near the 1887 fault scarp, since this is one of the most active areas of the region. Two felt events have occurred there in the last three years.

Conclusions

In summary, a magnitude of $7\frac{1}{4}$ has been calculated from studies of the 1887 northern Sonora earthquake. Intensity analysis demonstrates the problems of using the Modified Mercalli Intensity scale, and its modifications, in a sparsely populated region. More work must be done to standardize evaluations of ground failure and other geologic effects at the higher intensities (IX - XII). It is important to recognize the great impact of secondary geologic effects when considering potential damage from earthquakes, based upon the 1887 example. Finally, the 1887 event can be used in predicting damage should future major earthquakes occur in this region.

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NEW PROBABILISTIC HAZARD MAPS FOR THE UNITED STATES

A PROGRESS REPORT

by

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One of the responsibilities of the Geological Survey under the National Earthquake Hazards Reduction Program (Executive Office of the President, June 22, 1978) is the preparation of earthquake hazard maps on a regional and national scale. The current goal is to complete new probabilistic national maps of ground acceleration and velocity for the country by July 1981. The basic work involved is revision, where necessary, the 1976 probabilistic acceleration map of the continguous United States (Algermissen and Perkins, 1976) and preparation of a velocity map for the contiguous United States and new acceleration and velocity maps of Alaska and Hawaii. New probabilistic acceleration and velocity maps have already been prepared for Alaska (Thenhaus et al., 1980), the Pacific northwest (Perkins et al., 1980), western California (Thenhaus et al., 1980) and the eastern coastal area (Perkins et al., 1980). These maps were prepared in a project for probabilistic ground motion estimates. A special effort has been made to develop an expanded data base of seismotectonic information for use in the delineation of seismic source zone for hazard mapping. A series of workshops have been conducted to develop seismotectonic data and concepts that may have application in the probabilistic hazard mapping process. These workshops (as discussed in the paper by Bucknam and Anderson) have synthesized a considerable amount of useful information. Problems arise in devising the most effective methods of incorporating the seismotectonic data into the hazard maps. Various approaches to incorporating the seismotectonic data into one hazard mapping process are outlined together with the consequences (in terms of the resulting ground motion values) of each of several approaches.

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Seismic Zoning in Canada - Some Modifications to Current Maps

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INTRODUCTION:

Provisions for the seismic resistant design of specific buildings were first introduced in the National Building Code of Canada (NBC) in 1953. The seismic zoning map in this early edition of the code recognized the existence of significant earthquakes in Canada by defining zones of maximum risk around these few events. The remaining area of the country was subdivided into lesser seismic to aseismic zones in three stages. A modified seismic zoning map was introduced in the 1970 edition of NBC using new methods of analysis and an expanded historical and current seismicity data base. The methodology of seismic risk calculations is being revised once again for possible inclusion in a future edition of NBC. Research is concentrated on three topics that, when completed, should lead to new seismic risk maps that incorporate all available and relevant geological and seismic information to provide estimates of strong ground motion necessary for the safe design of buildings in the seismic areas of Canada.

Attenuation Functions

The 1953 NBC map did not require a ground motion parameter as the country was simply divided into four zones on a subjective basis with each having a factor assigned that was considered representative of seismic risk in the zone. These factors were included in the design of the structure by way of the equation for base shear. For the 1970 edition of NBC, zone boundaries were defined by a statistical determination of expected ground motion, and hence an attenuation function was required for peak horizontal acceleration. As no strong motion records were available in western

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Contribution from the Earth Physics Branch #885.

Canada it was necessary and reasonable to apply the few strong motion ground data available from California to western Canada. A function was developed from these data relating peak horizontal ground acceleration, magnitude and epicentral distance. As the calculation was made from available data it was said that the amplitudes were appropriate to sites with an average foundation material. The word "average" has since been translated into "firm soil". At the time, it was recognized that ground amplitudes from earthquakes were attenuated much less rapidly in eastern Canada than in the western part of North America. Consequently a function was developed for the east which related intensity, the only observed dependent variable available, to magnitude and epicentral distance. East and west were loosely defined as there were too few cataloged earthquakes in central Canada to require zoning consideration. The duration of strong shaking, and the focal depth of the earthquakes were not considered in the calculation, and data for determining near field amplitudes did not exist.

Since the calculations were made for the 1970 edition of NBC, many more strong motion records have been obtained in the western U.S. and a small amount of data has been obtained in western Canada (Weichert and Milne, 1980), but no significant data have been obtained from eastern Canada.

A common form for relating ground amplitude, A, and the relevant parameters is

$$A = b_1 \, e^{b_2 M} \, R^{-b_3}$$

where M is magnitude, R is hypocentral distance and b_1, b_2 and b_3 are empirical constants to be determined for the ground motion parameters and regions in question. A wide range of the b constants can be found in the literature depending on the data base used and the method of analysis. In some studies a constant (e.g. 25 km) has been added to the hypocentral distance in regression analysis, which has a large effect on the b_1 and b_3 constants. Milne (1977) using all the western United States data available to 1976 found values of b_1, b_2, and b_3 to be 0.04, 1.0 and 1.4, respectively, for peak acceleration in units of g, and 0.58, 1.17 and 1.2, respectively, for peak velocity in units of cm/sec.

A study is underway to derive new attenuation functions for Canada using results from the literature based on more recent data sets. The primary functions for peak acceleration and peak velocity are being developed for western Canada, and extrapolations to the east are being made on the basis of empirical comparisons of intensity attenuation in the two regions and other theoretical considerations. It is planned to produce seismic risk maps of both peak acceleration and peak velocity to provide the designer with ground motion information in both high and mid frequency ranges, respectively.

Peak horizontal ground acceleration data are the easiest and most reliable parameters to use for contouring maps for defining seismic zones. However, peak acceleration is not necessarily the ground motion parameter which provides realistic amplitudes for input to engineering designs. For instance, peak acceleration is usually associated with high frequency elastic waves, whereas modern tall slender buildings are out of this frequency range. A complementary seismic zoning map based upon peak horizontal ground velocity which is associated with lower frequency ranges can provide design criteria for these structures. The duration of maximum shaking also appears to be related to the damage suffered by a structure. Some relatively low magnitude earthquakes have produced high (near 1.0 g) accelerations for one or two cycles, but this duration of maximum shaking is so short that the effective acceleration level is much lower. Nuttli (1979) shows that sustained acceleration (or velocity) is a better figure, and Hasegawa shows that sustained acceleration (or velocity) attenuation curves are a constant multiplier lower than peak. Hasegawa also shows that the velocity response data are also a constant multiplier less than peak acceleration. Thus any of these may be used as alternates to peak values for design purposes, but the relative positions of seismic zones on a map is probably independent of the choice of parameter, and the zones can be drawn by using the most readily available parameters, that is peak velocity or peak acceleration.

Seismicity Patterns

The 1953 seismic zoning map was drawn by using epicentres of significant earthquakes to define large active areas which in turn became seismic zones. The 1970 zoning map was based upon the assumption that the future areal distribution of earthquake epicentres would reasonably follow the patterns of the historical past, and further that the time distributions could be statistically determined. Thus a new significant earthquake near the location of a previous event did not perturb the map significantly, but a similar earthquake at a new location could. Thus the 1970 map is stable for those regions where there is a good representative sampling of historical earthquakes but becomes progressively less so for other regions.

Cornell (1968) has proposed a different approach for the statistical calculation of earthquake risk which requires the identification of seismic source zones defined using the available geological, geophysical and tectonic knowledge together with the historical seismicity. Within Canada, the process of defining the source areas is continuing. The eastern seismic zones are described in an earlier paper (Basham et al., 1979). In the Arctic region, the distribution of epicentres combined with a tectonic/ geological map were used to produce a map of seismic source zones but it has not been possible, at this stage in the research, to relate source areas to active faults except in the SW Yukon.

The west coast region is subdivided into source areas, again partly using the distribution of earthquake epicentres, but in addition by reference to tectonic regimes. The spreading ridges and connecting transform fault formations clearly define an offshore area. The Queen Charlotte transform fault is a separate source of earthquakes. Northern Vancouver Island experiences severe, shallow strike slip earthquakes. Deep earthquakes are in the Puget Sound Basin, but this zone is defined as extending north to the 49th parallel of latitude, and to 40 km west of Victoria. Around and over this zone, shallow events occur at a lower level of probability.

The frequency of occurrence of earthquakes in each source area is usually expressed in the form:

$$\log_{10} N = a - b.M$$

where N is the annual number of earthquakes of magnitude M or greater. Weichert (1980) has discussed the maximum likelihood method of solving for a and b. In the zones which have been identified in Canada, most of the values of b are in the usual range of 0.7 to 1.0. However the northern Vancouver Island zone has a value as low as 0.4 owing to the absence of small earthquakes in recent years, and to the absence of earthquakes associated with the 1946 M 7.3 event in that source area.

The statistical calculation of risk requires that an estimate be made of the maximum magnitude earthquake in each source zone. This in many cases is a subjective selection, but there are some areas where fault dimensions and historical seismicity can be used to arrive at a reasonable value. For example, in the western offshore region the transform faults are small but very active, and the maximum historical earthquake has a magnitude of 6.5, in good agreement with estimates based on maximum fault dimensions.

Statistical Calculations

For the 1970 edition of the code, when earthquake epicentres were treated as point sources of uniform focal depth, peak horizontal ground acceleration values were calculated for a grid of sites across Canada using the attenuation curves available at that time. Gumbel's extreme value approach was applied to the set of largest annual amplitudes at each site, assuming a Gumbel Type II distribution. That is, no upper limits were applied. By this method, the peak horizontal acceleration with an annual probability of exceedance of 0.01 was calculated according to the requirements established for NBC 1970. During the research for the zoning methods eventually used in NBC, Milne and Davenport (1969) developed what was termed an average amplitude method as a companion estimator of ground motion. It can be argued (Weichert and Milne 1979) that with a complete data set, the latter is a more robust approach. For critical structures, where an annual probability of exceedance of at least an order of magnitude less than 0.01 is desired, neither of these approaches is entirely satisfactory without some limiting function on the amplitude of ground motion.

The new zoning map of Canada is being developed based upon the method developed by Cornell (1968) and used by Algermissen and Perkins (1976) to produce a risk map for the United States. McGuire's program (1976) is being used to compute strong motion parameters at sites across Canada for various probabilities of exceedance, including both the 0.01 per annum and 10% probability of exceedance in 50 years.

The selection of the probability to be used for the next edition of the National Building Code of Canada is important. NBC 1970 used an annual probability of exceedance of 0.01, in keeping with the philosophy that the level of ground motion should be such that minor damage can occur if the design earthquake happens, but that collapse will not occur if a larger event occurs. The level of risk chosen should be a reasonable value to accommodate the ability to design a structure to resist this ultimate earthquake without collapse without impractical cost levels.

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Comparative Ground Response Studies in Los Angeles Using NTS Nuclear Explosions and San Fernando Earthquake Data

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INTRODUCTION

Nineteen nuclear explosions have been recorded at about 100 sites in the Los Angeles region, including 28 strong motion stations, in order to study site response and explore the feasibility of seismic microzonation using a nuclear explosion data source (figs. 1-5). Site transfer functions and their mean values over several period bands have been computed for each recording site using stations at CIT, Palos Verdes Estates, and Griffith Park Observatory as the base rock sites. Alluvium-to-alluvium spectral ratios have also been computed in certain cases. An extensive geologic data base has been collected for the region using well-logs, engineering boreholes, and field studies. Near-surface compressional and shear wave studies have also been conducted using shallow boreholes at about 69 of the recording sites (Gibbs and others, 1980; Gibbs, personal communication).

In this study we discuss only that portion of the data set that addresses the comparison between ground response induced by earthquakes and nuclear explosions (table 1). Underground nuclear explosions (UNE) at the Nevada Test Site have been recorded at 28 strong motion sites in the Los Angeles region that also recorded the 1971 San Fernando earthquake in anticipation that the method of computing site transfer functions (STF) using UNE could be tested by comparison of computed STF's from the same station pair from both source types. Stations have been grouped geographically when computing spectral ratios (SR) because in some cases the earthquake SR's are contaminated by source and path effects when the azimuth and/or epicentral separation is too large. Station grouping has in some cases restricted the SR's to be formed from alluvium-to-alluvium site pairs; these pairs are not to be construed as STF's. An appropriate estimate of the site transfer function for each station has been given by Rogers and others (1980), where the nuclear derived SR's were formed using CIT as the base station. CIT is a site underlain by granitic rock. This station was not used as one of the base stations in this study because it is believed that the 1971 earthquake record contains source or path characteristics not present in the other strong motion records (Rogers and Hays, 1978). Because CIT was the only rock site that was recorded for all the nuclear events and some geographical groups contain stations that were recorded for several different nuclear explosions, the calculation of a SR using the group reference stations becomes more complicated. The lack of common nuclear events for a geographical group required that, in some cases, nuclear ratios first be formed using CIT as the base site ($STF^{sta\ i}_{cit}$); then the ratio to the geographical group reference station is formed by:

$$_{nuc}SR^{sta\ i}_{ref} = \frac{STF^{sta\ i}_{cit}}{STF^{ref}_{cit}} \quad .$$

This additional step unavoidably introduces some variability into the results. The earthquake SR's are computed straightforwardly by:

$$_{eq}SR^{sta\ i}_{ref} = \frac{S^{sta\ i}}{S^{ref}}$$ where S is the Fourier spectra at each site.

A complete discussion of the computational procedures and the statistical stability of the estimates of SR and STF is given by Rogers and others, (1980), and Rogers and others, (in preparation). This paper is restricted to a discussion of the results.

RESULTS

Comparison of the earthquake- and nuclear-derived SR's is shown in Figures 6-10. Figures 6 and 7 show the effect of grouping the data according to geographical region. For instance, stations HOI, 15250, and 15107 indicate nuclear and earthquake SR's that are more closely matched when the base station used is the nearby 14724 site than when the base station is GOC. Likewise, in Figure 7, stations in Long Beach show an improved comparison for data from the two source types when nearby station PVR is used as the base station. Rogers and others (in preparation) have shown that if the two SR's are within a factor of 5 at any given period, they are statistically equal at the 95-percent confidence level. The greatest portion of the variance contributing to the width of this confidence interval arises from the earthquake SR's which are considerably less stable than the nuclear data. On this basis, it can be seen that the two types of SR are statistically equal at individual periods in every case. It is possible to reduce the variance in the data to obtain a more meaningful comparison. By one method we compute means of the SR's over various period bands and plot scatter diagrams of these means as shown in Figures 11-14. The period bands used here are: total, 0.2-10.0 s; short, 0.2-0.5 s; intermediate, 0.5-1.0 s; and long, 1.0-3.3 s. Computing the correlation coefficient (r) for these scatter diagrams produces the values 0.69 (total), 0.65 (short), 0.85 (intermediate), and 0.41 (long). The hypothesis that r is drawn from a population with zero correlation can be rejected at the 1-percent significance level in every case. We also compute the ratio SR_{eq} to SR_{nuc} (R). If the two SR's are equal, R should equal 1. Again in order to decrease the variance in tests for R, we computed $\overline{R}$ over several period ranges. Table 2 shows the 95-percent confidence bands when there is no "drop-out" (Rogers and others, 1980) in the SR's, and the maximum number of independent estimates is available. If the confidence intervals are computed using the actual number of independent spectral estimates comprising $\overline{R}$, we find that 91 percent of the values in the total-period band are statistically equal to one; 89-95 percent of the short- and long-period values equal one, and 100 percent of the intermediate values equal one.

In conclusion, the nuclear and earthquake SR's are statistically equivalent with a few exceptions. We ascribe the observed scatter in the comparison of the mean SR's to two principal sources: (1) to the inherent instability in the SR (primarily the earthquake SR) that is related to the number of degrees of freedom in the spectra comprising the ratio; (2) to source and path effects remaining in the earthquake SR's even after they are grouped. These problems make it difficult to verify the proposed technique. The statistically significant results that have been found, however, are strong evidence that this method is a valid tool for estimating the site transfer functions.

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Rogers, A. M. and Hays, W. W., 1978, Preliminary evaluation of site transfer functions developed from earthquakes and nuclear explosions, in Proceedings of the Second International Conference on Microzonation for Safer Construction--Research and Application, v. 2, p. 753-764.

Rogers, A. M., Borcherdt, R. D., and Covington, P. A., in preparation, An evaluation of a method of seismic zonation: Comparative ground response studies in Los Angeles.

Table 1.--Station Abbreviations, Coordinates, and Cal. Tech. Strong Motion Station Codes

Station Name	Abbrev.	Cal. Tech. Strong Motion Station Code	Latitude (degrees)	Longitude (degrees)
Cal. Tech. Seism. Lab	CIT	G106	34.145	118.170
Holiday Inn	HOI	C048	34.220	118.471
Farmer's Ins. Bldg.	FIB	E072	34.062	118.331
Glendale Mun. Bldg.	GMB	F088	34.147	118.247
Century City, 1901 Ave of the Stars	CEN	R249	34.060	118.417
Athenaeum	ATH	G107	34.138	118.120
Millikan Library	MIL	G108	34.137	118.125
Hollywood Storage Bsmt.	HSB	D057	34.090	118.341
Griffith Observatory	GOC	0198	34.118	118.299
646 S. Olive Street	646	F098	34.047	118.254
800 W. First BH Towers	800	Q241	34.056	118.250
Long Beach Ter. Island	LBT	0205	33.756	118.233
Long Beach Utility Bldg.	LBU	0204	33.769	118.194
Palos Verdes Estates	PVR	N191	33.800	118.387
1760 Orchid Street Holiday Inn	1760	Q236	34.104	118.339
3838 Lankershime Blvd.	3838	L166	34.139	118.355
14724 Ventura Blvd.	14724	Q233	34.152	118.453
15250 Ventura Blvd.	15250	H115	34.153	118.462
6464 Sunset Blvd.	6464	R246	34.098	118.329
420 S. Grand	420	K157	34.051	118.252
445 Figueroa Street	445	C054	34.053	118.256
611 W. Sixth Street	611	G112	34.049	118.254
Lake Hughes #4	LH4	J142	34.648	118.481
Lake Hughes #1	LH1	J141	34.673	118.429
Lake Hughes #9	LH9	J143	34.608	118.562
Cal. State Univ., Long Beach	CSU	N196	33.777	118.111
Tishman Airport Ctr.	TAC	S267	33.947	118.385
15107 Vanowen Street	15107	J145	34.195	118.461

Table 2.--95-Percent Confidence Interval Factors for $\overline{R}$

Period Band	95-Percent Confidence Interval
Total	1.5
Short	1.6
Intermediate	2.5
Long	2.5

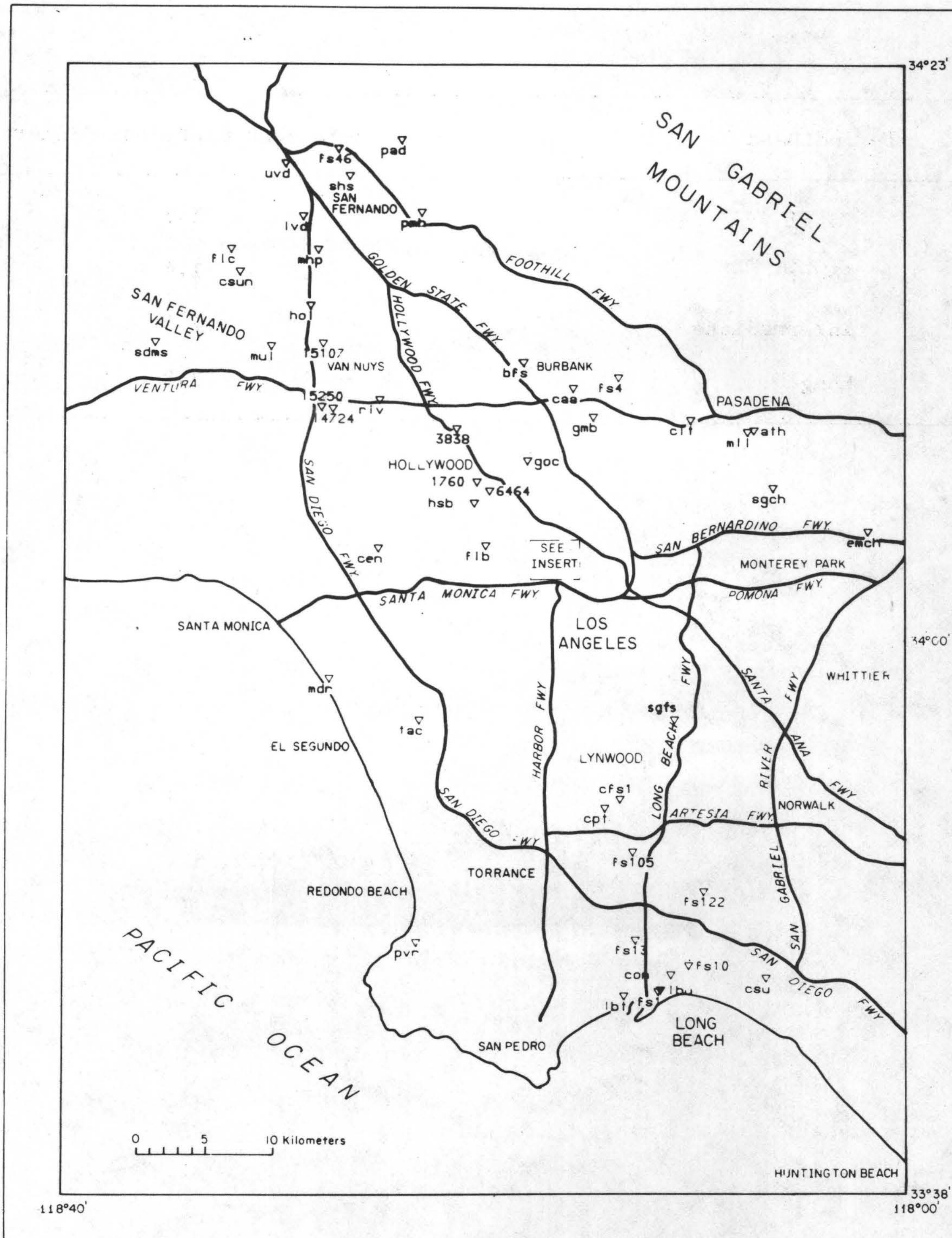

Figure 1.--Station locations in the greater Los Angeles region. Insert is shown in Figure 2.

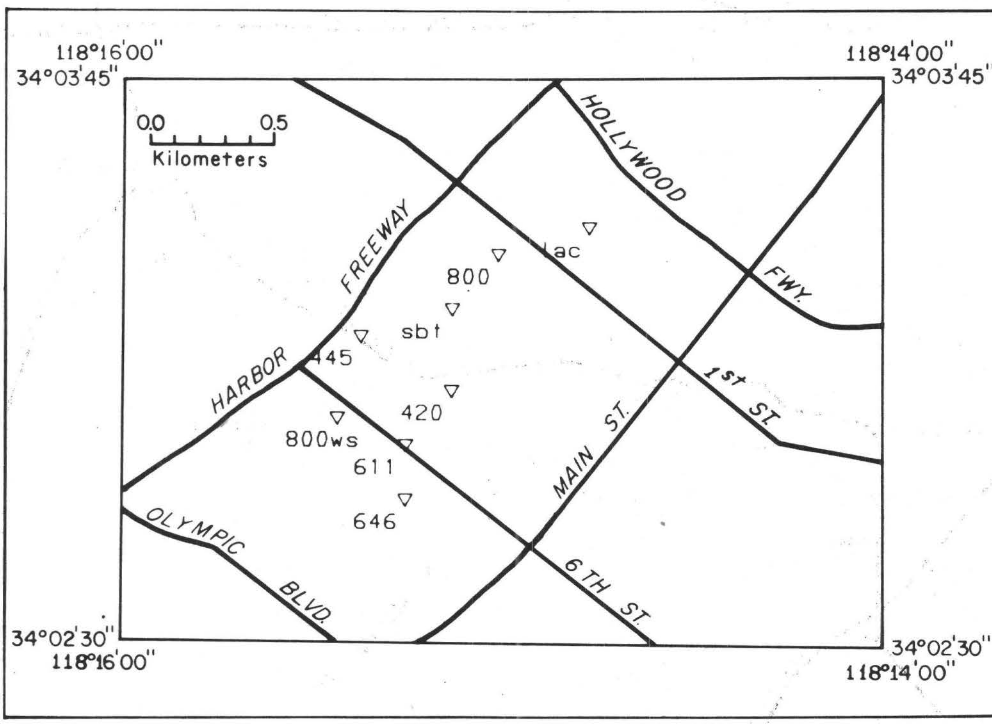

Figure 2.--Station locations in downtown Los Angeles.

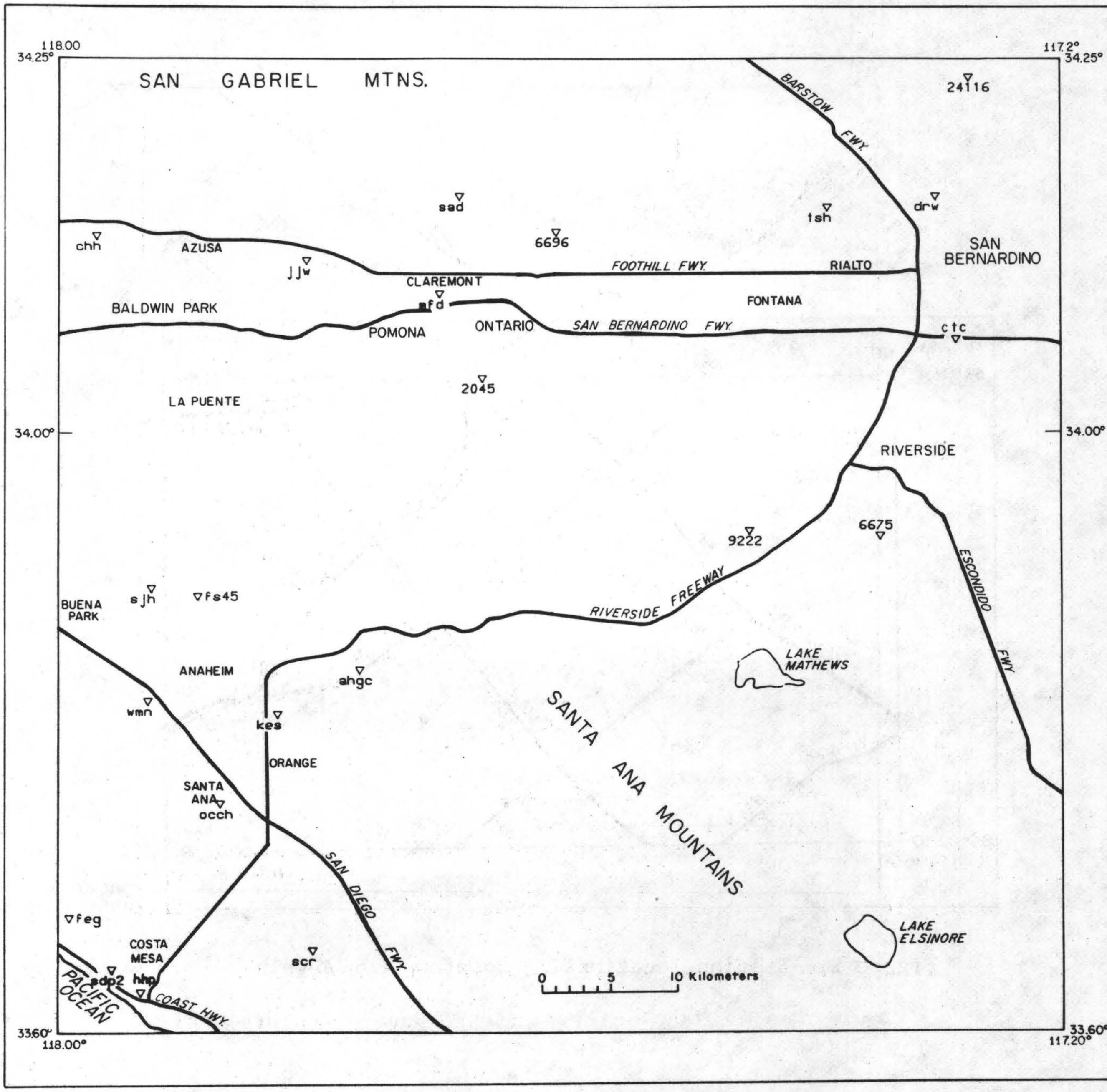

Figure 3.--Station locations in the San Bernardino and Santa Ana regions.

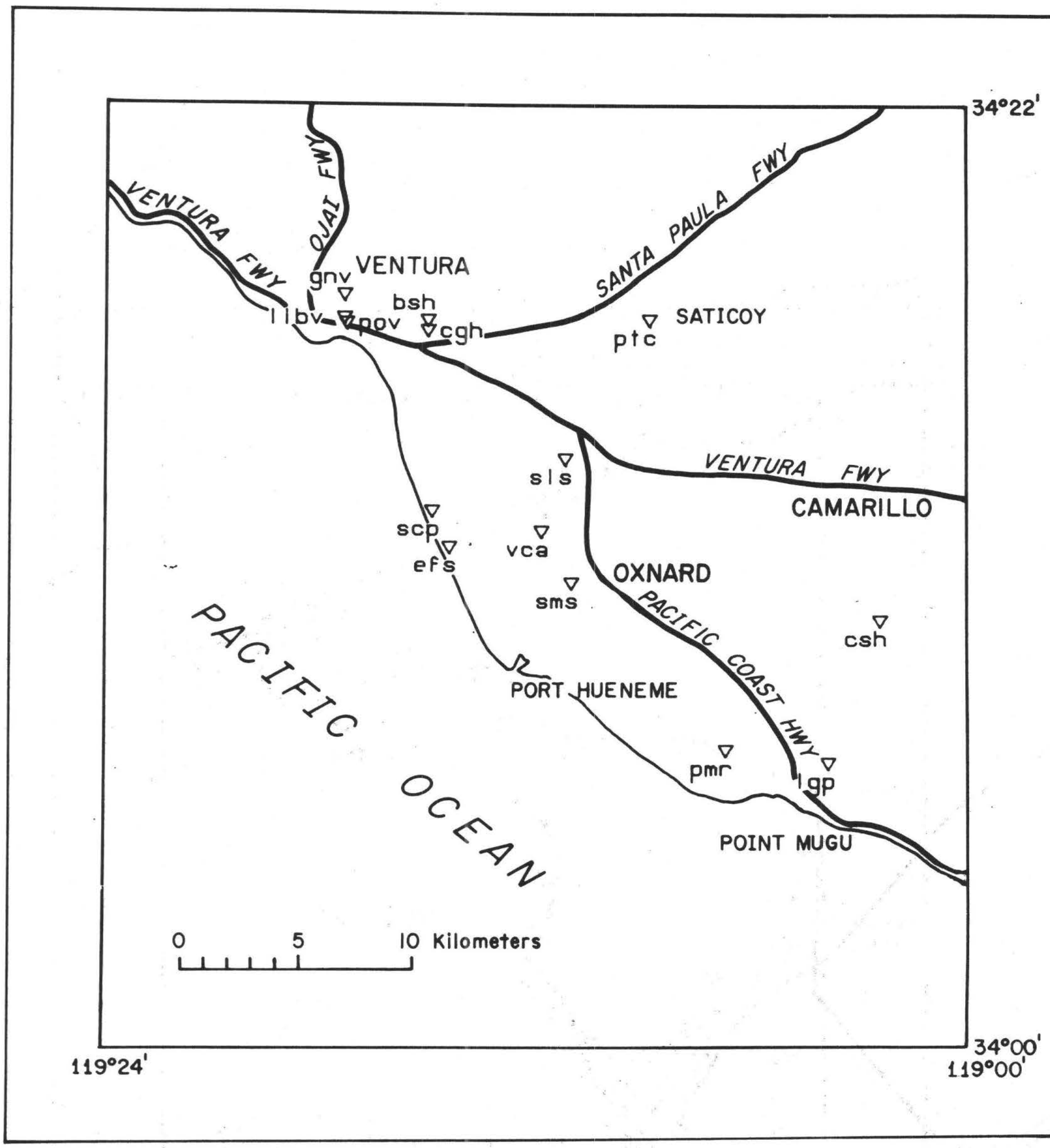

Figure 4.--Station locations in the Oxnard-Ventura region.

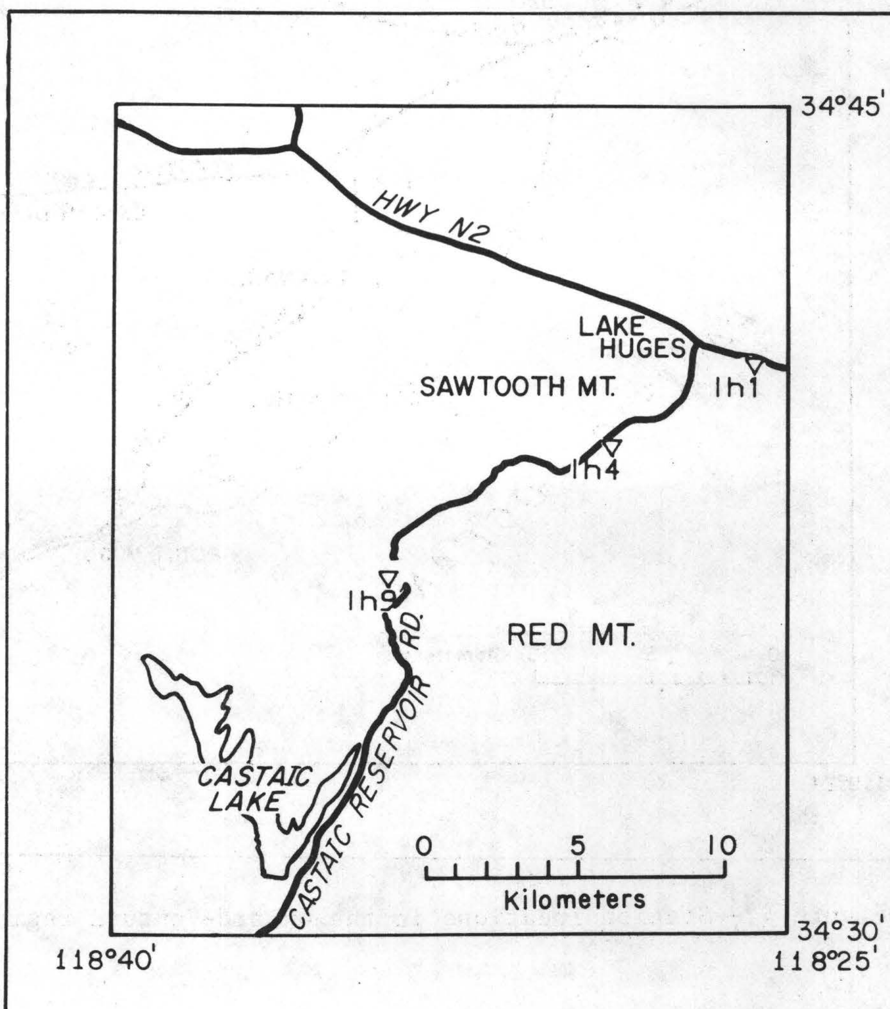

Figure 5.--Station locations in the Lake Hughes region.

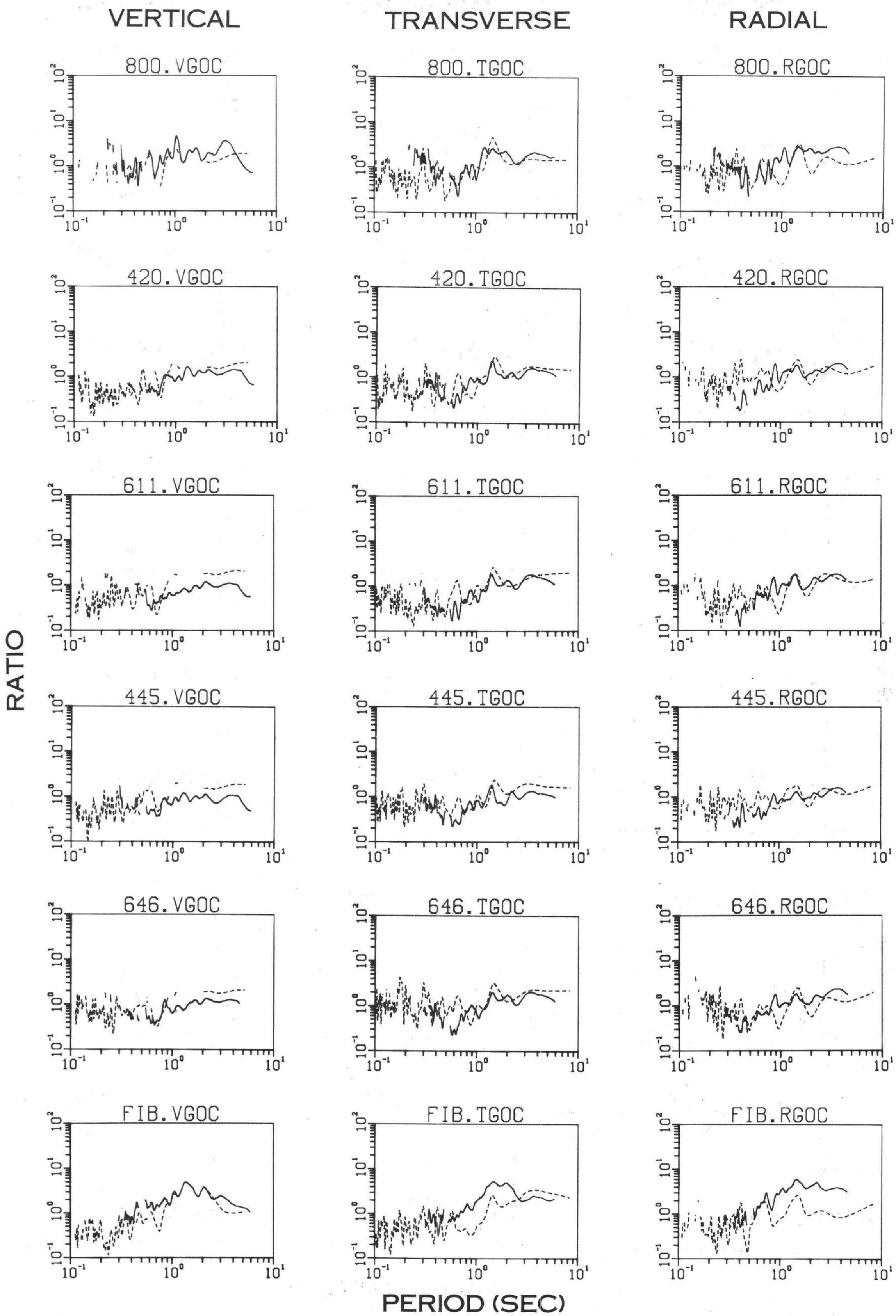

Figure 6.--Comparison of nuclear (solid line) and earthquake (dashed line) derived spectral ratios. Figure title notation indicates the station abbreviation of the spectral ratio numerator preceding the period. The single letter (V, T, R) following the period indicates the component of ground motion, and the remaining letters are the station abbreviation of the spectral ratio denominator. This figure shows the effect of grouping sites (see text).

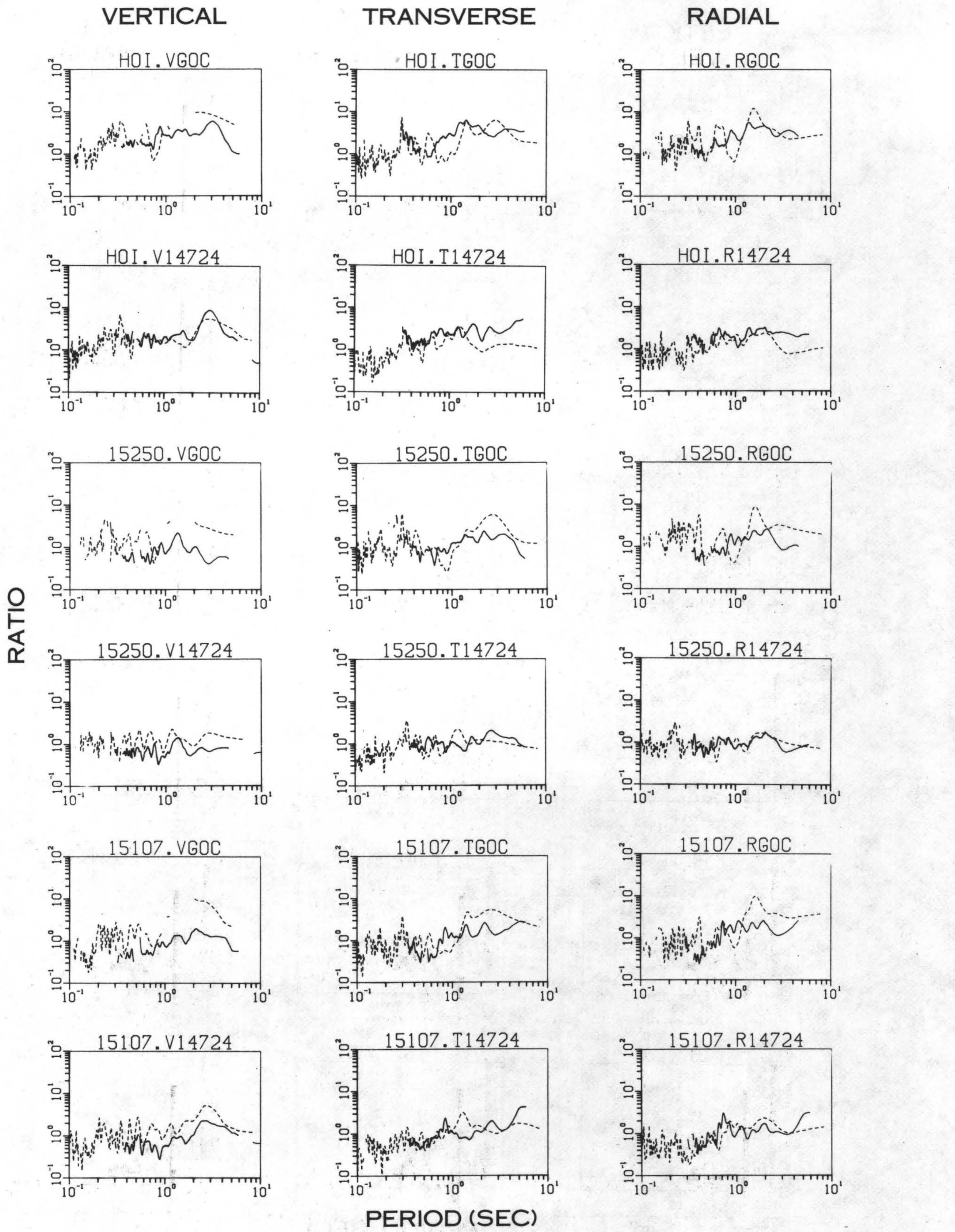

Figure 7.--Comparison of nuclear (solid line) and earthquake (dashed line) derived spectral ratios. Figure title notation indicates the station abbreviation of the spectral ratio numerator preceding the period. The single letter (V, T, R) following the period indicates the component of ground motion, and the remaining letters are the station abbreviation of the spectral ratio denominator. This figure shows the effect of grouping sites (see text).

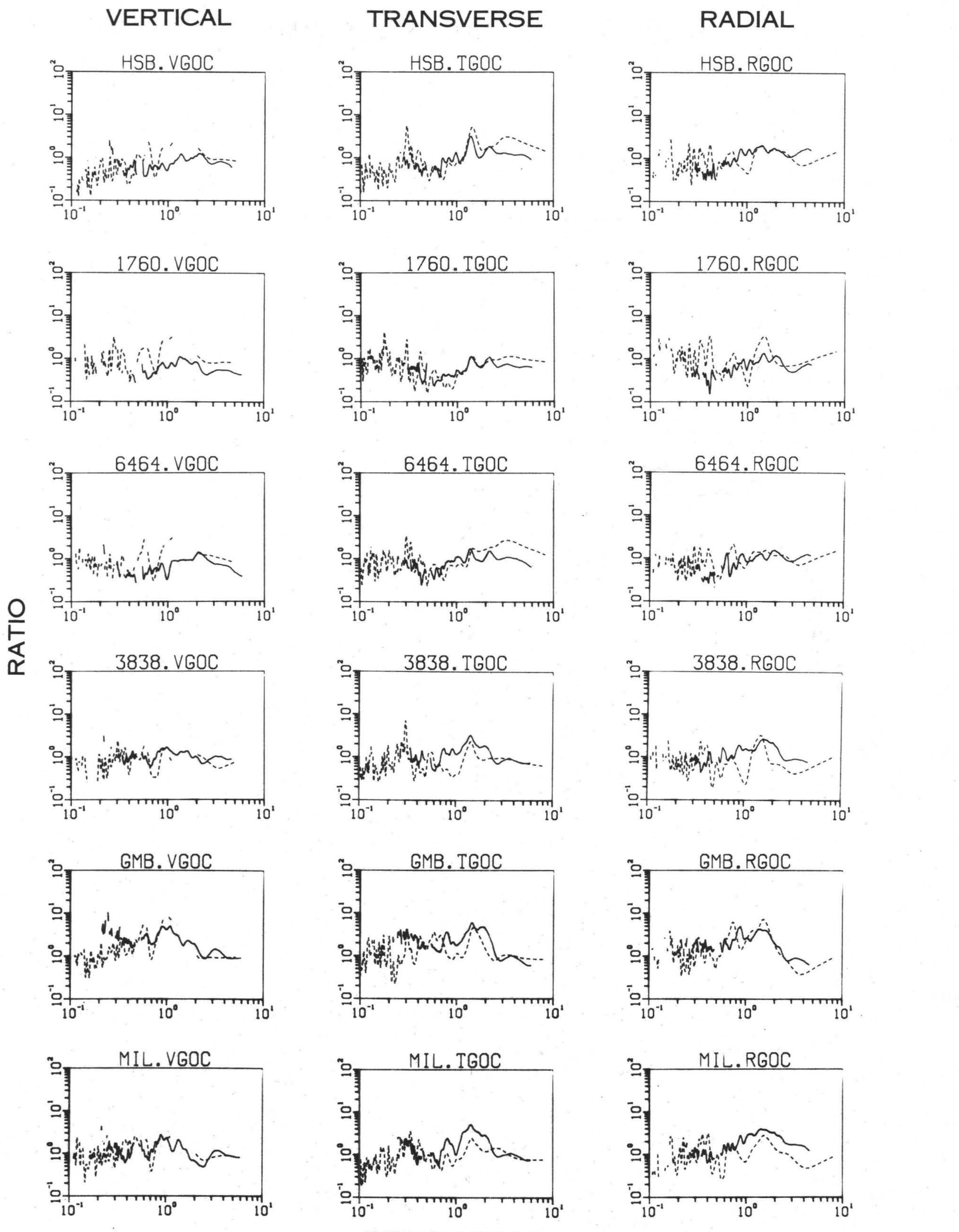

Figure 8.--Comparison of nuclear (solid line) and earthquake (dashed line) derived spectral ratios. Figure title notation indicates the station abbreviation of the spectral ratio numerator preceding the period. The single letter (V, T, R) following the period indicates the component of ground motion, and the remaining letters are the station abbreviation of the spectral ratio denominator.

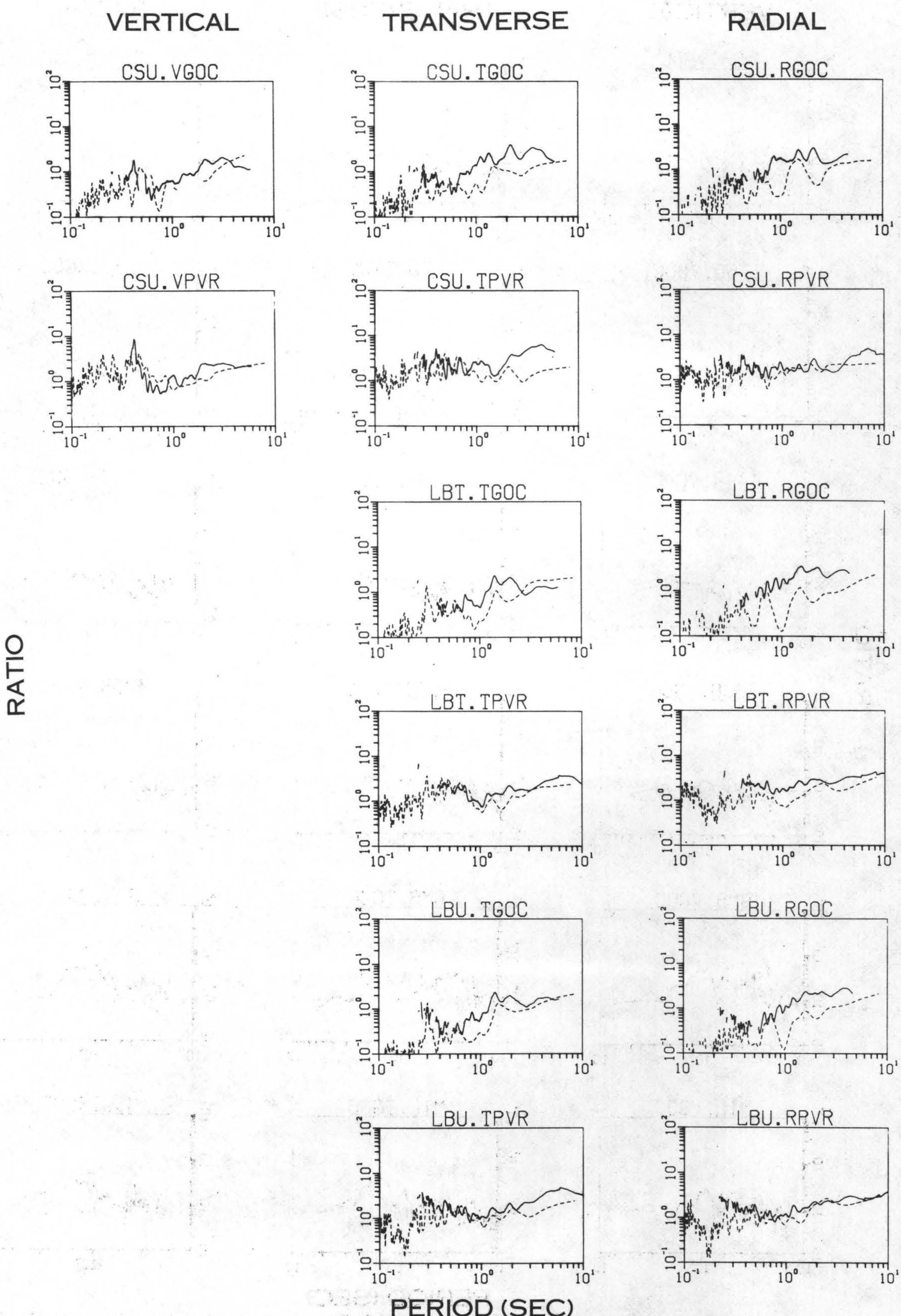

Figure 9.--Comparison of nuclear (solid line) and earthquake (dashed line) derived spectral ratios. Figure title notation indicates the station abbreviation of the spectral ratio numerator preceding the period. The single letter (V, T, R) following the period indicates the component of ground motion, and the remaining letters are the station abbreviation of the spectral ratio denominator.

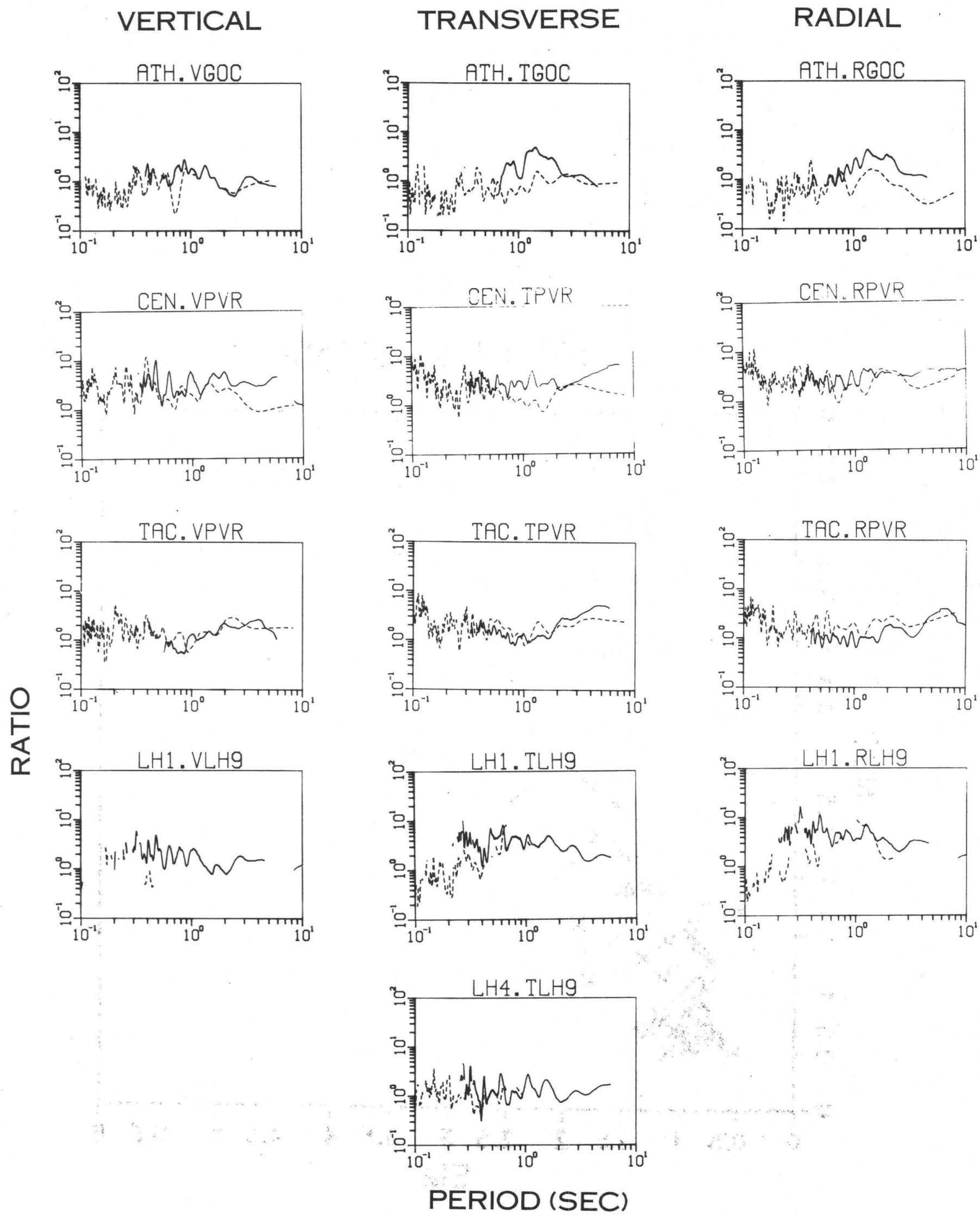

Figure 10.--Comparison of nuclear (solid line) and earthquake (dashed line) derived spectral ratios. Figure title notation indicates the station abbreviation of the spectral ratio numerator preceding the period. The single letter (V, T, R) following the period indicates the component of ground motion, and the remaining letters are the station abbreviation of the spectral ratio denominator.

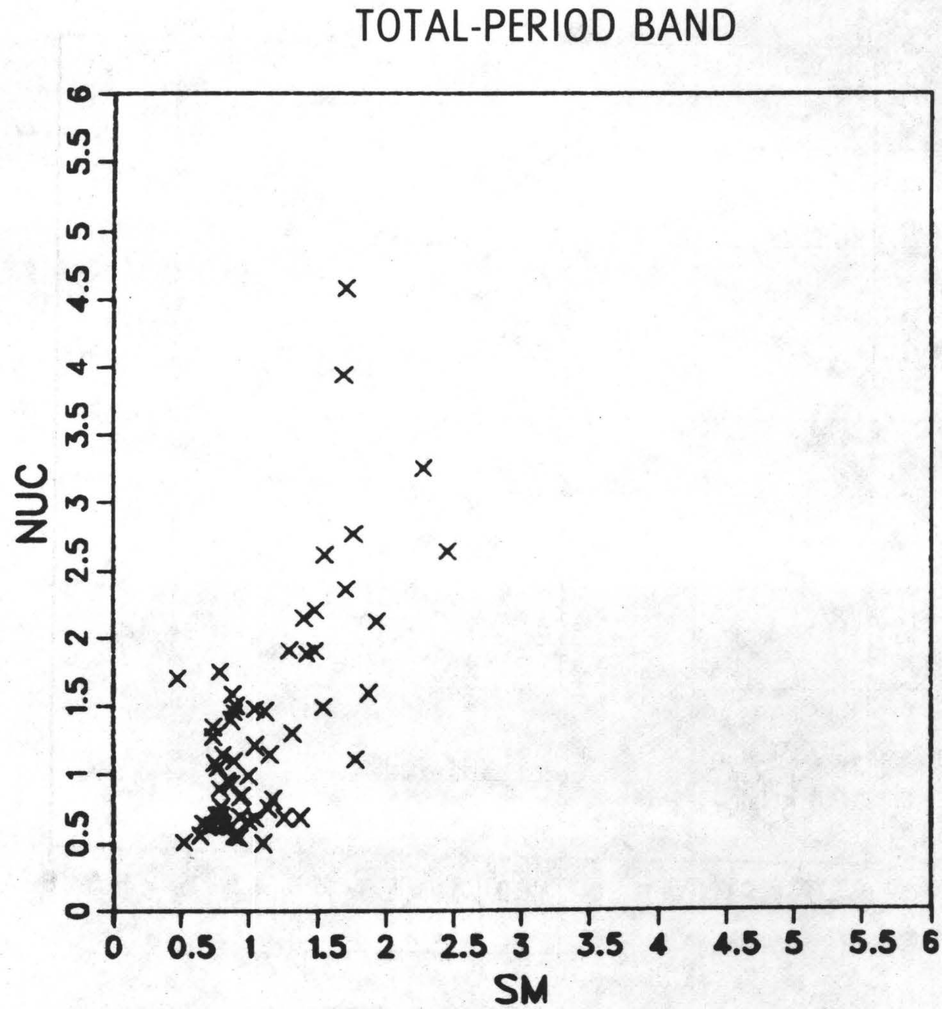

Figure 11.--A plot of the strong motion mean SR (SM) versus the nuclear mean SR (NUC) across the total-period band.

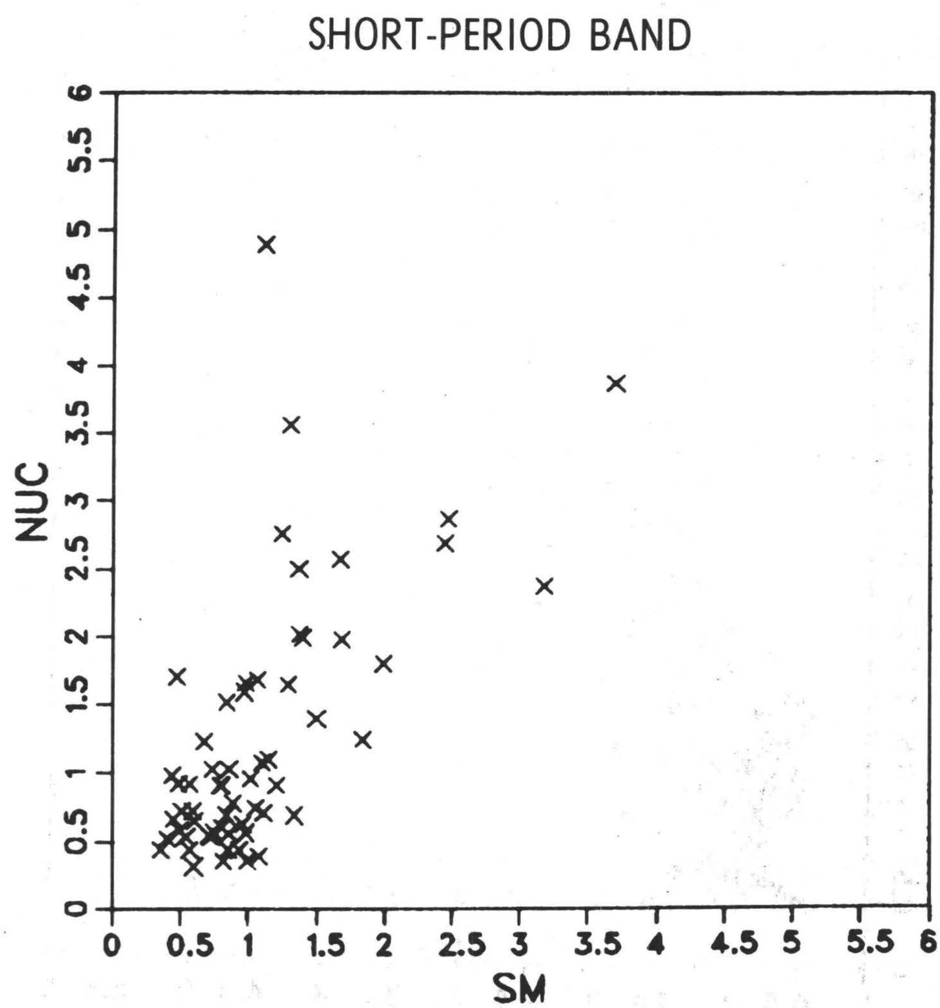

Figure 12.--A plot of the strong motion mean SR (SM) versus the nuclear mean SR (NUC) across the total-period band.

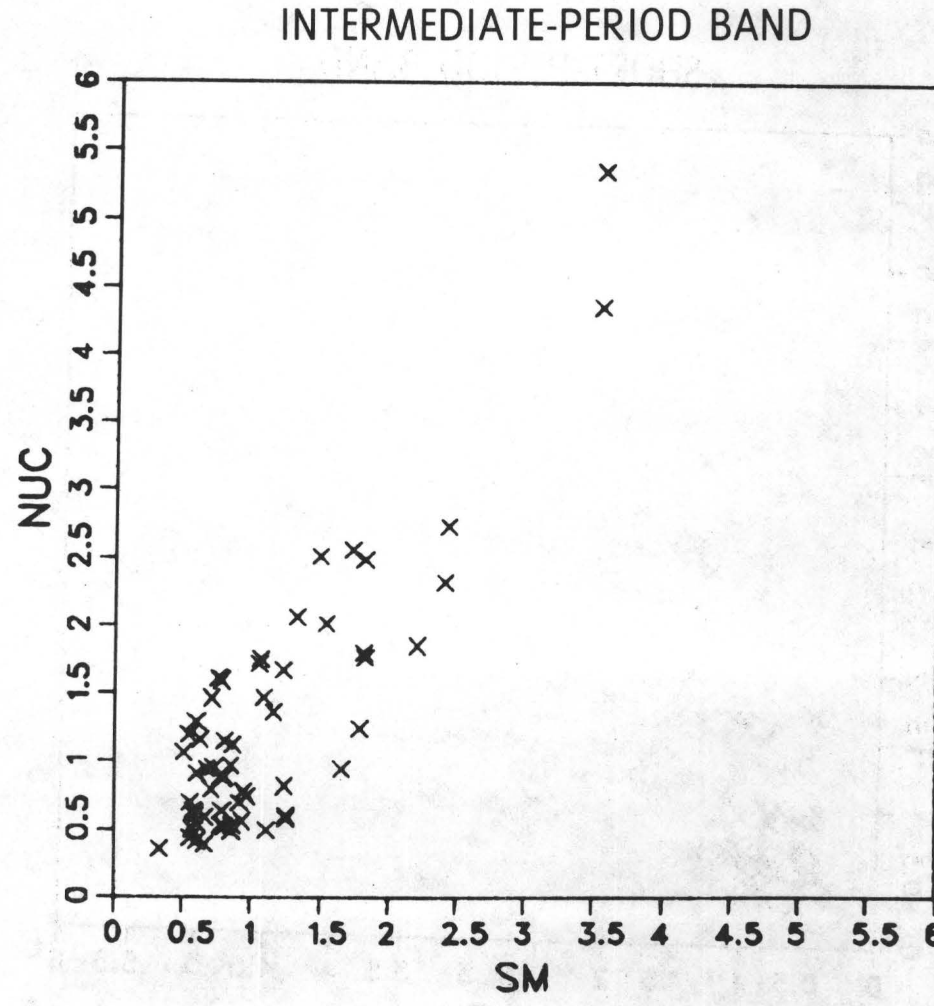

Figure 13.--A plot of the strong motion mean SR versus the nuclear mean SR (NUC) across the intermediate-period band.

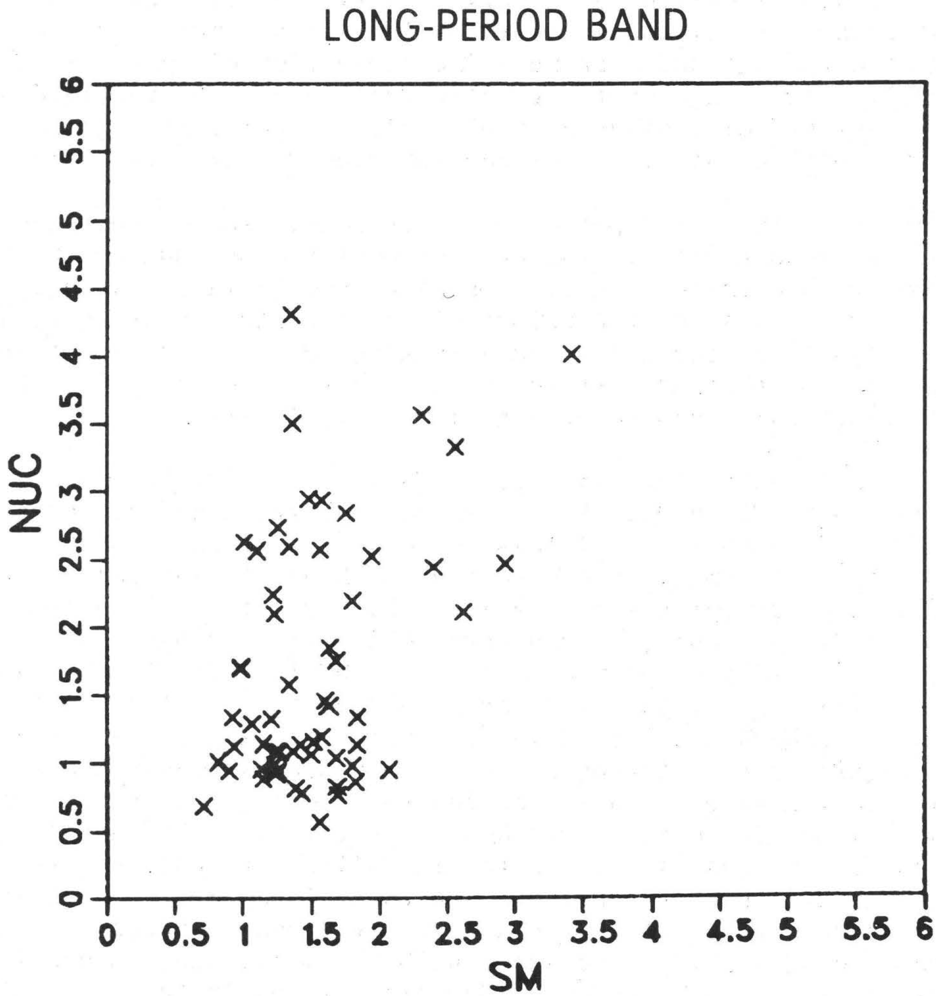

Figure 14.--A plot of the strong motion mean SR (SM) versus the nuclear mean SR (NUC) across the long-period band.

Estimation of Earthquake Losses

by S. T. Algermissen[1] and K. V. Steinbrugge[2]

Abstract

The general characteristics of earthquake losses and methods used to estimate economic and casualty losses are outlined. Ground shaking is the principal cause of economic loss in earthquakes while losses associated with surface faulting generally are a small percentage of the total loss. The significance of other kinds of ground failure such as liquefaction and landsliding are difficult to estimate and the extent of these types of ground failure may vary over wide limits. Very conservative estimates, however, indicate that losses associated with ground failure are much less than the losses associated with ground shaking. The definition of loss used here means the average percentage of the total actual cash value required to fully repair in kind any building of a particular class experiencing ground motion represented by a particular degree of the Modified Mercalli intensity scales.

Cumulative losses over a period of years to classes of buildings such as dwellings that result from earthquakes of moderate maximum intensity (VII-VIII) are greater than the cumulative losses caused by large earthquakes. This result occurs because of the nature of the magnitude distribution of earthquakes and the shape of the loss-ground shaking curves. It is not true for classes of structures that are very earthquake resistive, that is, classes of structures that are only damaged at high intensity levels.

Average annual losses to dwellings in California vary over a wide range depending upon geographic area. For example, in central and northern California the estimated average annual loss per dwelling is only about 1/100 the loss estimated for the remainder of the state. Economic and casualty losses for postulated large damaging earthquakes affecting the San Francisco Bay Area, Los Angeles and Orange County in southern California and Salt Lake County in Utah have been assessed and updated to 1980. Specifically, losses have been estimated for: (1) a magnitude 8.3 earthquake on the San Andreas fault and a magnitude 7.5 earthquake on the Haywood fault affecting the San Francisco Bay area; (2) a magnitude 8.3 earthquake on the San Andreas fault and a magnitude 7.5 earthquake on the Newport-Inglewood fault affecting Los Angeles and Orange counties; and (3) a magnitude 7.5 earthquake on the Wasatch fault affecting Salt Lake county. The casualty esitmates in California represent reassessment and updating of previous work, which estimates of total losses in California and Utah are the result of entirely new investigations. Losses to buildings resulting from ground shaking associated with the magnitude 8.3 and 7.5 earthquakes postulated in the San Francisco Bay area are estimated at $25.3 and $28.7 billions respectively, exclusive of contents. Included in these estimates are losses of $4.1 billion ($M_L$=8.3, San Andreas fault) and $3.1 billion ($M_L$=7.5, Haywood fault) to one to four family dwellings, exclusive of contents. For a magnitude 7.5 earthquake on the Newport-Inglewood fault replacement cost losses from ground shaking in Los Angeles and Orange counties are estimated at about $40.7 billion of which 8.1 billion is associated with 1-4 family dwellings. Shaking losses in Los Angeles and Orange county resulting from a magnitude 8.3 earthquake on the San Andreas fault in southern California are estimated at $16.3

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billion of which 4.8 billion are to 1-4 family dwellings. Losses in Salt Lake county, Utah depend heavily on assumptions of the earthquake potential of the Wasatch fault. Estimates of losses in Salt Lake county from surface faulting and ground shaking are made based upon the assumption of a magnitude 7.5 earthquake on the Wasatch fault. Casualties associated with large damaging earthquakes are difficult to estimate because of the many variables involved. It is, however, clear that the losses in past earthquakes in the United States have been unusually low and that substantially larger life losses may be expected in the future. Steinbrugge et al., 1980 have estimated life loss in the range of 3,000-11,000 with hospitalized injuries 11,000 to 44,000 for a large earthquake on the San Andreas fault near San Francisco. For a magnitude 7.5 earthquake on the Newport-Inglewood fault life loss is estimated at 3,000 to 11,000 depending upon the time of day. Hospitalized injuries would be about four times the life loss.

Many uncertainties are present in earthquake loss estimation but potential economic losses could be much more accurately evaluated if improved building inventories were available or could be developed. Both economic and life loss from earthquakes in the United States are likely to show substantial increases in the future because of: (1) the increase in the areal extent of populated areas; (2) the unusually low economic and life loss in previous United States earthquakes, and (3) the absence for many years of large damaging earthquakes near population centers in the country.

PROCEDURES AND DATA BASES FOR EARTHQUAKE DAMAGE PREDICTION AND RISK ASSESSMENT

by

Roger E. Scholl[1] and Onder Kustu[2]

INTRODUCTION

Efforts to assess earthquake losses have likely been made following major earthquakes for the entire history of mankind. The great San Francisco earthquake of 1906, however, is the earliest event for which useful documented quantitative evaluations of losses are available. Loss evaluations have been made with varying degrees of rigor following each of the subsequent major earthquakes in the United States, although loss statistics prior to 1971 have been of limited value because of the substantial effort involved in compiling detailed loss information and the sparsity of available ground motion data.

Efforts to develop quantitative loss prediction procedures have been made only in recent years. Fifteen years ago, predictive estimates of damage that might result from earthquakes were almost nonexistent. The development of various procedures for estimating losses caused by ground motion has been prompted by the increased potential loss resulting from increased population density near active faults, the availability of more complete data from recent earthquakes, and other factors. However, the development of a single, general yet rigorous damage prediction methodology is not presently feasible because of the complexity of the problem and the sparsity of data.

This paper discusses the various factors that must be considered in developing damage prediction procedures and reviews various procedures that are currently available. The paper also provides an example of a theoretically based loss prediction methodology and describes a specific damage factor model. Finally, a summary of research needs in the area of earthquake damage prediction is given.

CONSIDERATIONS FOR DEVELOPING DAMAGE PREDICTION PROCEDURES

Several factors must be considered in making comprehensive damage predictions:

- Reasons for making predictive estimates of earthquake losses
- Types of losses to be estimated
- Causes of earthquake damage
- Structure types and classifications

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- Structure elements, materials, and assemblage
- Ground motion and structure response
- Approaches to predicting damage
- Contemporary earthquake-resistant design philosophy
- Risk evaluation and hazard reduction
- Timing

These ten factors are discussed below.

Reasons for Making Predictive Estimates of Earthquake Losses

It is beneficial to identify the various reasons for making predictive estimates of earthquake losses. Although there are a variety of reasons, the most important are:

- Disaster preparedness planning
- Reduction of future losses
- Structure design optimization
- Determination of earthquake insurance needs and rates

Types of Losses to be Estimated

Consideration of the types of losses to be estimated -- or the manner in which the losses are to be identified -- is essential for making comprehensive damage predictions. Although there are many types of losses, the most important, consistent with the above reasons for estimating losses, are:

- Life loss
- Injuries
- Structural damage
- Nonstructural damage, e.g., partitions, glazing
- Mechanical and electrical equipment damage
- Damage to contents, e.g., furniture, merchandise
- Losses due to lost production and lost wages

Causes of Earthquake Damage

Earthquakes cause various types of physical phenomena to occur in the vicinity of a fault rupture -- and sometimes at great distances from the affecting fault. Because the occurrence of these phenomena may lead to earthquake losses, they must be considered in making damage predictions.

The primary physical phenomenon caused by earthquakes is ground shaking; other events are secondary phenomena caused by ground shaking. A list of these various causes of earthquake losses is as follows:

- Primary phenomenon: ground shaking

- Secondary phenomena:

  -- liquefaction
  -- landslide
  -- tsunami
  -- flood
  -- fire
  -- interrupted lifeline services

Structure Types and Classifications

A first step in predicting earthquake losses is to inventory structures that might be subjected to significant ground motion. For loss prediction purposes, the term *structure* can be defined as any object of value that can be damaged by ground motion. Typically, most structures will be buildings. Generally, the vast majority of buildings will be low-rise (1- and 2-story) structures; however, most affected areas will also have many other types of structures.

Establishing structure categories is only necessary for damage evaluations of large numbers of structures. For such evaluations, it is appropriate to categorize structures to minimize the overall work involved in making the damage evaluation. A list of typical categories and examples of structure types is given in Table 1. These structure types can be further classified into subcategories according to their physical and mechanical characteristics, such as their vibration properties, structural systems, materials of construction, architectural components, and building configurations. Of course, by creating structure categories and thereby lumping structures into groups, greater variability is introduced into the final damage evaluation because rarely are any two structures identical in all respects.

Structure Elements, Materials, and Assemblage

Because a completed structure consists of an assemblage of many elements, the effect of each element on the response of a structure to dynamic ground motion must be understood if damage prediction procedures are to be developed. A common aspect of the many elements that make up a typical structure is that all can be damaged -- as a result of either a primary or a secondary effect. For dynamic response and damage prediction purposes, the most important properties of the various elements are:

- Mass

- Force-deformation relationship

TABLE 1

TYPICAL CATEGORIES AND EXAMPLES OF STRUCTURES

A. Buildings

1. Residential (houses, apartments)
2. Agricultural (farmhouses, barns, outbuildings)
3. Commercial (stores, gasoline stations)
4. Institutional (schools, hospitals, churches)
5. Industrial (refineries, mills)
6. Special (shrines, ruins)

B. Utility and Transportation Structures

1. Electrical power structures (lines, transformers, switch gear converters, beacons)
2. Communication and microwave stations (reflectors, towers, equipment)
3. Roads, railroads, bridges, overpasses, tunnels, retaining walls
4. Air navigational facilities (beacons, marker stations)
5. Airfields and parking areas
6. Marine and waterfront structures (piers, bulkheads)

C. Hydraulic Structures

1. Earth, rock, or concrete dams, outlet works, control structures
2. Reservoirs, lakes, ponds, sumps, forebays, afterbays, and adjacent shores and slopes (for wave generation)
3. Canals, pipelines, siphons, surge tanks, elevated and surface storage tanks, distribution systems
4. Water storage, cisterns, distribution, processing stations
5. Petroleum products (liquid and gas) storage, handling, piping, processing stations

D. Earth Structures

1. Earth and rock slopes (for potential instability determinations and predictions of damage to roads, fields, stream contamination, hazards to persons)
2. Major existing landslides, land creep areas, snow, ice, or earth avalanche areas, subsidence areas
3. Natural or altered sites with scientific, historical, cultural, or ecological significance (pueblo dwellings, scenic rock formations, historical landmarks, archaeological sites)
4. Berms, dikes, banks

E. Special Structures and Items

1. Conveyor systems, tramways, cableways, flumes, ski lifts, trestles, headframes, personnel lifts
2. Ventilation systems, stacks
3. Mobile equipment, rolling stock, vehicles, drillrigs
4. Towers, poles, signs, frames, antennas
5. Material storage, ore heaps, elevated bulk storage, tailings piles, gravel plants, tailings ponds, corrosive fluid storage
6. Agricultural equipment, irrigation lines
7. Furnishings, shelf goods, roof-mounted air conditioners, bric-a-brac, dishes

- Energy absorption (damping)
- Damageability

Mass is important for determining inertial force magnitudes; the force-deformation relationship (stiffness), for determining the rate of element deformation and for determining limits of deformation (damage thresholds); energy absorption, for establishing the rate of decay of vibratory motion; and damageability, for determining the extent of damage for the level of structure response. Mass and force-deformation characteristics are also the principal parameters affecting a building's frequency and mode shape characteristics. Because these properties vary widely from element to element and because there are so many elements involved in structures, it is desirable to categorize and classify them.

For damage prediction purposes, classifying the elements of a structure according to function, i.e., structural or nonstructural, provides useful distinctions. Generally, a structural element is one that is important to the overall survival of a structure. Thus, damage to a nonstructural element would not be nearly as consequential as damage to a structural element. Examples of structural elements are foundations, beams, columns, vertical-load-bearing walls, and shear walls. Nonstructural elements include windows, partition walls, residential chimneys, and hung ceilings. Although damage to a nonstructural element might be hazardous to people, damage to structural elements is potentially much more serious because many more people could be endangered by building collapse.

For damage prediction purposes, materials are distinguished by the manner in which they deform under load. Materials are characterized as brittle or ductile, as flexible or stiff, and as strong or weak. Figure 1 shows schematic force-deformation relationships that define these six characterizations. The common definitions of these terms, except ductile and brittle, also apply in the field of structural dynamics. A ductile material is one that does not fail at the first sign of distress and also absorbs large amounts of energy when it deforms (Blume, 1960). Steel framing is a classic example of a ductile material. Brittle materials, such as glass, are generally understood to fail completely at or near the first sign of distress.

The assemblage of the elements that make up a structure is important to damage prediction in two respects. First, the manner in which various elements and materials are arranged in a building and their relative stiffnesses determine the order in which members are damaged. Plaster-surfaced, nailed, wood-frame construction provides a classic example of this. Nailed wood framing is generally characterized as a ductile material, while the plaster surface represents a brittle material. Knowing the general force-deformation relationship for the two materials, one can readily predict that the plaster will be severely cracked before any serious damage is done to the wood-frame timbers. Second, the degree of competency of element connections determines the extent to which an element participates in resisting lateral inertial forces caused by dynamic ground motion. Competency also involves the clearances between elements; for example, because of the relatively large tolerances in normal window installation, the RULISON underground nuclear explosion caused very little damage to window glass in low-

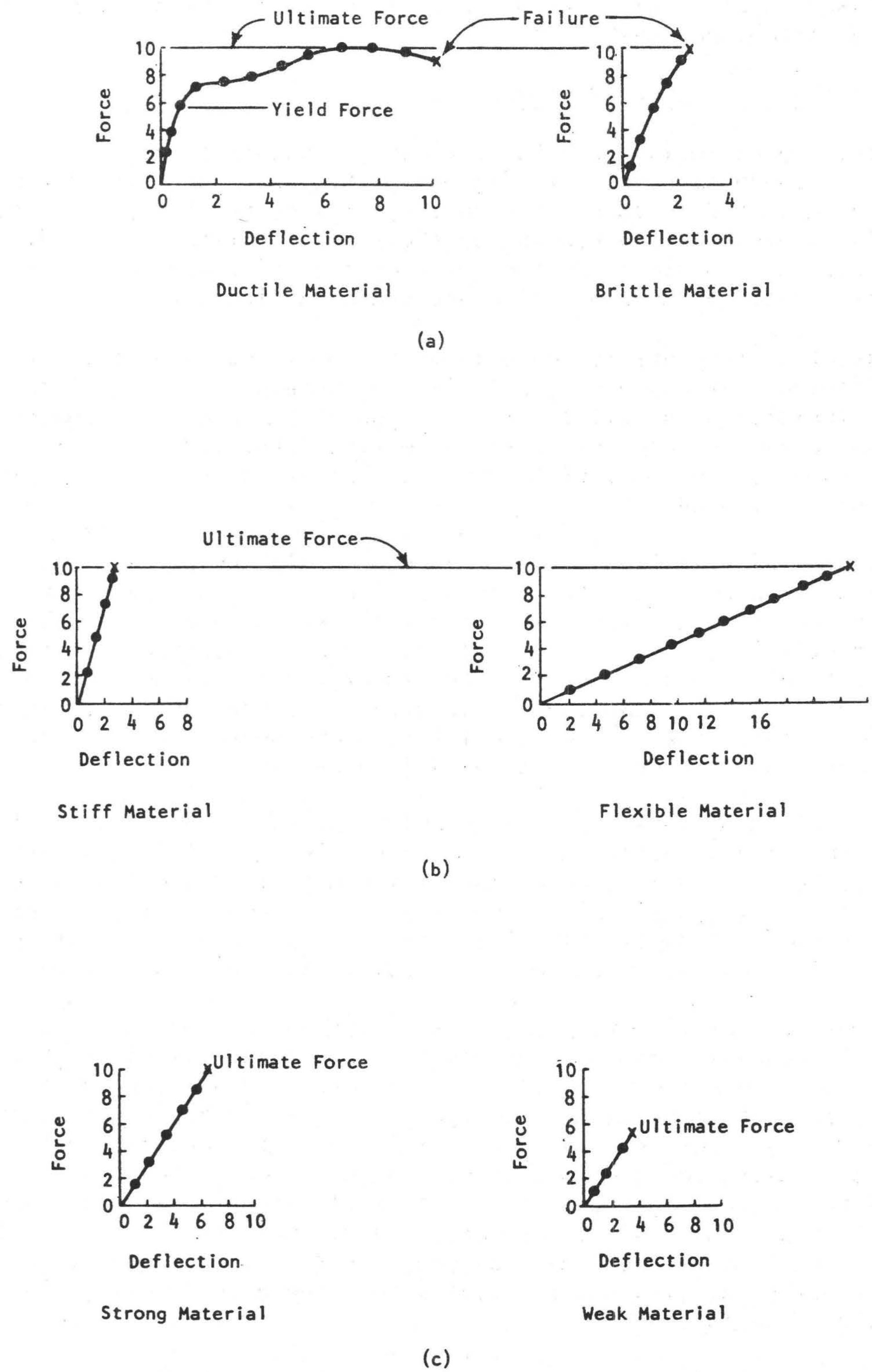


FIGURE 1 STRUCTURE MATERIAL CHARACTERISTICS (from URS/John A. Blume & Associates, Engineers, 1975)

rise buildings compared with damage to interior wall finish materials (Scholl and Farhoomand, 1973).

Ground Motion and Structure Response

Free-surface ground motions can be completely identified in terms of three independent orthogonal components (ignoring rotations about the three axes). These can be recorded in terms of time-varying acceleration (A), velocity (V), or displacement (D), depending on the type of seismometer used. Example acceleration recordings for the three orthogonal components representing moderate-amplitude earthquake motion are shown in Figure 2.

If the base of a structure is suddenly moved, other parts of the structure will not respond instantaneously but will lag because of inertial forces and structure flexibility, as illustrated in Figures 3 and 4. The concept of inertial forces is not new, of course -- Newton described it in his Second Law of Motion as the product of the mass of the structure (weight) times acceleration, or $F = mA$.

For simplicity, Figures 3 and 4 show motion in only one plane. Because the ground motion at a point on the earth's surface is three-dimensional, as described above, the structures affected will deform in a three-dimensional manner. Practically, however, the inertial forces generated by the horizontal components of ground motion are more important for seismic damage prediction than the vertical components because structures are less rigorously designed for lateral than for vertical forces and because of the factors of safety commonly used in vertical gravity load design.

A fundamental premise in structural dynamics is that structures are flexible or deform under load. Although the stiffness (inverse of flexibility) of different structures varies, depending on the materials and framing configuration involved, virtually all conventional civil engineering structures have some degree of flexibility. The elastic properties of structures and how their variations affect response and damage were discussed above.

The magnitudes of inertial forces induced by ground motion excitation are functions of the masses and accelerations of a structure. Although the masses of a structure can be easily and accurately identified, determining a structure's accelerations is more difficult. If a structure were perfectly rigid, i.e., if its entire mass moved precisely as the ground moves, establishing its acceleration and force distribution during ground motion excitation would be simple. However, because flexible structures deform under load, as Figures 3 and 4 show, the motion in various parts of a structure usually differs from that of the free ground surface. In some cases, a structure's motion amplitudes are greater than the ground motion; in other cases, the reverse is true.

For accurate calculation of a structure's motions, and therefore the acting inertial forces, a dynamic structure response analysis must be performed. At a minimum, the fundamental aspects of dynamic structure response must be included in damage prediction procedures that are to have any general applicability. Quantitative aspects of dynamic structure response are discussed in numerous text books.

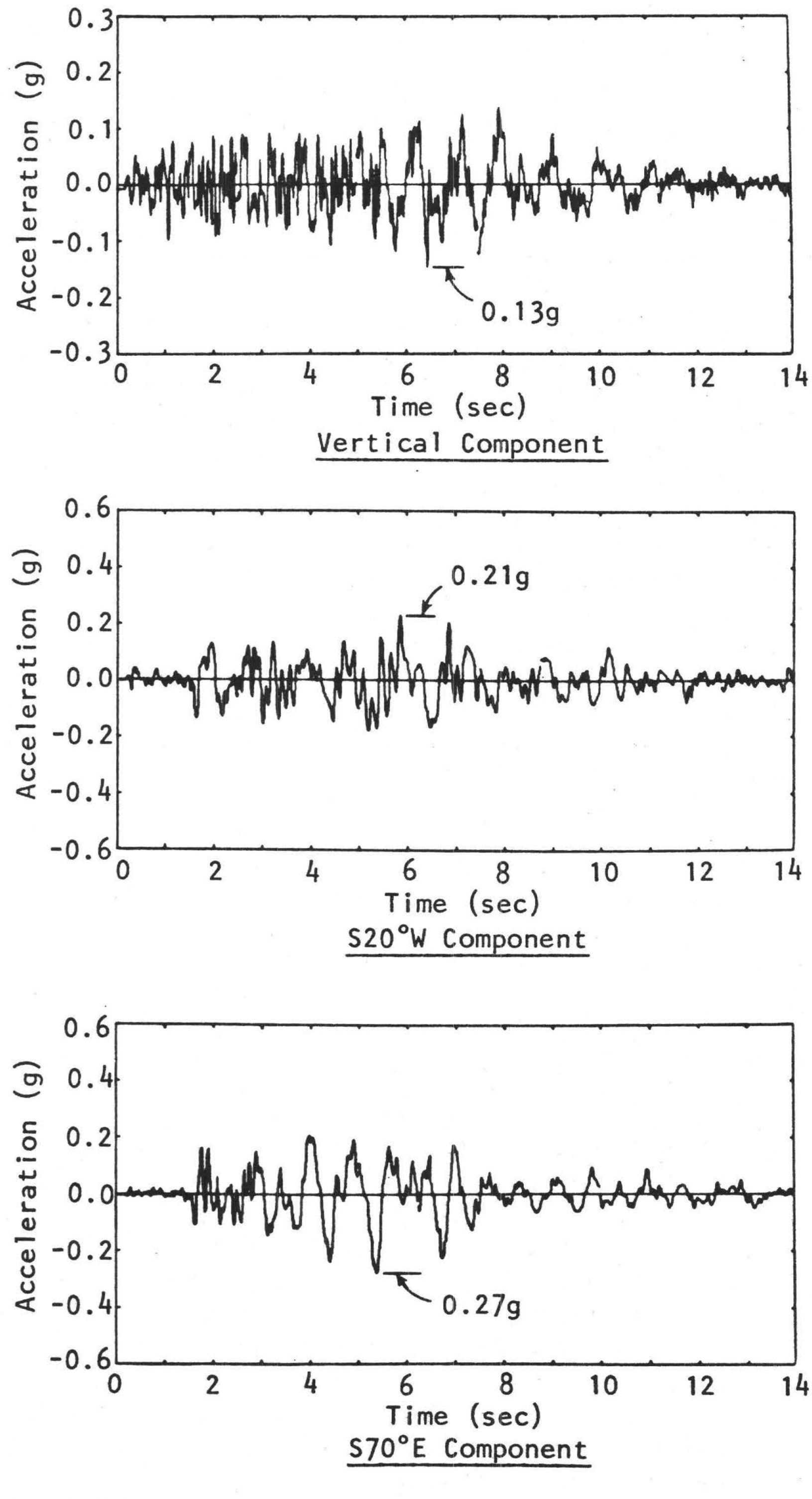


FIGURE 2 STRONG MOTION RECORDINGS FOR THE SAN FERNANDO EARTHQUAKE OF FEBRUARY 9, 1971, AT GLENDALE, CALIFORNIA, STATION 32 (from URS/John A. Blume & Associates, Engineers, 1975)

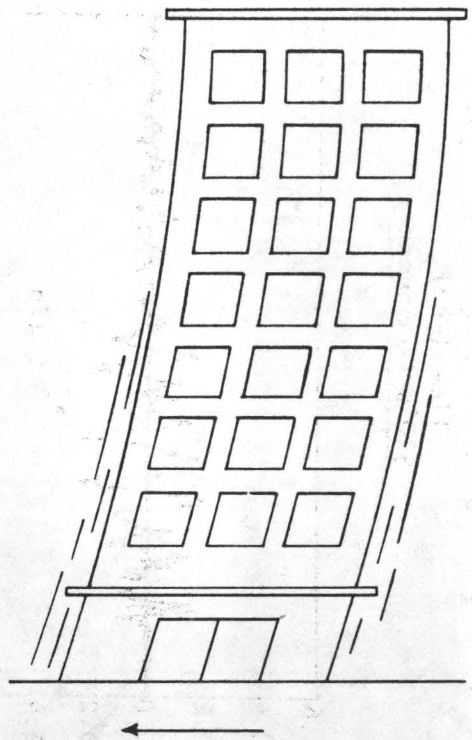


FIGURE 3 SCHEMATIC OF HIGH-RISE BUILDING SHEAR-TYPE INSTANTANEOUS DISTORTION CAUSED BY GROUND MOTION (from URS/John A. Blume & Associates, Engineers, 1975)

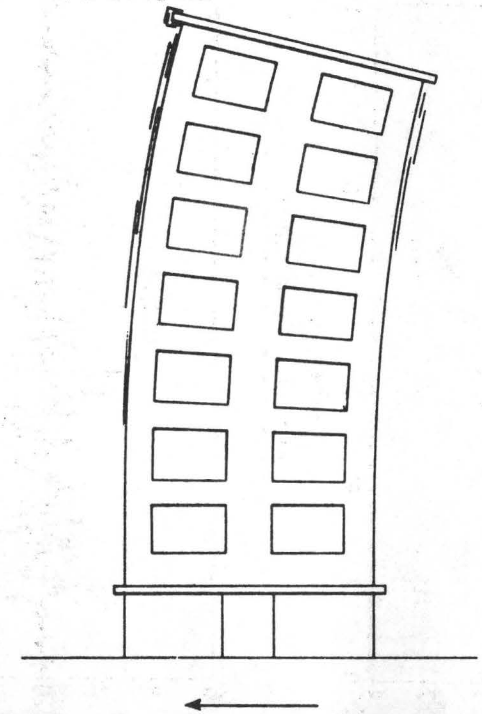


FIGURE 4 SCHEMATIC OF HIGH-RISE BUILDING BENDING-TYPE INSTANTANEOUS DISTORTION CAUSED BY GROUND MOTION (from URS/John A. Blume & Associates, Engineers, 1975)

The important characteristics of earthquake ground motion, as it affects structure response and damage, are:

- Amplitude
- Frequency content
- Duration
- Periodicity

All these characteristics, except for duration, are reflected in standard response spectrum plots. Duration can be revealed in three-dimensional response spectrum plots (Schopp and Scholl, 1972), but, because of the added complexity of presenting three-dimensional plots and because duration is less important than other ground motion characteristics, it is commonly not explicitly presented. However, for certain damage predictions (e.g., those involving liquefaction and low-cycle fatigue), knowledge of ground motion duration is crucial. In some cases, duration is presented by specifying the number of seconds during which the record shows that ground motion was greater than some given amplitude (e.g., acceleration $\geq$ 0.05g).

Currently, seismological intensity scales (e.g., Modified Mercalli and Rossi-Forel) are used extensively for damage prediction purposes. Although the various ground motion characteristics listed above are reflected in the seismological intensity scales, the scales present two serious limitations. First, the various ground motion characteristics are not independently distinguishable, and, second, the scales are not quantitatively applicable (except in a very approximate sense) to engineering analysis and design. The seismological intensity scales were first developed nearly two centuries ago and have been evolving ever since that time; however, they simply do not possess the quantitative precision compatible with modern seismic analysis and design technology.

Response spectrum plots facilitate distinguishing response amplitude as a function of frequency. This is important in engineering because different structures have different natural vibration frequencies, and thus dynamic amplification of various structures depends on amplitudes of ground motion at various frequencies.

Approaches to Predicting Damage

A comprehensive damage prediction methodology should satisfy the following criteria:

- It should be based on sound theory and engineering principles, and it should relate to and use commonly known engineering analysis and design methods and parameters. This would allow improvements to be made easily in the damage prediction methodology as the state of the art in engineering design and analysis advances. It should also facilitate the use of the methodology by most practicing professionals without requiring extensive experience with damage prediction technology.

- The methodology should be easily adaptable to all engineering structures. This criterion will be satisfied if the methodology is based on engineering principles and uses commonly known design and analysis methods and parameters.

- The methodology should have provisions for using the data from actual earthquakes and from laboratory experiments as they become available.

- The methodology should account for uncertainties in the ground motion demand, the structural capacity, and the analytical methods and assumptions. This requires the methodology to adopt a probabilistic approach.

- The methodology should be able to be conveniently automated for use of computers in real-world applications. This requires a modular structuring of the methodology. Basic modules, for example, can be ground motion prediction, structure response prediction, structure (or component) inventory, basic damage prediction, and economic factors. In addition, a decision analysis module can also be incorporated. The structure response — damage relationships or data can be stored as a separate module or as a damage data library.

Selection of an approach to predicting damage requires consideration of utilitarian factors, that is, whether damage is being predicted for a single structure, for a group of structures, or for a large urban area. These factors affect the degree of data-base structuring required for the methodology.

For the completely general case, an earthquake damage prediction methodology for structures would include the following steps:

1. Inventory methodology

2. Ground motion prediction methodology

3. Loss prediction methodology (loss algorithm)

Inventory methodology is relatively straightforward, and an example is given later in this paper. Ground motion predictions can be made in many ways. An outline of a general ground motion prediction methodology is also given later in this paper.

Loss algorithms, whether structure specific or for structure groups, can be developed from either empirical or theoretical procedures. The practical limitations of each approach require that information from both sources be used for developing loss prediction procedures involving real structures.

Empirical Procedures. Empirical procedures involve gathering and correlating ground motion information and loss information from past earthquakes or other sources of ground motion. Figure 5 shows an example of this type of information, giving a plot of mean damage factor (ratio of dollar loss to replacement value) versus 5%-damped response spectrum acceleration averaged over the period band of 0.05 to 0.20 sec. While this information is very useful, it has two serious limitations:

- It is almost impossible to gather the necessary volume of information for the wide variety of structures that exist in a large urban area.
- Changes in design and construction practice cause the information to have limited applicability for future events.

Theoretical Procedures. Theoretical procedures involve employing mathematical models, which include consideration of the physical and mechanical properties of a structure, for predicting damage. This approach is the avenue of choice from the perspective of generality and flexibility. If the procedures are based on the fundamental principles of structural engineering and dynamic response, engineers can use these procedures in future designs to reduce future earthquake hazards and can easily modify the methodology to reflect those design changes that would reduce future earthquake damage.

No single methodology will ever suffice for all damage prediction needs. All practicable methodologies are by necessity approximate, and various degrees of precision are required for different prediction needs. In addition, the variations in the many types of structures and structure components virtually dictate that different prediction approaches be used for different situations. For example, interstory drift is an important indicator of damage for structural and nonstructural components of a building, while floor acceleration is an important indicator of damage for equipment.

An example is provided below to illustrate one theoretical approach to predicting damage.

First, various interstory drift limits can be determined from test data or can be estimated for many types of structure configurations. Accordingly, information such as that given in Table 2 can be determined. The interstory drift information can then be used to calculate response spectrum amplitudes for the various drift limits as follows.

From fundamental considerations of dynamic response analysis, and considering only the fundamental mode response:

$$\delta_{roof} = S_d \gamma \tag{1}$$

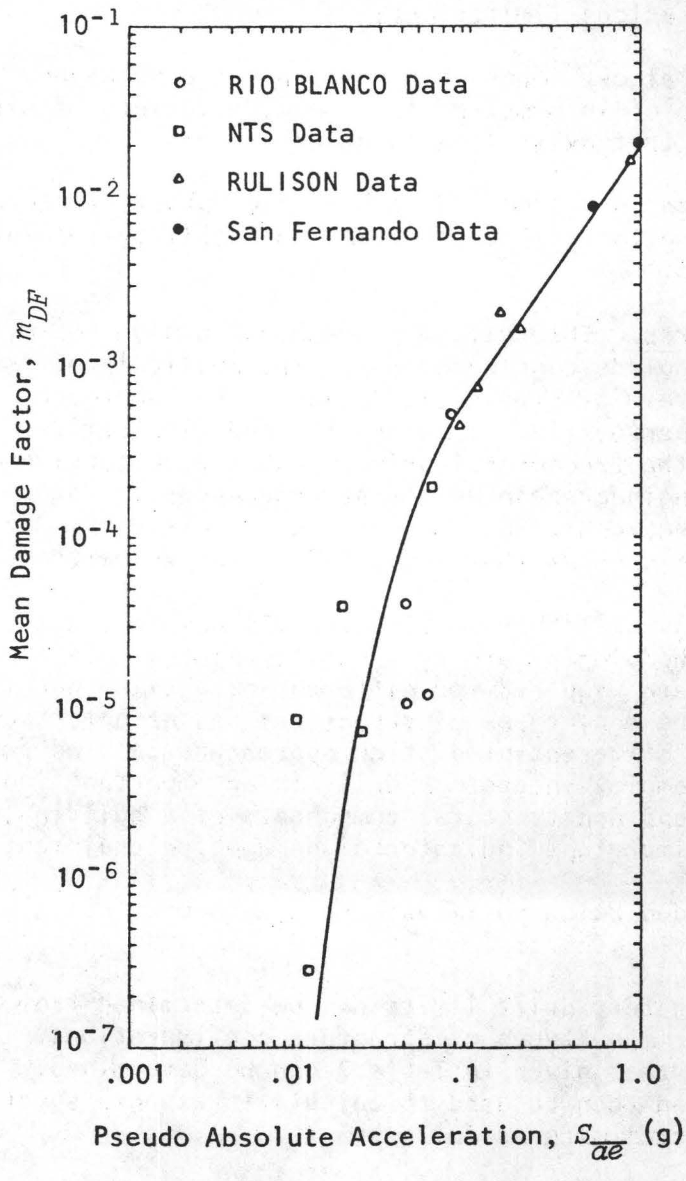


FIGURE 5 MEAN DAMAGE FACTOR VERSUS SPECTRAL ACCELERATION FOR LOW-RISE BUILDINGS (from URS/John A. Blume & Associates, Engineers, 1980)

TABLE 2

INTERSTORY DRIFT LIMITS FOR VARIOUS STRUCTURE TYPES

Lateral-Force-Resisting System	Interstory Drift (cm)		
	Observable Damage, Δu_1	Yield Capacity, Δu_2	Ultimate Capacity, Δu_3
Wood Frame			
Unreinforced Masonry			
Reinforced Masonry			
Reinforced Concrete Frame	.25*	1.0*	5.0*
Reinforced Concrete Shear Wall			
Steel Frame			
Steel Braced Frame			
Steel Eccentrically Braced Frame			

*Values assumed for this example. As further data are obtained, appropriate values can be filled in for each structure type.

where:

δ_{roof} = displacement of the roof relative to the ground

S_d = response spectrum displacement

γ = modal participation factor for fundamental mode with roof displacement normalized to unity

Then, assuming both a straight-line fundamental mode shape and that the fundamental building period, T, can be approximated by:

$$T = 0.1N \tag{2}$$

where:

N = the number of stories

it follows that:

$$\Delta u \cong \frac{\delta_{roof}}{N} = \frac{\gamma S_d}{N} \tag{3}$$

where:

Δu = average interstory drift

Finally:

$$S_d = \frac{N\Delta u}{\gamma} \tag{4}$$

Equations (2) and (4) facilitate plotting various interstory drift limits onto a response spectrum plot. In that form, damage can be crudely estimated by comparing a demand ground motion response spectrum with various structure component capacities developed from interstory drift limits. The calculated S_d values for the example assumed drift limits in Table 2 are given in Table 3. These S_d values are plotted in Figure 6, which also shows a plot of the 5%-damped response spectrum for the 1940 El Centro earthquake record.

Contemporary Earthquake-Resistant Design Philosophy

Structural analysis technology for prediction of earthquake response has advanced significantly in the past 15 years. Linear dynamic response analyses are commonplace today, and nonlinear dynamic response analyses are feasible for simple structures. These analyses are used for calculating structure member stresses and strains and are correspondingly used in design (i.e., structure members are sized by comparing calculated stresses and strains with those allowed by various codes and standards).

Unfortunately, the codes do not specify the degree of damage associated with various prescribed stresses and strains. In addition, the stated philosophy

TABLE 3

RESPONSE SPECTRUM DISPLACEMENTS FOR VARIOUS DAMAGE THRESHOLDS AND BUILDING HEIGHTS

Number of Stories, N	T (sec)	γ	$S_d = \frac{N\Delta u}{\gamma}$		
			Observable Damage	Yield Capacity	Ultimate Capacity
1	0.1	1.0	0.25	1.0	5.0
2	0.2	1.2	0.42	1.67	8.3
3	0.3	1.29	0.58	2.33	11.6
4	0.4	1.33	0.75	3.01	15.0
5	0.5	1.36	0.92	3.68	18.4
10	1.0	1.43	1.75	6.99	35.0
20	2.0	1.46	3.42	13.70	68.5
30	3.0	1.48	5.06	20.27	101.4
40	4.0	≈1.5	6.67	26.67	133.3

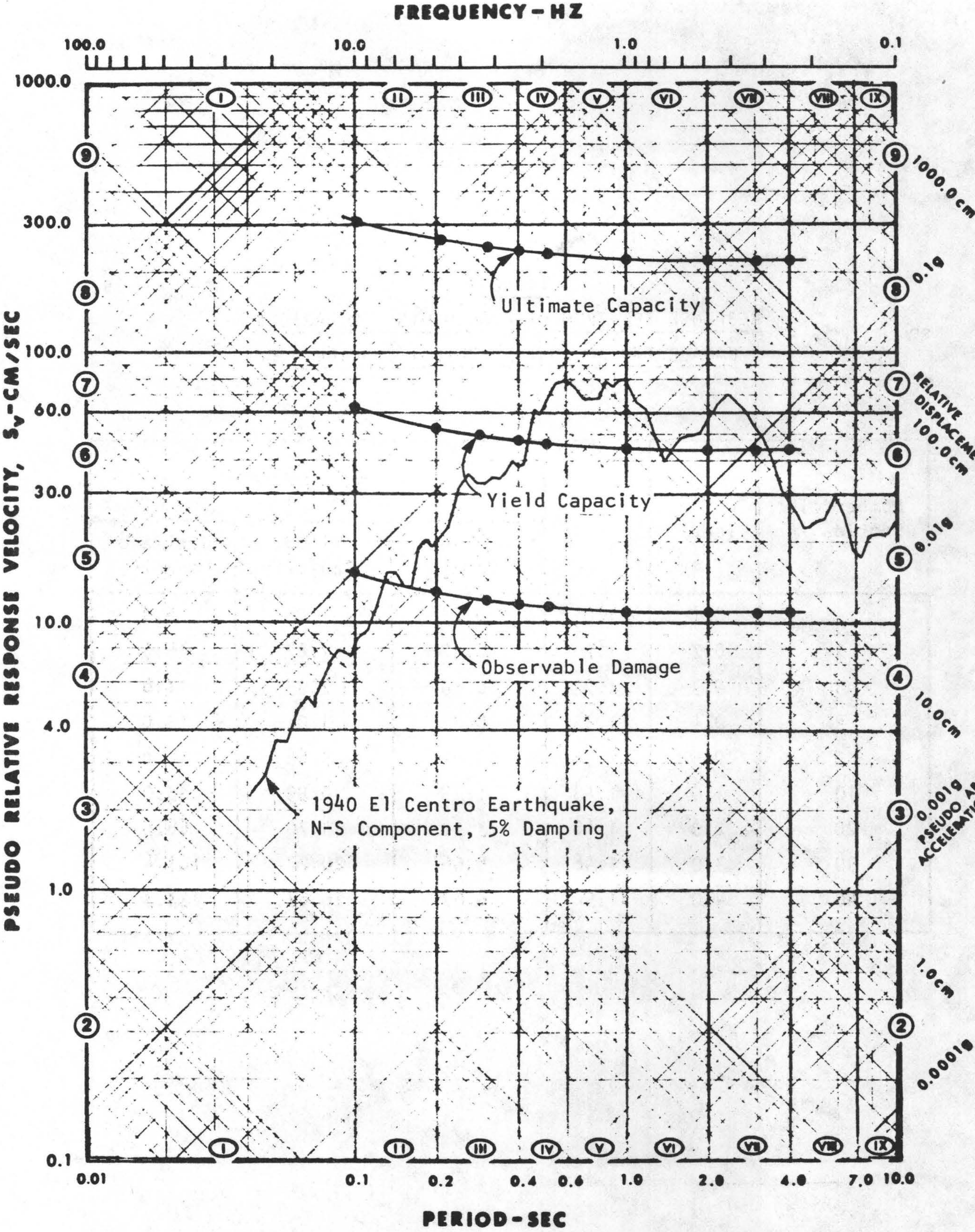


FIGURE 6 RESPONSE SPECTRUM AMPLITUDES FOR VARIOUS DAMAGE THRESHOLDS FOR REINFORCED CONCRETE FRAME STRUCTURES (from URS/John A. Blume & Associates, Engineers, 1980)

of contemporary earthquake design procedures (Structural Engineers Association of California, 1975) is that structures are expected to be damaged during major earthquakes but that collapse is to be precluded by using the recommendations prescribed. Finally, because structures are expected to be damaged during major earthquakes, they will respond nonlinearly.

Figures 7 and 8 illustrate (in an idealized sense) the contemporary code philosophy. Two important observations can be made from these illustrations. First, it is clear that nonlinear response considerations must be included in any attempt to relate response and damage. Second, once damage occurs, the structure behaves nonlinearly, and response is no longer easily tractable through acceleration. Accordingly, ultimate capacity is more appropriately gauged with displacement.

Risk Evaluation and Hazard Reduction

Risk evaluation implies determination of the probability of experiencing loss from some given hazard. Hazard reduction implies establishing ways and means for reducing or mitigating the loss. The processes involved in hazard reduction are, in general, similar to the process of optimization.

Figure 9 is an example of one possible optimization (or hazard reduction) scheme for earthquake-resistant design of structures. In this example, the risk mitigation scheme is to alter the structure capacity. Another risk mitigation scheme is to locate the structure at a site with a lower earthquake hazard.

Hazard reduction is most effectively achieved through the structure design process. The ability to distinguish quantitatively between the effects of one earthquake hazard and another is included in the process. Accordingly, loss evaluations based on the principles of dynamic structure analysis and design are the most expedient means for achieving earthquake hazard reduction.

Timing

The time of day and the time of year an earthquake occurs are significant for earthquake damage prediction. Although the time of day during which a major earthquake strikes has little effect on damage per se, it can affect the number of persons injured or killed. In the western United States, it is generally expected that life loss would be greater for an earthquake that occurs during business hours than for one that occurs during nonbusiness hours. This is simply because during nonbusiness hours a greater percentage of the affected population will be in wood-frame homes, which are generally expected to be more resistant to earthquakes than are typical commercial buildings. Recent earthquakes in other countries have not shown this to be true in all cases, however.

The time of year a major earthquake occurs affects both potential damage and life loss because of changes in climatic conditions. Foundation and soil failures (particularly landslides) are much more likely if an earthquake occurs when the ground is saturated than when the ground is dry.

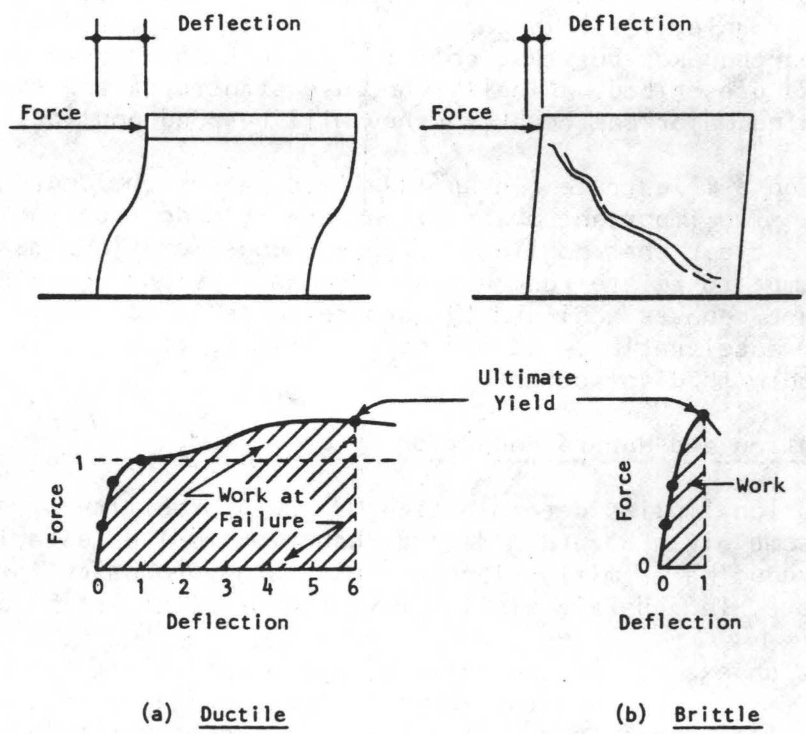


FIGURE 7 BUILDING MATERIAL FAILURE CHARACTERISTICS (from URS/John A. Blume & Associates, Engineers, 1975)

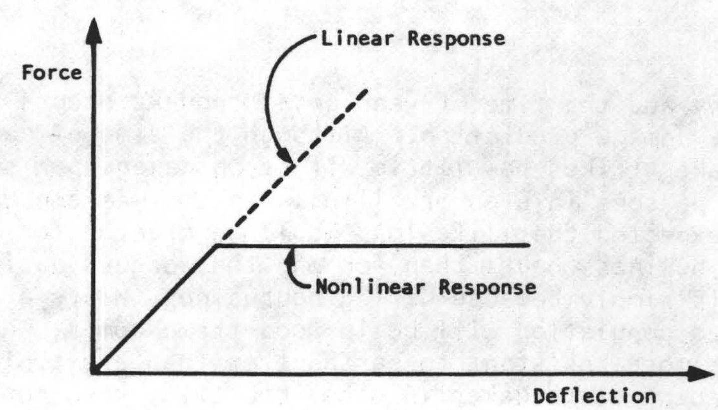


FIGURE 8 EXAMPLE BILINEAR FORCE-DEFLECTION CURVE

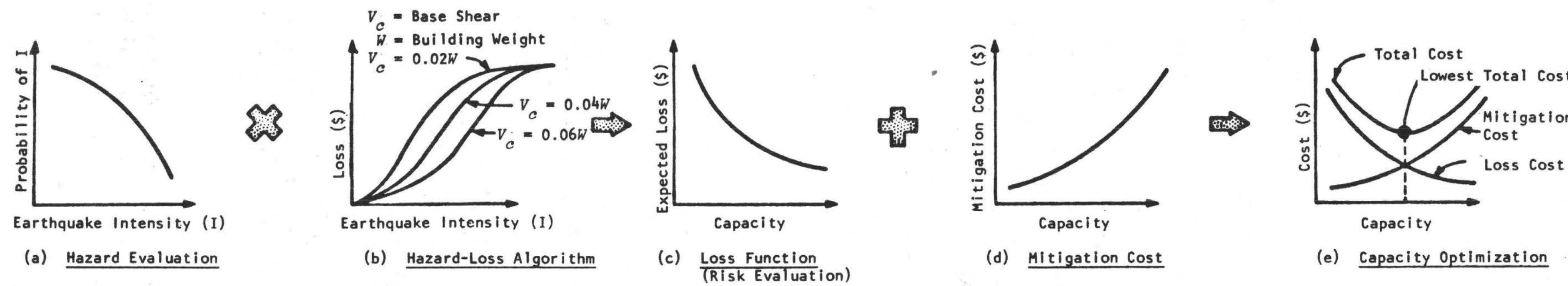


FIGURE 9 EARTHQUAKE-RESISTANT DESIGN OPTIMIZATION

REVIEW OF AVAILABLE PROCEDURES

Methodologies for predicting damage to structures due to ground vibrations have been developed by various investigators.

A methodology for estimating earthquake-induced economic losses to wood-frame dwellings in California was developed by a team led by Steinbrugge, McClure, and Snow (1969) to aid in analyzing the feasibility and effectiveness of earthquake insurance. This method uses the Modified Mercalli Intensity (MMI) scale to describe the intensity of ground motion. For a given earthquake, such as the maximum credible earthquake, empirical isoseismal maps are developed. These maps consider the rupture of the fault (hence, the ellipticity of the isoseismals) and the empirical relationship between magnitude and MMI. Empirical data are used to determine the area enclosed within a given MMI isoseismal line as a function of magnitude. Because MMI is used to represent the ground motion intensity and because MMI is directly related to damage, no structural response calculation is done.

Damage to wood-frame dwellings is estimated by four components: structure, interior finish, exterior finish, and chimney. These damage components are further subdivided to account for major variations within each component, such as age of dwellings. For each damage component, the degree of damage is described by such terms as slight, moderate, severe, and total loss.

The relationship of MMI to the degree of component damage is estimated by professionals and improved using the available data. These MMI-damage relationships are converted into relationships of MMI to repair cost, also estimated by professionals.

To predict losses to wood-frame dwellings within a region, the region under consideration is divided into standard location areas (SLAs). For a given earthquake, MMI is estimated for each SLA. Then losses for each SLA are calculated by using the MMI-loss relationships. Characteristics of the structure population within each SLA (inventory data) are derived mainly from data from the United States Bureau of the Census.

The method is a good one for the type of buildings for which it is intended. The sources of information identified are of great value for similar future studies. However, the method requires a great deal of expertise that can only be provided by experienced professionals from such diverse fields as engineering, statistics, and insurance. Also, the method cannot be applied to other types of structures without extensive modifications.

Studies have been performed to improve the above method and to apply it to other types of structures. Rinehart et al. (1976) have performed a sensitivity analysis to determine the relative significance of various parameters considered in the method with respect to losses. This analysis has led to improvements in the method.

Algermissen et al. (1978a) extended the previous work to cover buildings other than single-family dwellings. In their study, a building inventory methodology was formally introduced. A building classification, not necessarily related to engineering design parameters, was adopted from the Insurance Services Office (ISO) system and used in the method.

On the basis of their previous work, Algermissen et al. (1978b) developed a technique for rapid estimation of earthquake losses. This method entails development of a series of maps showing contours of the percentage of losses for specific building types at each MMI level. The method could be valuable for quick postearthquake loss estimates; however, the necessary data must be collected and processed before an earthquake occurs, and experts with specific understanding of the method must be available.

Culver et al. (1975) describe another method for surveying and evaluating existing buildings to determine the risk to life and to estimate the amount of expected damage. In their method, damage to both structural and nonstructural building components resulting from extreme natural hazards such as earthquakes, hurricanes, and tornadoes is considered. The method can treat a large class of structural types, including braced and unbraced steel frames, concrete frames with and without shear walls, bearing-wall structures, and long-span roof structures.

Culver et al. include three independent but related sets of procedures for estimating damage for each of the natural hazards. The first set of procedures (the Field Evaluation Method) provides a means for qualitatively determining the damage level on the basis of data collected in field surveys. The second set (the Approximate Analytical Evaluation Method) uses a structural analysis of the building to determine the damage level as a function of the behavior of critical elements. The third set (the Detailed Analytical Evaluation Method) is based on a computer analysis of the entire structure. The procedures are presented in a format that allows updating and refining.

The Field Evaluation Method and the Approximate Analytical Evaluation Method do not estimate the extent of damage quantitatively. In the Detailed Analytical Evaluation Method, the ground motion at a site is expressed in terms of a site particle-velocity spectrum, which is obtained by multiplying a hard-rock velocity spectrum by an appropriate soil amplification factor. Alternative procedures are described for obtaining the hard-rock velocity spectrum and the soil amplification factor for a given site.

A response spectrum approach with provisions for amplitude-dependent damping and stiffness characteristics is suggested for calculating the structure's response to the prescribed ground motion. The response parameters used in predicting damage are maximum floor accelerations, floor velocities, and interstory displacements. Three types of damage, namely, structural, nonstructural partition, and nonstructural window damage, are related to these parameters. Structural damage and window damage are assumed to be functions of interstory drift, whereas nonstructural partition damage is assumed to be related to the maximum floor velocity and acceleration.

The relationship between the percentage of structural damage at a given story level and the maximum drift at that level is assumed to be a normally distributed curve defined by a mean ductility to failure and an associated coefficient of variation. Ductility to failure is determined empirically, and professional judgment is exercised in selecting the proper coefficient of variation.

Nonstructural damage at a floor level is estimated by treating that level as a site on the ground subjected to an effective floor MMI, I_i, which is empirically related to maximum floor acceleration and velocity. The relationship between I_i and the percentage of nonstructural damage to the floor is also given by an empirical formula, which includes a parameter called *quality factor*, reflecting the damageability of the specific construction type. The relationship of story drift to glass damage is treated much like structural damage, with a defined drift-to-failure value, an associated coefficient of variation, and assumed normal distribution.

The method described by Culver et al. attempts to relate engineering parameters to the extent of damage suffered by the components of a given structure. However, damage is expressed in percentage only and is not related to monetary loss.

An extensive program, led by Whitman, Biggs, Cornell, and Vanmarcke, has been undertaken at Massachusetts Institute of Technology (MIT) to develop a method titled Optimum Seismic Protection and Building Damage Statistics. The title was later changed (Whitman, 1973) to Seismic Design Decision Analysis (SDDA).

To select the level of seismic resistance to be required for an individual structure or a group of structures, the SDDA considers (1) the cost of providing increased seismic resistance, (2) the damage that may occur during future earthquakes, and (3) the human and social consequences of such damage.

Many studies have been performed and reports published as part of the SDDA program. A description of the program as originally conceived is given in Report No. 1 (Whitman et al., 1972). Theoretical structure response studies are described in Reports No. 3 and No. 4 (Anagnostopoulos, 1972; Biggs and Grace, 1973). Damage data and statistics obtained from the 1971 San Fernando, California, earthquake are given in Report No. 7 (Whitman et al., 1973). Report No. 8, by Whitman (1973), gives damage probability matrices for multistory buildings. Two reports attempt to correlate earthquake damage to tall buildings with strong ground motion parameters (Wong, 1975; Whitman et al., 1977). In Report No. 30, Schumacker and Whitman (1977) apply the methods developed to the estimation of losses to cities and regions.

Czarnecki (1973) has developed a damage prediction method, as part of MIT's SDDA program, that is based on engineering principles and is oriented toward high-rise buildings. In this method, the damage is related to the structural response parameters. The building can be analyzed for a given earthquake using any acceptable dynamic analysis technique, such as response spectrum analysis or linear or nonlinear time-history analysis. Total damage to a given building is classified into components. Components suggested for high-rise buildings are structural damage (damage to steel frames, concrete frames, braced frames, shear walls), nonstructural damage (damage to drywall partitions, exterior glazing, brick masonry walls, concrete block walls), and other damage. Structural damage is fully attributed to the vertical structural elements (e.g., columns and shear walls) and is assumed to be proportional to the inelastic energy absorbed by those elements. Nonstructural damage is associated with maximum interstory drift. Drift-damage curves are developed on the bases of actual data and engineering design practices. No attempt is made to consider the variabilities of the parameters used in the damage prediction or of the final results.

The three distinct methods developed by Blume for predicting damage to structures due to large underground nuclear explosions are equally applicable to predicting damage due to earthquakes. These three methods -- the Engineering Intensity Scale (EIS) method, the Spectral Matrix Method (SMM), and the Threshold Evaluation Method (TEM) -- provide a means for making progressively more detailed predictions of structural effects due to seismic motions.

The EIS method (Blume, 1970) is used to estimate the extent of the area in which structures might be damaged and to make a general evaluation of the incidence and degree of damage to structures within that area. In the formulation of the EIS, ground motion is characterized by 5%-damped spectral velocity (S_v), and structures are characterized by their fundamental-mode vibration properties. Neglecting mode shape considerations, the important correlation variables for relating motion and damage are S_v amplitude and building period. The 5%-damping value is used because damping in many real structures varies from about 2% to 10%, and 5% has been made a standard reference level in the nuclear event structural response program conducted by URS/John A. Blume & Associates, Engineers (URS/Blume), for the Nevada Operations Office of the U.S. Department of Energy.

Engineering intensity (EI) numbers are assigned to various spectral velocity bands. The range of spectral velocities (S_v) and periods (T) applicable to civil engineering structures is divided into a 10 by 9 matrix with ten intensity levels, from 0 through 9, and nine period bands, I through IX, which range from 0.01 sec to 10 sec.

A significant amount of data on ground motion caused by underground nuclear explosions and corresponding damage data have been available for establishing the incidence and degree of damage for various EI ranges for low-rise buildings (Hafen and Kintzer, 1977; URS/Blume, 1975). In addition, motion and damage data from the 1971 San Fernando earthquake for low-rise (Hafen and Kintzer, 1977; Scholl, 1974) and high-rise (Hafen and Kintzer, 1977; Wong, 1975) buildings are available. Motion-damage relationship information for high-rise buildings from Whitman et al. (1977) and the additional correlation work currently in progress at URS/Blume will provide sufficient information for this class of buildings.

The SMM has been in continuous development and use by URS/Blume (or John A. Blume & Associates, Engineers) since 1966. The earliest version was presented in January 1967 (Blume, 1967). The method has subsequently been simplified and further developed (Blume, 1968; Blume and Monroe, 1971; URS/Blume, 1975). The method is based on observed data and theoretical considerations and is applicable to both high-rise and low-rise structures. The SMM uses physical and engineering characteristics of structures and ground motion spectra, including their variabilities, in relating ground motion to structural response and damage. Because of this characteristic, the SMM has potential for further development and application to a variety of structures. A detailed description of the SMM is given in a later section of this paper.

The TEM (Blume, 1969), which is used for predicting the effects of dynamic ground motion on structures, involves a systematic and detailed dynamic structural analysis of individual structures. This method is used to identify both the potential risk from a structure's failure caused by ground motion and modifications that might improve the resistance of that structure to failure. Basically, the TEM is an extension of conventional structural analysis procedures used in design. It requires the identification of various capacity thresholds and the evaluation of the probability of exceeding the thresholds for a given seismic event. It is intended to provide detailed insight into the structural behavior of an individual building under lateral loading and to take advantage of several mitigating factors that are normally ignored in structural design practice in the interest of providing additional, but realistic, margins of safety.

A fundamental step in conducting a threshold evaluation analysis is to develop a mathematical model of the building. Because the TEM considers both elastic and inelastic response, it is usually desirable to develop at least two mathematical models. The frequencies and mode shapes obtained from the mathematical models are used to estimate the response spectrum demand amplitude.

A capacity threshold is defined as the total lateral load that would be required to cause a building to reach a specified level of behavior. For example, a code-required threshold is the base shear coefficient required by an applicable building code. Similarly, a yield limit threshold is the smallest base shear coefficient causing a significant structural member to reach yield stress.

With this information, the probability of exceeding the various capacity thresholds for a particular seismic response spectrum can be evaluated. The significance of a high probability of exceedance depends on the threshold and the severity of the demand spectra being considered. For example, a high probability of exceeding the yield limit or observable damage threshold for a seismic event that is likely to occur several times during the building's useful life may be an unacceptably high risk. However, for the maximum credible seismic event, it may be acceptable to exceed all thresholds except story failure or collapse.

EXAMPLE LOSS EVALUATION METHODOLOGY

General Earthquake Loss Prediction Methodology for an Urban Area

Prediction of losses to an urban area from an earthquake involves a series of complex procedures with many steps requiring extensive computations and data handling. Accordingly, a systematic approach is necessary in which all significant procedures are identified and sequenced and all the necessary information described.

Figure 10 is a flowchart that shows the major procedures requiring well-defined methodologies for systematic prediction of losses to a city or to a designated urban area from an earthquake. These procedures are:

1. Zoning the city and classifying and taking an inventory of structures within each zone (inventory methodology)

2. Predicting the ground motion parameters for each zone (ground motion prediction methodology)

3. Predicting losses for individual structures and groups of structures (loss prediction methodology)

4. Summing losses from all structures and zones for the prediction of losses to the city

The methodologies for accomplishing procedures 1 through 3 are described below. The loss prediction methodology (procedure 3) can be established from empirical or theoretical considerations; the discussion provided below can accommodate either. The development of a theoretically based loss prediction algorithm, the Spectral Matrix Method, is presented in a later section.

<u>Inventory Methodology</u>. Cities are made up of many different types of structures. Even if it were possible to accurately predict losses that might be incurred by each structure, it would be an enormous task to come up with a citywide prediction. Therefore, the first step in the procedure is to divide the city (if it is so large or if the assumed earthquake is so close that the ground motion would vary substantially within the city) into zones for which the ground motion is defined. Then the structures in each zone can be classified into general groups. An example of extensive classification was given in Table 1. In general, conventional structures, such as low-rise, wood-frame residential houses or high-rise office buildings, can be classified so that, for each class, average structural characteristics can be estimated. Special structures, such as power stations, dams, and lifelines, might have to be studied individually to determine their characteristics. The next step is to inventory each classification to determine the number of structures in each class and their replacement values.

Figure 11 is a schematic description of the inventory methodology.

<u>Ground Motion Prediction Methodology</u>. Methods exist for estimating the characteristics of a maximum credible earthquake in regions where earthquake sources and mechanisms have been studied and are understood. Methods also exist for estimating the ground motion characteristics at a site due to a given earthquake with defined magnitude and source. Ground motion characteristics at a site can be described in terms of several parameters, such as:

- Peak values of ground acceleration, velocity, or displacement

- Response spectra for acceleration, velocity, or displacement

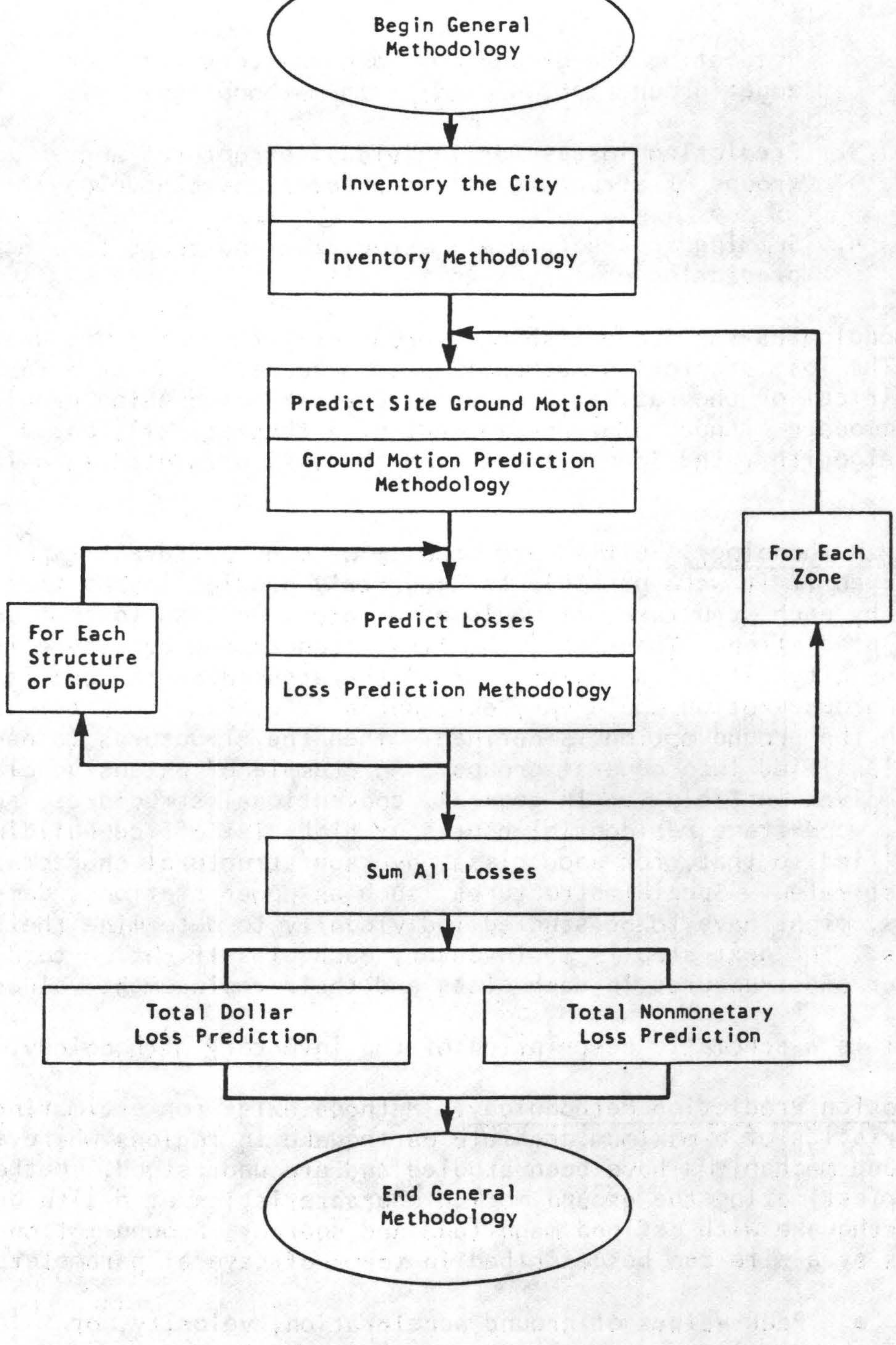


FIGURE 10 GENERAL EARTHQUAKE LOSS PREDICTION METHODOLOGY FOR A CITY

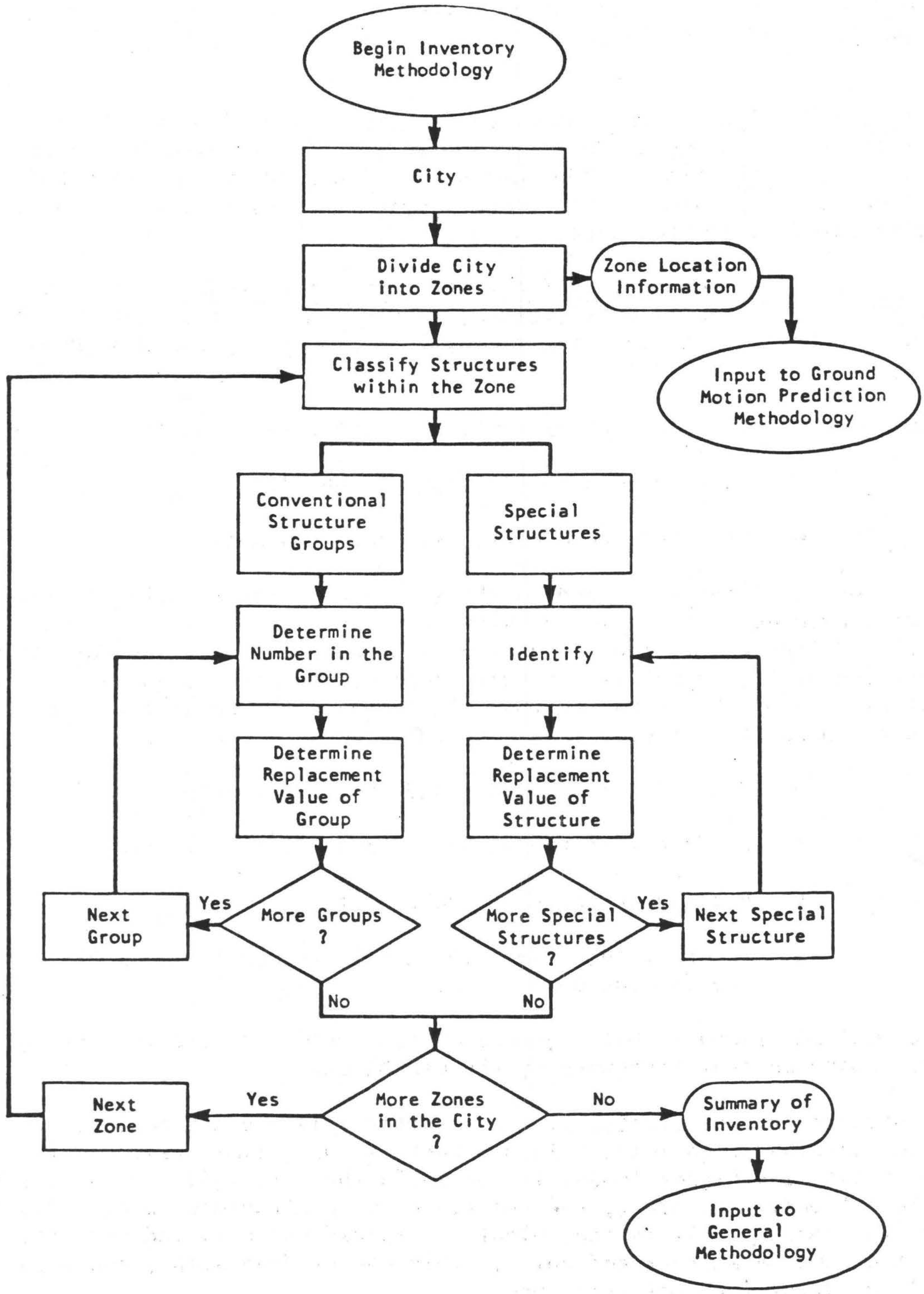


FIGURE 11 INVENTORY METHODOLOGY

- Intensity scales, such as the MMI scale or the EIS

Therefore, for an earthquake described by its magnitude, epicenter location, and depth, the ground motion parameters at a site some distance away with known local soil conditions can be estimated using state-of-the-art technology.

Figure 12, which presents a method for predicting site ground motion characteristics, shows how ground motion prediction is related to the general loss prediction methodology. The loss prediction procedure is independent of the ground motion prediction procedure; therefore, any of the available ground motion prediction procedures can be used.

Loss Prediction Methodology for a Structure or a Group of Structures. Earthquake losses for a given structure, or for a group of structures with common characteristics, can be estimated by following the procedure shown as a flowchart in Figure 13.

The characteristics of two elements are needed as input to this process:

- Earthquake ground motion at the site
- Structure, contents, use, and occupancy

Different structures respond to the same ground motion differently. Sometimes, even apparently similar structures may respond to the same ground motion differently. The response of a structure to a given earthquake is a function of the structure's dynamic response properties as well as the characteristics of the ground motion. The significant parameters that determine the response of a structure to a certain ground motion are:

- Mass of the structure and its distribution
- Stiffness of the structure and its distribution
- Damping capacity of the structure
- Interaction between the structure and the soil at the foundation

The maximum response that a ground motion would generate in a structure is the *demand* on that structure by the earthquake.

Structures are constructed of various materials and are designed to accommodate certain design loads. Each structure has a limit beyond which it cannot resist any higher loads; if forced further, it fails or suffers large deformations. Therefore, the resistance of a structure to earthquake loads is its *capacity*. It is convenient to express capacity and demand in terms of the same parameters and units. This can be done simply and with reasonable accuracy for most structures.

Damage that can be induced in a structure is a function of how large a demand is being made on the structure relative to its capacity. Damage is

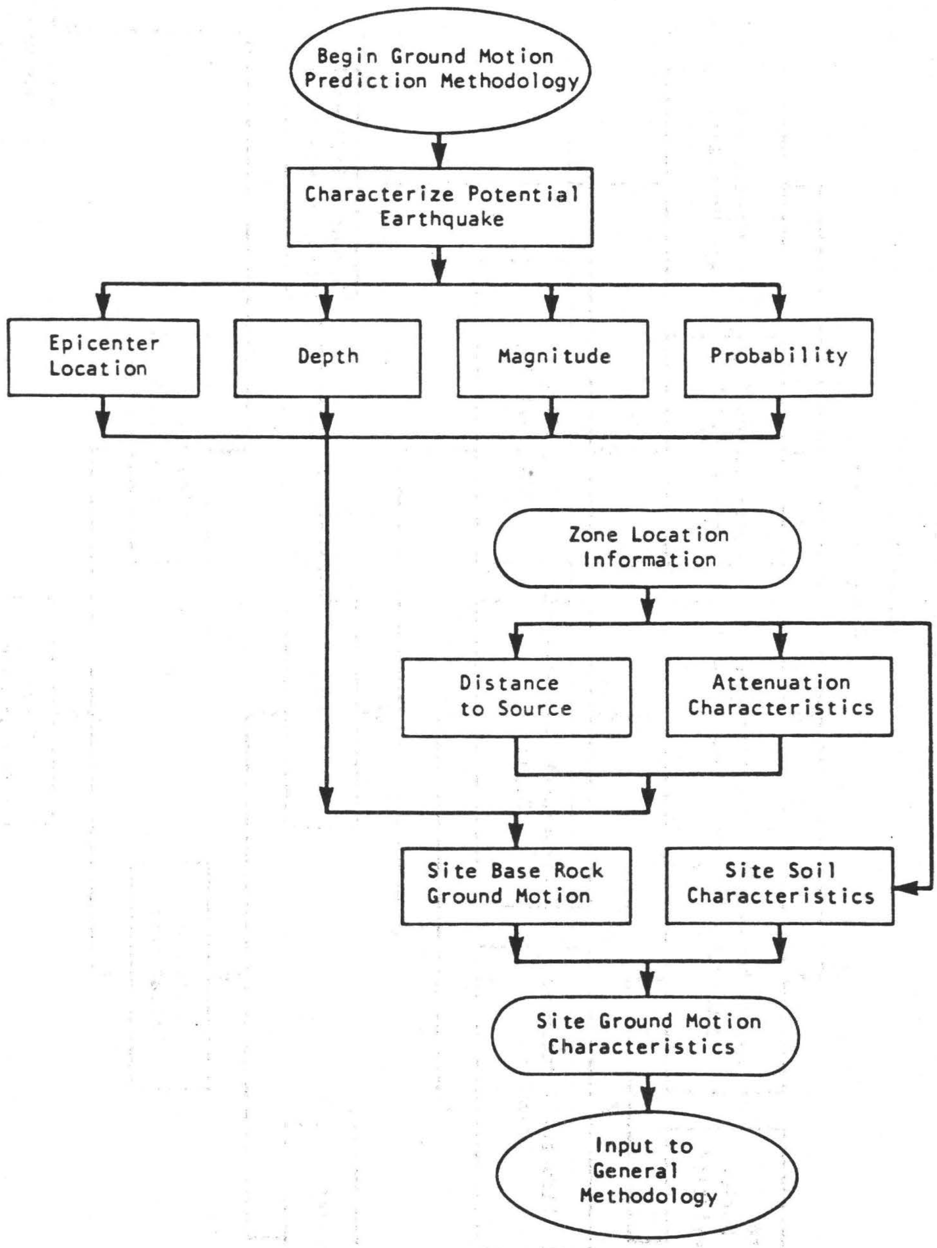


FIGURE 12 SITE GROUND MOTION PREDICTION METHODOLOGY

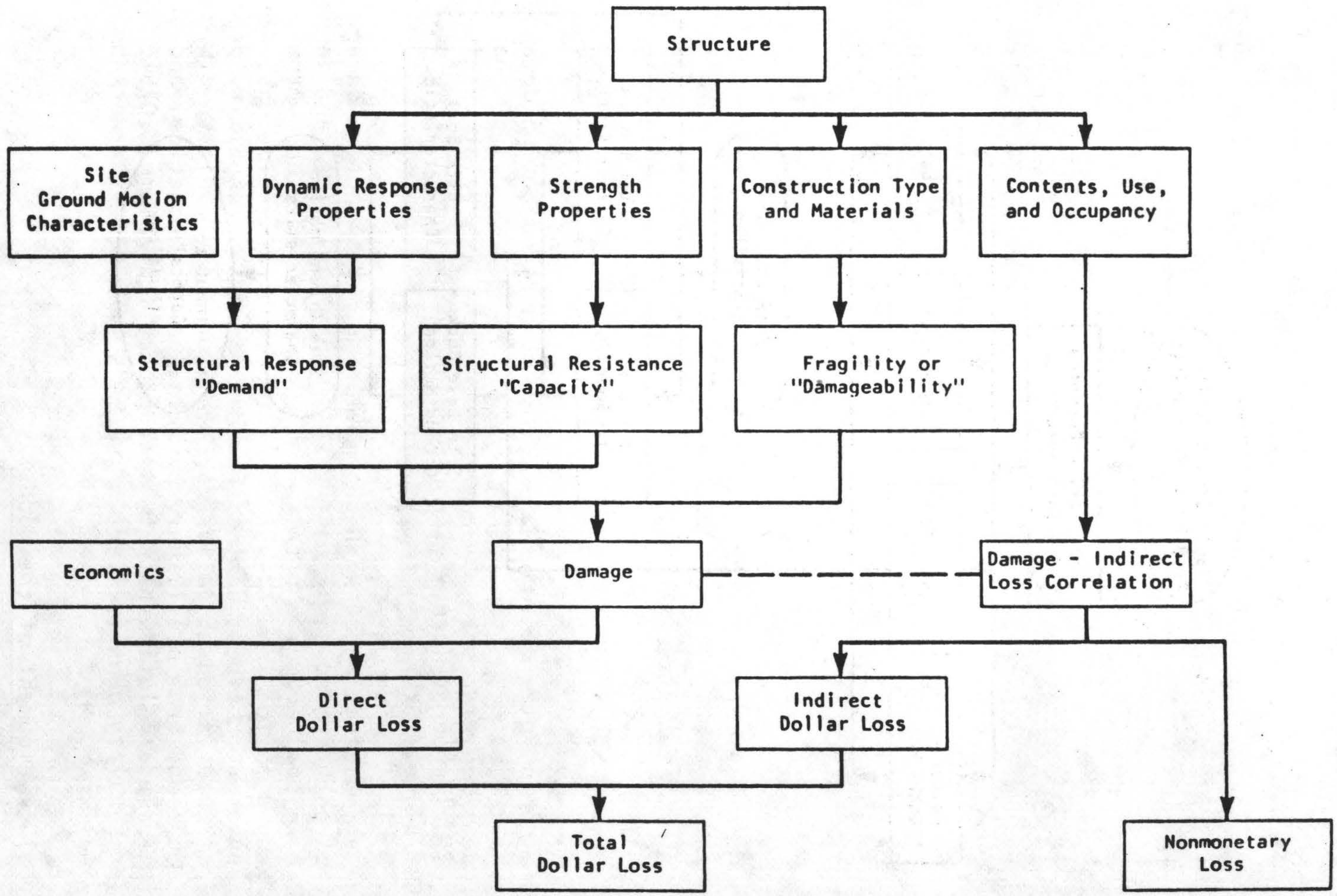


FIGURE 13 LOSS PREDICTION METHODOLOGY FOR A STRUCTURE OR GROUP OF STRUCTURES

also a function of the construction type and materials. Some structures are ductile and can deform without suffering much damage, whereas others are brittle and can suffer extensive damage with little deformation. This property of structures can be expressed in terms of a functional relationship that may be called damage function, fragility, or *damageability*.

Earthquake demand on the structure, the structure's capacity to resist, and its damageability determine how much physical damage may be incurred by the structure. The next two significant steps are to convert the damage into monetary losses and to determine the other possible effects of the damage.

The dollar value of the damage suffered by the structure can be estimated with relative ease, especially for certain types of structures for which data exist from past earthquake experiences. Damage is usually expressed in terms of a *damage factor*, which is the ratio of the estimated value of repairs to the replacement value of the total structure. Therefore, the direct dollar loss is obtained simply by multiplying the damage factor with the replacement value of the structure.

THE THEORETICAL DAMAGE FACTOR MODEL OF THE SPECTRAL MATRIX METHOD

General Considerations

The SMM was conceived as an orderly, standardized procedure for predicting damage to structures subjected to phenomena such as underground nuclear explosions, air blasts, earthquakes, tornadoes, hurricanes, and floods. While the SMM makes a number of contributions to damage prediction technology, its principal feature is its theoretical damage factor model. Initially proposed by Blume (1967), the model has been under continuous development by URS/Blume since 1966 (e.g., Blume, 1968; Blume and Monroe, 1971; URS/Blume, 1975; and Blume, Scholl, and Lum, 1977).

Fundamental principles of the SMM are that the ground motion demand, D, imposed on a structure and the damage-resisting capacity, C, of that structure can be identified by response spectrum values. These relationships are readily established by identifying demand and capacity in terms of base shear (URS/Blume, 1975).

Experimental observation of both ground motion and structure damage has revealed that both demand and capacity are random variables, and damage prediction therefore becomes a problem of joint probabilities. From observation of ground motion induced by underground nuclear explosions, demand variability appears to be best defined by the lognormal probability density function. From observation of failure testing for individual structure elements (Blume, 1967) and from preliminary correlation of theoretical and experimental motion-damage relationships for structures (URS/Blume, 1975), the Weibull probability density function appears to define the variability of capacity well.

Theoretical Development of the Model

The defining damage factor relationship between demand and capacity equates the energy absorbed by the inelastic capacity with an assumed equivalent elastic model. The basic assumption is that the amount of energy absorbed by an individual structure is independent of whether the building responds elastically or inelastically (Blume, 1960; Blume and Monroe, 1971). This relationship is shown in Figure 14.

For the elastic demand model:

$$E = \frac{1}{2}\frac{V_e^2}{K} \tag{5}$$

For the inelastic capacity model:

$$E = \frac{1}{2}\frac{V_y^2}{K} + \left(\frac{V + V_y}{2}\right)(\Delta - \Delta_y) \tag{6}$$

but

$$V = K\Delta_y + (\Delta - \Delta_y)\xi K$$

and

$$\mu = \Delta/\Delta_y$$

Therefore:

$$E = \frac{1}{2}\frac{V_y^2}{K}\left[2(\mu - 1) + (\mu - 1)^2\,\xi + 1\right] \tag{7}$$

By equating Equations (5) and (7), an expression for ductility, μ, can then be obtained:

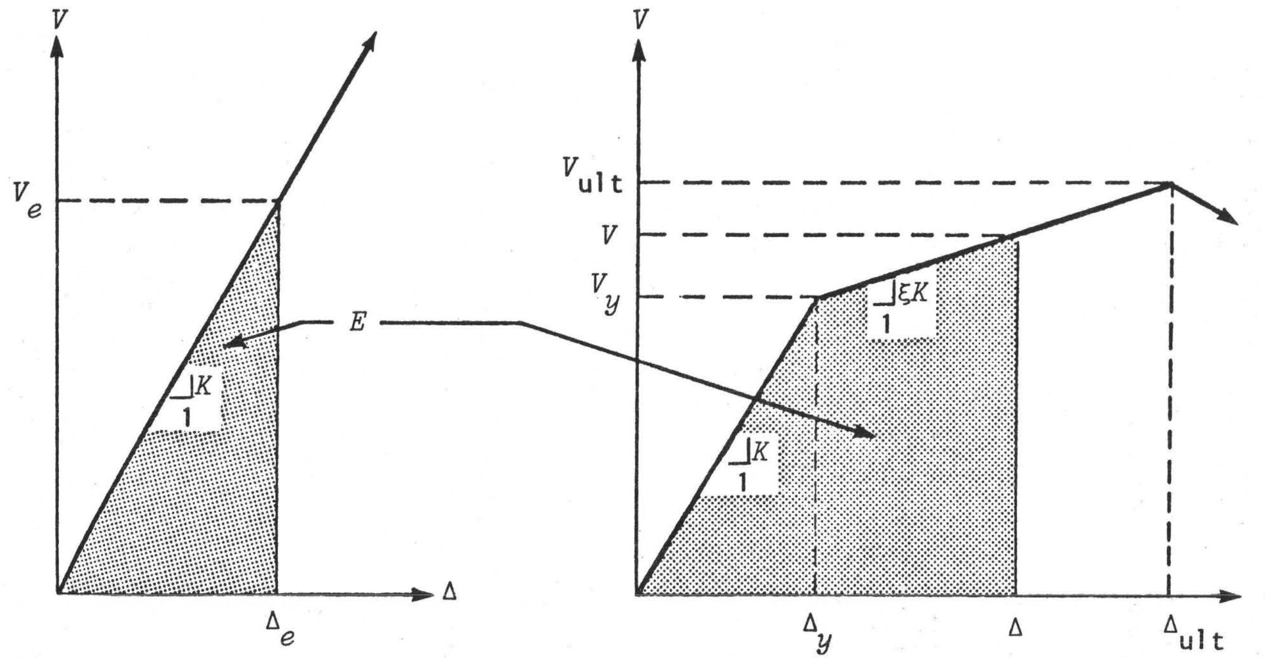
$$\mu = 1 - \frac{1}{\xi} + \sqrt{\frac{1}{\xi}\left[\frac{1}{\xi} + \left(\frac{V_e}{V_y}\right)^2 - 1\right]} \tag{8}$$

where ξ is the bilinear parameter as shown in Figure 14 and V_e/V_y is the ratio of demand over capacity, D/C.

Note that for the elastoplastic case ξ is equal to 0. If one substitutes $\xi = 0$ into Equation (8), numerical solution problems will be encountered. Therefore, derivation of ductility, μ, for the case of $\xi = 0$ is warranted.

The derivation is presented in Blume, Scholl, and Lum (1977). The result shows that for the elastoplastic case:

$$\mu = \frac{1}{2}\left[\left(\frac{V_e}{V_y}\right)^2 - 1\right] + 1 \tag{9}$$



Elastic Demand Model　　　　Inelastic Capacity Model

FIGURE 14 DEMAND AND CAPACITY ENERGY MODELS (from URS/John A. Blume & Associates, Engineers, 1975)

Damage factor is defined as the ratio of dollar damage for a building to the building's replacement value. In the SMM, it is also defined as a function of ductility:

$$DF = \frac{\text{repair cost}}{\text{replacement cost}} = \left(\frac{\mu - 1}{\mu_{ult} - 1}\right)^{\kappa} \tag{10}$$

where κ is an economic scale factor and μ_{ult} is the ultimate ductility.

Substituting Equation (8) into Equation (10) and substituting D/C for V_e/V_y, the formal definition of damage factor is:

$$DF = 0$$

$$\text{if } D/C < 1$$

$$DF = \left[\frac{\sqrt{1 + \xi\left[\left(\frac{D}{C}\right)^2 - 1\right]} - 1}{\xi(\mu_{ult} - 1)}\right]^{\kappa} \tag{11}$$

$$\text{if } 1 \leq D/C \leq \sqrt{2\mu_{ult} - 1 + \xi(\mu_{ult} - 1)^2}$$

$$DF = 1$$

$$\text{if } D/C > \sqrt{2\mu_{ult} - 1 + \xi(\mu_{ult} - 1)^2}$$

For the elastoplastic condition -- a special case -- damage factor is obtained by substituting Equation (9) into Equation (10):

$$DF = 0 \qquad \text{if } D/C < 1$$

$$DF = \left[\frac{\frac{1}{2}\left[\left(\frac{D}{C}\right)^2 - 1\right]}{\mu_{ult} - 1}\right]^{\kappa} \qquad \text{if } 1 \leq D/C \leq \sqrt{2\mu_{ult} - 1} \tag{12}$$

$$DF = 1 \qquad \text{if } D/C > \sqrt{2\mu_{ult} - 1}$$

Demand, D, and capacity, C, are considered to be random variables defined by appropriate probability density functions. The lognormal probability of demand, D, is defined by:

$$p_D(d) = \frac{1}{\sqrt{2\pi}\, d \ln(N)}\, e^{-\frac{1}{2}\left[\frac{1}{\ln(N)} \ln\left(\frac{d}{\check{D}}\right)\right]^2} \qquad d > 0 \tag{13}$$

where:

$\breve{D}$ = median demand value

N = geometric standard deviation

d = known value of demand

D = demand (as a random variable)

ln = log with base e

Figure 15 shows example demand lognormal probability density functions.

The Weibull probability of capacity, C, is defined as:

$$p_C(c) = \frac{k}{u}\left(\frac{c-\varepsilon}{u}\right)^{k-1} e^{-\left(\frac{c-\varepsilon}{u}\right)^k} \qquad c > \varepsilon \tag{14}$$

where:

$$\overline{C} = \varepsilon + u\Gamma\left(1+\frac{1}{k}\right)$$

$$V_C = \frac{u}{\overline{C}}\sqrt{\Gamma\left(1+\frac{2}{k}\right) - \Gamma^2\left(1+\frac{1}{k}\right)}$$

$\Gamma(\ )$ = the gamma function

c = known value of capacity

C = capacity (as a random variable)

Figure 16 shows example capacity Weibull probability density functions.

Using the standard procedure for computing the expectation of a function of a random variable (Benjamin and Cornell, 1970), the general expressions for mean, m_{DF}, and mean square, $E(DF^2)$, of the damage factor are:

$$m_{DF} = \int_1^{\lim} p_{D/C}(x)\left[\frac{\sqrt{1+\xi(x^2-1)}-1}{\xi(\mu_{ult}-1)}\right]^{\kappa} dx + \int_{\lim}^{\infty} p_{D/C}(x)\,dx \tag{15}$$

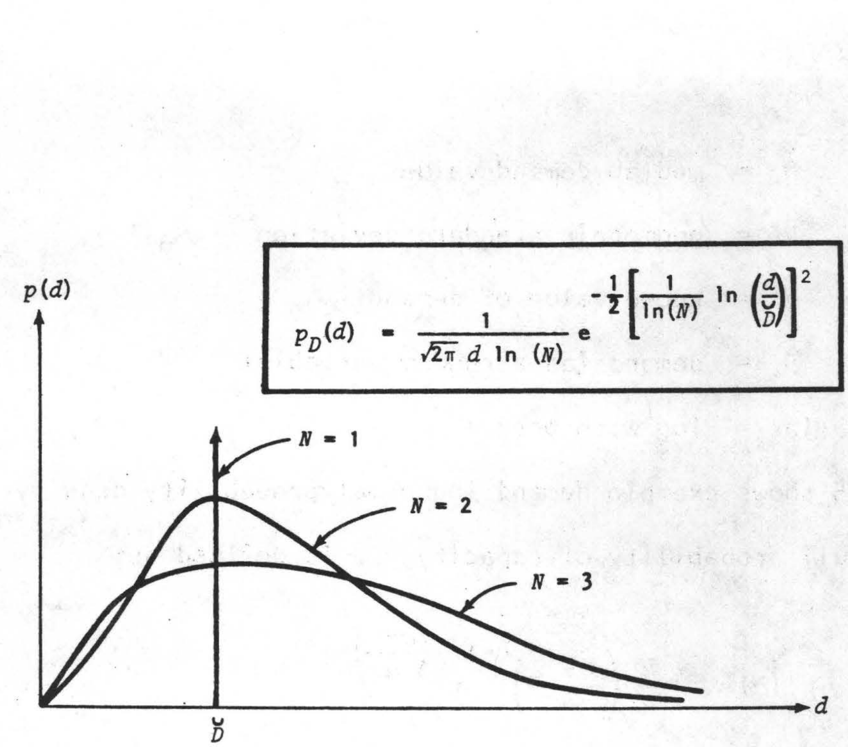


FIGURE 15 EXAMPLE DEMAND LOGNORMAL PROBABILITY DENSITY FUNCTIONS (from URS/John A. Blume & Associates, Engineers, 1975)

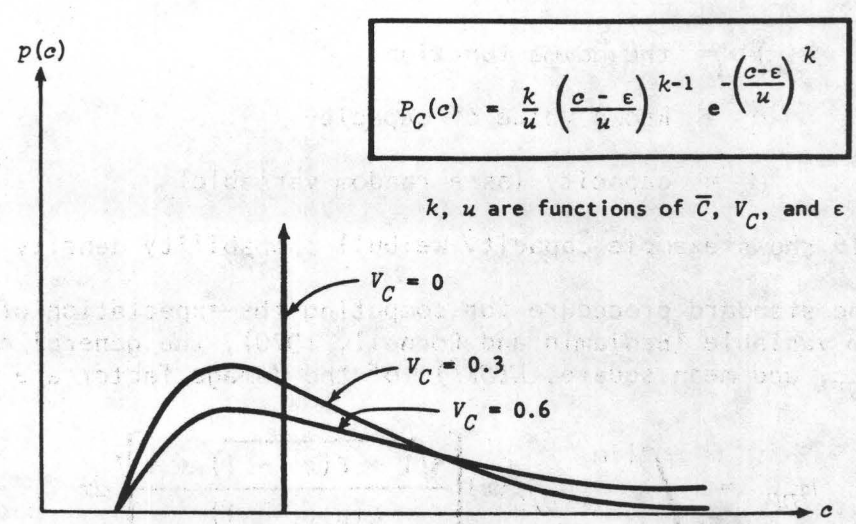


FIGURE 16 EXAMPLE CAPACITY WEIBULL PROBABILITY DENSITY FUNCTIONS (from URS/John A. Blume & Associates, Engineers, 1975)

$$E(DF^2) = \int_1^{\lim} p_{D/C}(x) \left[\frac{\sqrt{1 + \xi(x^2 - 1)} - 1}{\xi(\mu_{ult} - 1)} \right]^{2\kappa} dx$$

$$+ \int_{\lim}^{\infty} p_{D/C}(x)\, dx \qquad (16)$$

where:

$$\lim = \sqrt{2\mu_{ult} - 1 + \xi(\mu_{ult} - 1)^2}$$

$p_{D/C}(x)$ = probability density function of d/c equal to x

These expressions for m_{DF} and $E(DF^2)$ are derived in URS/Blume (1975).

The function $p_{D/C}(x)$ is the probability density function of the quotient D/C, which is derived to be:

$$p_{D/C}(x) = \int_{-\infty}^{\infty} |c| p_{D,C}(xc,c)\, dc \qquad (17)$$

where:

$p_{D,C}(xc,c)$ = joint probability density for demand and capacity

x = specified value of d/c

It is reasonable to assume that demand and capacity are independent, which allows $p_{D,C}(d,c)$ to be factored as follows:

$$p_{D,C}(d,c) = p_D(d)p_C(c) \qquad (18)$$

Combining Equations (17) and (18), and using the definitions for $p_D(d)$ and $p_C(c)$ given by Equations (13) and (14), $p_{D/C}(x)$ is expressed as follows:

$$p_{D/C}(x) = \frac{k\displaystyle\int_{\varepsilon}^{\infty} \left(\frac{c - \varepsilon}{u}\right)^{k-1} e^{-\left\{\left(\frac{c-\varepsilon}{u}\right)^k + \frac{1}{2}\left[\frac{1}{\ln(N)} \ln\left(\frac{xc}{\breve{D}}\right)\right]^2\right\}} dc}{\sqrt{2\pi}\, xu \ln(N)} \qquad (19)$$

Equation (19) is derived when probabilities of both demand and capacity are uncertain. However, when capacity is certain; that is, when

$$\int_{-\infty}^{\infty} P_c(c)dc = 1$$

then:

$$p_{D/C}(x) = \frac{1}{\sqrt{2\pi}\, x \ \ln\,(N)} \, e^{-\frac{1}{2}\left[\frac{1}{\ln(N)} \ln\left(\frac{x\overline{C}}{\breve{D}}\right)\right]^2} \tag{20}$$

and when demand is certain:

$$p_{D/C}(x) = \frac{D}{x^2}\,\frac{k}{u}\left(\frac{\frac{\breve{D}}{x} - \varepsilon}{u}\right)^{k-1} e^{-\left(\frac{\frac{\breve{D}}{x} - \varepsilon}{u}\right)^{k}} \tag{21}$$

The detailed derivations of Equations (19), (20), and (21) are presented in Blume, Scholl, and Lum (1977).

Values for m_{DF} and $E(DF^2)$ can be obtained by numerical integration using Equations (15) and (16) with the expression for $p_{D/C}(x)$ given above. Finally, the standard deviation of the damage factor, σ_{DF}, is obtained from the standard relationship:

$$\sigma_{DF} = \sqrt{E(DF^2) - m_{DF}^2} \tag{22}$$

From the derivations of mean and standard deviation of damage factors, it can be seen that several parameters have been introduced that distinguish various structure types for predictions. These structure-based parameters are included in the theoretical damage factor model to take best advantage of available structure-element test data and thus to facilitate application of the procedure to predictions involving structures for which no empirical data on motion-damage relationships exist. Specifically, these parameters are: ultimate ductility, μ_{ult}; mean capacity, $\overline{C}$; lower bound on damage, ε; coefficient of variation of capacity, V_C; bilinear parameter, ξ; and economic scale factor, κ. A detailed discussion of the ranges of these parameters for three important classes of structures -- high-rise, low-rise, and light industrial buildings -- is given in Blume, Scholl, and Lum (1977).

SMM Calibration for Low-Rise Buildings

Substantial empirical data pertaining to damage to low-rise, wood-frame buildings caused by ground motion have been documented in the past decade. (See, for example, Figure 5.)

Figure 17 shows example mean and standard deviation damage factor curves for low-rise buildings. Rather than plotting the damage statistics as functions of the median demand, $\breve{D}$, the normalized variable $\overline{\breve{D}/\breve{C}}$ is used. The mean and standard deviation damage factor curves for different values of the demand geometric standard deviation are plotted. The curves for N equal to 1 correspond to the situation where demand is known with certainty. Also shown are empirically derived data points. These points confirm the reasonableness of the N-equal-to-1 curve. The curves for values of N greater than 1 are for the more typical situation in which the ground motion demand is uncertain.

SUMMARY OF RESEARCH NEEDS

Methodologies proposed by various investigators for estimating earthquake losses have been briefly described in this paper. These methodologies have contributed significantly to earthquake loss prediction technology; however, none can be regarded as comprehensive because none provide sufficient detail to facilitate making changes to important structures based on engineering characteristics of ground motion that affect damage and because none can be used in comparing and selecting design strategies. Loss prediction procedures must be comprehensible and useful to the structure designer to facilitate earthquake hazard reduction.

Loss prediction procedures proposed to date that do consider all structures are so general that it is difficult, if not impossible, for all but their authors to make the changes necessary to account for updated seismic design criteria and industry technology developments. For example, new freeway bridges built according to updated seismic design criteria will likely not experience as much damage as the older bridges that were affected by the 1971 San Fernando earthquake. Therefore, a need exists for comprehensive loss prediction methodologies that are based on engineering analysis and design principles and that incorporate structure and ground motion parameters commonly used in design. These methodologies should be applicable to all types of structures and should have provisions for evaluating potential life loss or injury and for evaluating secondary economic losses on the basis of structure usage.

The Spectral Matrix Method developed by Blume appears to be the most highly developed of the general, theoretically based damage prediction methods. The SMM warrants further development and practical use because it considers, on a rational and realistic basis, most of the significant engineering parameters affecting damage. However, the ranges of these parameters for various types of structures need to be verified with actual data from past earthquakes, engineering analysis, and laboratory experiments.

Several reports of significant postearthquake damage investigations provide insight concerning earthquake losses: Lawson (1908), Freeman (1932), Martel (1936), Steinbrugge and Moran (1954), Steinbrugge et al. (1971), Scholl (1974), Whitman et al. (1977), and Hafen and Kintzer (1977). Most of the specific loss-ratio information available pertains only to low-rise and high-rise buildings -- yet these two classes of structures constituted only about one-half the total damage caused by the 1971 San Fernando earthquake (Steinbrugge et al., 1971). A significant need exists for earthquake damage

Factor, σ_{DF}

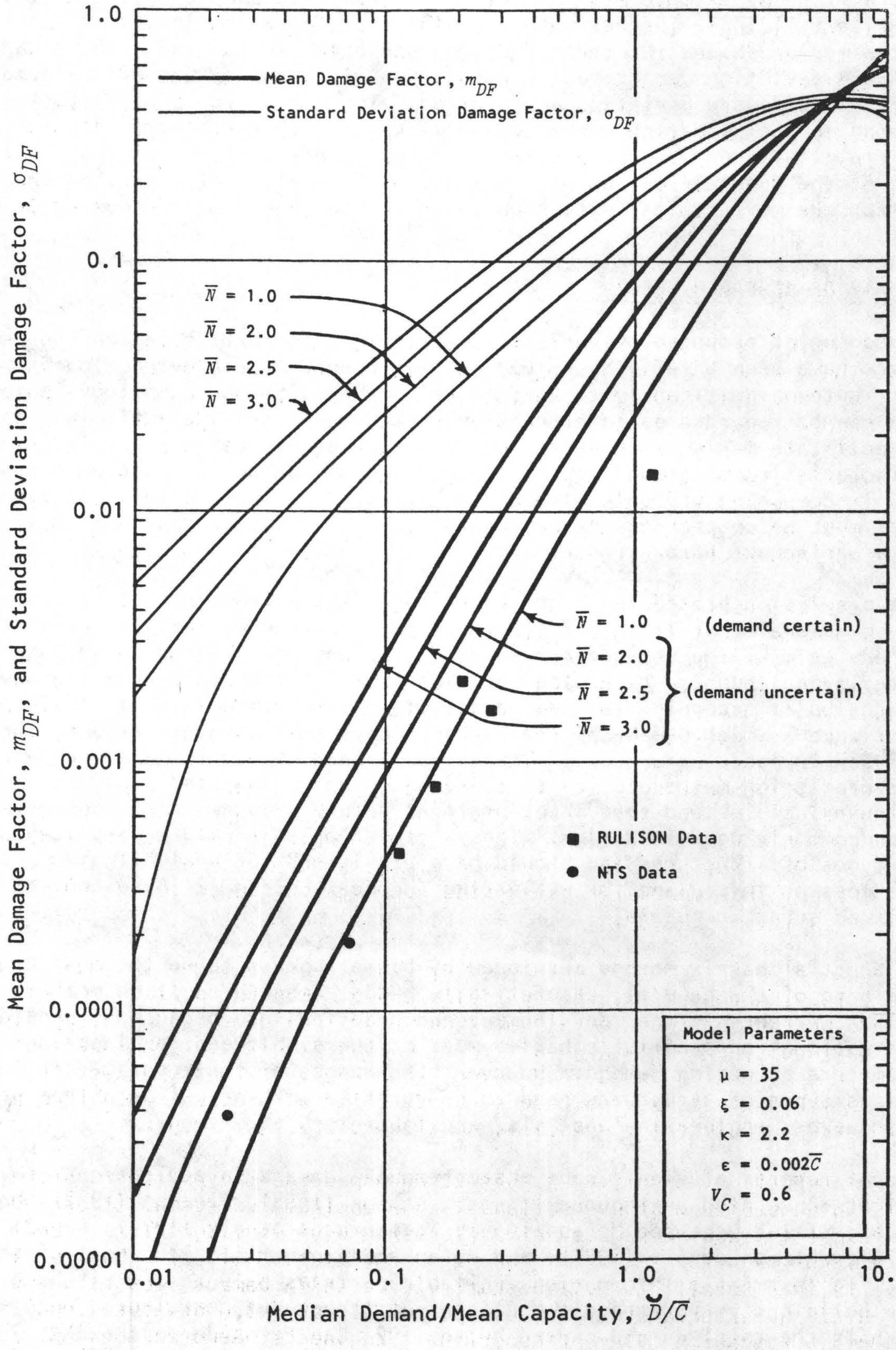


FIGURE 17 MEAN AND STANDARD DEVIATION DAMAGE FACTOR RELATIONSHIPS FOR INDIVIDUAL LOW-RISE BUILDINGS (from URS/John A. Blume & Associates, Engineers, 1975)

data for all other types of structures, in addition to low-rise and high-rise buildings.

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THE CONSEQUENCES OF EARTHQUAKE RISK MAPPING ON PUBLIC POLICY DECISIONS

by

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INTRODUCTION

Beginning in 1974, when the National Science Foundation requested the J. H. Wiggins Company to develop budgeting justifications for earthquake engineering research, studies have progressed in three major areas:

(1) Risk Mapping
(2) Loss Projections
(3) Public Policy Analysis

The early work resulted in seismic risk maps, which have been presented in terms of effective peak acceleration, effective peak velocity, and Modified Mercalli Intensity. Loss estimates were based on a detailed evaluation of the exposure located in over 3,000 counties in the United States for the years 1970 through 2000.

Others have developed risk maps for the United States. Principal among these are the Algermissen-Perkins and the ATC-3 Maps. Others have been produced by various investigators, however, these two are generally used the most to explain seismic risk in terms of "peak" acceleration. The J. H. Wiggins Company has also produced two risk maps. One represents risk for "hard rock" throughout the country. The second attempts to model the effect of soil conditions based on surface geology discriptors.

Though these four maps may look somewhat similar, the question remains: "What are the loss consequences that may be projected from each map, and how might these losses influence public policy?".

Using our exposure model and estimates about the quality of construction present throughout the United States losses projected from the different maps are computed and compared.

ISSUES

In developing the comparison of loss comparisons for the four different seismic risk maps, there are a number of different issues that can be raised by serious investigators. The first concerns the damage algorithms that may be appropriate for use in the various counties throughout the country. There are a number that have been published in the literature throughout the years as a function of MMI. All these are different and may be quite sensitive to the outcomes.

A second question that needs to be addressed, if not answered, concerns the uncertainties associated with the various parameters involved with the study, namely, the earthquake magnitude capabilities of certain seismic source zones, the attenuation properties of various regions throughout the country, the various qualities or classes of structures that exist in each county, not only for the year 1970, but through 2000. There are a number of other specific issues, but, these are central to this problem.

Another central issue that is not really an earthquake engineering problem, but, which concerns the policy question, is the one of the exposure. Most exposures are presented in economic terms and not in structural terms. There is, therefore, difficulty in ascribing the quality of construction or class of construction that might result from the economic profiles derived.

Lastly, in the public policy making business, it is important to consider all losses that may result. Those that are presented in this paper concern only the structural losses. However, there are others, some of which our model will define, but which have not been considered herein. These are losses to contents, business interruption, lifeline and other infrastructure, increased costs of repair, life loss, injury and health costs, secondary and higher order losses, benefits, etc. All of these must be balanced for certain kinds of earthquake events (namely size and amount of exposure within the highly shaken region) so as to know what kind of public policy is at issue.

ASSUMPTIONS

In the exposure model the county is the smallest grain that we have considered. Likewise, all the exposure by county has been placed directly at the longitude and latitude of the county seat. Smaller microzoning within a county was not accomplished.

The damage algorithms used were those developed by the J. H. Wiggins Company in 1975 using principally the published data of damages resulting from the San Fernando earthquake of 1971, the Bakersfield earthquake of 1952, and estimates made by a number of investigators who published prior to 1975.

Assumptions about the distribution of construction throughout the United States were as follows. Outside of California, and before the period of 1940, all construction was considered to be Q = 1 (that is the equivalent performance of average unreinforced masonry). After 1940, and also for regions outside of California, the quality of construction was considered to be equal to Q = 2 (a building resistance comparable to lie somewhere between a UBC zone 3 design and a Q = 1 quality of construction). Reasoning for this 1940 breakoff period was based on the fact that minimum property standards began to be exercised in 1940.

Inside of California, structures built prior to 1934 were regarded to be Q = 1. All construction built during or after 1934 was considered to be Q = 3 (equivalent to UBC zone 3 design).

The sources of the exposure data were developed through the Bureau of the Census, the Water Resources Research Council, and the Harvard School of Business reports on the economic sectors within the economy. All county records were researched for formulas for evaluating taxable property values and brought to current (1970) values. Eight private sectors of the economy were examined: agricultural, mining, construction, manufacturing, transportation, wholesale/retail trade, finance, insurance, real estate, and service. Likewise, multi-family and single family dwellings were identified as well as state, local, and Federal construction.

Other assumptions were made, in that the maps of each of the investigators studied were interpolated where no peak values were identified within a closed contour. Estimates were made as to what the "peak" values might be.

Another assumption that was made, and possibly a problem that might result, was in the definition of the term "acceleration". Necessarily accelerations had to be translated into terms of Modified Mercallian Intensity. In each case we identified acceleration as that value which would be developed from a regression equation of a number of values of accelerations taken from a number of accelerometers. Both the peak value of the high component and the peak value of the low component of each seismometer was used in the data. If the accelerations were meant to be otherwise in this assumption, differences in the comparisons would develop.

"Rock" assumptions were made by the author. What "rock" means to Algermissen-Perkins, ATC-3, and Wiggins may be three different things. Nevertheless, my assumptions were made as to what I thought the other authors meant and were applied in the comparisons.

RESULTS OF THE STUDY

The total annualized building losses, in terms of 1970 dollars for 1980 construction exposures, did not vary much between the four maps. The Algermissen-Perkins total was $450 million, ATC-3 was $339 million, Wiggins "rock" amounted to $742 million, while Wiggins "soil" amounted to $689 million. In other words, there is only about a two to one difference between the four different maps.

There are considerable differences between some of the major states that show annualized earthquake damages, however. The table below shows the high and low values estimated for the principal states involved.

State	High Loss Estimate	Low Loss Estimate	Ratio
California	$440,000,000	$250,000,000	1.8
Colorado	42,000,000	-0-	Very large
Illinois	20,000,000	280,000	71
Massachusets	25,000,000	480,000	52
Missouri	26,000,000	1,300,000	20
New York	23,000,000	2,200,000	10
New Mexico	13,000,000	140,000	93
Ohio	21,000,000	150,000	140
Tennessee	18,000,000	1,800,000	10
Utah	15,000,000	4,300,000	3.5
Washington	110,000,000	2,200,000	50

It can easily be seen that if the annualized losses are only $200,000 or so, only a mild public policy needs to be invoked for the state in question. Likewise, when there are variances as much as 140 to 1, it appears that considerable policy implications would result, depending on which risk map was used by the public policy maker. Naturally, east of the Rocky Mountains, policy makers will use that map which shows their state having the least impact from earthquake.

RECOMMENDATIONS

Because of the regional variances, I think it is very important to develop some consensus on methods of risk mapping and finally the development of a single risk map to be used for insurance purposes, public policy making, siting of structures, etc. I suggest this be done by accomplishing the following tasks.

(1) Convene a risk mapping committee to determine how risk mapping should be done.

(2) Convene an exposure mapping committee to determine how exposures should be reflected.

(3) Convene a damage algorithm committee to determine which damage algorithms are appropriate for the various kinds of economic descriptors used by the exposure researchers.

(4) Convene a loss definition committee which will describe the degree of depth losses should be defined as was listed above.

(5) Convene a committee which would be able to ascribe various qualities of construction to various economic sectors for different regions of the country.

(6) Perform sensitivity studies on the computer outcomes in order to identify specific research issues.

(7) Prepare a risk map and a loss analysis map for public policy makers.

EDUCATING PEOPLE TO UNDERSTAND AND DEAL WITH SEISMIC HAZARDS

by

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This paper presents an up date on the state of current federal, state, local and private programs designed to educate individuals to understand and deal with seismic hazards on a personal level. As the need for earthquake preparedness education becomes greater each year, several important questions must be addressed:

1. Who will take the responsibility and pay the price for educating people to understand and deal with seismic hazards?

2. What 'life-saving' and 'life-sustaining' information is needed by people who will have to deal with seismic hazards?

3. How is the best way to reach the general population and motivate them to become aware and prepared before a major seismic event occurs?

4. What effect will earthquake prediction capabilities have on the need for educating people to understand and deal with seismic hazards?

For an overview on the state of public preparedness education (through January, 1980) refer to Earthquake Predictions and their Effects on Preparedness: A Public Education Prespective by Shirley M. Smith in the Published Proceedings of Conference XII Earthquake Prediction Information (pages 307 - 314)*. At that time there was a general lack of public awareness and preparedness in the area of how to survive and cope in an earthquake. There was also no major effort toward earthquake education in the schools.

*Open-File Report 80-843 United States Department of the Interior, Geological Survey.

The federal and state governments were providing only minimal direction and no over-all program. A majority of their emphasis, budget and manpower, was being directed toward hazard reduction measures or post-impact response at the state and community levels.

Since that time, three bills related to educating people to deal with seismic hazards have come before the California State Legislature. SB 17-33 asked for $990,000 for earthquake awareness education and was tied into revenue from personalized California license plates. This bill died in the Finance Committee in May. AB 2201, a $128,000 project to update the planning guide used by California schools in writing their plans for procedures during and after an earthquake was defeated in the legislature in August. AB 2202 appropriated $750,000 in state funds and another $750,000 in matching funds from the Federal Emergency Management Agency for a project based in Southern California on preparations for a major earthquake with an eye toward prediction. Fortunately, this bill did pass the state legislature in the last week of the 1980 legislative session. This stepping up of efforts to improve the California earthquake readiness posture follows President Carter's visit to the volcanic eruption of Mt. St. Helens. This natural disaster impressed on the President, the importance of reviewing preparedness for other relatively infrequent, but potentially catastrophic, natural events such as earthquakes.

Due to the passage of AB 2202, "A cooperative study is getting underway with FEMA, Region IX, the California Seismic Safety Commission, state and local governments, other federal departments. voluntary agencies, practicing professions, business and commercial interests, labor, educators and researchers to develop an effective program to respond to an earthquake or a credible earthquake prediction in that part of the state. The emphasis is being placed on public safety, reduction of property damage, self-help on the part of individuals, socio-economic impacts, improved response and long-range recovery planning, mitigation activities, and public participation for both the post prediction and immediate post earthquake periods. This pilot effort is expected to be useable in other highly seismic areas of California as well as other states".*

It is interesting to note how the Federal Government intends to interface with state and local people in this project. Through a contact in the Department of Transportation it was learned that an ad hoc committee of federal agencies has been formed for this purpose. Unfortunately, I have no additional information to relate about this committee or any other specific federal actions. This is due to the fact that after five phone calls and a written request made to Dr. Charles Thiel of FEMA's Research and Hazard Mitigation, between May and September, there has been no response.

At the state and local level, Alex Cunningham of the Office of Emergency Services and John McLeod of the California Seismic Safety Commission report that the project is on its way. A Policy Advisory Board composed of 20 representatives from a variety of organizations and agencies in Southern California, has been selected and has met twice. The goal is to hire a staff and proceed with the ambitious project of developing a prediction response plan by October 1, 1981.

Until the passage of AB 2202, there had been no educational programs sponsored by either the federal or state governments. A few local communities and private organizations, along with the business sector, have so strongly felt the need for personal preparedness education that they have been

*NEWS, Federal Emergency Management Agency, Office of Public Affairs, 1725 I Street, N.W., Washington, D.C. 20472. September 29, 1980.

sponsoring their own programs. This has led to a wide variation in types of information presented, methods of dissemination and target audiences. Among the most successful have been the Girl Scout 'Quake Safe Badge' project developed in Santa Clara, California; the program for elementary education by Marilyn MacCabe at the U.S. Geological Survey, Menlo Park; the community college credit course taught by George Thyden from Huntington Beach; the employee preparedness program developed by Levi Strauss Co. in San Francisco; and the personal/family preparedness program by Creative Home Economics Services (CHES) in Southern California.

Being a partner in CHES and developer of this teaching concept, I would like to share with you some of the important elements for a successful earthquake preparedness education program. The initial phases of this personal/family preparedness program are described in the following excerpts from the report - Earthquake Predictions and their Effects on Preparedness: A Public Education Perspective by Smith.

> "The members of Creative Home Economics Services (CHES) gathered a body of detailed information about earthquake survival and needed home emergency supplies. We created a workshop format to teach the basics of home and family earthquake preparedness for impact and the days to follow. We incorporated as much valid information as we could find and used our imaginations to expand beyond existing literature.
>
> We then embarked on our preparedness education effort without sponsorship or funding. We continued to develop and refine our program over a three year period and expanded the workshop formats to include not only sessions suitable for the public but sessions for training the trainers to teach others. We sampled a broad cross section of potential future disseminating groups so we would be ready if community support ever materialized. Our list of contacts became wide and representative of opinion-making groups.

We produced an inexpensive spiral-bound handbook entitled HOW TO SURVIVE AN EARTHQUAKE: HOME AND FAMILY PREPAREDNESS. The Workshops presented the main points of preparedness using a table talk, videotapes, slides and transparencies. The handbook provided each family a more detailed guidebook for use at home with family discussions.

The overriding strength of a live education example like the CHES workshops is the relationship that develops between the teacher and his/her audience during that brief time. If education creates conviction, then live education does it better than passive education!

The CHES workshops were successful because they used the family as a primary self-interest motivator. People could see the benefits of home preparedness training. Workshop attendees were interested and/or concerned since they had chosen to come. The word spread from enthusiastic participants to other groups, so that it appeared the effort could have enjoyed a domino effect in requests to give more workshops. The striking contrasts with most public earthquake education to date were the manner and environment of delivery, the material presented, and the use of non-traditional developers and teachers."

This first phase of developing the personal/family preparedness program came to a close in August, 1979 when Shirley Smith, one of the three CHES partners moved to West Lafayette, Indiana with her family. With only two partners (Harriett Paine and Libby Lafferty) remaining in "earthquake country" it became obvious that our time must be directed toward activities that could produce the greatest number of educated and aware citizens. Therefore, we determined to concentrate our efforts in the area of training teachers while continually updating the program to reflect the state of earth science and home management technology.

To ensure that each presentation given by a teacher trained by CHES is highly credible and effective it became apparent that it would be wise to pre-package several of the well-tested components of our preparedness program. Many of the teachers we train are volunteers and they especially appreciate the teaching tools and helpful aids that make the job of teaching more enjoyable, as well as reducing the amount of time and energy necessary for preparation. It is also much more economical for cities to use this pre-packaged material than to prepare their own training package.

At this time, there are four components of the program available to the teacher:

1. Teachers manual that leads the teacher through the process of establishing goals and objectives; planning the presentation in terms of a specific audience; getting good publicity; using motivational techniques; audience participation and appropriate scenarios, as well as the basics of being sure that the physical set-up is going to work.

2. Traveling table talk with accompanying tent card messages. This tool consists of a suitcase filled with real props that relate to specific areas of earthquake preparedness. With the use of the tent card messages, this display of props exhibited on a table becomes a silent teaching aid. The table talk has proven to be valuable as a teaching tool that can be used in many different ways.

3. "Shake, Rattle & Roll" - A 17 minute Audio-Visual presentation packages a wide spectrum of information about earthquake readiness in a positive, fast-paced and colorful manner. It can become almost the whole program when making a 30-minute presentation to a service organization, or the final review to a more thorough two hour program.

This Audio-Visual is available as a filmstrip or slide set with audio cassette or video tape: VHS/BETA/3/4".

4. How to Survive an Earthquake: Home and Family Preparedness is the necessary guide to walking through the process of preparedness at home. The training sessions are designed to present basic information about earthquakes; create an understanding of realistic expectations during and immediately after a quake, and provide the needed motivation to change behaviour and become better prepared. The handbook becomes that important final link between the training session and follow through at home. It is also an excellent resource for the teacher to use in expanding or supporting the presentation.

Who is being trained to teach:

Many cities are now beginning to feel that preparedness education for citizens is an important element in their overall earthquake preparedness plan and have sponsored the training of teachers. Some police personnel are using the program through their Public Relations departments. One fire department is using the training in combination with CPR and Neighborhood Watch programs. Parks and Recreation personnel in several cities have taught the class as part of their public education program, and a community hospital is teaching the class as part of a community sponsored forum.

Private groups and organizations are often interested in having members become teachers and then providing service through sponsorship of the program. Members of the Junior League, Soroptimists, Kiwanis, Church groups, Scouts and Salvation Army personnel have all taken training and become involved at the community level.

Because of the lack of an adequate program for emergency/disaster preparedness education in the schools, teachers at many grade levels, as well as school administrators, are taking training. In one session, we trained 150 head start teachers and aids. They took the program back to 51 classes in 24 sites and have now given the program for 700 families and are holding earthquake readiness drills for the children each month. At the other end of the spectrum, the course has been approved by the State of California for community college credit and is now being taught at several community colleges in Southern California. The University of California at Davis uses "How to Survive an Earthquake" as a mini-text in Geology I. Nurses, optometerics and dental assistants can get continuing education credits for taking the course.

Business and industry is beginning to express the need for programs aimed at educating employees to better cope with impending emergencies, i.e. earthquakes. Some businesses are mandated to continue operations during an emergency and feel that employees will be more apt to stay on the job knowing that their families are prepared to be self-sufficient. We are finding that personal and family preparedness education, in combination with the program designed for the specific needs of the employee on the job, makes a very complete training package. Some companies, such as IBM in San Francisco, are presenting the program through the first level managers. Others, like the Federal Aviation Administration use it in selected offices as well as in new employee orientation.

Observations:

Several observations can be made from this experiment in earthquake preparedness education that help to begin to answer some of the initial questions enumerated in this paper.

- Public apathy is no longer epidemic. Many citizens are concerned enough to take the time to order books, come to programs or even volunteer to become trainers. One interesting indication is the number of "How to Survive an Earthquake" handbooks that have been sold. From April, 1977 to August, 1979 approximately 5,000 copies were sold but from September 1979 to October, 1980 over 10,000 books were sold.

- The responsibility and cost for public education programs must be shared by government, business and industry and the local community if it is going to have a significant impact. Government must provide the leadership needed to make the program available in all communities.

- The most successful methods of disseminating earthquake preparedness information is through already existing channels of communication. The community college system, employee education programs, fire and police educational programs, etc. have already developed extensive networks of communication that can be utilized effectively. Due to the lack of public education in the past it is important that many avenues of communication be used in order to teach the greatest number of people. Everything from highly visible mass media spots to live training sessions including audience participation, are necessary.

- Earthquake preparedness education will help people to become better prepared for all types of emergencies and disasters. Home preparedness education programs should define what types of services can be expected from government and what responsibilities citizens will have to assume. This information may have an influence upon community planning and preparedness. Some of the people who take this type of training go back into their community to better prepare their neighborhoods, schools, churches and organizations.

- In order to be effective and meaningful any broad brush public preparedness program must be flexible. It should be possible to tailor the program and information to meet the needs of various social, economic and age groups. For example, head start parents have different needs than a group of homeowners in a retirement community.

- The prospect of earthquake prediction capabilities has perhaps already had an effect upon future earthquake preparedness education with the passage of AB 2202. Hopefully, the careful use of these funds will allow the expansion of preparedness education programs to reach a significant number of citizens. We recommend that large numbers of teachers be trained at the local community level and that they be given access to well packaged, highly motivational training materials. This will insure that if and when an earthquake prediction becomes a reality, there will already be an extensive net work of trained teachers ready to meet the increased demand for public education for personal and home preparedness.

INCORPORATING HAZARD EVALUATIONS INTO BUILDING CODE REQUIREMENTS

by

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INTRODUCTION

The presentations during this workshop have described the results of extensive studies and research into seismic hazards. Summaries of a large amount of data have been presented. The question that arises is - How will the data be used - who is the end user or recipient? The bottom line is safer, more efficient and hopefully more economical buildings and structures. How do all these data go from research to practice? The usual way is through introducing modifications and improvements to building codes.

The purpose of most building codes is to provide minimum standards to safeguard life or limb, health, property, and public welfare by regulating and controlling the design, construction, quality of materials, use and occupancy, location and maintenance of all buildings and structures within its specified jurisdiction. In the United States the power and authority to regulate buildings resides with the individual states, except for federal government-owned facilities. There are four model codes which are used, generally with minor modifications, in the over thirteen thousand local jurisdictions with authority for building regulations. These codes are the Uniform Building Code,[1] Standard Building Code,[2] Basic Building Code,[3] and American National Standard ANSI.[4] In addition there are several codes at the federal government level which regulate construction of federal facilities, and nuclear power plants and related facilities. Further, many of the large industrial firms and manufacturers' associations have developed and adopted building design criteria that are often more stringent than the applicable governmental codes.

Because of the multiplicity of codes and the many interests involved, the development of changes and improvements to codes requires input from numerous groups and extensive efforts to convince the appropriate decision-making body(s) to adopt the proposed change(s). As a result, the process for transforming research results and other data into building code requirements has become lengthy. The purpose of this paper is to present the author's understanding of the process, how hazard evaluations (specifically earthquake hazards) are developed and adopted, and how this process might be improved.

To limit the extent of this paper and to better focus on the process for developing and adopting improvements to seismic design building code requirements, the procedures followed by the Uniform Building Code (UBC), Structural Engineers Association of California (SEAOC) and the Applied Technology Council (ATC) are presented and recommendations are offered.

PROBLEM STATEMENT

Most of the research conducted on seismic resistance of buildings and structures is either funded by governmental agencies, by industry, or jointly by government and industry. Some research is performed by professional society committees, although this effort, of necessity, is limited. The research is performed, the results are presented in a report, and then the results are presented to the building designers and code promulgating officials, usually with the request that certain changes or additions be made to the building code. Often there is resistance to accepting the change because the research does not directly apply to the problem presented, and extrapolation or interpolation has to be made to extend the research to the problem at hand. The applicability of the results then often becomes a matter of judgment between the designers and the industry representatives presenting the results. Further, the public has to be convinced - this is especially true for seismic. The present paper is intended to look at this process in some detail.

How do research needs arise? There seem to be two basic events that occur. First somebody has an idea to build in a more economical, safer, or efficient way. The other is that something happens: a building or structure is damaged due to variations in load, or imposition of transient loads such as earthquake and wind, or stress induced from thermal and moisture changes, and there is obviously a need to improve construction to resist these conditions. The research is performed to try to better define the loading functions(s) and improve the performance of the construction materials and buildings or structures as a whole. The research may be analytical, experimental, or combination of analytical and experimental.

The research results are needed because the building code promulgators, building designers and/or building officials need adequate "proof" or justification for the proposed improvement (either for loading function or material design requirement) because they have much of the responsibility for the end product. The building official is responsible for safeguarding public safety, property and welfare, and the building designer is responsible to the owner for a safe, economical structure.

The matter of modifying code provisions for seismic hazards is perhaps the most complex. The determination of the load function, i.e., ground acceleration, velocity, displacement, duration of motion, ground motion frequency content, frequency of occurrence to name a few, is not clearly understood. Presently, correlation of structural response to earthquake ground motion and to many of the parameters involved is inadequate. The development of seismic design requirements requires extensive study and analysis by experienced, qualified practitioners in several disciplines; seismology geophysics, geology, soil mechanics, and structural engineering. To date much of the effort has been fragmented with only a moderate amount of coordination between the disciplines. The present procedures generally followed including those followed by UBC (International Conference of Building Officials - ICBO) and SEAOC will be reviewed. UBC and SEAOC procedures are discussed because these two groups have been active in developing seismic design requirements for several decades.

PRESENT PROCEDURES

As noted previously, most earthquake engineering research is sponsored by governmental agencies or private industry. Government and/or industry groups might perform some of the research directly. Sometimes advisory groups are utilized; other times they are not used or they are ineffective. When industry performs the research, there can be problems of credibility. If an appropriate and effective advisory group is not factored in early in the project, there may be a lack of full understanding of the problem, the form the results should be presented in, and the way the results will be utilized by the building designer - the end user. Hence, the results might be suspect or argued, or misused. The other approach to research is where government and/or industry go to a university, private consultants, or a governmental research group. Again, these researchers may or may not use effective advisory groups.

The research program has to be carefully thought out and organized so as to hopefully produce the results in an economical manner and within the given budget. Because of restrictions in research facilities and budgets, the research experiment may not always be directly applicable to the problem. The results must be extrapolated or transformed in some manner so as to apply to the problem at hand.

The above approach seems to be rather straightforward, but what are some of the problems? Who should be involved in the process? The owner of the building is interested in a safe, economical structure (by economical is meant life cycle cost, including first cost and lifetime repairs and maintenance). The owner is normally represented by designers (architects and/or engineers). Then there is the manufacturer of the building materials, and closely allied with the manufacturer is the constructor or contractor, who furnishes the organization and assembles the required resources to build the structure. This includes materials and appropriate craftsmen.

There is the building official who has the responsibility to the public to see that any building constructed that comes within his purview safeguards the public safety, property and welfare. And, of course, there is the researcher, who has to set up the experiment. He has the background in experimental design; he knows the physical limitations of the various types of facilities; he presumably is unbiased and independent; and he will provide a factual, straightforward research program and corresponding report setting forth a description of the research, the results obtained, and his interpretation of same. Of course there is the public.

The difficulty is to bring all of these parties together to focus on the problem and to come up with a solution considering all of the restraints such as budget, physical facilities, knowledge about the phenomena, schedule, scale factors, workmanship, variation in materials, and other factors. Further, the resulting research program must, insofar as practical, be applicable to actual buildings and structures.

From the designer's viewpoint, there is a perception that the researcher and/or the funding group defines the program and often do not obtain all the required input from the designers. They may set up an advisory group, but often such advisory groups are not fully effective for a variety of reasons. They may be set up after the research program has been formulated or are not a good representative sample of the design professions, industry, and code officials, and participation is limited.

In transforming research needs into building code requirements, certain procedures have to be followed to revise a building code. The SEAOC Seismology Committee initiates changes to the seismic design provisions in the SEAOC Blue Book "Recommended Lateral Force Requirements" based on input from SEAOC members, researchers, and others, see Figure 1. The input may be in the form of observations of earthquake-induced damage, the results of research conducted by researchers at universities, governmental agencies, private firms, and/or industry, or recommendations from design professionals. The Committee studies the data in detail and then develops recommended revisions to the seismic design requirements. The revisions are then reviewed by local SEA committees, their comments considered and final text of the changes written. The SEAOC Seismology Committee may act independently of the SEAOC Board of Directors and publish its recommendations as revisions to the Blue Book without further SEAOC action. Changes to the SEAOC Blue Book are then considered by the appropriate ICBO committees for probable adoption into the next edition of the UBC.

For changes to the seismic design requirements in the Uniform Building Code, research results together with recommended provisions, must go first to the ICBO Code Changes Committee and then to the Seismology Subcommittee, which must review all provisions related to the design of structures to resist earthquakes, see Figure 2. For seismic provisions, ICBO usually refers all proposed code changes (except those proposed by

SEAOC) to the Structural Engineers' Association of California for review, required modifications, and/or recommendations. The code provision changes are assigned to the State Seismology Committee; on some occasions, the SEAOC State Code Committee is also involved. The SEAOC Seismology Committee reviews the proposed code changes in detail, generally relying partly on input from local committees and/or task groups. A SEAOC representative often attends ICBO committee meetings and hearings. For code changes applicable to other geographical areas, the local structural engineering associations may become involved.

It is evident there are a large number of people involved and if research is to be done with the aim of its results being incorporated into code requirements, a means of good communication with the appropriate groups should be set up early in the program to ensure that their ideas and judgements are involved at an early stage. If this is done, then the code development process should run much smoother.

RESULTS TO DATE

The results to date appear to be fair to good. However, it does not appear for the amount of research funds expended that the end results are necessarily commensurate with the effort and funds expended. Why is this? Some of the reasons were stated previously: inadequate design of the research program, and inappropriate use of advisory groups or their input. Another problem is the research program is set up to solve a specific problem and does not consider other factors or disciplines that might be interdependent with the problem. One result is there is considerable confusion about some research results, especially how they should be used, or can be used in building design. Difficulties are encountered in getting research results accepted and used by designers. If designers are not in favor it is usually very difficult to get the building officials to accept changes.

The question arises, can this situation be improved? There has been a lot of good work done, but it seems a number of improvements in the process can be made. Three projects that have been active over the past several years wherein the results of research and other data have been incorporated into building code provisions (or are in process) are of interest because some of the problems outlined above have been solved or at least partially solved. The projects involved development of "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC-3-06),[6] "Seismic Design Guidelines for Bridges" (ATC-6),[7] and "Guidelines for Seismic Design of Single Family Masonry Dwellings in UBC Zone 2" (ATC-5).[8] Each of these projects was initiated, organized and coordinated by the Applied Technology Council (ATC) under contracts with federal government agencies.

These projects, how they were organized, how they made use of research data (including seismic risk maps by USGS), and the problems encountered and successes are described and discussed in the following text. Suggestions and recommendations for improvements in the process are presented.

ATC-3-06 Project

The ATC-3 project involved 85 participants comprised of design professionals, researchers, code promulgating organizations and governmental agencies who worked over four years to complete the document, see Figure 3. The participants were assembled and coordinated by ATC under a contract with the National Bureau of Standards with funding by the National Science Foundation and NBS. Over 8,000 copies of the ATC-3-06 document have been widely distributed in the U.S. and many foreign countries. Since its publication in June 1978 it has become the subject of intensive review and study as a resource document, and has been adopted whole or in part, or is being used as the basis for seismic code changes in numerous countries. The seismic risk maps used in ATC-3-06 are based on maps developed by Algermissen and Perkins[9] and for velocity-related coefficients on work by McQuire.[10]

The ATC-3 project organization considered the interdependency of a number of disciplines including risk analysts, seismologists, geophysicists, researchers, practicing structural, mechanical and electrical engineers, architects, code promulgators and government representatives. There were five major task groups, one of which (Seismic Input) was charged with Risk Assessment, and Ground Motion and Site Effects. Numerous meetings were held between the two committees within the task group and with representatives of other committees in the project. Extensive correspondence was generated and in general there was reasonably good communication and interchange of ideas and information between pertinent committees. The analysis and design requirements in ATC-3-06 were developed by other task groups who gave detailed consideration to the seismic risk mapping and ground motion response spectra prepared by the Seismic Input task group.

The ATC-3 project involved most of the pertinent groups with the exception of construction materials industry representatives. These groups were not directly represented so as to ensure that the resulting provisions would be unbiased. The materials groups together with others in the construction industry and representatives of the public are now making a detailed and exhaustive review of the ATC-3-06 provisions. A major problem appears to be proper communication of the risks, costs and impact of the new design requirements. A lengthy educational process is part of the detailed review.

ATC-6 Project

The ATC-6 project, "Development of Seismic Design Guidelines for Bridges," is being completed by ATC under a contract with the Federal Highway Administration. A total of sixteen researchers and design professionals worked for nearly four years to develop the guidelines. A draft of the guidelines was used to redesign 21 bridges to determine the impact of the provisions. The redesign results were assessed and appropriate clarifications and changes made to the guidelines. The final guidelines will be submitted to the American Association of State Highway and Transportation Officials for adoption as part of the AASHTO Specifications. The input from and interchange between researchers, design professionals and users (state highway officials) during the project were major factors contributing to the successful development of the guidelines.

ATC-5 Project

Another project where researchers, design professionals and home builders are cooperating in development of future building code changes is the ATC-5 project "Development of Seismic Design Guidelines for Single Family Masonry Dwellings in UBC Seismic Zone 2" which is being conducted by ATC under a contract with the U.S. Department of Housing and Urban Development. The project was initiated to determine whether existing code requirements for reinforcing masonry construction were excessive. The research is being conducted by researchers at the University of California, Berkeley working in close cooperation with an advisory group (composed of design professionals and a home builder) appointed by ATC. The research experiments (shaking table tests) are reviewed and discussed in detail by the researchers and the advisory group to ensure that maximum benefit is gained from each test and the results are directly applicable (insofar as feasible) to design and construction. The resulting guidelines will be required (by HUD and FHA) to be used in construction of single family masonry dwellings in applicable areas of the U.S. Another shaking table test is being planned for early 1981 so the guidelines should be completed by late 1981.

RESEARCH NEEDS

Several areas of seismic design still need marked improvement although extensive research has been conducted and many improvements developed. A major area for improvement is that of seismic risk parameters to use in design. Presently all seismic design code provisions are based on ground acceleration. In the United States the acceleration maps developed by Dr. Algermissen and others at the Geological Survey are basic input for the UBC, SEAOC Blue Book, ANSI A58, ATC-3-06, and other codes. Yet it has become increasingly evident that there is very poor correlation between building response (damage) during an earthquake and the recorded instrumental accelerations especially for near field earthquakes. For example, commonly used analysis and design techniques would indicate much more extensive damage should have occurred to many structures during the October 15, 1979 Imperial Valley earthquake. The observed damage was relatively light considering the very high peak instrumental accelerations recorded. Observations from the Imperial Valley, Coyote Lake, San Fernando and other earthquakes are briefly summarized below.

Imperial Valley Earthquake

The lack of correlation of near-field instrumental peak accelerations with damage is evidenced by the EERI report (Ref. 11) on the Imperial Valley earthquake where there was minor damage to industrial facilities and, except for one collapsed tank, moderate damage to elevated water tanks. The El Centro Steam Plant was designed for 0.2g. No significant structural damage or reduction of structural integrity occurred. Instruments 0.85 km away from the plant recorded peak horizontal accelerations exceeding 0.5g and vertical accelerations of 0.93g.

There was limited damage to governmental and commercial structures. The major building damage occurred to the Imperial County Services Building, where four concrete columns at one end of the building failed (shortened about 12 inches, but the building did not collapse) largely due to excessive overturning forces and inadequate confinement of the vertical reinforcing steel. The building was subjected to ground motions in excess of 0.3g (the code static design factor was less than 0.1g). It is of interest that the County Courthouse (circa 1940) across the street from the Services Building incurred no structural damage and only limited plaster cracking.

The commercial structures on Main Street, El Centro are mostly one or two story masonry, concrete or light steel construction. These structures incurred limited structural damage (Ref. 11). Most of the damage was from fallen parapets, cracked window glass, and cracked plaster or finishes. Their design seismic resistance is quite nominal (the design basis was probably less than 0.1g) yet they suffered relatively minor structural damage from the earthquake and its aftershocks.

There are fifteen state highway bridges in the Imperial Valley; only one suffered damage sufficient to be closed to traffic (Ref. 12). Of the remainder, a few exhibited some minor cracking of concrete and settlement of approach fills. The New River bridge in Brawley, which was built in 1953, exhibited backfill settlement and some structural damage from the initial shock; the left bridge was closed to traffic. Three aftershocks of M5.0 to 5.8 with epicenters within 6 km of the bridge induced additional settlement of backfills and damage to abutments and supporting piles such that the right bridge was also closed to traffic.

It is of interest that there are nine bridges within about 20 km of the main shock epicenter. They are located from 0.2 miles to about 4 miles from the fault. Four of the nine suffered minor structural damage (concrete cracks and/or shearing of some welds) and settlement of backfills. Five bridges on Interstate 8 (which crosses the Imperial fault) had no structural damage although they are located from 0.2 to 3 miles from the fault. Instruments at the Meloland Road overcrossing (0.2 miles from the fault) recorded a peak free field horizontal acceleration of 0.32g and a peak vertical acceleration of 0.23g, and 0.52g maximum horizontal acceleration on the bridge. Considering the high recorded peak accelerations, the structural damage was slight.

Coyote Lake Earthquake

Accelerations exceeding 0.4g were recorded. Buildings and structures in the surrounding area were probably subjected to peak ground accelerations less than this value. There was very minor damage, generally only architectural or non-structural building components (Ref. 13).

San Fernando Earthquake

Damage to industrial structures was also relatively light in the San Fernando earthquake (Ref. 14). Most of the major damage to dams and large industrial structures resulted from ground movement such as settlement or lurching. Damage to highway overcrossing structures was generally due to excessive relative displacement of different elements. For example, abutments moved apart and the bridge spans dropped. The static seismic design factor for most of these structures was from 0.03 to 0.10g.

The damage to the Sylmar Converter Station was mostly to equipment and was generally due to inadequate anchorage or lack of design for seismic motions. The caretaker's cottage at Pacoima Dam, which was less than one-half mile from the recording station, suffered practically no damage. Its brick chimney remained standing. There was no damage to the dam; the instrument which recorded a peak of 1.2g was located near one abutment of the dam.

Other Earthquakes

Housner (Ref. 15) and Cloud (Ref. 16) note the small damage that occurred from the Parkfield earthquake. Lander (Ref. 17) describes the relatively light damage in the Melendy Ranch earthquake.

In summary, a large number of earthquake records have been obtained for near-field earthquakes, as shown in Table 1. In each of these earthquakes, the damage to buildings near the fault was substantially less than would have been predicted by using the recorded acceleration levels or response spectra calculated from these records. It is evident from the above that it is not realistic to use instrumental peak accelerations from near-field earthquakes to predict structure response and/or potential damage.

RECOMMENDATIONS

The preceding text discussed the processes and types of organizations involved in making changes or additions to building codes. The process of incorporating research results into building code provisions was outlined and suggested improvements were presented. Certain research needs were described. It is evident from the foregoing text that several recommendations are in order.

1. The conduct of research aimed toward improving building performance during earthquakes should involve more interdisciplinary communication, coordination and interaction.

2. Extensive study and research should be given to development of better risk mapping parameters. The correlation of building response and damage to recorded peak ground accelerations is very poor.

3. The code modification process is increasingly involving more segments of the public. Better translation of technical aspects into a format and text that lay public and others can understand is essential. Such translations will also help the earthquake engineering profession's relations with legislative bodies and public officials.

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TABLE 1
NEAR-FIELD PEAK EARTHQUAKE RECORDS

Record	Date	Horizontal Peak Acceleration	Reference
Pacoima Dam, CA	2/9/71	1.2g	19
Parkfield, CA	6/27/66	0.5g	15,16
Ancona, Italy	6/72	0.6g	18
Melendy Ranch, CA	9/4/72	0.7g	17
Imperial Valley, CA	10/15/79	0.8g	11
Coyote Lake, CA	8/6/79	0.42g	13

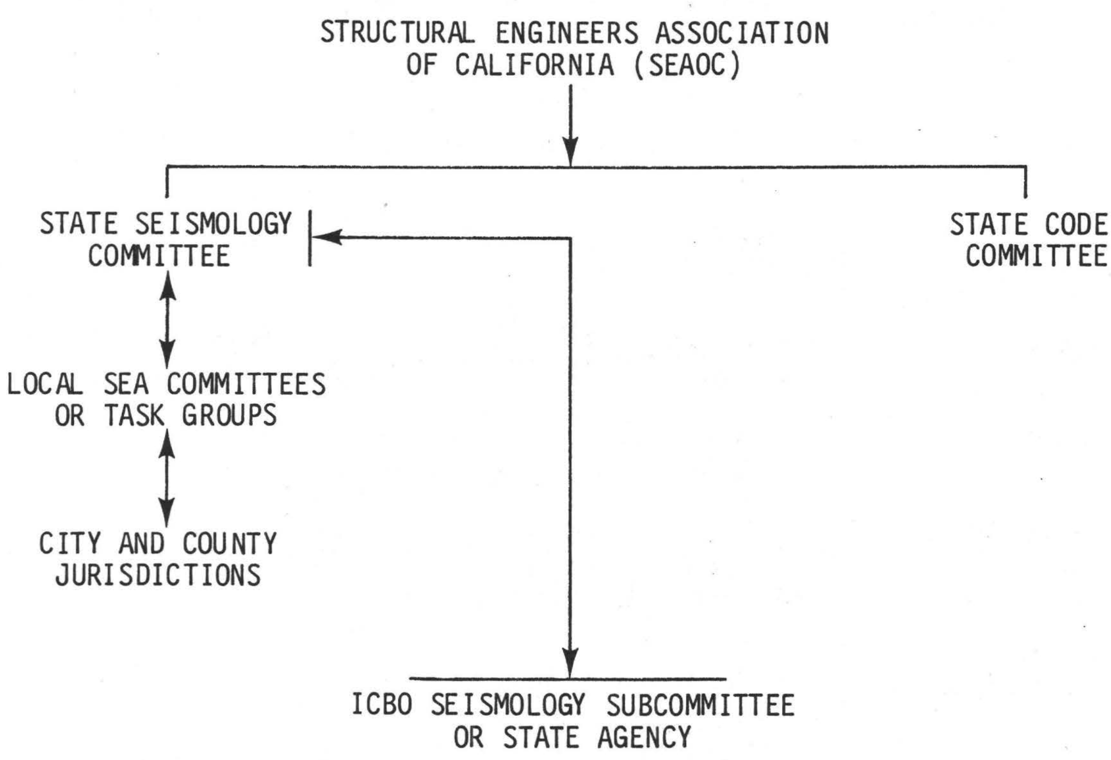

FIGURE 1. SEAOC PROCEDURE FOR SEISMIC CODE CHANGES

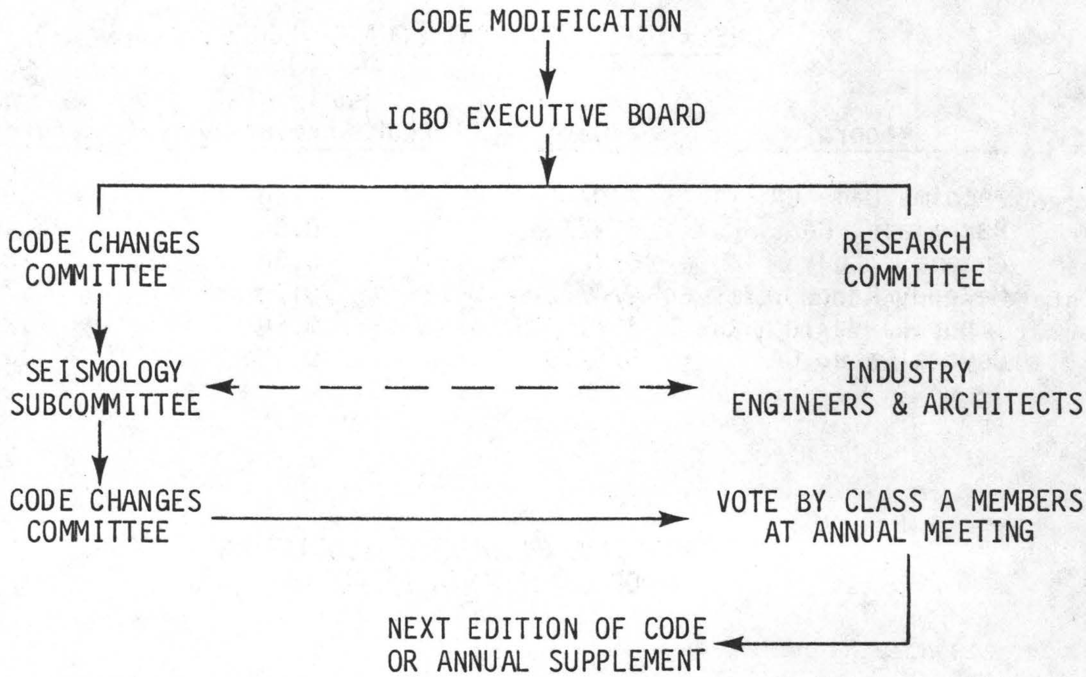

FIGURE 2. FLOW CHART FOR UBC CHANGES

PROJECT:	ATC-3-06, TENTATIVE PROVISIONS FOR DEVELOPMENT OF SEISMIC DESIGN REGULATIONS FOR BUILDINGS
SPONSOR:	CONTRACT WITH NATIONAL BUREAU OF STANDARDS WITH FUNDING BY NATIONAL SCIENCE FOUNDATION
OBJECTIVES:	1. REVIEW STATE OF ART AND EXISTING CODES 2. DEVELOP COMPREHENSIVE PROVISIONS FOR BUILDINGS 3. USE AS RESOURCE DOCUMENT
PROJECT PARTICIPANTS:	25 PROFESSORS (RESEARCHERS) 43 PRACTICING ENGINEERS 5 ARCHITECTS 8 CODE PROMULGATORS OR OFFICIALS 4 GOVERNMENT
DESIGN APPROACH:	STRENGTH DESIGN UTILIZING MORE REALISTIC GROUND MOTION INTENSITIES.
FINAL REPORT:	JUNE 1978

FIGURE 3

UTILIZATION OF EARTHQUAKE HAZARDS AND RISK EVALUATIONS BY LOCAL AND STATE GOVERNMENTS

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INTRODUCTION

In this paper, we describe how the Utah Seismic Safety Advisory Council, a state agency, has utilized earthquake hazards and risk information in formulating earthquake safety policy recommendations and programs for state and local governments. Included are discussions on the particular research information we have used, the adaptations that were necessary to allow practical application of the information, and the methodology developed for using the information. Two examples of Utah's use of such information are described--the preparation of a modified seismic zone map intended to guide building construction in the state, and development of earthquake safety recommendations and programs for existing school buildings in Utah.

THE UTILIZATION PROBLEM

The problem addressed in this paper is the practical use of earthquake hazards and risk evaluations both for formulating state earthquake safety policy and for guiding particular implementation programs. A sub-level problem is how such information can help to establish the feasibility and priority of particular hazards mitigation programs.

Meaningful definitions for the ambiguous terms "practical" and "earthquake hazards and risk evaluations" are essential elements both to this paper and to the efforts of the Utah Seismic Safety Advisory Council. Although earthquake hazards generally seem to have a common definition, the concept of risk is somewhat personal and largely judgemental. Acceptable risk is decided, in part, from that which is possible. In our use of the term, that which is practical satisfies the criteria of being (1) socially needed (2) politically acceptable, (3) economically feasible, and (4) understandable by the lay community. In the policy and program recommendations formulated by the Utah Seismic Safety Advisory Council, we have measured the various earthquake hazards and risk evaluations against these criteria.

Utah's earthquake environment has been studied quite intensely in recent years. Although the information is incomplete, such as regarding liquefaction susceptibility and ground vibration amplification in alluvial soils, useful earthquake hazards evaluations can be made. However, as recent as the mid-1970's, risk evaluations drawn from the hazards evaluations were limited to the seismic zone map contained in the Uniform Building Code and a loss study resulting from an assumed worst-case earthquake in the Salt Lake City area. Only a few attempts had been made to prepare detailed risk evaluations using available methodologies, such as suggested by Algermissen and Steinbrugge [Refs. 1 and 8]. Further, the limited information was not widely known outside the research community, and so public factual awareness was even more limited.

Given the situation described above, the Utah Seismic Safety Advisory Council in 1977 commenced studies to answer the following questions.

- Is the present research data describing Utah's earthquake environment adequate to make risk assessments for particular types of development and populations?

- Can the available methodologies (actually fragments of the whole problem) be synthesized into a comprehensive risk assessment methodology?

- Can the risk assessments for particular types of development (buildings, etc.) and populations be formulated so that policy recommendations may be developed, with consideration of the four criteria of practicality given above?

- Is the reliability of the risk evaluations suitable to justify establishing public policy for mitigation?

DISCUSSION

Mapping Utah's Seismic Environment

Earthquake hazards traditionally have been mapped to include occurrences, maximum probable bedrock accelerations, fault zones, susceptibility to landslides, and, more recently, susceptibility to liquefaction. Microzonation essentially is a consolidation and scale enlargement of such data.

Of these hazards, only earthquake occurrences [various references] and maximum probable bedrock accelerations [Ref. 1] and some fault zones [Ref. 2] had been mapped for Utah by 1977. The bedrock acceleration data was of macrozone scale and relatively unused beyond the scientific community. The seismic zone map contained in the Uniform Building Code was the reference for most seismic designs and often was used in ways not intended or appropriate. The Wasatch fault zone was mapped relatively thoroughly in the early 1970's by Woodward-Clyde, but other mapping was either fragmented or non-existent, and generally inconsistent both in technique and geographic area of coverage. Mapping of susceptibility to landslides and liquefaction potential are not available in Utah, although some studies now are underway.

In 1979, Arabasz, Smith, and Richins published a comprehensive compendium of historic seismicity in Utah [Ref. 3]. Also, in 1979 a consolidated fault map of Utah was produced and published by Fugro, Inc. [Ref. 4]. Soil amplification effects on ground vibrations were studied by W. Hays in the late 1970's for the Salt Lake Valley [Ref. 5].

The earthquake hazards data described above have served as the basis for earthquake risk assessments by the Utah Seismic Safety Advisory Council. These risk assessments have yielded better information and understanding of earthquake effects than heretofore has been available, but there remain voids in the data that will constrain the preparation of complete risk analyses until research provides more information.

Incomplete data causes two problems that must be recognized. First, and the most obvious, is that of uncertainty. Uncertainty can be handled by the technician, but it poses difficulties for the lay community. The second problem is that incomplete data may be used by the scientist to speculate on risk. The lay community often is unable to separate speculations from factual uncertainties, and such situations present new problems for policy formulation.

From the data sources named above, we have developed two new risk evaluation items that are described here. The first is an updated seismic zone map for the State of Utah that is intended to guide seismically resistant building construction and may be substituted for the seismic zone map contained in the Uniform Building Code. The second is a methodology for preparing risk assessments and which we here apply only to the evaluation of Utah's existing school buildings. The methodology has broader application, however, and has been applied to risk evaluations of other building classes and to utility lifelines.

Seismic Zone Map For Utah

The updated seismic zone map prepared by the Utah Seismic Safety Advisory Council is shown in Figure 1. This map was derived from work by S.T. Algermissen and D.M. Perkins [Ref. 1], with modifications based upon geologic investigations by L. Cluff [Ref. 6]. In particular, a new seismic risk zone coincident with the Wasatch fault has been established to account for a greater earthquake hazard potential that is inferred from geologic investigations but is not reflected in the historical record of seismicity for the region.

This new seismic zone map has been adopted by the State Building Board and other state agencies, and it is being used in lieu of the map contained in the Uniform Building Code. A detailed discussion of the development of the new map appears in Reference 7.

Of possible significance to this paper is that the new seismic zone map provides an improved level of information for users in Utah but actually is developed from fragments of available information drawn from several sources. The map demonstrates that useful new information can be assembled from existing data. Making the information more widely accessible to the users is one aspect of such usefulness.

Risk Assessment of Utah's Existing Schools

In reporting this risk assessment study, we begin with a brief description of the general methodology which has application to a variety of buildings, utility lifelines, and other types of development. We then describe specific application of the methodology to existing school buildings and the seismic safety policy recommendations that result from the evaluation.

The risk assessment methodology derives from data on building losses by S.T. Algermissen and K.V. Steinbrugge [Ref. 8] and from seismic source zones and related data developed by S.T. Algermissen and D.M. Perkins [Ref. 1]. The Algermissen and Perkins study provides a means to establish the earthquake potential of a region. The Algermissen and Steinbrugge material provides a methodology and essential information for estimating the effects of this earthquake potential upon various classes of building construction. Using other data about existing Utah school buildings obtained from various sources, we have derived estimates of life loss, casualties, and property losses for aggregate groups of school facilities and for various earthquake strengths and expected frequencies.

Earthquake frequency and recurrence estimates are derived from seismic source zone data for Utah taken from the work of Algermissen and Perkins. Calculations in Zone U-4 (See Figure 2) were based upon an estimate of maximum probable earthquake strength and recurrence inferred from geologic investigations of the Wasatch fault by L. Cluff. Other calculations used the standard equation $\log N = a + b_I I_o$ and historical seismicity data. In the end, a table of earthquake frequencies by seismic source zone were computed for various Modified Mercalli Intensities.

Estimates of building losses were based upon Figure 3 developed by K.V. Steinbrugge which charts percentages of loss for various classes of buildings and for various values of Modified Mercalli Intensities. The building classification system suggested by Steinbrugge was modified to fit known construction characteristics in Utah. Earthquake frequencies and percent losses for school buildings by class of construction and location combine to give estimated building damage losses. The resulting matrix also provides information on situations of greatest risk (see Table 2). Similar techniques were used to establish estimated life loss and casualties for school occupants.

We found that earthquake risk to existing schools in Utah although not insignificant, cannot justify the cost of extensive replacement or retrofit programs. For example, one would need to place a value of life at over $270 million in order to justify a major rebuilding program. This clearly is not a politically saleable program for investment of public money. On the other hand, selective retrofitting programs can be cost-effective, provided that the worst school buildings can be identified. Even more effective is a selective replacement or retrofit program that is based upon the normal aging and replacement cycle for school buildings. The success of such a program, of course, is heavily dependent upon one's ability to persuade the local school districts that earthquake safety should be considered along with the many other factors in evaluating the educational serviceability of a school.

An extension of the risk evaluation methodology, combined with appropriate assumptions, allows one to estimate the effectiveness of particular mitigation actions. For example, one can assume that all unreinforced masonry schools in the worst seismic zone are fully retrofitted to remove unsafe conditions, and the resulting property loss estimates can be compared with the estimated retrofit costs or with the reduced life loss and casualties. The extension, then, allows a benefit/cost evaluation in order to judge the merits of one mitigation program over another.

The risk assessment methodology we have described also utilizes information that is available in scientific reports. The methodology actually combines several single-purpose methodologies--earthquake hazards definition, loss estimates, and benefit/cost analysis--into a single technique for formulating earthquake safety policy and programs. Again, using fragments that are available, we have developed new and better information on which to make policy recommendations.

SUMMARY

We return now to the specific questions raised earlier in the paper and to our focus on the practical utilization of earthquake hazards and risk evaluations.

We have emphasized that available research information, although incomplete and sometimes in the wrong form, is sufficient to allow risk assessments to be made from which policy recommendations can be made, in turn. We note, however, that the risk evaluations prepared by the scientific community rarely can be used directly. Instead, these have served as methodology models for more rigorous analysis of specific risk studies. We note, also, that the resulting specific risk assessments have required that information and methodologies from several sources be combined and often restructured for our practical utilization.

Descriptions of Utah's earthquake environment are among the specific data used as input in the methodologies. Uncertainty regarding recurrence of large earthquakes remains as a problem that affects reliability of conclusions, as does incompleteness or absence of data on ground vibration amplification and liquefaction effects. However, these problems affect only the reliability of conclusions rather than the process of making the risk evaluations.

Benefit/cost techniques have been used to evaluate the social, political, and economic merits of possible earthquake safety policies. We cannot point to quantified techniques as a replacement for judgement in drawing conclusions from these risk evaluations, but we do believe that benefit/cost techniques are helpful for clarifying the judgements and for separating risk speculation from risk reality.

Reliability of conclusions is, perhaps, the most perplexing question that has been raised. Uncertainty of the input data on Utah's potential earthquake environment can cause seriously large variations in risk assessments that directly influence policy positions. We can foresee no near-term resolution of these uncertainties.

One other aspect of risk assessment reliability is unique to our application. The methodology that we have followed uses statistically derived information on aggregate classes of buildings and other types of development. Theoretically, conclusions about aggregate classes should not be extended to risk assessments of individual facilities that may be members of the particular class. Hence, our policy recommendations almost always include situations that require individual attention.

While we have obtained much valuable information from earthquake risk assessments in Utah and have formulated some important mitigation policies for the state, improved certainty of the information is deemed needed. Earthquake risk mitigation policies derived from speculation and probability scenarios are difficult to "sell" to a public whose perception of the risk is different than that of the technical or scientific community.

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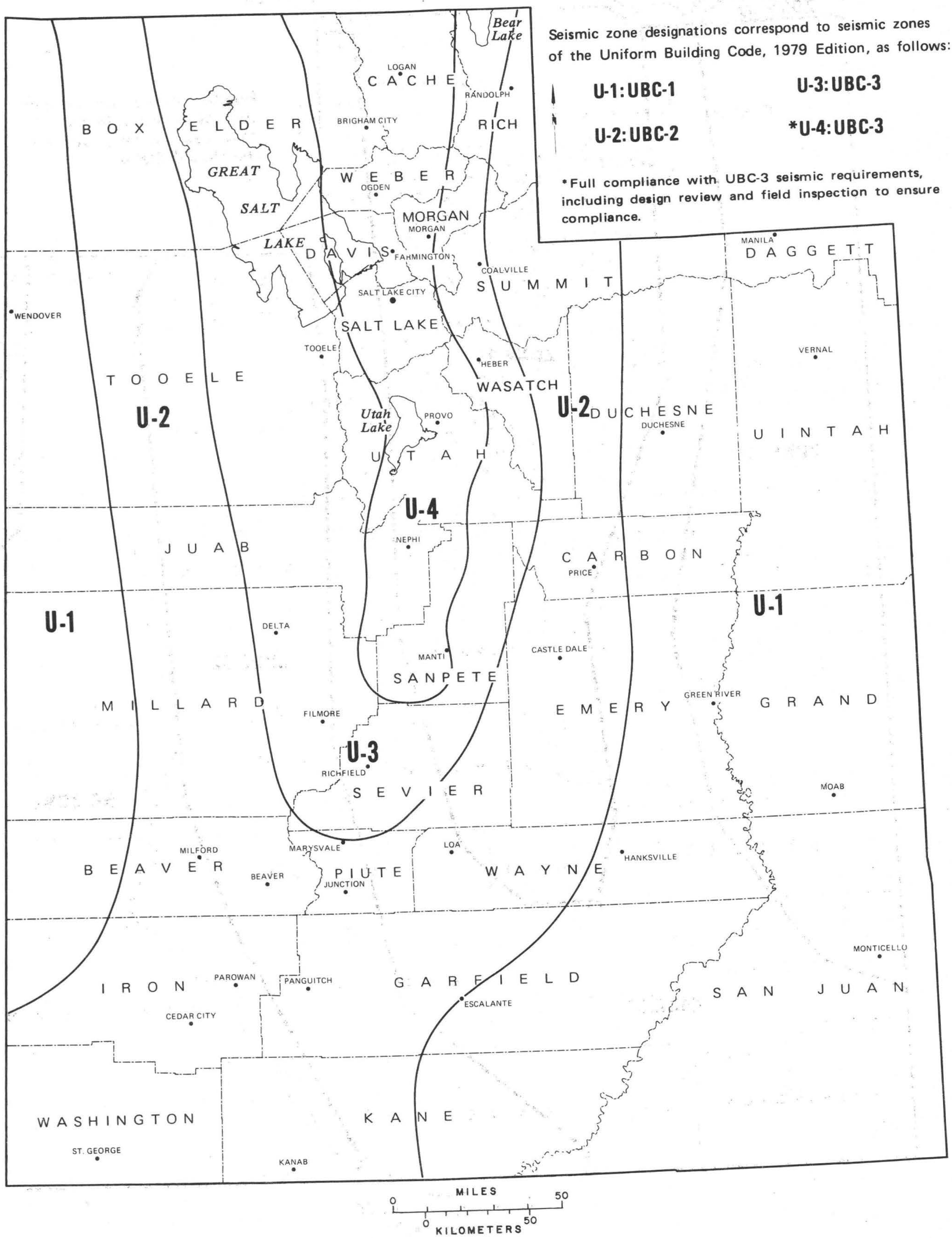

Figure 1
SEISMIC ZONES
January 1980
(Recommended by the Utah Seismic Safety Advisory Council)

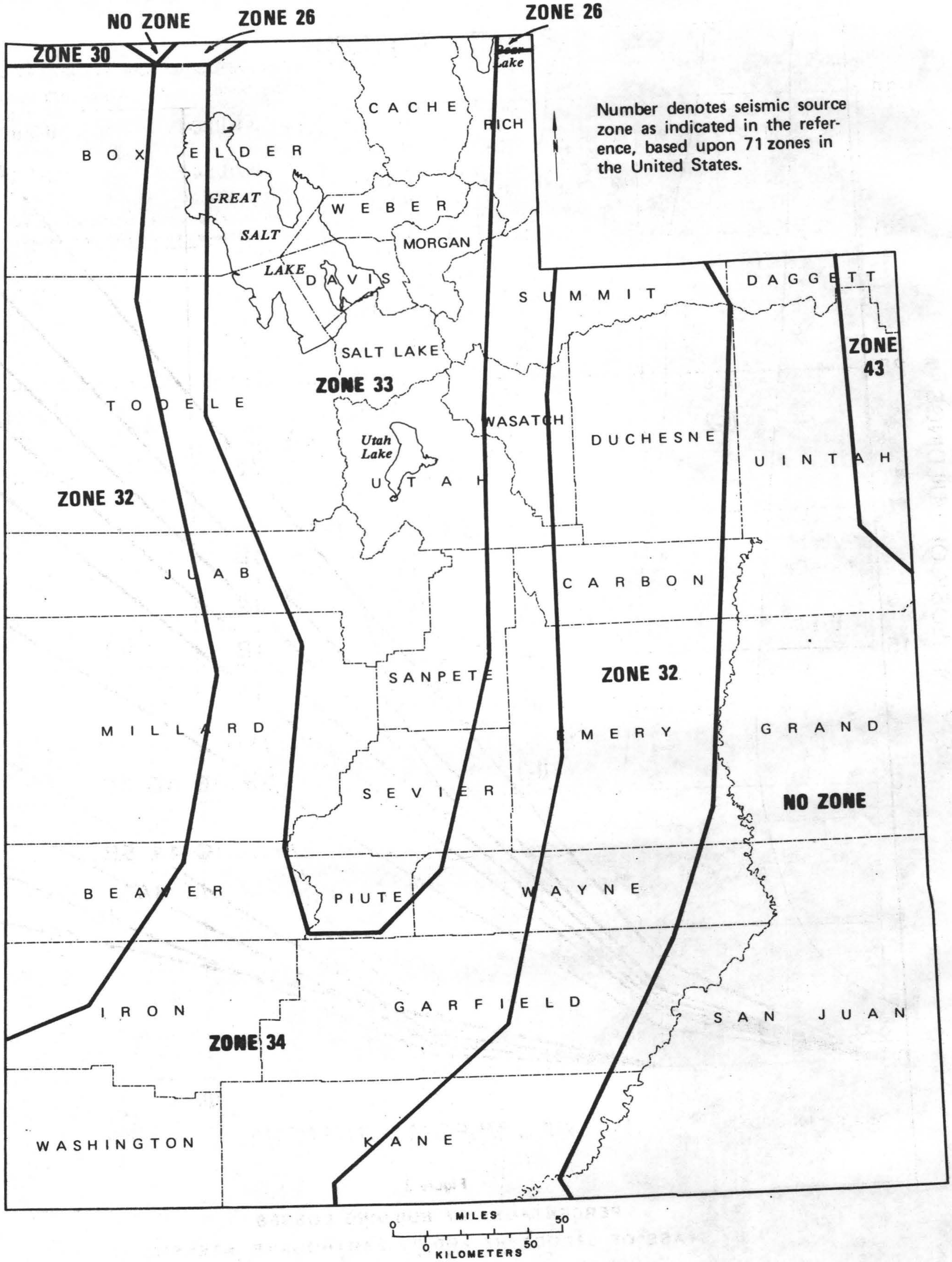

Figure 2
SEISMIC SOURCE AREAS IN UTAH
(Reference: S.T. Algermissen, and D.M. Perkins, USGS Open File Report 76-416)

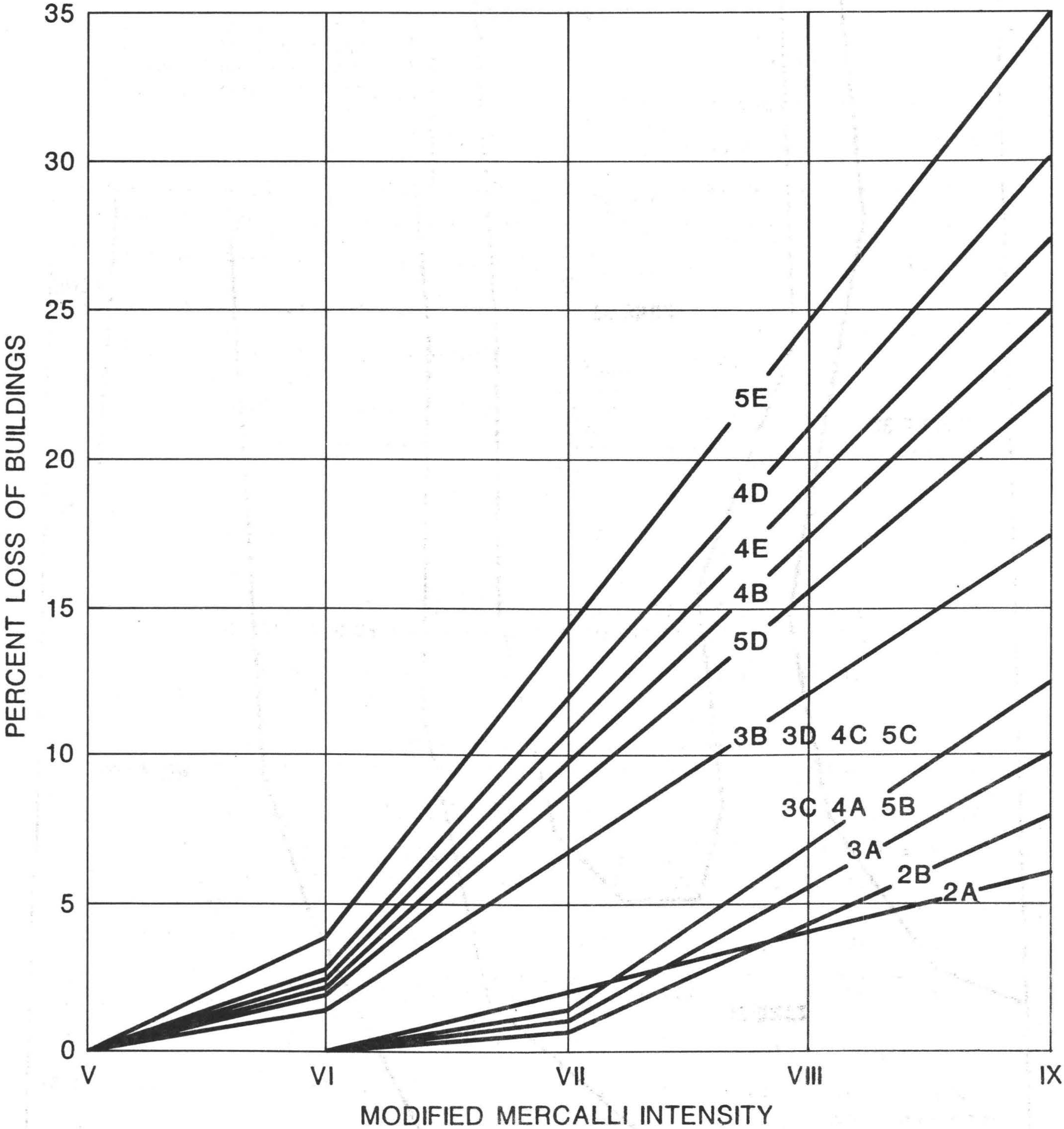

Figure 3

PERCENTAGES OF BUILDING LOSSES
BY CLASS OF STRUCTURE AND BY EARTHQUAKE INTENSITY

(From S. T. Algermissen and K. V. Steinbrugge)

Table 1

EARTHQUAKE FREQUENCIES BY SEISMIC SOURCE ZONE

STATE OF UTAH

Zone	Intensity X	IX	VIII	VII	VI	V
Zone 32	0	0	0.0006	0.0028	0.0124	0.0515
Zone 33A	0.0067	0.0353	0.1188	0.3976	1.2819	2.9376
Zone 33B	0.0002	0.0009	0.0111	0.0647	0.3764	1.5735
Zone 34	0.0001	0.0014	0.0083	0.0393	0.1726	0.7212

Table 2

EXPECTED 100-YEAR LOSSES TO BUILDINGS IN ZONE 33A
BY CLASS OF CONSTRUCTION EXPRESSED AS A PERCENT OF THE CLASS

(Based on Algermissen and Steinbrugge Loss Estimates)

Percent Loss at a Given Intensity

Intensity	Construction Class 5E	4D	4E	4B	5D	3B,3D 4C,5C	3C,4A 5B	3A	2B	2A
X	50%	42%	37%	33%	30%	23%	18%	15%	12%	8%
IX	35%	30%	27.5%	25%	22.5%	17.5%	13%	11%	8%	7%
VIII	25%	22%	19%	18%	16%	12.5%	7.5%	6%	4.5%	4%
VII	14.5%	12.5%	11%	10%	9%	7%	2%	1.5%	1%	2.5%
VI	4%	3%	2.5%	2.5%	2.5%	2%	0	0	0	0

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