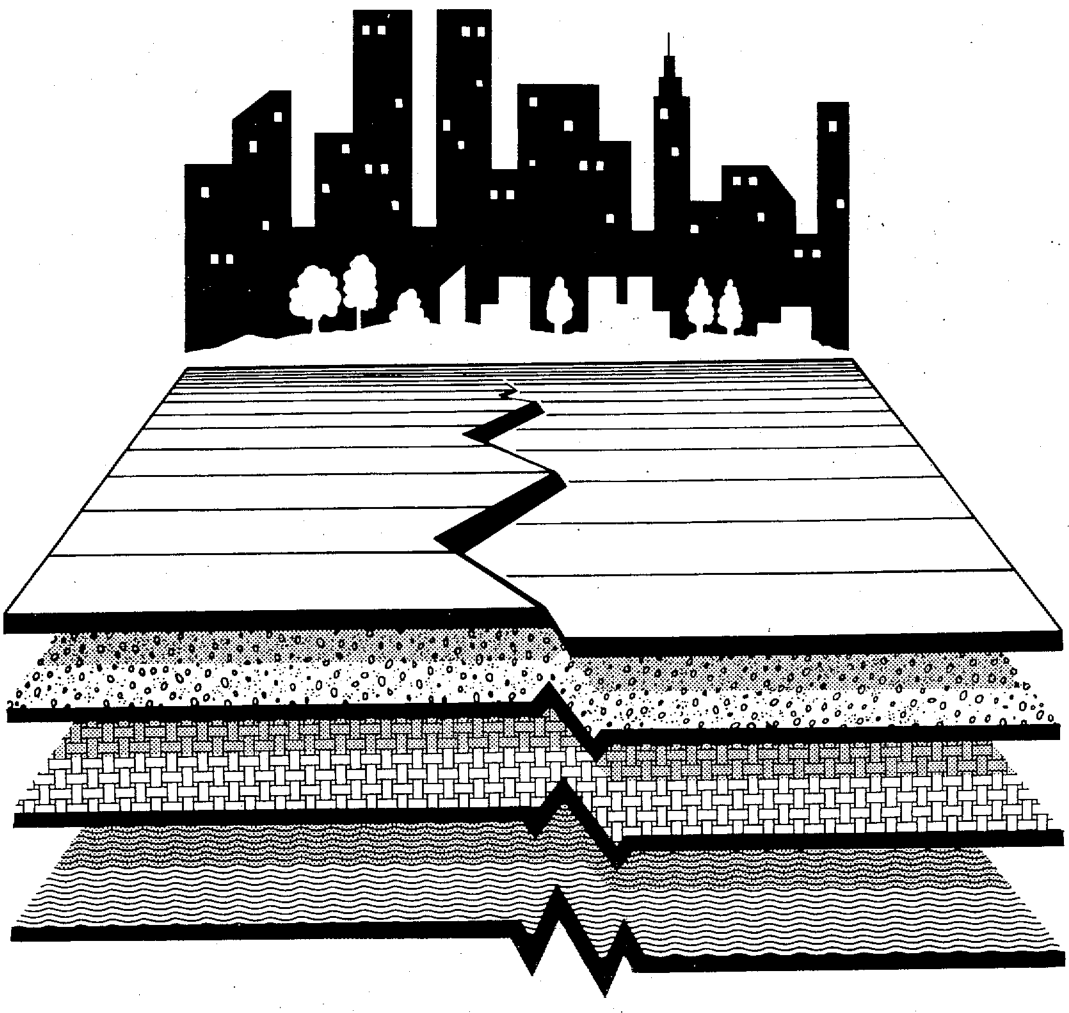
FEDERAL EMERGENCY MANAGEMENT AGENCY

FEMA-84/June,1985

Societal Implications: Selected Readings



EARTHQUAKE HAZARDS REDUCTION SERIES 14

Issued by FEMA in furtherance of the Decade for Natural Disaster Reduction.

Societal Implications: Selected Readings FEMA - 84

BUILDING SEISMIC SAFETY COUNCIL

The Building Seismic Safety Council (BSSC) is an independent, voluntary body that was established under the auspices of the National Institute of Building Sciences (NIBS) in 1979 as a direct result of nationwide interest in the seismic safety of buildings. It has a membership of 57 organizations representing a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings. To fulfill its purpose, the BSSC:

1. Promotes the development of seismic safety provisions suitable for use throughout the United States;

2. Recommends, encourages, and promotes the adoption of appropriate seismic safety provisions in voluntary standards and model codes;

3. Assesses progress in the implementation of such provisions by federal, state, and local regulatory and construction agencies;

4. Identifies opportunities for improving seismic safety regulations and practices and encourages public and private organizations to effect such improvements;

5. Promotes the development of training and educational courses and materials for use by design professionals, builders, building regulatory officials, elected officials, industry representatives, other members of the building community, and the public;

6. Advises government bodies on their programs of research, development, and implementation; and

7. Periodically reviews and evaluates research findings, practices, and experience and makes recommendations for incorporation into seismic design practices.

The BSSC's area of interest encompasses all building-type structures and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations.

The BSSC believes that the achievement of its purpose is a concern shared by all in the public and private sectors; therefore, its activities are strutured to provide all interested entities (e.g., government bodies at all levels, voluntary organizations, business, industry, the design profession, the construction industry, the research community, and the general public) with the opportunity to participate. The BSSC also believes that the regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach to seismic safety that allows for consideration of the relative risk, resources, and capabilities of each community.

The BSSC is committed to continued technical improvement of seismic design provisions, assessment of advances in engineering knowledge and design experience, and evaluation of earthquake impacts. It recognizes that appropriate earthquake hazard reduction measures and initiatives should be adopted by existing organizations and institutions and incorporated, whenever possible, into their legislation, regulations, practices, rules, codes, relief procedures, and loan requirements so that these measures and initiatives become an integral part of established activities, not additional burdens. The BSSC itself assumes no standards-making and -promulgating role; rather, it advocates that standards-formulation organizations consider BSSC recommendations for inclusion into their documents and standards.

BSSC PROGRAM ON IMPROVED SEISMIC SAFETY PROVISIONS

SOCIETAL IMPLICATIONS: SELECTED READINGS

Prepared for the
Federal Emergency Management Agency
by the
Building Seismic Safety Council
Committee on the Societal Implications
of Using New or Improved
Seismic Safety Design Provisions

BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.
1985

NOTICE: Any opinions, findings, conclusions, or recommendations expressed in this publication do not necessarily reflect the views of the Federal Emergency Management Agency. Additionally, neither FEMA nor any of its employees make any warranty, expressed or implied, nor assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, product, or process included in this publication.

This report was prepared under Contract EMW-C-0903 between the Federal Emergency Management Agency and the National Institute of Building Sciences.

For further information regarding this document, contact the Executive Director, Building Seismic Safety Council, 1015 15th Street, N.W., Suite 700, Washington, D.C. 20005.

Reports in the series prepared by the Building Seismic Safety Council as part of its Program on Improved Seismic Safety Provisions include the following:

Societal Implications: A Community Handbook, 1985
Societal Implications: Selected Readings, 1985

Overview of Phases I and II, 1984
Appendixes to the Overview, 1984

NEHRP Recommended Provisions for the Development of Seismic Safety Provisions for New Buildings (draft version for ballot by the BSSC membership), 1984:
- Part 1--Provisions,
- Part 2--Commentary,
- Appendix--Existing Buildings

Trial Designs, 1984:
- Charleston Designs by Enwright Associates, Inc.,
- Chicago Designs by Alfred Benesch and Company,
- Chicago Designs by Klein and Hoffman, Inc. (Parts 1-4),
- Ft. Worth Designs by Datum/Moore Partnership,
- Los Angeles Designs by S. B. Barnes and Associates,
- Los Angeles Designs by Johnson and Nielsen Associates,
- Los Angeles Designs by Wheeler and Gray,
- Memphis Designs by Allen and Hoshall,
- Memphis Designs by Ellers, Oakley, Chester, and Rike, Inc.,
- New York Designs by Weidlinger Associates (Parts 1-2),
- New York Designs by Robertson, Fowler, and Associates,
- Phoenix Designs by Magadini-Alagia Associates,
- Phoenix Designs by Read Jones Christoffersen, Inc.,
- Seattle Designs by ABAM Engineers, Inc.,
- Seattle Designs by Bruce C. Olsen,
- Seattle Designs by Skilling Ward Rogers Barkshire, Inc.,
- St. Louis Designs by Theiss Engineers, Inc. (Parts 1-2)

Printed in the United States of America.

BSSC BOARD OF DIRECTION
1984-85

**BSSC COMMITTEE
ON THE SOCIETAL IMPLICATIONS
OF USING NEW OR IMPROVED
SEISMIC SAFETY DESIGN PROVISIONS**

<u>Chairman</u>

Christopher W. Arnold, Building Systems Development, Inc., San Mateo, California

<u>Members</u>

Warner Howe, Gardner and Howe Structural Engineers, Memphis, Tennessee
James E. Lapping, AFL-CIO Building and Construction Trades Department, Washington, D.C.
Joseph J. Messersmith, Jr., Portland Cement Association, Rockville, Virginia
Charlene Sizemore, Consumer Representative, Huntington, West Virginia

<u>Consultants</u>

Claret M. Heider, Technical Editor-Writer, Sterling, Virginia
Claire B. Rubin, Program of Policy Studies in Science and Technology, George Washington University, Washington, D.C.
Stephen F. Weber, Center for Applied Mathematics, National Bureau of Standards, Gaithersburg, Maryland

<u>BSSC Staff</u>

James R. Smith, Executive Director
Elaine Griffin, Secretary

This volume of selected readings is intended to accompany the volume Societal Implications: A Community Handbook, one of a series of publications prepared by the Building Seismic Safety Council (BSSC) under contract to the Federal Emergency Management Agency (FEMA). The objective of the handbook is simply to provide between two covers a synthesis of what is known about the most significant societal implications of adopting new or improved seismic regulations for new buildings in those communities across the land that are considering such a step. This accompanying volume of selected readings provides a sampling of more detailed information.

The handbook is a companion publication to the NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings. Both are intended for voluntary use by interested parties in the nonfederal sector. Comments and suggestions for improvement of the handbook are earnestly solicited. Similar publications are scheduled for completion in the next several months.

FEMA is grateful to the BSSC Board of Direction and its Executive Director, to the BSSC committee members and consultants who prepared the handbook and assembled the selected readings, and to the many other volunteers whose contributions to and participation in the BSSC study have enriched the content of these publications. Similar acknowledgment is due the U.S. Geological Survey for the geotechnical information and the National Bureau of Standards for the structural engineering and cost information contained in the handbook as well as for their support at the four BSSC meetings with building process participants (in Charleston, South Carolina; Memphis, Tennessee; St. Louis, Missouri; and Seattle, Washington) during which many useful insights were obtained.

Federal Emergency Management Agency

This volume of selected readings and the handbook it accompanies have been developed to provide participants in the building process at the local, state, and regional levels with the information they need to adequately address the potential effects on their communities of using new or improved seismic safety design provisions in the development of regulations for new buildings. It represents one product of an ongoing program conducted by the Building Seismic Safety Council (BSSC) for the Federal Emergency Management Agency (FEMA). A brief description of this program is presented below so that readers of the handbook and these selected readings can approach their use with a fuller understanding of their purpose and limitations.

BSSC PROGRAM ON IMPROVED SEISMIC SAFETY PROVISIONS

The BSSC was established in 1979 as an independent, voluntary body with a membership of 57 organizations representing the full spectrum of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings. The BSSC Program on Improved Seismic Safety Provisions is structured to assist FEMA in achieving national seismic safety goals.

Phases I and II

Phases I and II of the BSSC program have focused on new construction. During these phases Tentative Provisions for the Development of Seismic Regulations for Buildings, originally developed by the Applied Technology Council (ATC), were reviewed and revised (in cooperation with the National Bureau of Standards). To assess the economic impact, usability, and technical validity of the amended provisions, 17 design firms in 9 major cities,[1] where the seismic risk varies from high to low, were retained to prepare trial designs of the structural systems of various types of buildings. The trial design effort included 46 buildings and each was designed twice--once according to the amended ATC document and once according to the prevailing local code for the particular location of the design.

The amended ATC document was further revised in light of the results of these trial designs and in late 1984 was submitted by the BSSC for ballot

[1]Charleston, South Carolina; Chicago, Illinois; Ft. Worth, Texas; Los Angeles, California; Memphis, Tennessee; New York, New York; Phoenix, Arizona; St. Louis, Missouri; and Seattle, Washington.

to its members (see inside back cover) as The NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings.

Phase III

During Phase III of the BSSC program, modifications are being made as a result of this first ballot. The document that results, NEHRP Recommended Provisions--1984, will reflect the consensus approval of virtually all segments of the building community and its publication is expected in late 1985. Since the NEHRP Recommended Provisions document is to present the most up-to-date data and technology in the context of a rational, nationally applicable approach to seismic safety design, its continuous revision and the issuance of subsequent editions are to be expected.

The BSSC also has examined the societal implications that could be expected as a consequence of utilizing the NEHRP Recommended Provisions as a source document in the development of local regulations, especially in communities east of the Rocky Mountains that have, to date, been largely unconcerned about the seismic safety aspects of building design. The handbook and this accompanying volume of selected readings present the results of that study.

Related Efforts

In related efforts the BSSC is examining the likely impact of the NEHRP Recommended Provisions on building regulatory practices and is developing materials and plans for encouraging maximum use of the NEHRP Recommended Provisions. In a joint venture with the Applied Technology Council and the Earthquake Engineering Research Institute, the BSSC is also examining the issues involved in improving the seismic safety of existing buildings and critical facilities. Information on these subjects will be published separately.

SCOPE OF THE HANDBOOK

The potential societal impacts of using new or improved seismic safety design provisions in developing regulations for new buildings are varied and difficult to quantify definitively. Nevertheless, after meeting with building process participants and seismic safety experts and pooling the expertise of its members, the BSSC Committee on Societal Implications has identified a number of potential impacts that require community consideration. The emphasis is on new buildings, and existing facilities are discussed only to the extent that seismic safety provisions for new buildings affect them.

DEVELOPMENT OF THE HANDBOOK

To develop the needed information, the BSSC Societal Implications Committee attempted to identify the many principal concerns, issues, and problems connected with utilization of the NEHRP Recommended Provisions by meeting with building process participants in four selected areas:

- Charleston, South Carolina
- Memphis, Tennessee
- Seattle, Washington
- St. Louis, Missouri

Charleston and Seattle already enforce seismic safety provisions for new buildings while Memphis and St. Louis do not. Although these four communities have somewhat different physical, social, and economic characteristics and different degrees of seismic risk, they are representative of a broad range of seismic conditions and urban characteristics that exist in the United States.

The committee supplemented the information it gathered in the four communities with information from the literature and with the expertise and experience of its individual members so that it could present the users of the handbook with relatively authoritative, if not completely comprehensive, guidance.

CONTENT OF THE HANDBOOK AND THESE SELECTED READINGS

In the chapters included in the handbook:

- The potential impacts identified by the committee are described.

- Information sources and data bases that may be able to provide communities with general as well as specific information and guidance are listed.

- General terms related to earthquakes are defined and the modified Mercalli intensity (MMI) scale and the Richter magnitude scale are described.

In this accompanying volume of selected readings, the committee has assembled a series of papers that address various aspects of the seismic safety issue. A number of these papers were prepared specifically for the BSSC study and several were presented at the BSSC committee meetings with building process participants. Several other papers were originally presented at a 1984 FEMA workshop but were not published. One other paper was suggested for inclusion by a BSSC committee member. Included are:

- An estimate of the impact of the NEHRP Recommended Provisions on design and construction costs developed for the BSSC study

  "Cost Impact of the NEHRP Recommended Provisions on the Design and Construction of Buildings" by Stephen F. Weber, National Bureau of Standards

- Descriptions of the seismic hazard in various areas of the United States developed for the BSSC study

  "Earthquake at Charleston in 1886" by G. A. Bollinger, Virginia Polytechnic Institute and State University

  "Earthquake Hazards in the Memphis, Tennessee, Area" by Arch C. Johnston and Susan J. Nava, Tennessee Earthquake Information Center

  "Evaluation of the Earthquake Ground-Shaking Hazard for Earthquake Resistant Design" by Walter W. Hays, U.S. Geological Survey

  "Introduction to Seismological Concepts Related to Earthquake Hazards in the Pacific Northwest" by Stewart W. Smith, University of Washington

  "Nature of the Earthquake Threat in St. Louis" by Otto W. Nuttli, St. Louis University

- Explanations of seismic safety codes

  "Development of Seismic Safety Codes" by Robert M. Dillon, American Council for Construction Education

  "The Purpose and Effects of Earthquake Codes" by Theodore C. Zsutty, San Jose State University, and Haresh C. Shah, Stanford University

- Descriptions of current seismic hazard mitigation practices and programs

  "Current Practices in Earthquake Preparedness and Mitigation for Critical Facilities" by James E. Beavers, Martin Marietta Energy Systems, Inc.

  "Management of Earthquake Safety Programs by State and Local Governments," by Delbert B. Ward, Structural Facilities, Inc.

- A description of recent seismic safety policy research developed for the BSSC study

  "Summary of Recent Research on Local Public Policy and Seismic Safety Mitigation" by Claire B. Rubin, George Washington University

- A summary of the BSSC committee meetings with building process participants in Charleston, Memphis, St. Louis, and Seattle

- A relatively extensive set of references to serve as the basis for more detailed research

- The list of information sources and the glossary of terms that also appear as Chapters 7 and 8 of the handbook

Although the readings presented herein are far from comprehensive, they are intended to give the handbook user some idea of the sorts of information that are available. In addition, the set of references and the list of information sources, which are included in both the handbook and the selected readings volume, will give interested readers some guidance about what to look for and where to find it when they pursue topics of special interest.

ACKNOWLEDGMENTS

The BSSC and its Committee on Societal Implications is grateful to the many individuals who contributed to this project. The committee is especially grateful to the building process participants in Charleston, Memphis, St. Louis, and Seattle who attended its meetings and so articulately identified issues for committee attention. Special thanks go to those who spoke at and/or developed presentations for the committee's meetings: in Charleston, Charles Lindbergh of the South Carolina Seismic Safety Commission, G. A. Bollinger of Virginia Polytechnic Institute and State University, and Joyce B. Bagwell, of Baptist College; in Memphis, Warner Howe of Gardner and Howe Structural Engineers and Arch Johnston and Susan Nava of the Tennessee Earthquake Information Center; in St. Louis, Otto Nuttli of St. Louis University and John Theiss of Theiss Engineers, Inc.; and in Seattle, Bruce Olson, Consulting Engineer, and Stewart Smith of the University of Washington. The committee also wishes to thank Stephen Weber of the National Bureau of Standards for conducting an economic analysis of the cost impact of the NEHRP Recommended Provisions and for presenting a summary of his findings at each of the four meetings. Walter Hays of the U.S. Geological Survey deserves special recognition for arranging for the speakers and for preparing a special background paper for their use. Finally, the committee wishes to acknowledge the contribution of its consultants and the other authors who graciously allowed their work to be included in the selected readings volume.

REQUEST FOR FEEDBACK

Because every community is unique in some way, FEMA and the BSSC urge those using the handbook and these accompanying readings to provide feedback on their experiences. If the handbook is to serve its purpose as one means for providing up-to-date, experience-based seismic design information, reports from its users are essential.

A "Feedback Sheet" is included at the back of both the reports to make the response process easier and to permit users to request additional information. Every attempt will be made to integrate what is learned into future publications and to inform those who respond about the experiences of other communities and about subsequent BSSC and FEMA efforts.

TABLE OF CONTENTS

APPENDIXES

COST IMPACT OF THE NEHRP RECOMMENDED PROVISIONS ON THE DESIGN AND CONSTRUCTION OF BUILDINGS

STEPHEN F. WEBER

ABSTRACT

This paper provides some information on the approximate cost impacts resulting from implementation of the NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions (Building Seismic Safety Council 1984 a) and proposes research to obtain improved estimates of cost impacts. The information is derived from the 52 case studies of the Building Seismic Safety Council (BSSC) trial design program conducted in 1983-84 and based on an amended version of the Applied Technology Council's Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC Tentative Provisions). The NEHRP Recommended Provisions are the result of the revisions and amendments to the ATC Tentative Provisions that were recommended during the trial design program. For the 29 trial designs conducted in the 5 cities (Chicago, Ft. Worth, Memphis, New York, and St. Louis) whose local building codes currently have no seismic design provisions, the average projected increase in total building construction costs was 2.1 percent. For the 23 trial designs conducted in the 4 cities (Charleston, Los Angeles, Phoenix, and Seattle) whose local codes currently do have seismic design provisions, the average projected increase in total building construction costs was 0.9 percent. The average increase in cost for all 9 cities combined was 1.6 percent. Although these case study results cannot be directly projected to the U.S. building population, they do reflect the order of magnitude of the cost impacts.

INTRODUCTION

This paper provides information on the approximate cost impacts resulting from implementation of the National Earthquake Hazards Reduction Program NEHRP Recommended Provisions and proposes research to obtain improved estimates of these cost impacts. The information presented here summarizes the results of 52 case studies which compared the costs of constructing the structural components of a wide variety of buildings designed according to two distinct criteria: (1) the prevailing local

---

Dr. Weber is an Economist for the Center for Applied Mathematics, National Bureau of Standards, Gaithersburg, Maryland. He developed this paper for the BSSC Study of Societal Implications and presented this information at the BSSC meetings in Charleston, Memphis, St. Louis, and Seattle.

building code; and (2) a proposed set of improved seismic safety provisions similar to the NEHRP Recommended Provisions. Some of the case studies also compared the structural engineering design time required for the two design criteria. The case studies included multifamily residential, office, industrial, and commercial building designs in nine U.S. cities.

The case studies that serve as the primary data source for this paper are the result of the Building Seismic Safety Council (BSSC) trial design program that was conducted in 1983-84. This trial design program was established to evaluate the usability, technical validity, and cost impact of the application of a somewhat amended version the Applied Technology Council (ATC) Tentative Provisions for the Development of Seismic Regulations for Buildings. The NEHRP Recommended Provisions, which currently are being balloted by the BSSC membership, include additional amendments made in response to the results of the trial design program.[1] It is important to note, therefore, that the trial design program data on potential cost impacts of seismic design summarized here are based on the amended Tentative Provisions and not directly on the NEHRP Recommended Provisions themselves and that, as noted by the BSSC: "Some buildings showing high cost impacts will be significantly affected by new amendments to the amended Tentative Provisions that should tend to reduce the impact (BSSC, 1984 b)."

The framework for selecting the specific building designs included in the trial design program is first described. The major factors considered in that selection framework include building occupancy type, structural system, number of stories, and the cities for which the designs were developed. The types of cost data reported by the participating engineering firms also are described. The cost impact data results of the trial designs then are presented in summary form by building occupancy type and by city as well as in detail for each of the four cities visited by the BSSC Committee on Societal Implications (Charleston, South Carolina; Memphis, Tennessee; St. Louis, Missouri; and Seattle, Washington). In presenting the cost data, a distinction will be made between two separate cases: (1) building communities not currently using a seismic code of any kind (e.g., Memphis and St. Louis) and (2) building communities that currently are using a seismic code (e.g., Charleston and Seattle). The paper closes with some conclusions regarding the cost impact of seismic design and suggestions for further research.

DESCRIPTION OF THE TRIAL DESIGN DATA

The construction cost impact of the amended Tentative Provisions generally depends on two major groups of factors: those related to characteristics of the building itself and those related to the location in which the building is to be constructed. The first group includes such

[1]See Volume 1, Overview of Phase I and II, of the 1984 BSSC report, BSSC Program on Improved Seismic Safety Provisions, for a full description of the trial design effort.

factors as the planned occupancy of the building, the structural system used to support the building, the general shape of the building in terms of number of stories and floor plan, and the total size of the building. The second group includes such factors as the seismic hazard of the building site and the degree to which that hazard is reflected in the current local building code. Because each of these six cost impact factors can assume several different values, the number of potentially unique trial designs is very large indeed. A statistically valid experimental design that would adequately sample from each of these unique cases (combinations of cost impact factors) would have required a total sample size that was well beyond the budget and time available for the trial design program.

Framework for Selecting Trial Designs

Because of the necessary limit on the number of trial designs, the case study approach was used as an alternative to statistical sampling. In order to make the case studies as representative as possible, a framework was developed distributing the trial designs over the broad range of values for each of the cost impact factors mentioned above. This overall framework used for selecting the specific building designs included in the trial design program is best illustrated by referring to Table 1. Beginning with the left-hand column, there are four types of building occupancy included in the framework: residential, office, industrial, and commercial. As the next four columns show, the structural system was divided into four elements, each of which has a number of different types: vertical load system, seismic resisting system components, other vertical components, and floor or roof components. For example, the vertical load system could use either bearing walls or a complete vertical load carrying frame. The method of resisting seismic forces could employ such systems as plywood walls, concrete masonry walls, brick walls, precast concrete walls, reinforced concrete shear walls, prestressed moment frame, or steel braced frame. The number of stories varied from single-story to a high-rise building with 40 stories. Between these extremes there were buildings with 2, 3, 5, 10, 20, and 30 stories. As indicated in the far right-hand columns, the trial designs were distributed over nine cities: Los Angeles, Seattle, Memphis, Phoenix, New York, Chicago, Ft. Worth, Charleston, and St. Louis. These cities cover the range of seismic hazard levels found in the United States and they vary in the degree to which seismic provisions are contained in their local building code. For example, Los Angeles is in a very high seismic hazard area while New York City is in a low hazard area. Similarily, Seattle has adopted the Uniform Building Code (1979) seismic provisions while the city of Memphis, although exposed to considerable seismic hazard, has no seismic provisions in its building code.

There are a total of 468 possible combinations of the 9 cities with the 52 building types. Each of these combinations constituted a potential candidate for inclusion in the trial design program. Each candidate is represented by one of the cells in the nine columns on the right-hand side of Table 1. From all these potential candidates, 46 were selected as the building design/city combinations used in the trial design pro-

TABLE 1 Framework for Selecting BSSC Trial Designs

Plan Form	Vertical Load System	Seismic Resisting System Components	Other Vertical Components	Floor or Roof Components	Bldg. No.	No. of Stories[1]	Cities: Phase I: Los Angeles	Seattle	Memphis	Phoenix	Phase II: New York	Chicago	Ft. Worth	Charleston	St. Louis
Residential	Bearing Walls	Plywood wall		Wood + plywood diaphragm	1	3	o	o							
		Concrete masonry wall		Wood + plywood diaphragm	2	3				o					
		Brick and concrete masonry		Wood + plywood diaphragm	2A	3						o			
		Concrete masonry wall		Prestressed slab	3	5				o			o		
		Brick wall		Reinforced concrete slab	4	6									
				Reinforced concrete slab	5	12	o				o				
				Reinforced concrete slab	5A	10									o
		Brick and concrete masonry wall		Steel joist	6	5								o	
				Steel joist	7	12									
		Reinforced concrete wall		Reinforced concrete slab	8	5			o						
				Reinforced concrete slab	9	12						o			
				Post-tensioned slab	10	5				o					
		Precast concrete wall		Prestressed slab	11	5									
				Prestressed slab	12	12									
	Complete Vertical Load Carrying Frame	Steel, braced frame (transverse)/ moment frame (longitudinal)	Steel framing	Steel joists	13	10									
			Steel framing	Steel beam & RC slab	14	20			o						
		Reinforced concrete shear wall	RC framing	RC flat plate	15	10	o						o		
			RC framing	Post-tensioned flat plate	16	20						o			o
			RC framing	Post-tensioned flat plate	17	30									
		Reinforced concrete moment frame (perimeter)	RC framing	RC flat plate	18	10	o		o						
			RC framing	RC flat plate	19	20									
		RC, MF (perimeter) & SW (dual)	RC framing	RC flat plate	20	20									
			RC framing	RC flat plate	20A	30					o				
Office	Bearing walls	Reinforced concrete wall (core)	RC framing	RC flat slab	21	10									
			RC framing	RC flat slab	22	20				o	o				
		PC wall (interior & exterior)	PS framing	Prestressed slab	23	10									
	Complete Vertical Load Carrying Frame	Reinforced concrete shear wall	Steel framing	Steel beam & RC slab	24	10		o						o	
			Steel framing	Steel beam & RC slab	25	20									
		Steel braced frame	Steel framing	Steel beam & RC slab	26	30									
			Steel framing	Steel beam & RC slab	26A	5									o
		Steel moment frame	Steel framing	Steel beam & RC slab	27	10	o		o						
			Steel framing	Steel joist	27A	5					o				
			Steel framing	Steel beam & RC slab	27A	5							o		
			Steel framing	Steel beam & RC slab	28	40									
			Steel framing	Steel beam & RC slab	28A	30					o				
		Steel MF & RC SW (dual)	Steel framing	Steel beam & RC slab	29	20	o								
		Steel MF and BF (dual)	Steel framing	Steel beam and RC slab	30	20		o							
		Reinforced concrete moment frame	RC framing	Post-tensioned flat slab	31	10									
			RC framing	RC pan joist & waffle	32	10				o	o				
			RC framing	PT pan joist	33	20									
		RC, MF & SW (dual)	RC framing	RC pan joist & waffle	34	20	o								
Industrial	Bearing walls	Concrete masonry wall	Steel framing	Steel joist	35	L				o					
		PC wall (maybe PS)	PS framing	Prestressed slab	36	H									
			PS framing	PC double tees girders & beam	36A	H						o			
		PC tilt-up wall	Wood framing	Wood (plywood)	37	L	o								
	Complete Vertical Load Carrying Frame	PC tilt-up wall	Steel framing	Steel joist	38	L			o						
		Steel moment frame (transverse)/ braced frame (longitudinal)	Steel framing	Steel purlins & deck	39	L	o							o	
			Steel framing	Steel long-span truss	40	H		o							
		Concrete masonry wall	Steel framing	Steel joist	41A	2					o				
Commercial	Complete Vertical Load Carrying Frame	Concrete masonry wall	Steel framing	Steel joist	41	3	o								
			Steel framing	Steel joist (irregular plan form)	42	2			o						
		PS moment frame	PS framing	Prestressed slab	43	2									

1 - All office buildings will have a high first story, the industrial buildings are all on one story (with the exception of Building No. 41A) and for them the L indicates a low clearance, and H indicates a high clearance.

2 - BF = braced frame MF = moment frame RC = reinforced concrete
PC = precast concrete PT = post-tensioned concrete SW = shear wall (non-bearing)
PS = prestressed, precast concrete

3 - With the exception of the industrial building with purlins and steel deck (the metal building) all moment frames in Los Angeles, Seattle, and Memphis are to be Special. All moment frame in dual systems must also be Special. All other moment frames may be Ordinary.

gram. These selected combinations are represented by dots that appear in the cells of Table 1. For 6 of these 46 buildings, alternative designs were also developed to provide 6 additional cost impact estimates. As a result, there are 52 data points for which cost impact estimates are available.

For each of the 52 building designs included in the trial design program, a set of building requirements or general specifications was developed and provided to the responsible design engineering firm. An example of such building requirements specifications is presented in Table 2. Within these requirements designers were given latitude to assure that building design parameters such as bay size were compatible with local construction practice. The designers were not permitted, however, to change the basic structural type. For example, they could not change from a reinforced concrete frame system specified in the building requirements to a reinforced concrete shear wall system. Such changes were not permitted even if an alternative structural type would have cost less under the amended Tentative Provisions than the specified type. This constraint may have prevented the designer from selecting the most economical system for the amended Tentative Provisions, and consequently may have resulted in overestimates of the cost impacts for some of the trial designs. The 17 design firms involved in the trial design program and the building designs for which each was responsible are identified by city in Table 3.

Data Reported for Trial Designs

For each of the trial designs, the engineering firms developed two individual designs for the structural components of the buildings. One design was based on the prevailing local building code and the other was based on the amended Tentative Provisions for the city in which the building was to be located. The former will be referred to as the Local Code Design and the latter will be referred to as the Tentative Provisions Design. Both of these designs are described in considerable detail for each trial design in the engineering reports submitted by the firms (BSSC, 1984c). It should be noted that only structural components were included in the analysis for the 52 trial designs summarized here. Consequently, the Tentative Provisions Design did not include those requirements for nonstructural elements described in Chapter 8 of the amended Tentative Provisions. The engineering reports also include detailed estimates of the construction costs for the structural components of each of the two designs (Local Code Design and Tentative Provisions Design). These cost estimates were derived using standard, nationally recognized cost estimating guides that take into account local cost factors. The estimates were made on the basis of current construction costs at the time the designs were completed, which ranged from early 1983 through the middle of 1984. The percentage differences in these structural component cost estimates for the two designs (i.e., cost of the Tentative Provisions Design minus cost of the Local Code Design divided by cost of the Local Code Design times 100) provide the

TABLE 2 Typical Building Requirements[a]

- Plan Form - as per that shown for each building type
- Number of Stories - 20
- Clear Structural Height - 11 feet except that: (a) the first story shall have a 20 - foot clear structural height, and (b) the clear structural height does not apply along the perimeter
- Plan Story Area - 7,500 to 25,000 sq ft
- Plan Aspect Ratio - 1:1 to 2:1
- Bay Size - 20 foot minimum dimension; 600 sq ft minimum area (minimum bay size does not apply to perimeter column spacing)
- Roof - nominally flat but with a 1/4 in 12 slope for drainage
- Window Areas - 30 to 40 percent of exterior wall areas
- Core Size - proportional to the building height
- Core Walls and Floors - include openings for doorways, stairs, and elevators; core wall may be structural
- Foundation Conditions - selected as representative of those that could be anticipated in the local, consistent for all designs, and included in design presentations
- Vertical Load Systems - complete vertical load-carrying frames
- Seismic Resisting Systems Components - dual system[b] - steel moment frame (Special) and braced frame
- Other Vertical Components - steel framing
- Floor and Roof Components - steel beams and reinforced concrete slabs
- Similarity should be maintained in paired studies, such as local requirements for live loads and assumed dead loads
- Other - not applicable

[a]Requirements vary with building type.
[b]As defined in Chapter 2 amended Tentative Provisions.

TABLE 3 Design Firms and Types of Building Designs

City/Design firm	Type of Building/No.
Seattle	
Abam Engineers, Inc.	o 10-Story Steel Frame with RC Shear Wall (O)/S-24
Bruce C. Olsen	o 3-Story Wood with Plywood Walls (R)/S-1
	o 1-Story Long Spa Steel, 30' Clear Height-MF and Braced Frames (I)/S-40
Skilling, Ward, Rogers, Barkshire	o 20-Story Steel Frame-Dual Special & Braced Frames (O)S-30
Los Angeles	
S. B. Barnes & Associates	o 3-Story Wood with Plywood Walls (R)LA-1
	o 1-Story Wood Frame with Precast Concrete Tilt-Up Walls (I)/LA-37
	o 1-Story Steel with Moment and Braced Frames (I)LA-39
	o 2-Story Steel Frame with RC Block Walls (C)/LA-41
Johnson & Nielsen	o 20-Story Steel Moment Frame with Shear Walls (Dual) (O)LA-34
Wheeler & Gray	o 12-Story Reinforced Brick Bearing Wall with RC Slabs (R)LA-5
Phoenix	
Magadini-Alagia Associates	o 5-Story RC Bearing Wall (R)/P-10
	o 20-Story RC Bearing Wall with Core Shear Walls (O)P-22
	o 10-Story RC Frame (Ordinary) (O)/P-32
Read, Jones, Christoffersen Inc.	o 3-Story RC Block Bearing Wall (R)/P-2
	o 5-Story RC Block Bearing Wall (R)/P-3
	o 1-Story Steel Frame with RC Block Shear Walls (I)/P-35

TABLE 3 Continued

City/Design Firm	Type of Building/No.
Allen & Hoshall, Inc.	o 5-Story Bearing Wall (R)M-8 o 1-Story Steel Frame with RC Tilt-Up Exterior Shear Walls (I)/M-38 o 2-Story Steel Frame with Non-Bearing RC Block Walls (C)M-42
Ellers, Oakley, Chester & Rike, Inc.	o 20-Story Steel Moment and Braced Frame with RC Floors (R)/M-14 o 10-Story RC Moment Frame (Perimeter) (R)/M-18 o 10-Story Steel Moment Frame (Special) with RC Slabs (O)/M-27
Ft. Worth, Texas	
Datum-Moore Partnership	o 5-Story RC Block Walls with Pre-stressed Slabs (R)/FW-3 o 10-Story RC Frame with RC Shear Walls (R)FW-15 o 5-Story Steel Moment Frame (O)FW-27A
St. Louis	
Theiss Engineering	o 10-Story Clay Brick Bearing Wall (R)/SL-5A o 20-Story RC Frame with RC Shear Walls (R)SL-16 o 5-Story Steel Frame with Braced Framed at Core (O)/SL-26A
Chicago	
Alfred Benesche & Co.	o 3-Story Brick and RC Block Bearing Walls with Plywood Floor & Roof Diaphragms (R)/C-2A o 20-Story RC Frame with RC Shear Walls (R)/C-16
Klein & Hoffman	o 12-Story RC Bearing Wall (R)/C-9 o Parametric Study of Steel Moment and/or Braced Frames (O)C-26, C-27, & C-30 o 1-Story Precase RC Bearing Walls with PC Double Tee Roof (I)/C-36A

TABLE 3 Continued

City/Design Firm	Type of Building/No.
Klein & Hoffman	o 12-Story RC Bearing Wall (R)/C-9 o Parametric Study of Steel Moment and/or Braced Frames (O)/C-26, C-27, & C-30 o 1-Story Precast RC Bearing Walls with PC Double Tee Roof (I)/C-36A
New York City	
Weidlinger Associates	o 12-Story Brick Bearing Wall (R)/NY-5 o 30-Story RC Moment Frame and Non-Bearing Shear Wall (Dual) (R)/NY-20A o 10-Story RC Moment Frame (O)/NY-32
Robertson and Fowler	o 20-Story RC Bearing Wall (O)/NY-22 o 5-Story Steel Moment Frame (O)/NY-27A o 30-Story Steel Moment Frame (O)/NY-28A o 2-Story Steel Frame with RC Block Walls (I)/NY-41A
Charleston, S.C.	
Enright Associates	o 5-Story Brick and RC Block Bearing Walls (R)/CSC-6 o 10-Story Steel Frame with RC Shear Walls (O)/CSC-24 o 1-Story Steel Moment and Braced Frame (I)/CSC-39

R = Residential
O = Office
I = Industrial
C = Commercial

primary raw data on which this paper is based. Because the focus of this paper is on percentage cost differences rather than absolute estimates, the slight changes in construction costs during the study period can be reasonably ignored.

In addition to the estimates of the construction costs for the structural components of the two designs, the engineering firms also submitted rough estimates of the additional design time that would be required to use the amended *Tentative Provisions*. Typically these estimates were reported as percentage changes in design time required for the structural components assuming the design engineer was already familiar with the amended *Tentative Provisions*. These design time cost percentage change estimates are also summarized below.

SUMMARY OF COST IMPACTS

This section summarizes the cost impact data reported by the 17 design engineering firms that participated in the trial design program. The first subsection provides an overview of the construction cost impacts organized first by type of building occupancy and then by city. In the overview by city, the data are presented in two groups: cities not currently using any seismic provisions in their local building codes and cities currently using seismic provisions in their codes. The first subsection also summarizes the design time percentage change estimates provided by the engineering firms. The second subsection reports the construction cost impacts for each individual trial design in the four cities that were visited by the BSSC Committee on Societal Implications (Charleston, Memphis, St. Louis, and Seattle).

Overview of Cost Impacts

Table 4 presents an overview of the construction cost impacts by type of building occupancy. The five classes of buildings were derived from the orginal four classes found in the framework for selecting trial designs by dividing the residential designs into low-rise (five stories or fewer) and high rise (more than five stories). Because only three of the office building designs have fewer than ten stories (and those three have five stories), the office building class is not divided. Similarly, all seven of the industrial building designs have just one story and the three commercial designs all have two stories. The third column in Table 4 presents the percentage change in construction costs for the structural components of the building, with the Local Code Design as the base, as estimated by the BSSC trial design engineering firms. As can be seen, the average change for the structural costs is 5.6 percent, with by far the largest change (11.2 percent) reported for the high-rise residential designs. This high average for residential buildings is significantly influenced by the extremely high estimates reported for four of these building designs: LA1B (17 percent); M14 (16 percent); M18 (46 percent); and NY20A (20 percent).

TABLE 4 Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs by Building Occupancy Type

Building Occupancy	Number of Designs	Estimated Change In Structural Cost (%)[a]	Projected Change in Total Cost (%)[b]
Low-rise residential[c]	9	3.6	0.7
High-rise residential[d]	12	11.2	3.3
Office	21	4.7	1.3
Industrial	7	1.5	0.5
Commercial	3	5.6	1.7
Average Percentage Change		5.6	1.6

[a]Percentage change in structural construction cost from the local code to Amended Tentative Provisions, as estimated by the BSSC trial design engineering firms, 1983-1984.

[b]Projected percentage change in total building construction cost from the local code to Amended Tentative Provisions, derived from estimated structural cost changes by using the following McGraw-Hill's, Dodge Construction Systems Cost (1984) data on structural cost as a percent of total building cost:

Low-rise residential	18.1%
High-rise residental	30.0%
Office	28.1%
Industrial	33.7%
Commercial	29.5%

[c]Five or fewer stories.

[d]More than five stories.

The fourth column of Table 4 presents the projected percentage change in total building construction costs for each building occupancy type. These total cost changes were projected from the structural cost percentage changes by using data on structural cost as a percentage share of total building cost for each building occupany type. The percentage shares are based on data from McGraw-Hill's, Dodge Construction System Costs (1984), which reports the structural percentage share of total building cost for a large number of typical building designs. The shares for three of these typical building designs were averaged for each of the building occupancy types to derive the percentage shares used in Tables 4 and 5 and reported in the footnotes to the tables. The average projected change in the total construction cost over all 52 of the trial designs is 1.6 percent. The high-rise residential building designs have the highest total building cost impact with 3.3 percent, both because of the four outliers mentioned above and the relatively high structural percentage share used for this type of building (30.0 percent).

Table 5 presents the same type of data as Table 4 but reported for each city grouped according to whether the city currently has a seismic building code or not. As expected, the average estimated change in the structural cost is considerably higher (more than twice as high) for those cities with no seismic provisions in their local codes than for those with seismic provisions: 7.6 percent versus 3.1 percent. A similar relationship holds for the projected change in total building cost: 2.1 percent for cities without seismic provisions versus 0.9 percent for those already having some seismic provisions in their local codes.

Table 6 summarizes the estimates made by the engineering firms of the change in structural design time that is expected to be required once the firms are familiar with the amended Tentative Provisions. The 52 responses are divided into the four categories: negligible change, positive but unspecified change, positive specified change, and negative specified change. The fourth category means that the amended Tentative Provisions, once adopted and familiar to the design firms, would require fewer design hours than the current codes do. The first response category of negligible change was the most common with 28 designs.

Detailed Cost Impacts for Selected Cities

Tables 7 through 10 present the cost impact data for each of the individual trial designs in the four cities visited by the BSSC Committee on Societal Implications. The first two cities (presented in Tables 7 and 8), Memphis and St. Louis, are examples of cities with no seismic provisions in their current building code even though the amended Tentative Provisions place them in relatively high seismic hazard zones. The last two cities (presented in Tables 9 and 10), Charleston and Seattle, are two examples of cities that do have seismic provisions in their local building codes. The point made in reference to Table 6 regarding greater cost impact for the cities without seismic codes can also be

TABLE 5 Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs, by City and City Group With and Without Seismic Provisions in Current Local Codes

City	Number Of Designs	Estimated Change In Structural Cost (%)[a]	Project Change in Total Cost (%)[b]
		Cities Without Seismic Provisions	
Chicago	10	2.5	0.7
Fort Worth	3	6.1	1.5
Memphis	6	18.9	5.2
New York	7	7.3	2.1
St. Louis	3	4.5	1.3
Average Percentage Change		7.6	2.1
		Cities With Seismic Provisions	
Charleston	3	-2.5	-0.6
Los Angeles	10	4.2	1.3
Phoenix	6	6.9	1.9
Seattle	4	-1.1	-0.3
Average Percentage Change		3.1	0.9
Overall Average Percentage Change		5.6	1.6

[a]Percentage change in structural construction cost from the local code to the amended Tentative Provisions, as estimated by the BSSC Trial Design engineering firms, 1983-1984.

[b]Projected percentage change in total building construction cost from the local code to Amended Tentative Provisions, derived from estimated structural cost changes by using the following McGraw-Hill's, Dodge Construction Systems Costs (1984) data on structural cost as percent of total building costs:

Low-Rise Residential	18.1%
High-Rise Residential	30.0%
Office	28.1%
Industrial	33.7%
Commercial	29.5%

TABLE 6 Possible Effects of the Amended Tentative Provisions on Structural Engineering Design Time as Reported by the Trial Design Firms[a]

- For these 28 building designs negligible change was reported:

  LA1, S1, P2, P3, LA5, SL5A, CSC6, C9, P10, LA15, FW15, SL16, LA18, NY20a, S24, CSC24, SL26A, LA27, FW27A, NY28A, NY32, P35, C36A, LA37, CSC39, S40, LA41

- For these 11 building designs positive but unspecified change was reported:

  C2A, FW3, NY5, C26A, C26, C27, C27A, S30, C30A, C30, NY41A

- For these 11 buildin designs positive specified change ranging from 5% to 50% was reported:

  M8, M14, C16, M18, P22, NY22, M27, NY27A, P32, M38, M42

- For these 2 building designs negative specified change of -5% was reported:

  LA29, LA34

[a]For descriptions of the individual building designs listed here, see Table 3.

TABLE 7 Design Description and Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs of Memphis

Design Code	Stories	Structural Cost Change (%)[a]	Total Building Cost Change (%)[a]	Design Code Description
M8	5	25.0	4.5	Residential, reinforced concrete wall and slab
M14	20	16.0	4.8	Residential, steel frame/ moment frame, composite floor
M18	10	46.0	13.8	Residential, reinforced concrete moment frame, flat plate
M27	10	11.0	3.1	Office, steel moment frame, composite floor
M38	1	5.4	1.8	Industrial, tilt-up shear wall, steel framing
M42	2	10.0	3.0	Masonry shear wall, steel framing

[a]See note on Tables 4 and 5 for definition.

TABLE 8 Design Description and Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs of St. Louis

Design Code	Stories	Structural Cost Change (%)[a]	Total Building Cost Change (%)[a]	Design Description
SL5A	10	6.0	1.8	Residential, masonry walls, reinforced concrete slab
SL16	20	3.8	1.1	Residential, reinforced shear wall, flat plate
SL26A	5	3.6	1.0	Office, steel braced frame, composite floor

[a]See note on Tables 4 and 5 for definition.

TABLE 9 Design Description and Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs of Charleston, S. C.

Design Code	Stories	Structural Cost Change (%)[a]	Total Building Cost Change (%)[a]	Design Description
CSC6	5	-3.5	-0.6	Residential, masonry walls, steel joists
CSC24	10	-4.0	-1.1	Office, reinforced concrete shear wall, composite floor
CSC39	1	0.0	0.0	Industrial, steel braced frame/moment frame

[a]See note on Tables 4 and 5 for definition.

TABLE 10 Design Description and Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs of Seattle

Design Code	Stories	Structural Cost Change (%)[a]	Total Building Cost Change (%)[b]	Design Description
S1	3	-1.1	-0.2	Residential, wood frame, plywood walls & dDiaphragms
S24	10	-4.6	-1.3	Office, reinforced concrete shear wall, composite floor
S30	20	1.3	0.4	Office, dual steel braced frame/moment frame, composite floor
S40	1	0.0	0.0	Industrial, steel braced frame/moment frame (metal building)

[a]See note on Tables 4 and 5 for definition.

made here by comparing the average projected change in total building costs for Memphis (the highest at 5.2 percent) and St. Louis (1.3 percent) with the corresponding percentages for Charleston and Seattle (both negative).

CONCLUSIONS AND SUGGESTIONS FOR FURTHER RESEARCH

The results of the BSSC trial design program presented here provide some idea of the approximate cost impacts expected from implementation of the NEHRP Recommended Provisions. For the 29 trial designs conducted in the 5 cities (Chicago, Ft. Worth, Memphis, New York, and St. Louis) whose local building codes currently have no seismic design provisions, the average projected increase in total building construction costs was 2.1 percent. For the 23 trial designs conducted in the 4 cities (Charleston, Los Angeles, Phoenix, and Seattle) whose local codes currently do have seismic design provisions, the average projected increase in total building construction costs was 0.9 percent. The average increase in costs for all 9 cities combined was 1.6 percent. Although these case study results cannot be directly projected to the U.S. building population, they do reflect the order of magnitude of the cost impacts of the NEHRP Recommended Provisions.

In spite of the limited sample size of the trial design program, these data do offer several avenues for further research. The first is an analysis of variance test to see whether the difference in the cost impact estimates for the cities with and without current seismic provisions is statistically significant. Because of the rather large variance in the cost impact estimates, it may be that the difference between the two categories (2.1 percent versus 0.9 percent) is not significant. Other analyses could be conducted to see whether the factors such as building occupancy type and number of levels have a significant effect on the cost impact estimates.

Another major effort could be undertaken to normalize the data by controlling for the effect of the local seismic hazard and the presence of seismic provisons in the current code from city to city. If a seismic design value could be established for the Local Code Design cases that is comparable (i.e., on the same numeric scale) to the Seismic Design Coefficient used in the amended Tentative Provisions cases, then such a normalization could be accomplished. This would make possible the use of regression analysis techniques to develop a statistically valid method for estimating seismic design cost impacts for any city.

REFERENCES

Building Seismic Safety Council. 1984a. NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions. Volume 2 of BSSC Program on Improved Seismic Safety Provisions. Washington, D.C.: Building Seismic Safety Council.

Building Seismic Safety Council. 1984b. Overview of Phases I and II. Volume 1 of BSSC Program on Improved Seismic Safety Provisions.Washington, D.C.: Building Seismic Safety Council.

Building Seismic Safety Council. 1984c. Trial Designs. Volume 3 of BSSC Program on Improved Seismic Safety Provisions. Washington, D.C.: Building Seismic Safety Council.

CURRENT PRACTICES IN EARTHQUAKE PREPAREDNESS AND MITIGATION FOR CRITICAL FACILITIES

JAMES E. BEAVERS

In this paper an attempt is made to briefly address the broad issues of earthquake preparedness and mitigation for critical facilities. Critical facilities considered herein are divided into two major groups: industrial and public.

Critical industrial facilities are defined as those facilities that, if damaged by an earthquake occurrence, could result in the release of substances harmful to the public, employees, or the environment or that could result in what owners consider as unacceptable financial losses. Examples of such facilities are nuclear power plants, chemical processing plants, research and development facilities, and high-technology manufacturing plants.

Critical public facilities are defined as those facilities that, if damaged by an earthquake occurrence, could result in large numbers of the public experiencing life, life-support systems, or financial losses. Examples of such facilities are hospitals, schools, stadiums, fire stations, dams, and bridges.

CURRENT PRACTICES

Practice vs Hazard

Current practice today is actually based on the perception of the earthquake hazard. All one has to do to recognize this is to compare earthquake design practice in the State of California to that in the State of Tennessee for example. In California, the perception is that there is an earthquake hazard, rightfully so. As a result, there are uniformly accepted seismic preparedness and mitigating practices, primarily in the form of accepted seismic design codes. In Tennessee, the perception is that there is no earthquake hazard, which is wrongfully so. As a result, not only are there no uniform seismic preparedness and mitigating practices, they are virtually nonexistent.

Dr. Beavers is Manager, Civil and Architectural Engineering, at Martin Marietta Energy Systems, Inc., Oak Ridge, Tennessee. He presented this paper at the FEMA Earthquake Education Curriculum Workshop held at the National Emergency Training Center, Emmitsburg, Maryland, June 27-29, 1984.

Four Levels of Practice

Regardless of the general perception of the earthquake hazard, today's practice in earthquake preparedness and mitigation for critical facilities from an engineering point of view can be divided into four general levels:

> Level I--Complex earthquake hazard evaluation and facility seismic analysis and design as is conducted for nuclear power plants (U.S. Nuclear Regulatory Commission, 1975).
>
> Level II--Earthquake hazard evaluation and seismic analysis and design as is conducted for an important chemical plant or, on occasion, possibly a hospital (Manrod et al., 1981).
>
> Level III--Normal earthquake hazard evaluation and facilities analysis and design procedures as is conducted using the Uniform Building Code (UBC) or similar codes (International Conference of Building Officials, 1982; Structural Engineers Association of California, 1975).
>
> Level IV--No earthquake hazard evaluation or facility seismic analysis or design provisions except for the inherent lateral resistance provided by wind analysis and design requirements.

Level I provides for a thorough evaluation of the earthquake hazard at the location of interest to the point of simulating the expected ground motions. The ground motions are then used as input to a rigorous seismic analysis of the facilities followed by detail design and documentation procedures. In many cases, Level I is considered as a very conservative approach to earthquake preparedness and mitigation.

Level II generally represents an adjusted medium between the approach in Level I and the approach used in Level III. The Applied Technology Council provisions (Applied Technology Council, 1978) represent a Level II approach for buildings. Manrod and co-workers (1981) discuss a Level II approach for preparedness and mitigation of existing critical industrial facilities.

Unfortunately, the preparedness and mitigation actions taken for most structures built in the United States today, many of which may be considered critical, fall under Level IV.

Except in California and one or two other states, there are virtually no adopted earthquake hazard evaluation or seismic analysis and design guidelines or codes in the cities, counties, or municipalities.

Levels of Application vs Critical Facilities

All nuclear power plants being constructed today fall under the strict seismic evaluation, analysis, and design requirements set forth by the U.S. Nuclear Regulatory Commission identified herein as Level I. Other similar critical facilities, such as plutonium facilities, generally fall under the same requirements.

Chemical processing facilities, uranium enrichment facilities, and high technology manufacturing plants usually will fall into the Level III approach and, in some circumstances, Level II at the discretion of the owners--be they government or private industry. However, in many cases, using the minimum requirements of the UBC seismic design provision (the Level III application) may not be adequate for such facilities.

Critical public facilities such as dams and bridges may also fall under Level II and III seismic provisions depending upon the perceived earthquake hazard of the builder/owner. Schools, hospitals, fire stations, and stadiums will fall under the seismic provisions as described in either Level III or IV. Since the mid-1970s, most hospital designs fall under the Level III procedures. However, hospitals built before the mid-1970s and schools (except California), fire stations, and stadiums built today may actually fall under Level IV.

All critical facilities, as a minimum, should meet earthquake preparedness and mitigation requirements as defined in the UBC and, in many cases, go beyond the requirements of the UBC. However, as a cautionary note, it must be remembered when using the UBC, especially for industrial facilities, that it is a building code and judgment must be used where the code does not directly apply.

Today's Application

Although it was stated above that most structures built in the United States today are not designed to earthquake preparedness and mitigation provisions (a Level IV approach), nor are such provisions required by law, a process is occurring in this country where such provision are being applied more and more each day. This process is happening because of the educational program occurring within the professional groups (engineers, architects, scientists, etc.) and the liability responsibilities of such professionals. For example, most engineers are now aware of the need for earthquake hazard preparedness and mitigation practices in the design of any new facility. Although no local enforcement codes may require such procedures, architects and engineers are acutely aware of recent decisions in the courts where following the minimum requirements of building codes is not justification for not using prudent engineering judgment. As a result, many architects and engineers are now applying earthquake hazard preparedness and mitigation provisions in their facility design. For critical facilities, architects and engineers usually have no trouble convincing the builder/owner of the necessity for such provisions and the builder/owner is willing to accept the additional costs. However, for noncritical facilities, it is extremely difficult for the engineer or architect to convince the builder/owner of the long-term cost benefit of applying such provisions, and in many cases, the builder/owner will refuse--creating a professional dilemma for the architect or engineer.

TODAY'S TECHNOLOGY

Progress

Today's technology can best be described as a "forever changing state of the art." After each major earthquake, scientists and engineers seem to gain new insights as to how earthquake ground-shaking occurs and how man-made structures respond. The state of the art has advanced tremendously during the past 20 years as a result of the 1964 Alaskan Earthquake, the 1971 San Fernando Earthquake, other large but less notable earthquakes (e.g., Coalinga 1983), engineers' and scientists' success at obtaining instrumental recordings of earthquake motions and structural response, the "national" emphasis placed on understanding the earthquake phenomena to provide safe nuclear power plants, and the passage of the Earthquake Hazards Reduction Act of 1977.

The nuclear power industry can be contributed with being the catalyst that sparked a strong earthquake and earthquake engineering research program in the mid-1960s that may have peaked as we entered the 1980s.

Although a lot has been learned during the past 20 years, our current understanding of the earthquake phenomena and how man-made structures respond to such events still has many shortcomings.

Understanding the Problem

We now understand the general phenomena of what causes earthquakes based on the concept of plate tectonics. This concept applies very well on the West Coast of the United States. However, understanding the concept of earthquake occurrences at intra-plate locations like the midwestern and eastern parts of the United States is extremely lacking. The lack of understanding can be based on two primary reasons: infrequent earthquake occurrences and earthquake occurrences at depth with no surface faulting. We do know enough about intra-plate earthquakes to know that the same design and analysis principles that are used on the West Coast may not be directly applicable in the Midwest and East because of the infrequency of such events and the attenuation rates.

From a purely engineering point of view, a such high state of technology exists regarding our ability to analyze complex structures to great detail. The phenomenal growth of the computer industry has provided us with this capability. However, our understanding of material properties and our ability to construct structures to such precise detail is far behind. In fact, our ability to analyze and design structures to earthquake ground motions far exceeds our ability to understand what the motions might be.

Lag Time

As engineers and scientists learn more about preparedness and mitigation of the earthquake hazard and our development of technology, they begin the process of adopting this new found knowledge to practice. Like any industry, when trying to put new technology into practice, there is a lag time. However, in the case of nuclear power plants where the Level I approach to preparedness and mitigation occurs, technology has been placed directly into practice with little or no lag time. The Level I approach to preparedness and mitigation has been the leader of the "earthquake industry." In the Level II approach, an assessment would be made of the new developments in the Level I approach and these developments would be either rejected or accepted as deemed appropriate and practical for the particular critical facility under consideration. For those developments deemed appropriate for a Level II application, the lag time was usually relatively short. Those developments not deemed appropriate for a Level II application have been put aside--it may take years before such developments become practice.

The lag time in getting new developments into practice at the Level III stage of application usually is several years unless the development results in the awareness of a serious deficiency in the Level III approach. Even then it would probably take one or two years to get the code bodies changed.

Dynamic Analysis--Practice

As an example of the difficulty of taking technological development and applying it to practice, let's consider the case of dynamic analysis. Dynamic analysis capability has been around for 30 years and engineers recognize that structures subjected to earthquake loads are more properly analyzed using some form of dynamic analysis. But in the UBC, which is an accepted nationwide Level III type application, there are no provisions for such analyses. This exists for several reasons including, for example, perceived added costs of doing such analyses which are more complex than a simple static analysis, an undergraduate engineering educational level that does not require a dynamic analysis background (reserving it for graduate students), perceived low earthquake hazards by engineers and the public, and the tendency to keep legislated codes as simple as possible in an attempt to insure more uniform application of such requirements.

Applied Technology Council

In an attempt to overcome the obstacles to placing current technology into the hands of practice in as practical a way as possible, the Applied Technology Council (1978) developed the Tentative Provisions for the Development of Seismic Regulations for Buildings. This effort began in the early 1970s and when the result was published in 1978, it represented a very good recommendation for earthquake technology transfer to practice. Excellent work is still going on to substantiate and justify the cost benefits of this technology transfer. However, except for isolated cases on a voluntary basis, none of this technology transfer has actually occurred.

EXISTING CRITICAL FACILITIES

Although earthquake hazard preparedness and mitigation practices have been occurring for new critical facilities during recent years, very little has been done to retrofit existing critical facilities. Most owners are not willing to provide the funds to retrofit such facilities because of the high cost involved. The high costs occur when the retrofit requirements are based on bringing the existing facilities under total compliance of a Level I, II, or III approach.

To avoid the high costs of total retrofit, much can still be done in costing critical facilities to minimize the earthquake risks. For example, anchoring equipment and piping systems in existing facilities is an effective way to conduct earthquake hazard preparedness and mitigation procedure.

TECHNOLOGY TRANSFER COMMITMENTS

Several technology initiatives could be developed for the transfer of earthquake hazard preparedness and mitigation technology to practice. However, to be successful, several commitments must be made.

There must be a commitment by government, industry, and the public to appropriate the funds required for such initiatives. In addition, the public, industrial and government managers, and political representatives must have a reasonable understanding of what the earthquake hazards are in their area of concern. As stated earlier, the problem here is that other than in, say, California, the earthquake hazard is perceived by these groups to be no hazard. The professional groups--architects, engineers, and scientists--must do their utmost to understand the earthquake hazard and develop proper preparedness and mitigation procedures--technology transferred to practice. The political and industrial communities must be committed to support and promote the initiatives.

For critical industrial facilities, today's social and political environment in the United States is very conductive for obtaining the commitment of the public and the political community. To get the same level of commitment for many critical public facilities is, and will be, considerably more difficult and will not occur until the public has some understanding of the earthquake hazard. However, because critical facilities are "critical," there is an ever-increasing commitment by architects, engineers, builders, and owners to transfer today's earthquake technology to practice.

SUMMARY

Although scientists and engineers continue to strive for a better understanding of earthquake hazard preparedness and mitigation, the technological state of the art seems far ahead of that technology, except for highly visible and critical facilities, used in current practice.

An education program involving all phases of training is needed. However, public information and awareness programs should be placed at the top of the list. Until the public has a better understanding of what the earthquake hazards are, progress toward earthquake preparedness and mitigation will be slow unless regulation occurs--and regulators are the public.

REFERENCES

Applied Technology Council. 1978. Tentative Provisions for the Development of Seismic Regulations for Buildings. Washington, D.C.: U.S. Government Printing Office.

International Conference of Building Officials. 1982. Uniform Building Code. Whittier, California: ICBO.

Manrod, W. E., W. J. Hall, and J. E. Beavers. 1981. "Seismic Evaluation Criteria for Existing Industrial Facilities." In Earthquakes and Earthquake Engineering: The Eastern United States, edited by J. E. Beavers. Ann Arbor, Michigan: Ann Arbor Science Publishers.

Structural Engineers Association of California, Seismology Committee. 1975. Recommended Lateral Force Requirements and Commentary. Los Angeles, California: Structural Engineers Association of California.

U. S. Nuclear Regulatory Commission, Office of Nuclear Reactor Regulation. 1975. Standard Review Plan. Washington, D.C.: U.S. Nuclear Regulatory Commission.

DEVELOPMENT OF SEISMIC SAFETY CODES

ROBERT M. DILLON, AIA, M.ASCE, A.AIC

The history of the codes and standards system in the United States is an interesting one; however, of greater importance in this context is what it can tell us about the likely future course of codes and standards development, and the wisdom of working within that system to effect nationwide change in building hazard mitigation practices.

The first model code, the National Building Code, was prepared in 1905 by the National Board of Fire Underwriters, now the American Insurance Association. Concerned about the huge fire losses in American cities and towns, the Board drafted the code with the hope that it would be adopted into law by these cities and towns. Of course, the code dealt with more than fire safety, so it also held the promise of helping reduce the wide variations in the content of building codes--a problem that already was becoming apparent as community after community made a tailored response to perceived public health and safety needs and to public demands for such protection. As early as 1921, a U.S. Senate committee called attention to the high costs of construction that it felt were a consequence of the growing number of municipal codes and the lack of uniformity among those codes. Therefore, the lack of uniformity in building codes, as well as the extent and adequacy of their coverage, is hardly a new concern--just one that is rediscovered from time to time.

In 1927, the first edition of the Uniform Building Code was published by what today is the West Coast headquartered International Conference of Building Officials (ICBO).

In 1939, it was the U.S. National Bureau of Standards that issued a report calling for greater code uniformity. At the same time, it called for the use of nationally recognized building standards in building codes and for the development of means for the acceptance of new materials and methods--the concept of a total system for both regulation and the introduction of technology.

Following World War II (in 1946), the Southern Building Code Congress (SBCC), headquartered in Alabama, was formed and its model code, the Standard Building Code, was first published. Then, in 1950, the Building

---

Mr. Dillon, AIA, M.ASCE, A.AIC, is Executive Vice President of the American Council for Construction Education, Washington, D.C. He presented this paper at the FEMA Earthquake Education Curriculum Workshop held at the National Emergency Training Center, Emmitsburg, Maryland, on June 27-29, 1984.

Officials and Code Administrators (BOCA), which was created in 1915 and is headquartered in Chicago, published its model code, the Basic Building Code.

There now were four model codes--the National Building Code, the Uniform Building Code, the Standard Building Code, and the Basic Building Code. The latter three were and are prepared by building officials with input from the building community.

The National Building Code was last revised in 1976, and in 1980, the National Conference of States on Building Codes and Standards--a body that received its impetus from the National Bureau of Standards--obtained the rights to the code and proposed to develop it as a consensus document in the manner of standards of the American Society for Testing and Materials (ASTM) and the American National Standards Institute (ANSI). Although the concept of a consensus code--as distant from a document produced with building officials as the sole decision-makers--was lauded by many and a degree of progress was made in organizing for the task, the concern for the creation of yet another model code, just as it appeared that the number would be reduced to three, led to the ultimate abandonment of the effort. Today, BOCA has the rights to the national building code name.

The three model code bodies have been quite aggressive and competitive in seeking adoptions of their respective codes. Nevertheless, there still are communities across the country that have no code, particularly communities in rural and newly developing areas, and areas where the code treats only or principally facilities involving public use or occupancy. Also, many of the communities that have adopted one of the model codes have not done so without additions, deletions, and modifications --not infrequently, extensive such deviations. Further, not all codes are up to date by any means, which leads to even further lack of uniformity among various jurisdictions.

The difficulty was compounded by a move in the late 1960s and early 1970s to foster more state rather than local codes--leaving us with a greater mixture of both. Finally, many of our nation's largest cities continue to have their own code. Thus, the dream of uniformity or, what is perhaps a better way of phrasing the need, harmony of provisions is far from a reality.

As early as 1949, the model code organizations, together with several national organizations such as ASTM, the American Insurance Association and the Underwriter's Laboratories, several federal agencies, and the National Research Council of Canada formed the Joint Committee on Building Codes (JCBC) to seek greater code uniformity. In 1959, the JCBC became the Model Codes Standardization Council (MCSC) and the design professions became advisory members. The MCSC was further expanded in 1970 to include construction industry representatives, also as advisory members.

With all of this, progress was still painfully slow on the issue of uniformity and/or harmonization. The nation and building technology were growing rapidly and there still were strong feelings that codes were growing rapidly and there still were strong feelings that codes were a major deterrent to progress and a cause of increased building costs. As a result, Congress created the National Commission on Urban Problems--more popularly known as the Douglas Commission after its chairman, the late Senator Paul Douglas of Illinois. The Douglas Commission made a rather exhaustive study of the codes and standards situation across the United States. Its findings were detailed in a 1969 report, and one of those findings was that an entirely new instrument was needed to address the problem--one that would have the backing of the Congress and the clear mission of bringing about a more rational and responsive building regulatory environment and a nationwide system for facilitating the introduction of new technology. The new instrument was designated the National Institute of Building Sciences (NIBS) by the Commission.

NIBS was a long time coming into being. Not only did the Congress have to be convinced that it was needed--particularly in the form of a private, nongovernmental body authorized by the Congress--but the many diverse and divided public and private interests in the building community itself had to be convinced that NIBS was necessary or at least worth a try.

It took from 1969 until 1974 to be authorized by the Congress, and until mid-1976 for the President of the United States to appoint its first Board of Directors. NIBS received its first of five start-up capital appropriations from the Congress in late 1977 and effectively began operations at the beginning of 1978. And, during these years, the building community and the code bodies were not idle.

In 1972, the three model code bodies formed the Council of American Building Officials (CABO), and CABO in turn created the Board for the Coordination of Model Codes (BCMC) and the National Research Board (NRB) to begin a process for reviewing and recognizing building products and systems. This was not the first effort made by the three model codes to find a way to work together but it has been the only one to have withstood the test of time to date. No doubt the creation of NIBS and the events that surrounded it provided considerable impetus to succeed.

One example of CABO achievements is that it succeeded in creating a one- and two-family dwelling code that, because of its adoption by reference by the three parent model code bodies, has become a nationwide model. It must be pointed out at this juncture, however, that there are few who are familiar with the regulatory scene in this country who would like to see a national model code--or, perhaps it would be more to the point to say that there are a few who would want to see a single national model code that could easily become a national building code by legislative action. The building community has gained a healthy respect for the value of divided authority whether private or public. This is not to say, however, that there is not a desire for greater harmonization of the provisions of both model and actual codes. The same can be said for working to eliminate needless overlap, duplication, and conflict among the standards referenced and available for referencing in codes.

For example, when NIBS recommended the gradual phasing-out of the HUD Minimum Property Standards in favor of an improved CABO One- and Two Family Dwelling Code for that type of housing and any of the three nationally recognized model codes or their equivalent for multifamily housing, a great opportunity was created for achieving increased harmonization of code provisions, at least in this one area of building regulation. Both HUD and CABO have followed through with this recommendation. Further, because the One- and Two-Family Dwelling Code process is more open to building community participation than is the case with the model codes themselves, there has been the opportunity to bring a diversity of building industry talents to bear on at least one area of model code formulation in a manner akin to that of voluntary consensus standards development.

With this gradual movement toward greater harmonization of the model codes, there also has been a gradual movement toward the adoption of these model codes by the nation's states and communities. However, it must be stressed again that adoptions are by no means universal and certainly not adoptions without modification; that most of the major cities continue to have a code that is in many ways unique to that city and reflective of its history and political character, that not all jurisdictions keep their codes up to date, and that appeals and resulting variances make it virtually impossible to be able to say that provisions that even appear to be the same are truly the same at any given point in time.

Therefore, with perhaps as many as 16,000 code issuing jurisdictions in the country, some at the state level, some at the local level and some at both, and with all of these forces at work, there remains a great deal of disharmony among the resulting codes and code provisions in force. It also is the case that many federal agencies have their own construction requirements which add to the lack of harmony. As an aside, the relatively recent action of the Office of Management and Budget in issuing a bulletin that calls upon all federal agencies to rely on voluntary consensus standards to the maximum extent possible is helping the cause of harmonization significantly.

It should be clear at this point that there is no one point of entry for effecting code changes even though input through the model code change process can have a significant effect on the whole of code practice. It always must be remembered that ultimately it is the body having political jurisdiction that must decide what performance level will be sought and what specific requirements will be imposed to achieve that level of performance. This applies to the location, design, construction, and rehabilitation of its own facilities as well as to those under private ownership.

These decisions--that is, whether and how to provide protection against any potential natural or man-made destructive force--are political simply because determining the level of risk and the costs and benefits that are likely to flow from taking any given set of protective measures is so much a matter of judgment. The challenge to the professional community, then, is to provide political decision-makers with ever more reliable information and recommendations to assist them in their awesome task of assessing the risks and establishing the costs and benefits of one decision over the other. This implies, of course, that the professional community will be able to reach a reasonable agreement on what information and recommendations are to be provided. And in this regard, the nation is at a turning point with regard to earthquake technology and its proper application.

Today, there is a major debate concerning how realistic the risk of damaging earthquakes is in much of the eastern two-thirds of the country and an even greater debate on what regulatory provisions can best address those perceived risks.

It is important to recognize that perhaps 80 percent of a building code is made up of reference standards or materials that have come from standards. In the United States, most of these standards are either voluntary consensus standards or industry standards; however, there continues to be reliance on a number of government standards as well, particularly standards promulgated by federal agencies for their own use or for regulatory purposes. Therefore, it is to these criteria and standards that one also must look if building practices are to be changed or influenced. It was not too many years ago that the sources of information and data on seismicity and seismic effects were numerous. Today, these sources are fewer.

At this point it might be best to refer to the June 1978 publication, Tentative Provisions for the Development of Seismic Regulations for Buildings, prepared by the Applied Technology Council of the Structural Engineers Association of California. Popularly known as ATC 3-06, this document has become the focus of proposed changes in seismic standards and codes because of its sponsorship by the National Science Foundation and wide participation by design professionals and representatives of code bodies, governmental agencies at all levels, and the materials industry.

The program effectively began with a workshop on disaster mitigation sponsored by NSF and the National Bureau of Standards (NBS) in Boulder, Colorado, in August 1972. Therefore, the current effort to upgrade disaster mitigation through improved codes and standards is already 12 years old. After ATC 3-06 was published, there was much debate as to the appropriateness of some of the proposed provisions, as to the extent of the proposed application of the provisions, and as to the usefulness of the document itself for the purpose implied in its title--i.e., as provisions for regulatory purposes--because of its mixture of criteria, design procedures, and commentary. Actually, it is clearly stated in the foreword to the document that:

> These provisions are tentative in nature. Their viability for the full range of applications should be established. We recommend this be done prior to their being used for regulatory purposes. Trial designs should be made for representative types of buildings from different areas of the country and detailed comparisons made with costs and hazard levels from existing design regulations.

Concern for a better way to assure consensus among all of the interested parties became a significant issue toward the end of the 1970s; therefore, in 1979, after much discussion among the key building community organizations and federal agencies, the Building Seismic Safety Council (BSSC) was created under the auspices of the aforementioned National Institute of Building Sciences. Today, BSSC operates within NIBS as an independent, voluntary body of some 58 separate organizations. The trial designs recommended by ATC are some 58 separate organizations. The trial designs recommended by ATC are well under way with funding by FEMA--indeed, the second series of these designs is now nearing completion. The next phase of the program will entail getting agreement of the members of the Council on any changes proposed by its committees as a result of previous balloting on the tentative provisions and any changes that seem needed as a result of the trial designs. Publication of the agreed upon seismic safety provisions will follow. It also will include an assessment of the socio-economic impact that could be expected as a consequence of implementing and utilizing the provisions, especially in communities east of the Rocky Mountains that to-date have been largely unconcerned with the seismic safety aspects of building design; a study of the likely impact of the provisions on building regulatory practices; and development of materials and plans for encouraging maximum use of the provisions. Next will come the arduous tasks of seeking changes in the model and actual codes and the appropriate reference standards and educating designers and other building community participants in their use. A good start on this latter task will already have been made because of the involvement of local firms across the country in the trial designs.

In the meantime, the federal government, working through an interagency committee, has been proceeding with applications for federal construction. And, it appears that the National Bureau of Standards, as the Secretariat for an American National Standards Institute standards committee known as A-58.1, already has introduced elements of ATC 3-06 into the 1982 edition of A58.1. For example, the A58.1-1982 seismic zone maps--i.e., maps of the 50 states and Puerto Rico which identify geographic areas of differing earthquake hazard (from 0 to 4)--is derived from maps contained in ATC 3-06.

It appears likely that seismic design procedures will be considerably different if the current work stays on course. At present, the seismic force factors used in ANSI A58.1-1982 are quite similar to those used in the 1982 edition of the <u>Uniform Building Code</u> (UBC) and, because the UBC is the model code most used in the West where earthquakes of significant magnitude are a matter of fairly recent memory, the UBC is typically the most responsive to changes in earthquake engineering technology. The <u>Standard Building Code</u> (SBC) simply references the provisions of A58.1 and must be updated to reference new editions or to introduce other provisions. The lateral force factors in the <u>Basic Building Code</u> (BBC) are specified and are somewhat different from those in the UBC and A58.1-1982. The risk maps in the SBC and BBC are different than those in A59.1-1982. It might be reasoned that all of these standard reference works will come into greater harmony if not actually share the same provisions once the work of BSSC is finished and a reasonable consensus has been achieved on the seismic safety provisions thus recommended. However, even if this does occur, that is not to say that all states and communities will readily adopt the provisions appropriate to their area.

It does seem, however, that with the greater acceptance of decision-making processes such as those employed by the Building Seismic Safety Council and A58.1 (which deals with all dead, live, and environmental loads on buildings and not just earthquakes), the opportunity exists to influence those political bodies that ultimately must make the risk-taking decisions in the areas of public health, safety, and welfare. By bringing together representatives of all vital interests and expertise, the likelihood of finding adequate authority outside the process to challenge the collective judgments of those involved decreases dramatically.

One would think that concern for the potentially devastating effects of earthquakes would engender an eagerness to apply the regulatory provisions offered by technical experts. This simply has not been the case. Regardless of what the technical experts say, the evidence has not been sufficient to convince a lay public that has never experienced an earthquake or is aware that there has not been an earthquake of significance in their area in recorded history, that one of potentially devastating effect could occur tommorrow. And, perhaps more to the point, the lay public may not perceive the odds that such an earthquake will occur in their area during their lifetime to be great enough to justify spending large sums of public and/or private funds to provide or upgrade protection. A finding that the costs of providing adequate protection are minimal or within reason, would go a long way toward allaying these concerns--at least with new construction.

Unfortunately, much the same skepticism can be found with many design professionals and others directly involved with the building community who have never been taught seismic design and who are not required to possess such knowledge to be able to practice or fulfill their other roles in building. Such knowledge simply is of little use in an area where it is not needed for survival in the marketplace.

The answer to the question of whether there are problems that can be addressed by education, therefore, is a resounding yes. There is a big job of public education to be done. There is need to expand the education of building design professionals in seismic design practices. There is need to educate all those who would participate in housing, building, and planning on the state of the art in seismic technology. And, there is need to continue to educate everyone on the importance of achieving a voluntary consensus--one that includes the executive branches of government--on the standards and regulatory provisions that are to be recommended to the appropriate legislative bodies.

It appears that the knowledge and tools will soon be ready for making the next step up on seismic building design, construction, and rehabilitation practice. What is needed is a game plan for bringing those tools into play in an atmosphere of rationality--something that has not been done too well in the building arena in the past. Experience has shown that once a change is perceived as desirable or possible by those directly involved, the federal government has all too frequently agreed to lead the charge--not in a studied manner but in a rush and with an outsized and often frantic program with unreal goals and timetables. I hope I have indicated that the building community and the body politic as it deals with housing, building, and planning issues simply does not respond well to this kind of pressure.

What usually happens after one of these frantic efforts has been tried and fails is that the legislators that voted the resources and the consumers that have been stimulated to great expectations either become convinced that one cannot get from here to there or simply fall back to sleep. The effort is aborted and the goal is farther from achievement than if the program had never been launched--witness Operation Breakthrough and the Building Energy Performance Standards.

A continuation of the cooperative program already under way, with a steady hand on the tiller, will undoubtedly prove in the long run to have been the best course to follow. The old adage "haste makes wastes" certainly should not be forgotten in the case of the earthquake hazard reduction program. Its going well. Let's not break it.

THE EARTHQUAKE AT CHARLESTON IN 1886

G. A. BOLLINGER

At about 9:50 p.m. on August 31, 1886, a large earthquake occurred in Charleston, South Carolina. Its magnitude (M_s) has been estimated at 7.5, its modified Mercalli intensity (MMI) was X, and it was sensibly felt by people over an area of some 2 million square miles. There was extensive damage to the city of Charleston ($5 million in 1886 dollars) and death estimates ranged between 60 and 100 (1886 population density). In Milwaukee, Wisconsin, large buildings were shaken violently, windows were broken, and people fled into the streets. At Brooklyn, New York, buildings were also shaken to the extent that people were frightened; chandeliers rattled. On the sixth floor of a Chicago hotel, plastering was thrown from ceilings and guests were nauseated and fled the hotel in terror. The shock was felt as far away as Boston, Massachusetts; Bermuda; and Cuba.

The 1886 earthquake was certainly the largest known for the southeastern United States and one of the largest historic earthquakes in all of eastern North America. The following will first discuss three important factors that can be derived from consideration of the 1886 shock in the context of the historical seismicity of the region. Each of those factors then will be seen to have one or more important, associated questions. Finally, the physical effects from this large earthquake will be presented in some detail.

IMPORTANT FACTORS AND ASSOCIATED QUESTIONS

The important factors are:

1. The fact that a magnitude 7.5 earthquake occurred in Charleston, South Carolina, demonstrates the presence in the area of a seismogenic structure capable of generating such a shock. In principle, such a structure could occur elsewhere, but at the present time Charleston is the only locale in the Southeast that has its presence confirmed.

2. The earthquake activity in the eastern United States was at a much higher level prior to the turn of the century than it has been subsequently. In addition to the 1886 shock, there was a

---

Dr. Bollinger is a member of the faculty of Virginia Polytechnic Institute and State University. He developed this paper for presentation at the BSSC Meeting in Charleston, South Carolina, on February 13, 1985.

magnitude 5.7 (M_S) earthquake located in western Virginia in 1897 and a series of magnitude 8-8+ earthquakes in southern Missouri during 1811-1812. None of those three states, South Carolina, Virginia or Missouri, or their neighboring states has experienced such large shocks during the twentieth century. Thus, we have documentation that the level of earthquake energy release in the region can change with time.

3. The decrease of earthquake vibrations with increasing distance from an earthquake epicenter in the eastern United States has been shown by numerous studies during the past decade to be very slow, especially with respect to the western part of the country. What this means is that larger areas of structural damage and other earthquake effects can be expected in the East than in the West. The 1886 Charleston earthquake is a good example of those larger than average affected areas.

Some direct questions that follow from the above factors are:

1. Is the Charleston area the only area in the region capable of generating a 7.5 magnitude earthquake? The answer is that it probably is not since it is geologically reasonable for other such seismogenic structures to be present. Also, there are zones of persistent, low-level earthquake activity in the eastern United States. Those zones are candidates for larger shocks in the future.

2. Although the seismicity of the region is currently at a low level, is it going to continue that quiescence or are we in a lull before another period of increased earthquake occurrences?

3. Can the 1886 Charleston earthquake be used as a "type example" of what to expect from a future occurrence of a large earthquake in the region? Yes, but the soil and bedrock geology are certainly different in the Appalachian highlands (Valley and Ridge and Blue Ridge provinces) than in the Atlantic Coastal area that was host to the 1886 shock. These differences as well as the difference in construction practices and materials between 1886 and 1985 need to be taken into account. The differences in type and degree of land utilization also are relevant.

The preceding questions cannot be answered in a deterministic fashion. We just do not have enough data of all kinds--geologic, geophysical, seismological, and engineering--to develop precise answers. What can be done, however, is to approach the problem from a probabilistic point of view. The U.S. Geological Survey (USGS) has been very active in such studies for the past decade. (For summary a overview of the USGS results see the paper by Walter W. Hays.)

DESCRIPTION OF THE EFFECTS FROM THE 1886 EARTHQUAKE

Epicentral Region

At least 80 kilometers of railroad track was seriously damaged and more than 1,300 km^2 of extensive cratering and fissuring occurred as a result of the 1886 earthquake. In Charleston, the railroad-track damage and cratering were virtually absent, but many buildings on both good and poor ("made") ground were destroyed. Specifically, Dutton (1889) reports:

> There was not a building in the city which had wholly escaped injury, and very few had escaped serious injury. The extent of the damage varied greatly, ranging from total demolition down to the loss of chimney tops and the dislodgement of more or less plastering. The number of buildings that were completely demolished and leveled to the ground was not great. But there were several hundred which lost a large portion of their walls. There were very many also which remained standing, but were so badly shattered that public safety required that they be pulled down altogether. There were not, so far as is at present known, a brick or stone building which was not more or less cracked, and in most of them the cracks were a permanent disfigurement and a source of danger or inconvenience. A majority of them, however, were susceptible to repair by means of long bolts and tie-rods.

Also see the reprint of USGS Professional Paper 1028 (1977) that concludes this paper.

At a Distance of 100 Kilometers (60 miles)

Most severely affected at this range from the epicenter of the 1886 shock were coastal locations such as Port Royal and Beaufort to the southwest and Georgetown to the northeast. At Port Royal (MMI of IX), the shock was described by the United Press as "very violent." Houses were moved on their foundations and people were thrown to the ground. At Beaufort (Associated Press) and Georgetown (Dr. M. S. Iseman, M.D.), both with an MMI of VIII, chimneys and chimney tops were thrown down, brick parapets were dislodged, and brick buildings "undulated." Residents fled their houses and remained in the streets and fields all night, many praying. At Beaufort, the Charleston Yearbook described the shock as "very severe," lasting 30 seconds, cracking some large buildings, and causing a 2-foot depression over an area some 60 feet in circumference.

Noncoastal location such as Manning to the north and Orangeburg and Bamberg to the northwest were shaken at a MMI level of VII. All reported damage to brick houses and brick walls and the falling of plaster. The response of the populace at these northerly sites was also one of terror and many camped in the open air overnight.

At a Distance of 200 Kilometers (120 miles)

Reports from Augusta, Georgia, 200 kilometers from the epicenter, deal extensively with the response of the citizenry. The Savannah Morning News of September 2, 1886, gave a September 1 communication from Augusta citing: "...two ladies lie at the point of death from fright," "...an old lady died from fright," and "many ladies fainted and thousands of men were completely unnerved. The citizens remained in the streets all night."

The following paragraphs from Dutton (1889) comment on the pronounced psychological effects at Augusta as well as the structural damages suffered there:

> Thus Augusta, in Georgia, just beyond the 100-mile circle, was shaken with great violence. Many buildings were seriously damaged. At the arsenal two heavy walled buildings used as officer's quarters were so badly shattered that reconstruction was necessary. Many cornices were dislodged and it is estimated that more than a thousand chimneys were overthrown. People residing in brick dwellings refused for several days to enter them and found lodgings in wooden houses or camped in the streets and gardens. So great was the alarm felt that business and society were for two days fully paralyzed as in Charleston. Everyone was in a state of apprehension that the worst was yet to come and the only thing to be thought of was safety. Indeed, among all the large cities of the South, the general tenor of the reports indicates that Augusta stands next to Charleston in respect to the degree of violence of the shocks and the consternation of the people.
>
> Augusta is built in close proximity to the contact of the new and older strata, and starting from that city it will be of interest to follow this line of contact northeastward. In detail the course is more or less sinuous. A few miles to the northeast of Augusta is a little railway station named Langley, where a small tributary of the Savannah River has been dammed to secure water power. The ground in this neighborhood, which is a loose soil thinly covering harder rocks below, was in many places fissured by the earthquake and opened in many cracks, some of which were several inches in width. A number of large cracks passed through the dam, opening passage for the water in the reservoir, which quickly enlarged the fissures. The county below was quickly aflood. The railway track was swept [away], and before warning could be given a passenger train ran into the flood and upon the broken track, where it was wrecked, with some loss of life. In this neighborhood the towns of Bath, Graniteville, and Vaucluse, which stand upon outcrops of crystalline rocks, report shocks of very great severity. Still farther to the northeastward, Batesburg, Leesville, and Lexington give similar reports. Passing beyond Columbia along the same line of contact, we find reports of very violent shocks at Blythwood, Camden, Chesterfield, and Cheeraw.

The Savannah Morning News report also noted that "the most severe damage was done on the Sand Hills in Georgia and in Aiken County, South Carolina." Specific localities mentioned were Langley and Bath, just across the Savannah River from Augusta, some 10 kilometers to the east. At Langley, on the South Carolina Railroad, 24 kilometers (15 miles) from Augusta, Georgia, and 200 kilometers (125 miles) from Charleston, "the earthquake destroyed the mill dam and the water washed away the roadbed. A train dashed into the flood, and the engineer and fireman were drowned. The engine is now 40 feet under water." Dutton (1889) reported: "Houses badly shaken and glasses broken; dams broke loose destroying 1,000 feet of railroad; terrible suffering among the inhabitants." An MMI of X is assigned to the Langley, South Carolina, locale (Bollinger and Stover, 1975).

At a Distance of 400 Kilometers (240 miles)

At an epicentral distance of 400 kilometers, the level of ground-shaking continued to cause panic among the people: "a state of terror and excitement; people left their houses and many stayed in the streets all night (Beaufort, North Carolina); "streets rapidly filled with people, screams of frightened persons could be heard" (Raleigh, North Carolina); "rushed frightened from their houses into the streets; terror-stricken men, women and children, in night dress, crowded the streets in a moment; a number of ladies fainted" (Ashville, North Carolina); and "people rushed into the streets in indescribable confusion, each looking for an explanation from the others; the streets at 10 o'clock are full of people, who fear to return to their houses" (Atlanta, Georgia).

Buildings and household items (mirrors, pictures, lamps, dishes, window glass, etc.) were shaken at a MMI level of VIII or less. Atlanta, in northern Georgia, reported one house (Marrietta Street) "shaken to pieces," all the chimneys fell from the six-story Construction building in the city, window glass was broken, chimneys were knocked down, and dishes and glasses were smashed to pieces. However, Valdosta, to the south-southeast and near the Georgia-Florida border, reported only falling of plaster (MMI VI).

Across the entire state of North Carolina, MMI effects ranged from V to VII. Examples of the highest levels were seen at Beaufort on the coast, Raleigh in central North Carolina and Waynesville in the extreme southwestern part of the state. The seismic waves at those locations caused chimneys to be overthrown or have their tops shaken off, some walls to crack, plastering to be thrown down, buildings to rock, and some floors to break "loose from their supports." Additionally, church bells were rung, clocks stopped, mirrors and pictures were thrown from walls, and lamps were overturned. At Asheville, North Carolina, houses were violently shaken, but no buildings were "shaken down" (MMI of VI). In Black Mountain (20 kilometers to the east of Asheville), the vibrations were accompanied by loud explosive sounds and heavy rumblings, and large masses of rock were dislodged from several steep slopes and rolled into the valleys below.

<u>THROUGHOUT THE COUNTRY</u>

The following pages are a reprint of a study of the effects of the 1886 earthquake throughout the United States that was published in 1977 as part of <u>Studies Related to the Charleston, South Carolina, Earthquake of 1886--A Preliminary Report</u>, USGS Professional Paper 1028, edited by Douglas W. Rankin (Washington, D.C.: U.S. Government Printing Office).

Reinterpretation of the Intensity Data for the 1886 Charleston, South Carolina, Earthquake

By G. A. BOLLINGER

STUDIES RELATED TO THE CHARLESTON, SOUTH CAROLINA, EARTHQUAKE OF 1886—A PRELIMINARY REPORT

GEOLOGICAL SURVEY PROFESSIONAL PAPER 1028-B

CONTENTS

ILLUSTRATIONS

TABLES

REINTERPRETATION OF THE INTENSITY DATA FOR THE 1886 CHARLESTON, SOUTH CAROLINA, EARTHQUAKE

By G. A. BOLLINGER [1]

ABSTRACT

In 1889, C. E. Dutton published all his basic intensity data for the 1886 Charleston, S.C., shock but did not list what intensity values he assigned to each report, nor did he show the distribution of the locations of these data reports on his isoseismal map. The writer and two other seismologists have each independently evaluated Dutton's 1,300 intensity reports (at least two of the three interpreters agreed on intensity values for 90 percent of the reports), and the consensus values were plotted and contoured. One map was prepared on which contours emphasized the broad regional pattern of effects (with results similar to Dutton's); another map was contoured to depict the more localized variations of intensity. As expected, the latter map shows considerable detail in the epicentral region as well as in the far-field. In particular, intensity VI (Modified Mercalli (MM)) effects are noted as far away as central Alabama and the Illinois-Kentucky-Tennessee border area. Dutton's "low intensity zone" in West Virginia appears on both isoseismal maps.

A maximum MM intensity of X for the epicentral region and IX for Charleston appears to be appropriate. Epicentral effects included at least 80 km of railroad track seriously damaged and more than 1,300 km ^{2} of extensive cratering and fissuring. In Charleston, the railroad-track damage and cratering were virtually absent, whereas many, but not most, buildings on both good and poor ground were destroyed.

The epicentral distances to some 800 intensity-observation localities were measured, and the resulting data set was analyzed by least-square regression procedures. The attenuation equation derived is similar to others published for different parts of the eastern half of the United States. The technique of using intensity-distance pairs rather than isoseismal maps has the advantages, however, of completely bypassing the subjective contouring step in the data handling and of being able to specify the particular fractile of the intensity data to be considered.

When one uses intensities in the VI to X range, and their associated epicentral distances for this earthquake, body-wave magnitude estimates of 6.8 (Central United States intensity-velocity data published by Nuttli in 1976) and 7.1 (Western United States intensity-velocity data published by Trifunac and Brady in 1975) are obtained.

[1] Virginia Polytechnic Institute and State University, Blacksburg, Va.

INTRODUCTION

The problems associated with the description of seismic ground motion in a minor seismicity area such as the Southeastern United States are well known. In that region, the largest events took place before instruments were available to record them, so that only qualitative descriptions of their effects exist. During the past few decades, when instruments began to be used, no event having $m_b > 5$ has taken place. Thus we have quantitative data only for small events, and we need to analyze the qualitative data, which are all that is available for larger events.

The purpose of this study is to review thoroughly the data that do exist and to derive as much information as possible concerning regional seismic ground motions. Fortunately, the largest earthquake known to have occurred in the region, the 1886 Charleston, S.C., earthquake, was well studied by Dutton (1889) and his coworkers. An excellent suite of intensity information is thus available for that important earthquake. Secondly, the Worldwide Standard Seismograph Network (WWSSN) stations in the Eastern United States provide data on the radiation from the regional earthquakes that have occurred since installation of the stations. Finally, intensity-particle-velocity relationships as well as attenuation values for various seismic phases have been proposed that can be utilized in an attempt to synthesize the above data types.

The initial part of this paper is concerned with a reevaluation of the intensity data for the 1886 Charleston earthquake, and the second part, with a consideration of the attenuation of intensity as distance from the epicenter increases. (The distance from the epicenter is hereafter called epicentral distance.) The concluding section presents a magnitude estimate for the 1886 shock.

This research was conducted while the author was on study-research leave with the U.S. Geological Survey (U.S.G.S.) in Golden, Colo. Thanks are extended to the members of the Survey, particularly Robin McGuire and David Perkins, for their many helpful discussions. Robin McGuire did the regression analysis presented in this paper, and Carl Stover provided a plot program for the intensity data. Thanks are also due to Rutlage Brazee (National Oceanographic and Atmospheric Administration, N.O.A.A.) and Ruth Simon (U.S.G.S.) for interpreting the sizable amount of intensity data involved in this study.

This research was sponsored in part by the National Science Foundation under grant No. DES 75–14691.

INTENSITY EFFECTS IN THE EPICENTRAL REGION

Dutton assigned an intensity X as the maximum epicentral intensity for the 1886 shock. He used the Rossi-Forel scale; conversion to the Modified Mercalli (MM) scale results in a X–XII value. However, the revised edition (through 1970) of the "Earthquake History of the United States" (U.S. Environmental Data Service, 1973) downgraded Dutton's value to a IX–X (MM). Because of this revision, it is appropriate to compare the scale differences between these two intensity levels (IX and X) with the meizoseismal effects as presented by Dutton.

Ground effects, such as cracks and fissures, and damage to structures increase from the intensity IX to the intensity X level, whereas damage to rails is first listed in the MM scale at the X level. Taken literally, rail damage is indicative of at least intensity-X-level shaking. Richter (1958, p. 138) also listed "Rails bent slightly" for the first time at intensity X. However, he instructed (p. 136) that, "Each effect is named at that level of intensity at which it first appears frequently and characteristically. Each effect may be found less strongly, or in fewer instances, at the next lower grade of intensity; more strongly or more often at the next higher grade." Thus, widespread damage to rails is a firm indicator of intensity-X shaking.

In discussing building damage, it is convenient to use Richter's (1958, p. 136–137) masonry A, B, C, D classification:

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

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

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

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

At the IX level, masonry D structures are destroyed, masonry C structures are heavily damaged, sometimes completely collapsed, and masonry B structures are seriously damaged. Frame structures, if not bolted, are shifted off their foundations and have their frames racked at IX-level shaking, whereas at intensity X most such structures are destroyed. Nearly complete destruction of buildings up to and including those in the masonry B class is a characteristic of the intensity-X level.

Only in Charleston do we have a valid sample of the range of structural damage caused by the 1886 earthquake. It was the only nearby large city, and it contained structural classes up to the range between masonry C and masonry B. Many of the important public buildings, as well as mansions and churches, had thick walls of rough handmade bricks joined with an especially strong oyster-shell-lime mortar. The workmanship was described as excellent, but nowhere in Dutton's (1889) account is reference made to special reinforcement or design to resist lateral forces. Structures outside the Charleston area (as in Summerville, see p. 21) were built on piers, some 1–2 m (3–6 ft) high, thereby making the structures inverted pendulums. Dutton's report for Charleston indicates that although the damage was indeed extensive (see below), most masonry buildings and frame structures were not destroyed. This fact plus Dutton's report on the absence of rail damage and extensive ground effects in the Charleston area indicates an intensity level of IX.

The following quotations from Dutton's report (1889, p. 248–249, 253) contain detailed descriptions of the structural damage in Charleston caused by the earthquake of 1886:

> There was not a building in the city which had wholly escaped injury, and very few had escaped serious injury. The extent of the damage varied greatly, ranging from total demolition down to the loss of chimney tops and the dislodgment of more or less plastering. The number of buildings which were completely demolished and leveled to the ground was not great. But there were several hundred which lost a large portion of their walls. There were very many also which remained standing, but so badly shattered

that public safety required that they should be pulled down altogether. There was not, so far as at present known, a brick or stone building which was not more or less cracked, and in most of them the cracks were a permanent disfigurement and a source of danger or inconvenience. A majority of them however were susceptible of repair by means of long bolts and tie-rods. But though the buildings might be made habitable and safe against any stresses that houses are liable to except fire and earthquake, the cracked walls, warped floors, distorted foundations, and patched plaster and stucco must remain as long as the buildings stand permanent eye-sores and sources of inconveniences. As soon as measures were taken to repair damages the amount of injury disclosed was greater than had at first appeared. Innumerable cracks which had before been unnoticed made their appearance. The bricks had "worked" in the embedding mortar and the mortar was disintegrated. The foundations were found to be badly shaken and their solidity was greatly impaired. Many buildings had suffered horizontal displacement; vertical supports were out of plumb; floors out of level; joints parted in the wood work; beams and joists badly wrenched and in some cases dislodged from their sockets. The wooden buildings in the northern part of the city usually exhibited externally few signs of the shaking they received except the loss of chimney tops. Some of them had been horizontally moved upon their brick foundations, but none were overthrown. Within these houses the injuries were of the same general nature as within those of brick, though upon the whole not quite so severe.

The amount of injury varied much in different sections of the city from causes which seem to be attributable to the varying nature of the ground. The peninsula included between the Cooper and Ashley Rivers, upon which Charleston is built, was originally an irregular tract of comparatively high and dry land, invaded at many points of its boundary by inlets of low swampy ground or salt marsh. These inlets, as the city grew, were gradually filled up so as to be on about the same level as the higher ground. * * * As a general rule, though not without a considerable number of exceptions, the destruction was greater upon made ground than upon the original higher land. [p. 248–249] * * *

In truth, there was no street in Charleston which did not receive injuries more or less similar to those just described. To mention them in detail would be wearisome and to no purpose. The general nature of the destruction may be summed up in comparatively few words. The destruction was not of that sweeping and unmitigated order which has befallen other cities, and in which every structure built of material other than wood has been either leveled completely to the earth in a chaos of broken rubble, beams, tiles, and planking, or left in a condition practically no better. On the contrary, a great majority of houses were left in a condition shattered indeed, but still susceptible of being repaired. Undoubtedly there were very many which, if they alone had suffered, would never have been repaired at all, but would have been torn down and new structures built in their places; for no man likes to occupy a place of business which suffers by contrast with those of his equals. But when a common calamity falls upon all, and by its very magnitude and universality renders it difficult to procure the means of reconstruction, and where thousands suffer much alike, his action will be different. Thus a very large number of buildings were repaired which, if the injuries to them had been exceptional misfortunes instead of part of a common disaster, would have been replaced by new structures. Instances of total demolition were not common.

This is probably due, in some measure, to the stronger and more enduring character of the buildings in comparison with the rubble and adobe work of those cities and villages which are famous chiefly for the calamities which have befallen them. Still the fact remains that the violence of the quaking at Charleston, as indicated by the havoc wrought, was decidely less than that which has brought ruin to other localities. The number of houses which escaped very serious injuries to their walls was rather large; but few are known to have escaped minor damages, such as small cracks, the loss of plastering, and broken chimney tops. [p. 253]

Damage to the three railroad tracks that extend north, northwest, and southwest from Charleston began about 6 km (3.7 mi) northwest of the city and was extensive (fig. 1*A*). More than 80 km (62 mi) of these tracks was affected. The effects listed were: lateral and vertical displacement, formation of S-shaped curves, and the longitudinal movement of hundreds of meters of track. A detailed listing of the effects along the South Carolina Railroad tracks, which run northwest from Charleston directly through the epicentral region, is given in table 1.

Ground cracks from which mud or sand are ejected and in which earthquake fountains or sand craters are formed begin on a small scale at intensity VIII, become notable at IX, and are large and spectacular phenomena at X (Richter, 1958, p. 139). The formation of sand craterlets and the ejection of sand were certainly widespread in the epicentral area of the 1886 earthquake. Many acres of ground were overflowed with sand, and craterlets as much as 6.4 m (21 ft) across were formed. Dutton (1889, p. 281) wrote: "Indeed, the fissuring of the ground within certain limits may be stated to have been universal, while the extravasation of water was confined to certain belts. The area within which these fissures may be said to have been a conspicuous and almost universal phenomenon may be roughly estimated at nearly 600 square miles [1,550 sq. km]." By comparison, the elliptical intensity-X contour suggested by the present study encloses an area of approximately 1,300 km^2.

The distribution of craterlets taken from Dutton (1889, pl. 28) is also shown in figure 1*A*. In a few localities, the water from the craters probably spouted to heights of 4.5–6 m (15–20 ft), as indicated by sand and mud on the limbs and foliage of trees overhanging the craters.

Other ground effects indicating the intensity-X level are fissures as much as a meter wide running parallel to canal and streambanks, and changes of

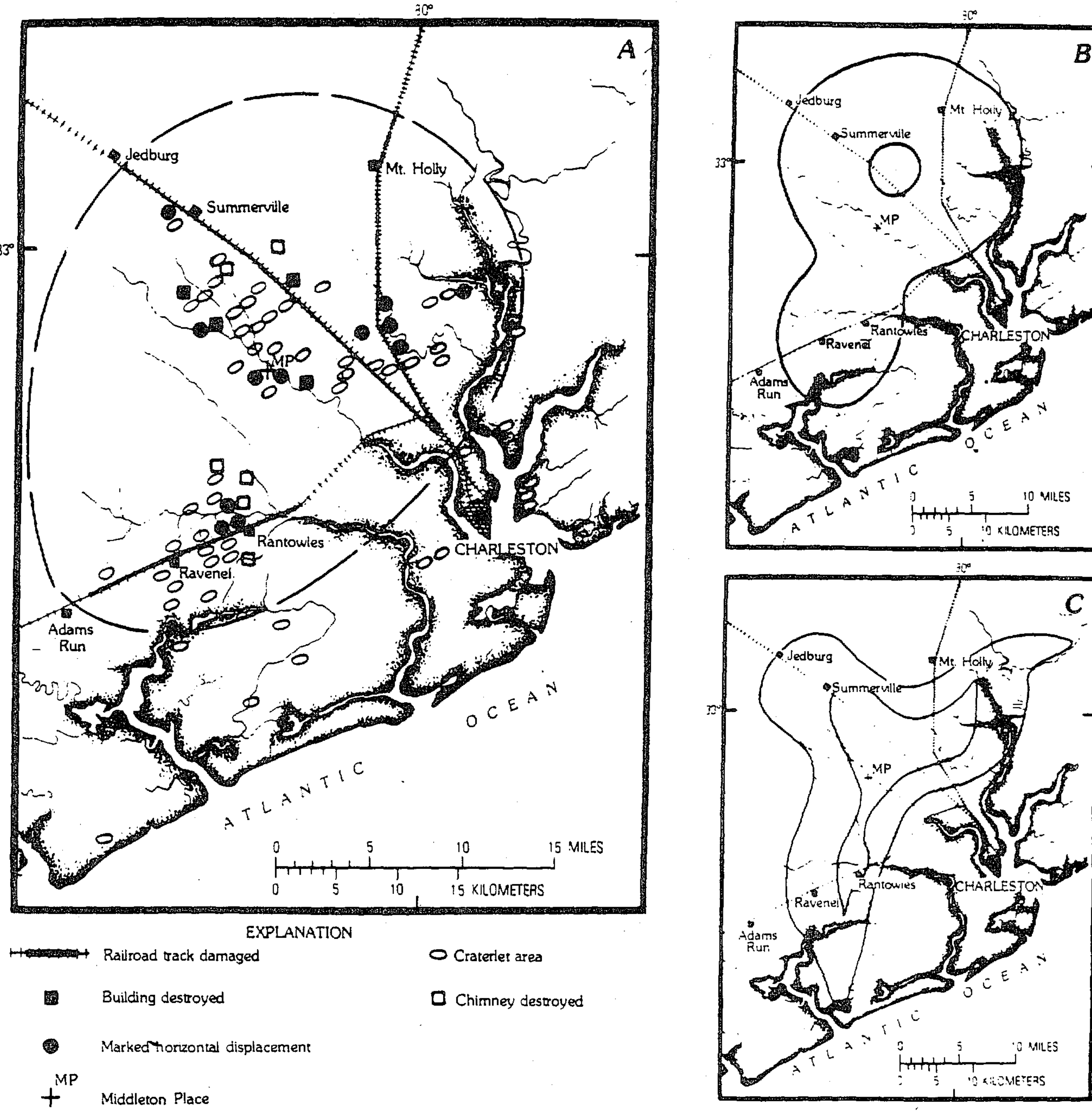


FIGURE 1.—Epicentral area maps for the 1886 Charleston, S.C., earthquake. *A*, This study. Dashed contour encloses intensity-X effects. *B*, Dutton's map and *C*, Sloan's map (modified from Dutton, 1889, pls. 26 and 27, respectively) show contours enclosing the highest intensity zone, although neither Dutton nor Sloan labeled his contours. Base map modified from Dutton (1889). Rivers flowing past the Charleston peninsula are the Ashley River flowing from the northwest and the Cooper River flowing from the north.

the water level in wells (Wood and Neuman, 1931). Dutton (1889, p. 298) reported that a series of wide cracks opened parallel to the Ashley River (see caption, fig. 1) and that the sliding of the bank riverward uprooted several large trees, which fell over into the water. His plate 23 shows a crack along the bank of the Ashley River about a meter wide and some tens of meters long across the field of view of the photograph.

In a belt of craterlets (trend N. 80° E., length ~5 km) about 10 km (6.2 mi) southeast of Summerville, Sloan reported (Dutton, 1889, p. 297) that

TABLE 1.—*Variation of intensity effects along the South Carolina Railroad*

[Based on Dutton, 1889, p. 282–287. Refer to fig. 1 for locations mentioned]

Distance from Charleston (km)	Distance from Charleston (mi)	Effects
<5.8	<3.66	Occasional cracks in ground; no marked disturbance of track or roadbed.
5.8	3.66	Rails notably bent and joints between rail opened.
5.8–8	3.66–5	Ground cracks and small craterlets.
8	5	Fishplates torn from fastenings by shearing of the bolts; joints between rails opened to 17.5 cm (7 in.).
9.6	6	Joints opened, roadbed permanently depressed 15 cm (6 in.).
14.4	9	Lateral displacements of the track more frequent and greater in amount; serious flexure in the track that caused a train to derail; more and larger craterlets.
16	10	Craterlets seemed to be greater in size (as much as 6.4 m (21 ft) across) and number; many acres overflowed with sand.
16–17.6	10–11	Maximum distortions and dislocations of the track; often displaced laterally and sometimes alternately depressed and elevated; occasional severe lateral flexures of double curvature and great amount; many hundreds of meters of track shoved bodily to the southeast; track parted longitudinally, leaving gaps of 17.5 cm (7 in.) between rail ends; 46 cm (18 in.) depression or sink in roadbed over a 18-m (60-ft) length.
17.6–24	11–15	Many lateral deflections of the rails.
24–25.6	15–16	Epicentral area—a few wooden sheds with brick chimneys completely collapsed; railroad alinement distorted by flexures; elevations and depressions, some of considerable amount, also produced.
29–30.6	18.5–19	Flexures in track, one in an 8.8-m (29-ft) section of single rails had an S-shape and more than 30 cm (12 in.) of distortion.
32	≈20	"... a still more complex flexure was found. Beneath it was a culvert which had been strained to the northwest and broken" (p. 286); a long stretch of the roadbed and track distorted by many sinuous flexures of small amplitude.
33.9	21	Tracks distorted laterally and vertically for a considerable distance.
34.9	21.66	At Summerville—many flexures, one of which was a sharp S-shape; broken culvert under tracks in a sharp double curvature.
35.4–44.3	22–27.5	Disturbance to track and roadbed diminishes rapidly.
44.3	27.5	At Jedburg—a severe buckling of the track.

wells had been cracked in vertical planes from top to bottom, and that the wells had been almost universally disturbed, many overflowing and subsequently subsiding, others filling with sand or becoming muddy.

In Summerville, whose population at that time was about 2,000, the structures were supported on wood posts or brick piers 1–2 m high and, though especially susceptible to horizontal motions, the great majority did not fall. Rather, the posts and piers were driven into the soil so that many houses settled in an inclined position or were displaced as much as 5 cm. Chimneys, which were constructed to be independent of the houses, generally had the part above the roofline dislodged and thrown to the ground. Below the roofs, many chimneys were crushed at their bases, both bricks and mortar being disintegrated and shattered, allowing the whole column to sink down through the floors. This absence of overturning in piered structures plus the nature of the damage to chimneys was interpreted by Dutton as evidence for predominantly vertical ground motions.

The preceding discussion indicates an intensity-X level of shaking in the epicentral area. Figure 1*A* depicts the approximate extent of this region along with the locations of rail damage, craterlet areas, building damage, and areas of marked horizontal displacements. Dutton and his coworkers did not map the regions of pronounced vertical-motion effects, but they did emphasize the importance of these effects in the epicentral region. Also shown in figure 1 (*B* and *C*) is the extent of the highest intensity zone, as given by Dutton and by Sloan. Because of the sparsely settled and swampy nature of the region, the meizoseismal area cannot be defined accurately.

INTENSITY EFFECTS THROUGHOUT THE COUNTRY

Dutton (1889) published all his intensity reports, some 1,337, but he did not list the intensity values that he assigned to each report, nor did he show the location of the data points on his isoseismal map. By using the basic data at hand, a reevaluation was attempted to present another interpretation of the data (in the MM scale) and to determine whether additional information could be extracted concerning this important earthquake. The writer and two other seismologists (Rutlage Brazee, N.O.A.A., and Ruth Simon, U.S.G.S.) each independently evaluated Dutton's intensity data listing according to the MM scale. For the resulting 1,047 usable reports, ranging from MM level I to X, at least two of the three interpreters agreed on intensity values for 90 percent of the reports. As would be expected, most of the disagreement was found at the lower intensity levels (II–V). A full listing of the three independent intensity assignments for each location was made by Bollinger and Stover (1976).

The consensus values, or the average intensity values, in the 10 percent of the reports where all three interpreters disagreed were plotted at two different map scales and contoured (figs. 2–5). When multiple reports were involved, for example, those from cities, the highest of the intensity values obtained was assigned as the value for that location.

The greatest number of reports (178) for an individual State was from South Carolina. Figure 2 presents the writer's interpretation of these data. Even

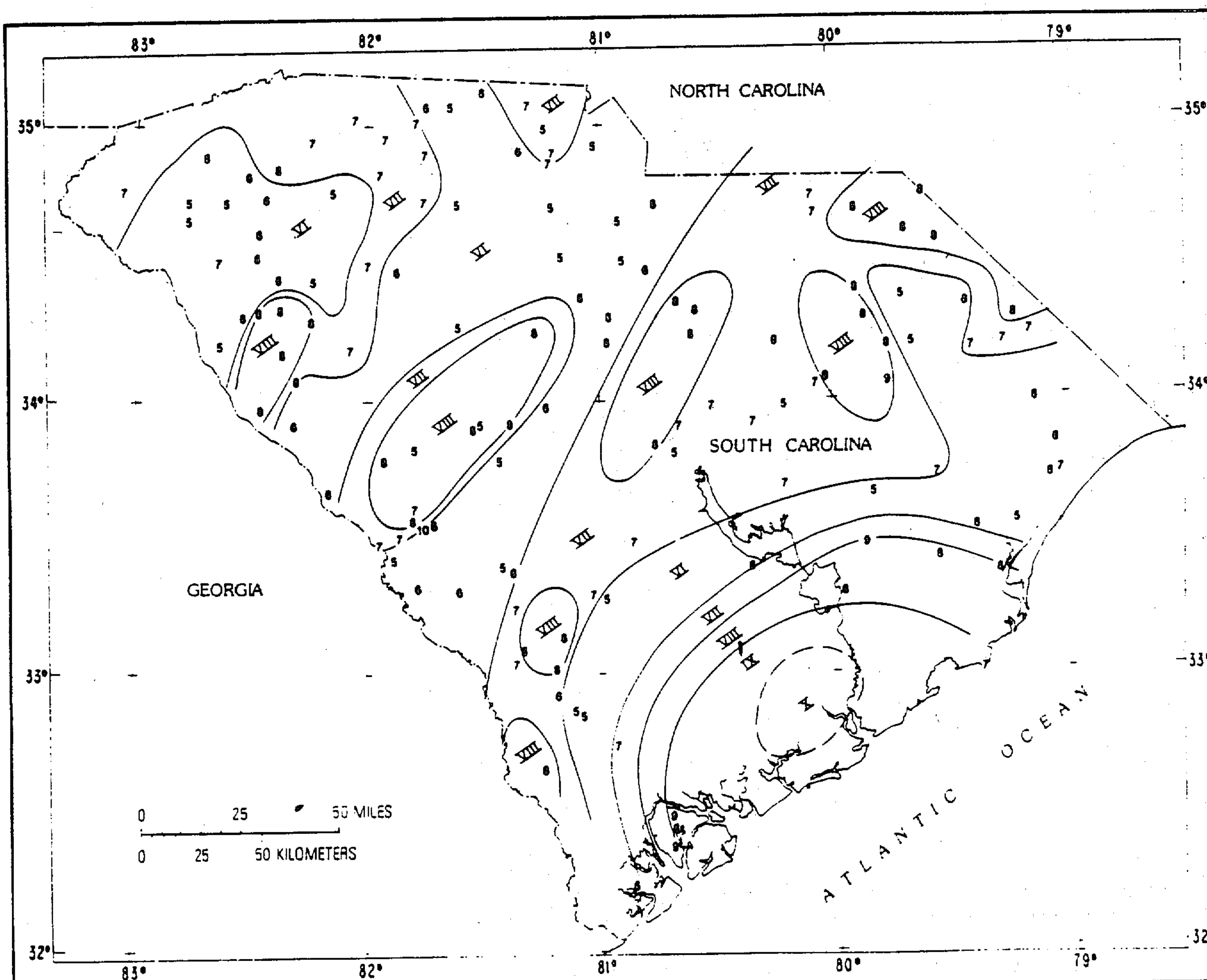


FIGURE 2.—Isoseismal map showing the State of South Carolina for the 1886 Charleston earthquake. Intensity observations are indicated by Arabic numerals, and the contoured levels are shown by Roman numerals.

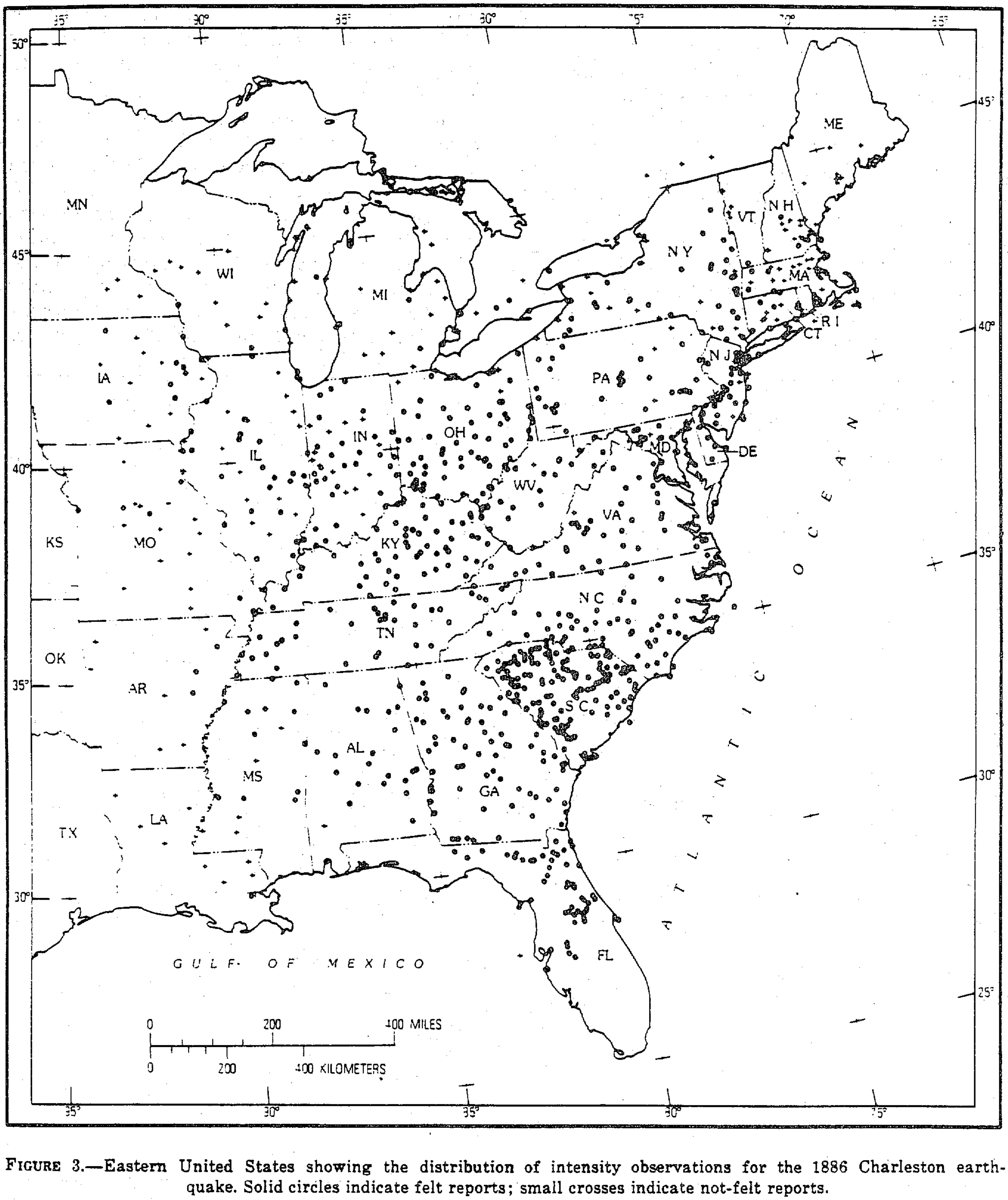


FIGURE 3.—Eastern United States showing the distribution of intensity observations for the 1886 Charleston earthquake. Solid circles indicate felt reports; small crosses indicate not-felt reports.

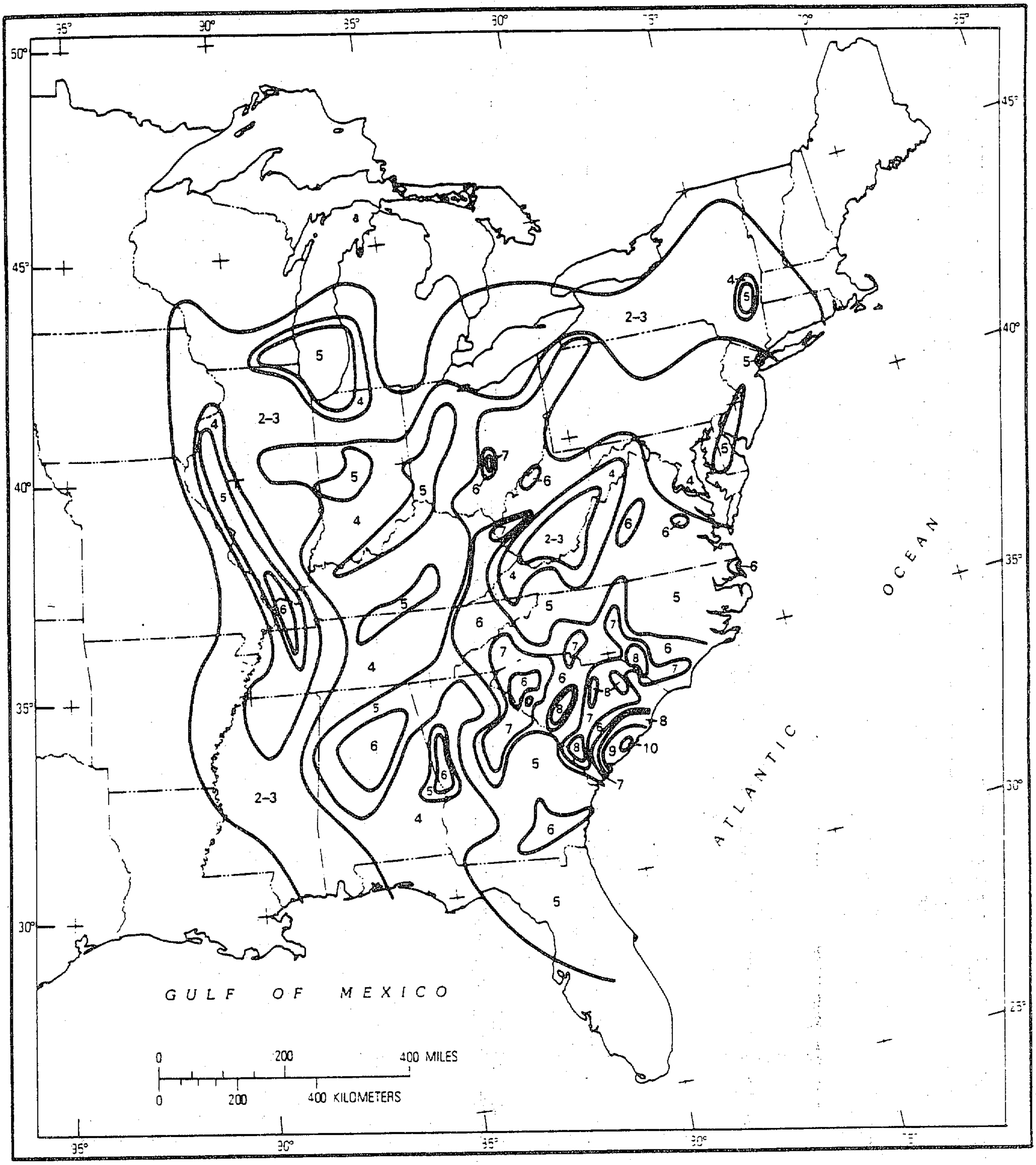


FIGURE 4.—Isoseismal map of the Eastern United States contoured to show the more localized variations in the reported intensities for the 1886 Charleston earthquake. Contoured intensity levels are shown by Arabic numerals.

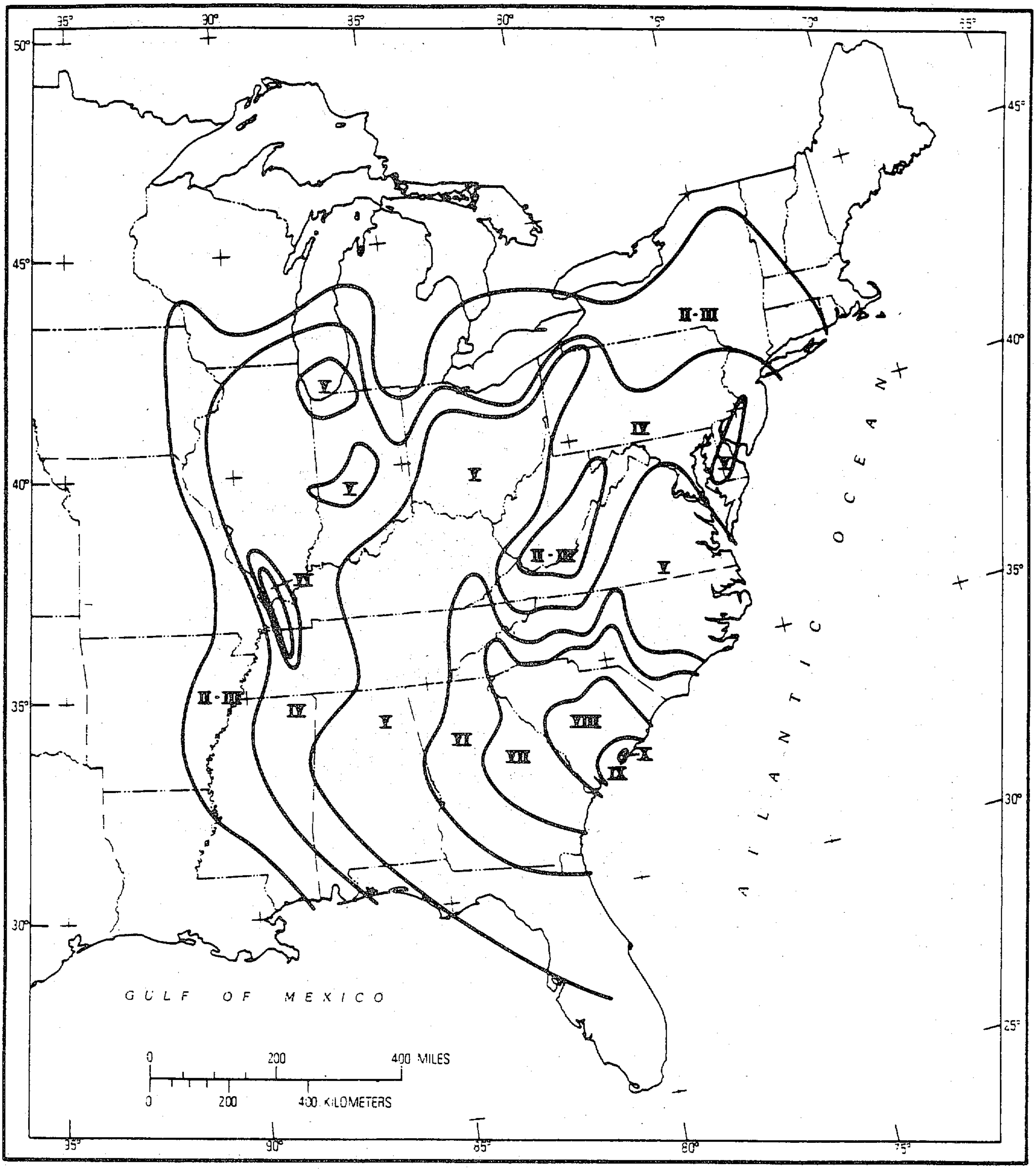


Figure 5.—Isoseismal map of the Eastern United States contoured to show the broad regional patterns of the reported intensities for the 1886 Charleston earthquake. Contoured intensity levels are shown in Roman numerals.

in contouring the mode of the intensity values, as was done here, intensity effects vary considerably with epicentral distance within the State. In particular, two intensity-VI zones are shown that trend northeastward across the State and separate areas of intensity-VIII effects. Although some of this variation may be due to incomplete reporting and (or) population density, it seems more likely that the local effects of surficial geology, soils, and water-table level are being seen. Interpreted literally, a very complex behavior of intensity is seen in the epicentral region.

The intensity data base and interpretive, isoseismal lines throughout the Eastern United States are shown in figures 3–5. In figure 4, the data are contoured to emphasize local variations, whereas figure 5 depicts the broad regional pattern of effects. Richter (1958, p. 142–145), in discussing the problem of how to allow for or represent the effect of ground in drawing isoseismal lines, suggested that two isoseismal maps might be prepared. One map would show the actual observed intensities; the other map would show intensities inferred for typical or average ground. The procedure followed here was to contour the mode of the intensity values (figs. 2 and 4) so as to portray the observed intensities in a manner that emphasizes local variations. Those isoseismal lines were then subjectively smoothed to produce a second isoseismal map showing the regional pattern of effects (fig. 5). The two maps that result from this procedure seem to the writer to represent reasonable extremes in the interpretation of intensity data. The subjectivity always involved in the contouring of intensity data is well known to workers concerned with such efforts. The purpose of the dual presentation here is to emphasize this subjectivity and to point out that, depending on the application, one form may be more useful than the other. Both local and regional contouring interpretations are to be found in the literature for U.S. earthquakes.

Figures 4 and 5 show that a rather complex isoseismal pattern, including Dutton's low-intensity zone (epicentral distance $=\Delta\cong 550$ km (341 mi)) in West Virginia, was present outside South Carolina. Intensity-VIII effects were observed at distances of 250 km (150 mi) and intensity-VI effects were observed 1,000 km (620 mi) from Charleston. Individual reports, given below, are all paraphrased from Dutton (1889). They note what took place in areas affected by intensity VI (MM) or higher at epicentral distances greater than about 600 km (372 mi). Some of these reports were ignored in the contouring shown in figure 4.

Intensity VI–VIII in Virginia ($\Delta\cong 600$ km (372 mi)):

Richmond (VIII)—Western part of the city: bricks shaken from houses, plaster and chimneys thrown down, entire population in streets, people thrown from their feet; in other parts of the city, earthquake not generally felt on ground floors, but upper floors considerably shaken.

Charlottesville (VII)—Report that several chimneys were overthrown.

Ashcake (VI)—Piano and beds moved 15 cm (6 in.); everything loose moved.

Danville (VI)—Bricks fell from chimneys, walls cracked, loose objects thrown down, a chandelier swung for 8 minutes after shocks.

Lynchburg (VI)—Bricks thrown from chimneys, walls cracked in several houses.

Intensity VII in eastern Kentucky and western West Virginia ($\Delta\cong 650$ km (404 mi)):

Ashland, Ky. (VIII)—Town fearfully shaken, several houses thrown down, three or four persons injured.

Charleston, W. Va.—"A number of chimneys toppled over" (p. 522).

Mouth of Pigeon, W. Va.—Chimneys toppled off to level of roofs, lamps broken, a house swayed violently.

Intensity VI in central Alabama ($\Delta\cong 700$ km (434 mi)):

Clanton (VII)—Water level rose in wells, some went dry and others flowed freely; plastering ruined.

Cullman—House wall cracked, lamp on table thrown over.

Gadsden—People ran from houses.

Tuscaloosa—Walls cracked, chimneys rocked, blinds shaken off, screaming women and children left houses.

Intensity VII in central Ohio ($\Delta\cong 800$ km (496 mi)):

Lancaster—Several chimneys toppled over, decorations shaken down, hundreds rushed to the streets.

Logan—Bricks knocked from chimney tops, houses shaken and rocked.

Intensity VI in southeastern Indiana and northern Kentucky ($\Delta\cong 800$ km (496 mi)):

Rising Sun, Ind.—Plaster dislodged, ornaments thrown down, glass broken.

Stanford, Ky.—Some plaster thrown down, hanging lamps swung 15 cm (6 in.).

Intensity VI in southern Illinois, eastern Tennessee, and Kentucky (Δ≅950 km (590 mi)):

Cairo, Ill.—Broken windows, "houses settled considerably" (p. 430) in one section, ceiling cracked in post office.

Murphysboro, Ill.—Brick walls shook, firebell rang for a minute, suspended objects swung.

Milan, Tenn.—Cracked plaster, people sitting in chairs knocked over.

Clinton, Ky.—Some bricks fell from chimneys.

Intensity VI in central and western Indiana (Δ≅1,000 km (620 mi)):

Indianapolis—Earthquake not felt on ground floors; part of a cornice displaced on one hotel, people prevented from writing at desks, clock in court house tower stopped, a lamp thrown from a mantle.

Terre Haute—Plaster dislodged, sleepers awakened; in Opera House, earthquake felt by a few on the ground floor, but swaying caused a panic in the upper galleries.

Madison—Several walls cracked, chandeliers swung.

Intensity VI in northern Illinois and Indiana (Δ≅1,200 km (744 mi)):

Chicago, Ill.—Plaster shaken from walls and ceilings in one building above the fourth floor; barometer at Signal Office "stood 0.01 inches higher than before the shock for eight minutes" (p. 432); earthquake not felt in some parts of City Hall, especially noticeable in upper stories of tall buildings, not felt on streets and lower floors.

Valparaiso, Ind.—Plaster thrown down in hotel, chandeliers swung, windows cracked, pictures thrown from walls.

The preceding reports indicate that structural damage extended to epicentral distances of several hundred kilometers and that apparent long-period effects were present at distances exceeding 1,000 km (620 mi). Persons also frequently reported nausea at these greater distances.

Dutton apparently contoured his isoseismal map in a generalized manner, which is an entirely valid procedure. The rationale in that approach is to depict not the more local variations, as was presented in the above discussion, but rather the regional pattern of effects from the event. Figure 5 is the writer's attempt at that type of interpretation, and the resulting map is very similar to Dutton's.

ATTENUATION OF INTENSITY WITH EPICENTRAL DISTANCE

The decrease of intensity with epicentral distance is influenced by such a multiplicity of factors that it is particularly difficult to measure. The initial task in any attenuation study is to specify the distance (or distance range) associated with a given intensity level. Common selections are: minimum, maximum, or average isoseismal contour distances or the radius of an equivalent area circle. In all these approaches, the original individual intensities are not considered; rather, isoseismal maps are used. Perhaps a better, but more laborious, procedure has been suggested by Perkins (oral commun., 1975), wherein the intensity distribution of observations is plotted for specific distance intervals. In this manner, all the basic data are presented to the reader without interpretation by contouring. He is then in a position to know exactly how the data base is handled and thereby to judge more effectively the results that follow. Once the intensity-distance data are cast in this format, they are then also available for use in different applications.

The epicentral distances to some 800 different locations affected by the 1886 shock were measured and are listed in table 2. For these measurements, the center of the intensity X (fig. 1) area was assumed to be the epicenter. Figure 6 presents the resulting intensity distributions as functions of epicentral distance. The complexity present in the isoseismal maps (figs. 4 and 5) is now transformed to specific distances, and the difficulty of assigning a single distance or distance interval to a given intensity level is clearly shown. The approach followed here was to perform a regression analysis on the intensity-distance data set, using an equation of the form,

TABLE 2.—*Number of intensity observations as a function of epicentral distance intervals for the 1886 Charleston, S. C., earthquake*

Epicentral distance (km)	IX	VIII	VII	VI	V	IV	II–III	Number of observations
50– 99	3	4	3	3	3	---	---	16
100– 199	2	18	18	17	18	1	---	74
200– 299	-	9	22	25	30	5	---	91
300– 399	-	3	16	12	31	8	---	70
400– 499	-	2	3	10	26	19	12	72
500– 599	-	1	3	11	13	19	7	54
600– 699	-	1	3	3	14	33	11	65
700– 799	-	--	3	4	22	16	22	67
800– 899	-	--	1	2	29	20	20	72
900– 999	-	--	--	3	18	17	30	68
1,000–1,249	-	--	--	4	24	19	48	95
1,250–1,499	-	--	--	--	6	6	20	32
1,500–1,749	-	--	--	--	---	1	3	4
Totals	5	38	72	94	234	164	173	780

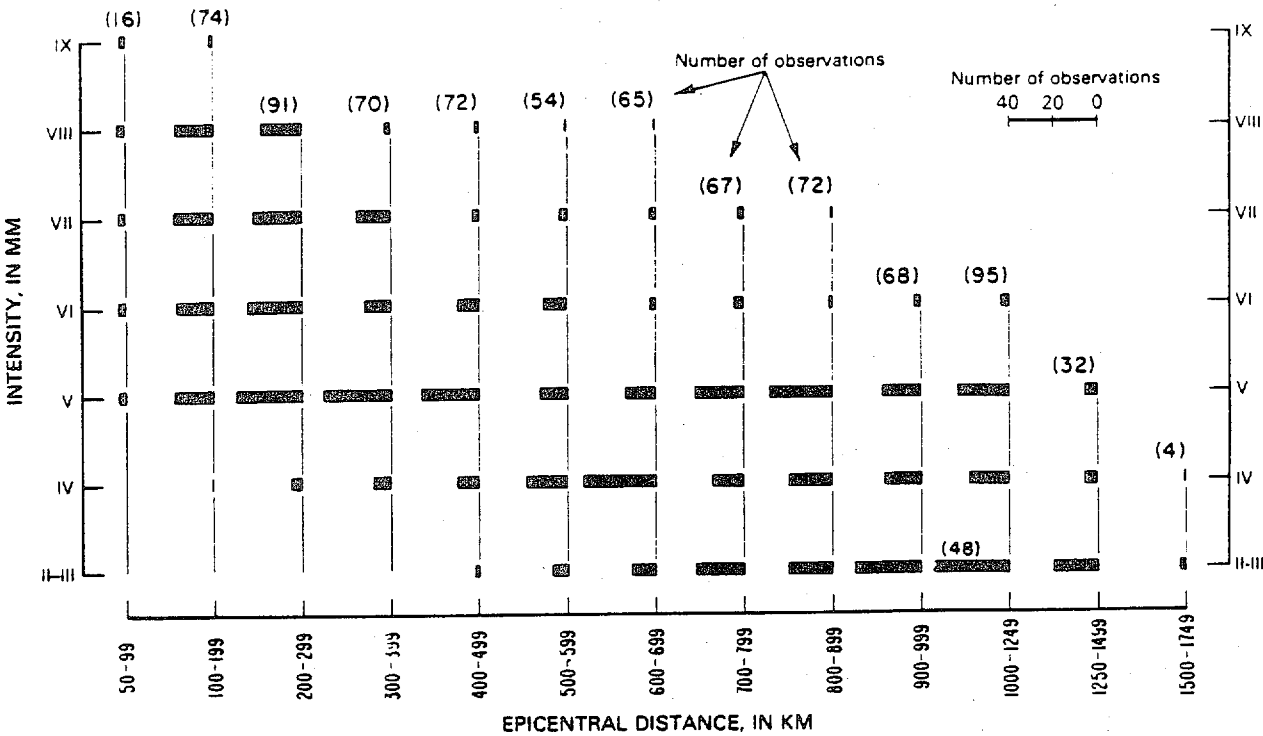


FIGURE 6.—Distribution of intensity (Modified Mercalli, MM) as a function of epicentral distance (km) for the 1886 Charleston earthquake. Intensity distribution is shown for specific distance intervals.

$$I = I_0 + a + b\Delta + c \log \Delta,$$

where a, b, c are constants, Δ is the epicentral distance in kilometers, I_0 is the epicentral intensity, and I is the intensity at distance Δ. This equation form was selected because it has been found useful by other investigators (for example, Gupta and Nuttli, 1976). The resulting fit for the median, or 50-percent fractile, was,

$$I = I_0 + 2.87 - 0.00052\Delta - 2.88 \log \Delta.$$

The standard deviation, σ_I, between the observed and predicted intensities, is 1.2 intensity units for these data. For the 75-percent fractile, the a constant is 3.68; for the 90-percent fractile, the a constant is 4.39. The b term is very small and could perhaps be deleted, as it results in only half an intensity unit at 1,000 km. The minimum epicentral distance at which the equation is valid is probably 10–20 km. The intensity-distance pairs extend to within only 50 km of the center of the epicentral region, but that region (fig. 1) has a diameter of approximately 20 km.

The curves for the 50-, 75-, and 90-percent fractiles are shown in figures 7 and 8 along with other published intensity attenuation curves for the Central and Eastern United States. Isoseismal maps

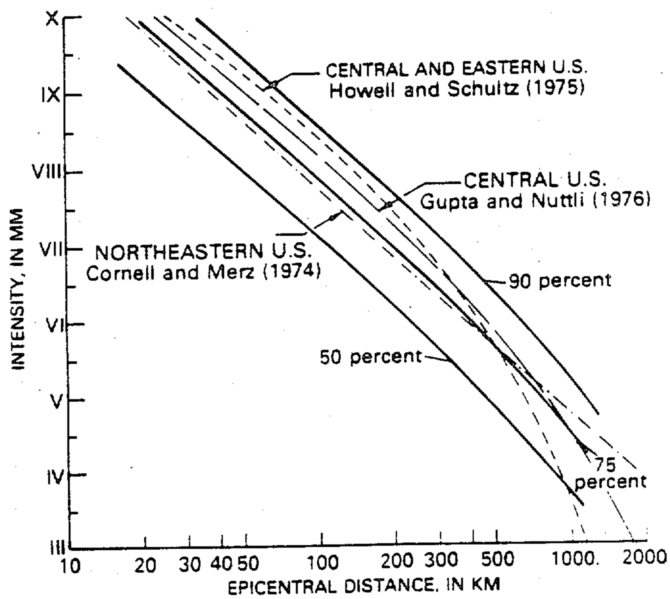


FIGURE 7.—Attenuation of intensity (MM) with epicentral distance (km) for various fractiles of intensity at given distance intervals for the 1886 Charleston earthquake (heavy solid curves). Attenuation functions by Howell and Schultz (1975), Gupta and Nuttli (1976), and Cornell and Merz (1974) are shown by light dashed curves.

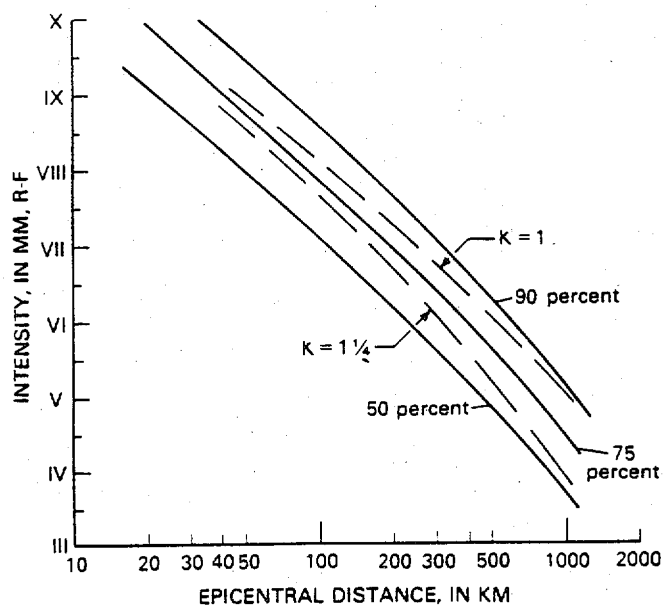


FIGURE 8.—Attenuation of intensity (MM) with epicentral distance (km) for various fractiles of intensity at given distance intervals for the Charleston earthquake (solid curves). Evernden's attenuation curves (1975) (Rossi-Forel intensity scale; L=10 km, C=25 km, k=1 and 1¼) are shown by dashed curves for I_0=X.

were utilized to develop these latter curves, and the general agreement between the entire suite of curves is remarkable. A direct comparison between curves, which may not be valid because of different data sets and different regions, would suggest that the Howell and Schultz (1975) curve is at about the 85-percent fractile, the Gupta and Nuttli (1976) curve is at the 80-percent fractile, and the Cornell and Merz (1974) curve is at the 70-percent fractile. At the intensity-VI level and higher, note that there is less than one intensity-unit difference among the Central United States, Central and Eastern United States, and Northeastern United States curves and the 75- and 90-percent fractile curves of this study.

Evernden's (1975) curves (fig. 8) for his k=1 and k=1¼ factors lie between the 50- and 90-percent fractile curves of this study. Evernden used k factors to describe the different patterns of intensity decay with distance in the United States. A value of k=1¼ was found for the Gulf and Atlantic Coastal Plains and the Mississippi Embayment and a k=1 for the remainder of the Eastern United States. Evernden prefers to work with the Rossi-Forel (R–F) intensity scale. The difference between the R–F and MM scales is generally about half an intensity unit, and conversion to R–F values would essentially result in translating the fractile curves of this study upward by that amount. This would put the 75-percent fractile curve in near superposition with Evernden's k=1 curve. Such a result is perhaps not surprising because approximately two-thirds of the felt area from the 1886 shock is in Evernden's k=1 region, and isoseismal lines are often drawn to enclose most of the values at a given intensity level. Although differences in intensity attenuation may exist between various parts of the Eastern United States, it would appear from this study that the dispersion of the data (σ_I=1.2) could preclude its precise definition. If, indeed, significant differences do exist between the various regions, then the curves given here would apply to large shocks in the Coastal Plain province of the Southeastern United States.

The advantages of the method presented herein are that it allows a prior selection of the fractile of the intensity observations to be considered and that it eliminates one subjective step, the contouring interpretation of the intensity data. Furthermore, the dispersion of the intensity values can be calculated.

Neumann (1954) also presented intensity-versus-distance data in a manner similar to that described above. However, Neumann did not consider the intensity distribution for specific distance intervals as was done herein, but rather plotted the distance distribution for each intensity level. To illustrate the difference in the two approaches, the 1886 earthquake data were cast in Neumann's format (fig. 9).

MAGNITUDE ESTIMATE

Nuttli (1973), in arriving at magnitude estimates for the major shocks in the 1811–1812 Mississippi Valley earthquake sequence, developed a technique for correlating isoseismal maps and instrumental ground-motion data. Later, he (1976) presented specific amplitude-period $(A/T)_z$ values for MM intensities IV through X for the 3-second Rayleigh wave. Basically, Nuttli's technique consists of:

(1) Determination of a relation between $(A/T)_z$ and intensity from instrumental data and isoseismal maps,

(2) Use of the $(A/T)_z$ level at 10-km epicentral distance derived from the m_b value for the largest well-recorded earthquake in the region. That level will serve as a reference level from which to scale other m_b magnitudes,

(3) For the historical event of interest, assign epicentral distances (Δ) to each intensity level from the isoseismal map for the event. Convert from intensity to $(A/T)_z$, according to the relationship of (1) above, then

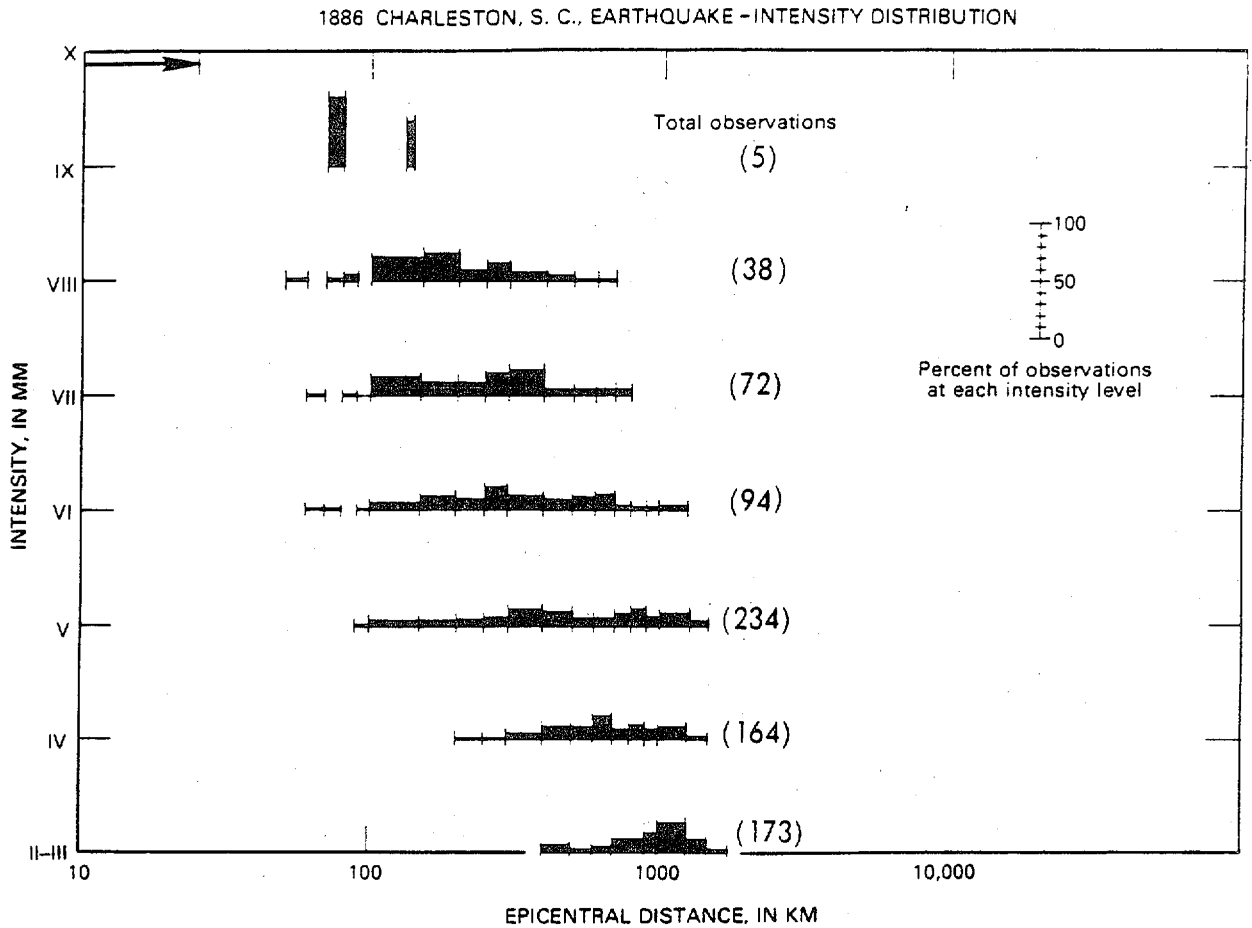


FIGURE 9.—Distribution of epicentral distances (km) for given intensity (MM) levels of the 1886 Charleston earthquake.

(4) Plot $(A/T)_z$ versus Δ and fit with a theoretical attenuation curve. Next, scale from (2) above to determine the Δm_b between the historical shock and the reference earthquake.

In the $(A\ T)_z$ versus intensity of (1) and the curve fitting of (4), Nuttli found that surface waves having periods of about 3 seconds (s) were implied. He justified the use of m_b (determined from waves having periods of about 1 s) by assuming that the corner periods of the source spectra of the earthquakes involved are no less than 3 s. This implies a constant proportion between the 1- and 3-s energy in the source spectra. Nuttli used m_b rather than M_s because he felt that, for his reference earthquake, the former parameter was the more accurately determined.

If we apply Nuttli's technique to the 1886 earthquake and use the distances associated with the 90-percent fractile intensity-distance relationship, the resulting m_b estimate is 6.8 (fig. 10). Nuttli (1976) obtained a value of 6.5 when he used Dutton's isoseismal map and converted from the Rossi-Forel scale to the MM scale. If the Trifunac and Brady (1975) peak velocity versus MM intensity relationship, derived from Western United States data, is taken with the 90-percent fractile distances, then the m_b estimate is 7.1 (fig. 10). Because the 90-percent fractile curve is the most conservative, it results in the largest intensity estimate at a given distance. The magnitude estimates in this study would be upper-bound values.

My magnitude estimates, as well as those of Nuttli, are based primarily on three previously mentioned factors: intensity-distance relations, intensity-particle velocity relations, and reference magnitude level (or, equivalently, the reference earthquake, which in this instance is the November 9, 1968, Illinois earthquake with $m_b = 5.5$). In the Central and Eastern United States, the data base for the later two factors is very small. It is in this context that the magnitude estimates should be considered.

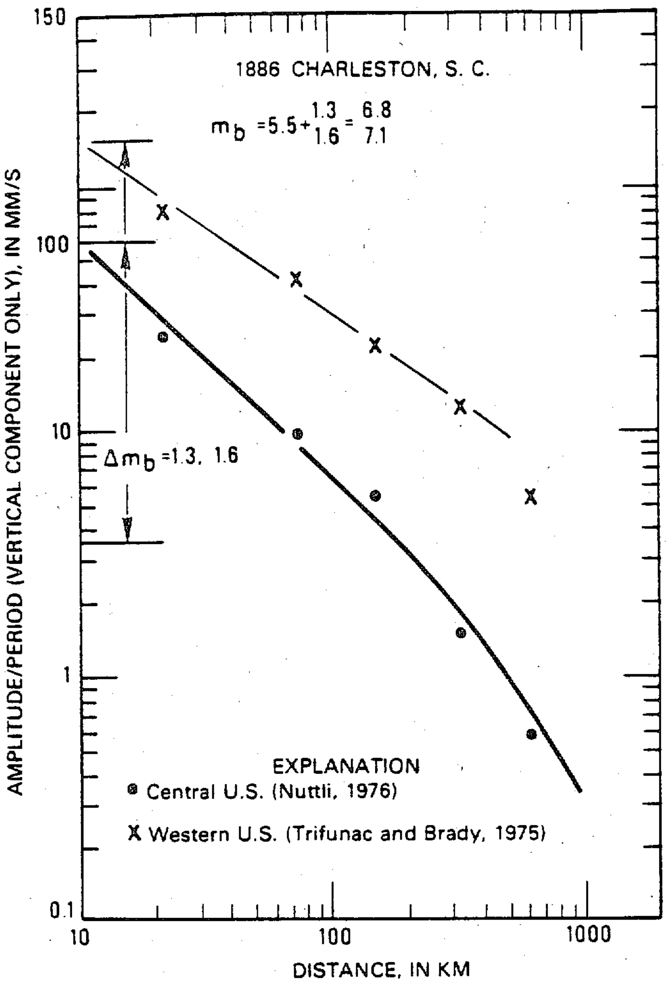


FIGURE 10.—Body wave magnitude (m_b) estimates for the 1886 Charleston earthquake based on Nuttli's (1973, 1976) technique. Nuttli's Central United States particle velocity-intensity data are indicated by solid circles. Trifunac and Brady's (1975) Western United States particle velocity-intensity data are indicated by X's. Distances are from the 90-percent fractile curve of this study. Heavy curve is Nuttli's (1973) theoretical attenuation for the 3-s Rayleigh wave. Western United States data fit with a straight line (light curve).

CONCLUSIONS

The intensity data base published by Dutton (1889) has been studied, and the principal results of that effort are as follows:

1. The maximum epicentral intensity was X (MM), and the intensity in the city of Charleston was IX (MM).
2. The writer verified that Dutton's isoseismal map was contoured so as to depict the broad regional pattern of the effects from ground shaking.
3. When contoured to show more localized variations, the intensity patterns show considerable complexity at all distances.
4. The epicentral distance was measured to each intensity observation point and the resulting data set (780 pairs) was subjected to regression analysis. For the 50-percent fractile of that data set, the equation developed was

$$I = I_0 + 2.87 - 0.00052\Delta - 2.88 \log \Delta$$

   with a standard deviation (σ_I) of 1.2. For the 90- and 75-percent fractiles, the 2.87 constant is replaced by 4.39 and 3.68, respectively. This variation of intensity with distance agrees rather closely with relationships obtained by other workers for the central, eastern, and northeastern parts of the United States. It thus appears that the broad overall attenuation of intensities may be very similar throughout the entire Central and Eastern United States.
5. Using intensity-particle velocity data derived from Central United States earthquakes, the writer estimates a body-wave magnitude (m_b) of 6.8 for the main shock of August 31, 1886. However, the data base upon which this estimate is made is very small; therefore, the estimated m_b should be considered provisional until more data are forthcoming. Use of Western United States intensity-particle velocity data produces an m_b estimate of 7.1.

REFERENCES CITED

Bollinger, G. A., and Stover, C. W., 1976, List of intensities for the 1886 Charleston, South Carolina, earthquake: U.S. Geol. Survey open-file rept. 76-66, 31 p.

Cornell, C. A., and Merz, H. A., 1974, A seismic risk analysis of Boston: Am. Soc. Civil Engineers Proc., Structural Div. Jour., v. 110, no. ST 10 (Paper 11617), p. 2027–2043.

Dutton, C. E., 1889, The Charleston earthquake of August 31, 1886: U.S. Geol. Survey, Ninth Ann. Rept. 1887–88, p. 203–528.

Evernden, J. F., 1975, Seismic intensities, "size" of earthquakes and related parameters: Seismol. Soc. America Bull., v. 65, no. 5, p. 1287–1313.

Gupta, I. N., and Nuttli, O. W., 1976, Spatial attenuation of intensities for Central United States earthquake: Seismol. Soc. America Bull., v. 66, no. 3, p. 743–751.

Howell, B. F., Jr., and Schultz, T. R., 1975, Attenuation of Modified Mercalli intensity with distance from the epicenter: Seismol. Soc. America Bull., v. 65, no. 3, p. 651–665.

Neumann, Frank, 1954, Earthquake intensity and related ground motion: Seattle, Wash., Univ. Washington Press, 77 p.

Nuttli, O. W., 1973, The Mississippi Valley earthquakes of 1811 and 1812—Intensities, ground motion and magnitudes: Seismol. Soc. America Bull., v. 63, no. 1, p. 227–248.

——— 1976, Comments on "Seismic intensities, 'size' of earthquakes and related parameters," by Jack F. Evernden: Seismol. Soc. America Bull., v. 66, no. 1, p. 331–338.

Richter, C. F., 1958, Elementary seismology: San Francisco, Calif., W. H. Freeman Co., 768 p.

Trifunac, M. D., and Brady, A. G., 1975, On the correlation of seismic intensity scales with the peaks of recorded strong ground motion: Seismol. Soc. America Bull., v. 65, no. 1, p. 139–162.

U.S. Environmental Data Service, 1973, Earthquake history of the United States: U.S. Environmental Data Service Pub. 41-1, rev. ed. (through 1970), 208 p.

Wood, H. O., and Neumann, Frank, 1931, Modified Mercalli intensity scale of 1931: Seismol. Soc. America Bull., v. 21, no. 4, p. 277–283.

EARTHQUAKE HAZARD IN THE MEMPHIS, TENNESSEE, AREA

ARCH C. JOHNSTON and SUSAN J. NAVA

There is a difference to be marked between hazard and risk. The two are most easily distinguished by answering the question: Can the actions of people have any effect on the situation? Hazard cannot be lessened or increased but risk can. The earthquake hazard in Memphis, Tennessee, is an inheritance of geographic location and is due to the city's proximity to the New Madrid seismic zone; it cannot be changed by man. Earthquake risk is the immediate danger posed to the population and it can be substantially altered by a number of actions, most significantly, improved construction and siting of buildings. The purpose of this paper is to give a brief introduction to the seismic hazard in Memphis, Tennessee.

THE NEW MADRID SEISMIC ZONE

The New Madrid seismic zone is depicted in Figures 1 and 2. Figure 1 shows the instrumentally located epicenters for the past nine years; the main branches of the seismic zone are delineated by the concentrated pattern of epicenters within the small box of Figure 1. Figure 2 shows the relationship of the zone to Memphis and Shelby County and to the major critical facilities in the surrounding region. The generalized modified Mercalli isoseismals of Algermissen et al. (1983) are superimposed; the contours are estimated as combined effects of maximum magnitude events in the northern and southern portions of the zone. A single event would not produce these estimated intensities at all locations.

The New Madrid seismic zone is regarded by seismologists and disaster response planners as the most hazardous zone east of the Rocky Mountains (Johnston, 1982) There are three basic reasons for this estimation:

1. In the winter of 1811-1812, the zone produced three of the largest earthquakes known to have occurred in North America (M_S 8.5, 8.4, and 8.8) and hundreds of damaging aftershocks (Nuttli, 1983).

2. A major geological structure--an ancient crustal rift--has been identified through a decade of extensive research (McKeown and Pakiser, 1982). The rift underlies the shallow

The authors are members of the staff of the Tennessee Earthquake Information Center in Memphis. They developed this paper for presentation at BSSC meeting in Memphis on January 22, 1985.

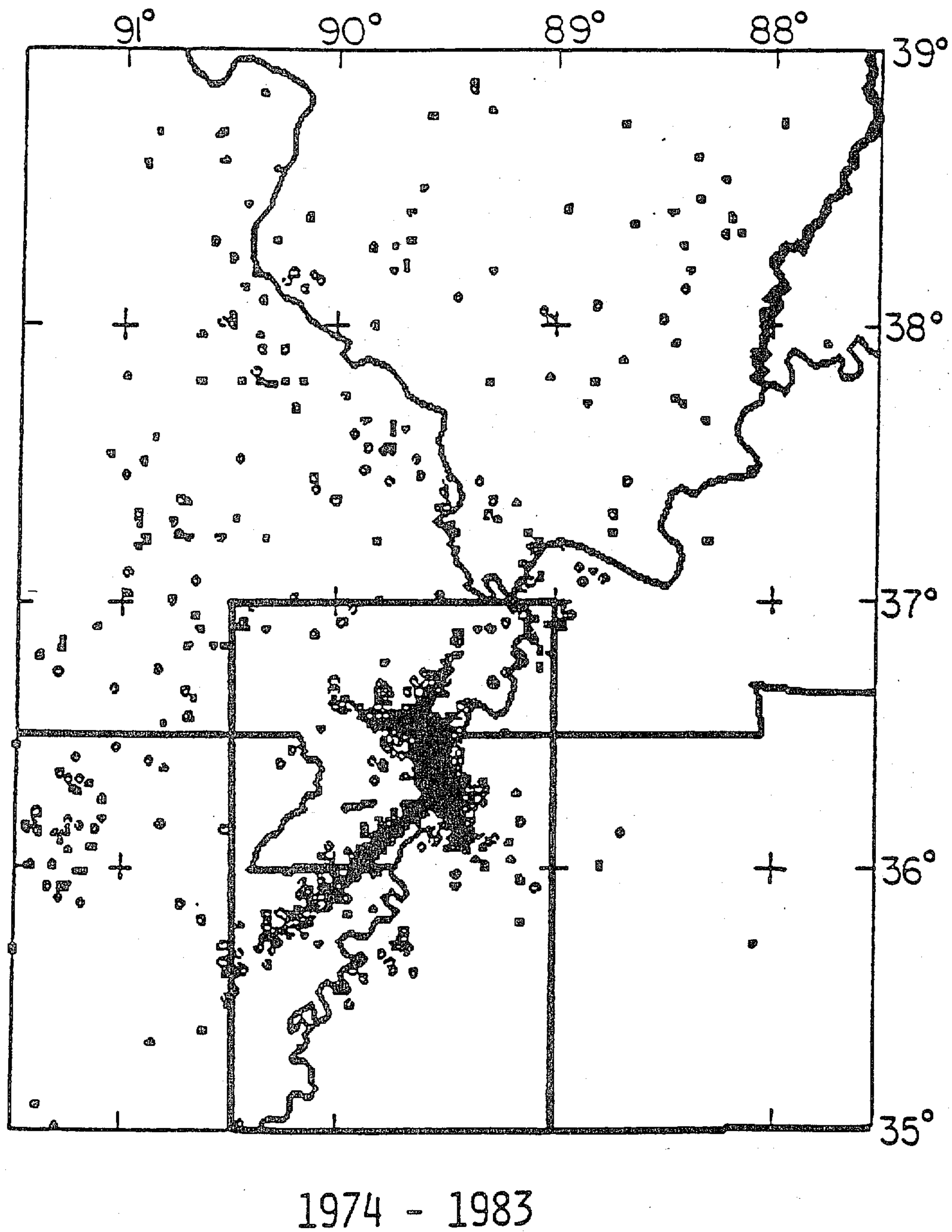


FIGURE 1 Map of the central United States with the 1974-1983 instrumental seismicity data set (Stauder and others, 1974-1983). The boundaries of the two source zones used for frequency-magnitude determination are: Large zones, 35.0 -37.0 N/89.0 -91.5 W.

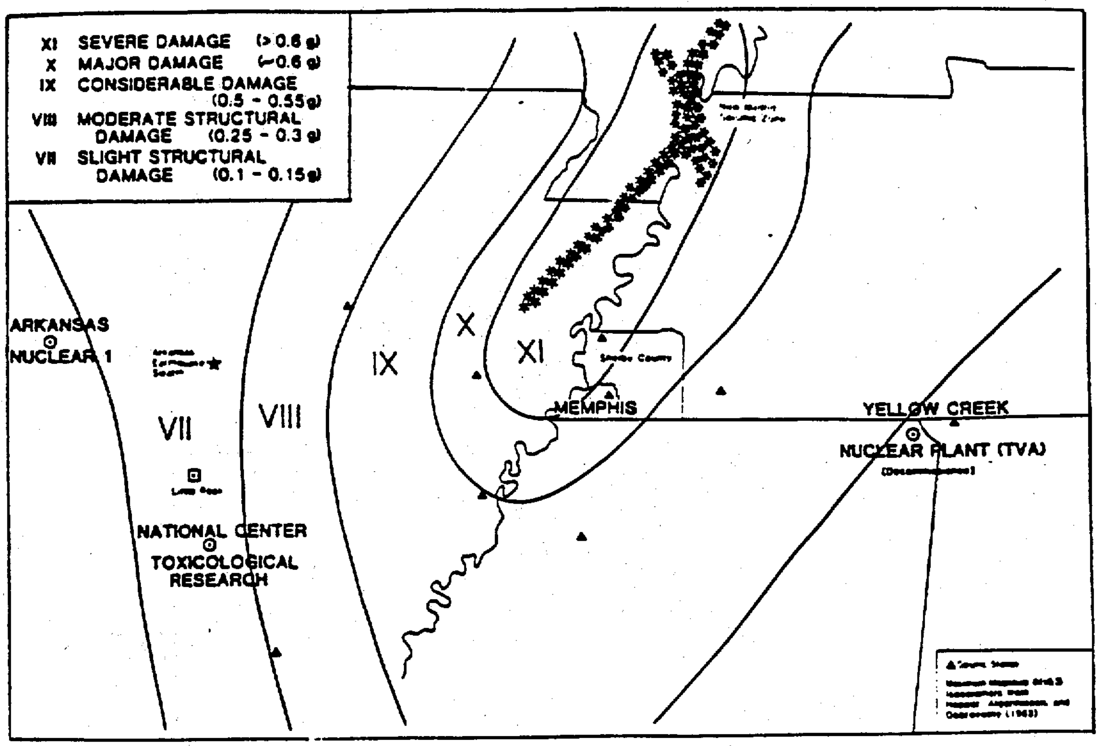


FIGURE 2 The relation of Memphis, Tennessee, and Shelby County to the New Madrid seismic zone. Also shown are major critical facilities in the region and Modified Mercalli isoseismals for a "composited" maximum magnitude New Madrid earthquake.

sediments of the Mississippi embayment and is of such character and dimension that it could generate major earthquakes.

3. The zone is still quite seismically active (Figure 1). More than 2,000 earthquakes (of which 97 percent have been too small to be felt) have been detected in the zone since 1974.

These three observations--past great earthquakes, identified geological structure, and continuing activity--constitute the reasons for the high hazard potential with which the New Madrid zone is presently regarded.

EARTHQUAKE PROBABILITY

Without a doubt, the most frequently asked and least satisfactorily answered question concerning the earthquakes of the New Madrid seismic zones is: When is the next major earthquake going to happen? Seismology cannot now (nor in the near future) answer this question in a deterministic fashion (i.e., accurately predict earthquakes), but a probabilistic assessment is possible. In a recent study, Johnston and Nava (1985) estimated the probability of occurrence of large New Madrid earthquakes for two time periods--by the end of the century and within a representative lifetime (15 and 50 years, respectively) The estimates are based on magnitude: (1) a body-wave magnitude, m_b, of 6.0 (or equivalently a surface-wave magnitude, M_s, of 6.3) which could be destructive over an area of one or more counties and (2) a body-wave magnitude of 7.0 (surface-wave magnitude of 8.3) which is considered equivalent to a repeat of one of the great New Madrid events of 1811-1812. Using these magnitude categories, the determined probabilities are as follows:

Body Wave Magnitude	Probability (%) 1985 to 2000	Probability (%) 1985 to 2035
m_b 6.0 (M_s 6.3)	40-63	86-97
m_b 7.0 (M_s 8.3)	0.3-1.0	2.7-4

A number of assumptions about the seismic behavior of New Madrid were necessary in order to generate the above probability ranges. The approach used and the assumptions that went into the final probability estimates are described briefly below.

Probability estimates require that the seismic zone behaves in a roughly predictable or period manner. This cannot be proven for large New Madrid events because of an incomplete data set over many seismic cycles, but smaller earthquakes exhibit a well behaved recurrence pattern. Therefore, the authors took instrumentally recorded data from the past nine years (see Figure 1) and a historical list of earthquakes of the past 158 years, determined the recurrence relationships for this data set, and then extrapolated to large magnitudes. This yielded an estimate of the average recurrence or repeat time in years between New Madrid earthquakes for a given magnitude range. For m_b 6.0, the average repeat time is 70 years. (The last such event occurred 90 years ago in 1895.) For m_b 7.0 (M_s 8.3). the average repeat time is 550 years. (The last such event was in 1812, 173 years ago.) These estimates apply to data from the entire region shown in Figure 1. If only the small region is considered (within the rectangle of Figure 1), repeat times approximately double. There are sound geophysical reasons for choosing the larger source zone.

Once the average repeat time is established, both cumulative and conditional probabilities can be determined. Cumulative probability tells us the likelihood that a quake of a certain magnitude would have occurred by now (the present) given the date of the last occurrence and the average recurrence interval. Conditional probability estimates the likelihood of occurrence during a future specified time period (i.e., 15 and 50 years--this study). Obviously, conditional probabilities are of greater interest than cumulative and are therefore emphasized in this study.

In order to make the final probability computations it is necessary to know the manner in which actual earthquake repeat times, for a given magnitude range, are dispersed about the estimated mean repeat time. This is described statistically in terms of a probability distribution with a given standard deviation. Such information for large magnitude New Madrid events is lacking; the authors' approach, therefore, was to take a number of different distributions and a range of standard deviations from the literature of studies of other active earthquake zones and apply these to New Madrid. This approach allowed for a large uncertainty in the actual (but unknown) behavior of New Madrid. This results in a range of probability values as quoted above rather than a single number.

Figures 3-5 are graphs of Gaussian conditional probabilities from m_b 6.0, m_b 6.6, and m_b 7.0 earthquakes (M_s 6.3, M_s 7.6, and M_s 8.3, respectively), graphs on which one can see the effect that the standard deviation exerts on the probability values. The types of probability distribution employed also have an effect but to a lesser degree. The date of last occurrence, the present (1985), and the mean recurrence time are indicated on the horizontal time axis. Shading illustrates the probability range as standard deviation is varied from 33 percent to 50 percent of the mean repeat time. Calculations were done for four different statistical representations--Gaussian, log-normal, Weilbull, and Poisson--but only Gaussian is shown here. Poisson statistics, which yield a constant conditional probability, are not appropriate for this analysis; therefore, only the Gaussian, log-normal, and Weibull distributions were used to obtain the probability ranges quoted above.

In conclusion, the authors estimate that there is a medium probability of a locally destructive New Madrid earthquake in the next 15 years (40 percent to 63 percent) and a high probability (86 percent to 97 percent) in the next 50 years. The probability for a great New Madrid event is less than 1 percent by the turn of the century and less than 4.0 percent during the next 50 years. These estimates are of necessity based on a number of unproven assumptions about the New Madrid zone; however, every effort was made to take an appropriate and comprehensive range of estimates in order to bracket the actual probability for future destructive earthquakes in the central United States.

PROBABILITY OF A MAGNITUDE (Ms) 8.3 EARTHQUAKE

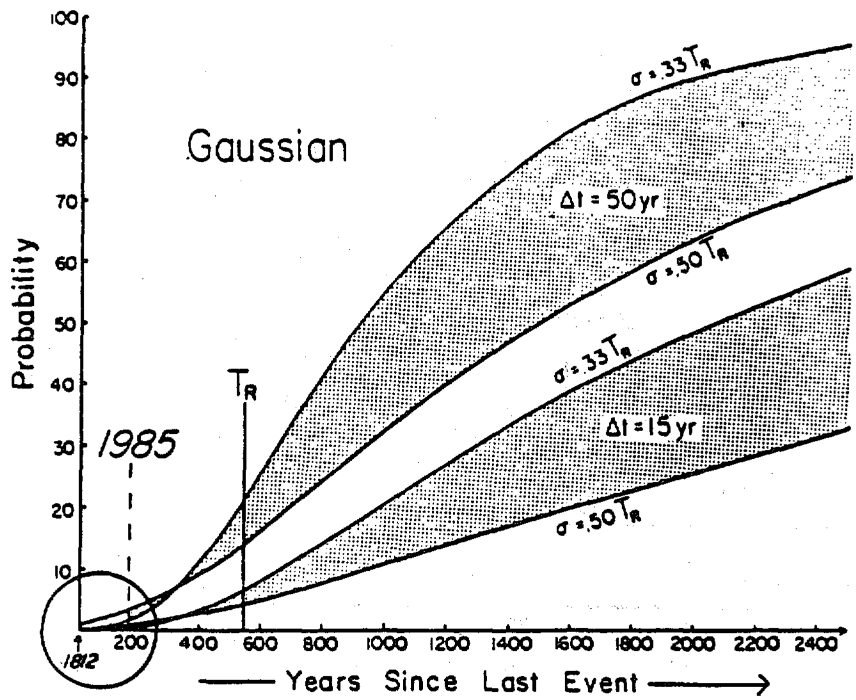


FIGURE 3(a) Gaussian conditional probability computed for magnitude m_b 7.0 (M_S 8.3) earthquake. The last such event occurred in 1812 and the mean repeat time (T_R) is 550 years. The shaded region represents the range of conditional probability as the standard deviation is varied from 33 percent to 50 percent of T_R. Future time intervals (Δt) of 15 and 50 years are depicted.

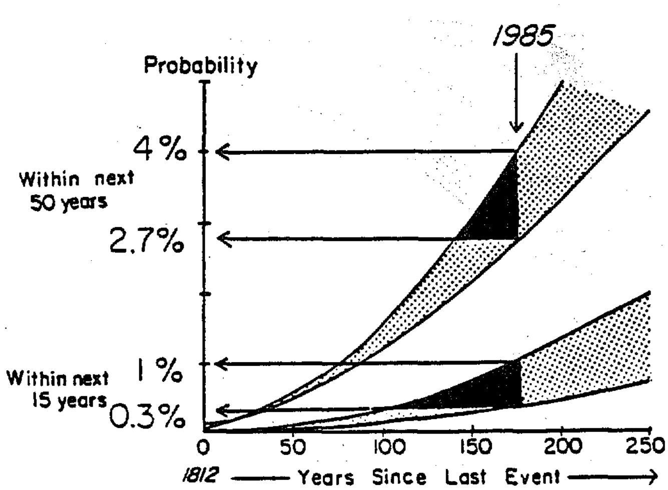


FIGURE 3(b) An expanded view of the circled region near the origin of Figure 3(a). The probability ranges are those quoted in the text.

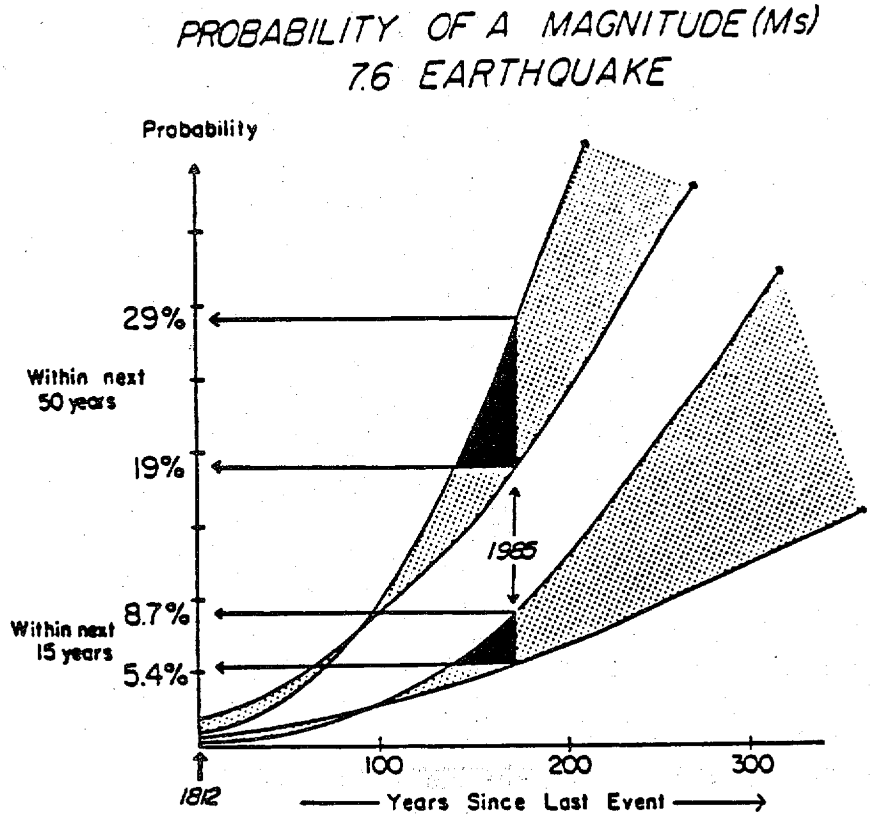


FIGURE 4 Conditional probability representation of an m_b 6.6/M_S 7.6 earthquake. Graph description follows Figure 3(a).

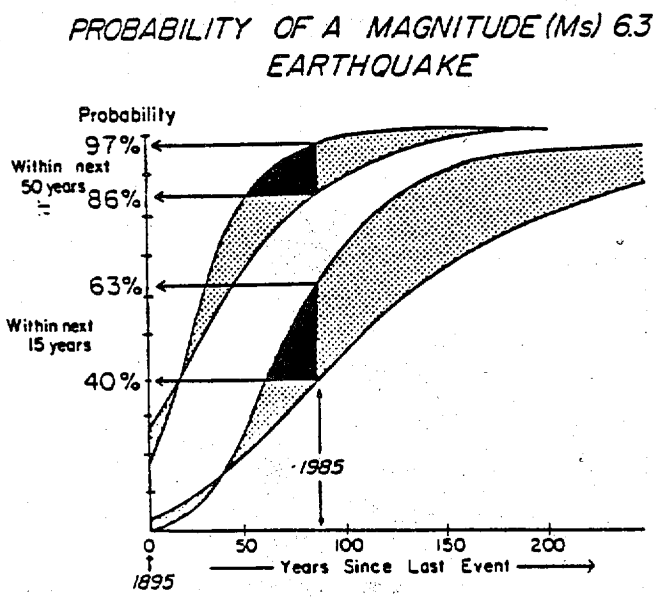


FIGURE 5 Conditional probability representation of m_b 6.0/M_S 6.3 earthquake. Graph description follows Figure 3(a).

REFERENCES

Hopper, M. G., S. T. Algermissen, E. E. Dobrovolny, 1983. Estimation of Earthquake Effects Associated with a Great Earthquake in the New Madrid Seismic Zone. USGS Open File Report 83-179. Washington, D.C. : U.S. Geological Survey.

Johnston, A. C. 1982. "A Major Earthquake Zone on the Mississippi." Scientific American 246 (4):60-68.

Johnston, A. C., and S. J. Nava. 1982. Investigations of the New Madrid Seismic Zone. Submitted to the Journal of Geophysical Research.

Mekeon, F. A. and L. C. Pakiser, Editors. 1982. Investigation of the New Madrid, Missouri, Earthquake Region. USGS Professional Paper 1236. Washington, D.C.: U.S.Geological Survey.

Nuttli, O. W. 1983. "Average Seismic Source-Parameter Relations for Mid-Plate Earthquake." Bulletin of the Seismic Society of America 73(2): 519-535.

EVALUATION OF THE EARTHQUAKE GROUND-SHAKING HAZARD FOR EARTHQUAKE-RESISTANT DESIGN

WALTER W. HAYS

This paper describes current research that can be applied to evaluate the earthquake ground-shaking hazard in any geographic region. Because most of the spectacular damage that takes place during an earthquake is caused by partial or total collapse of buildings as a result of ground shaking or the triggering of geologic effects such as ground failures and surface faulting, an accurate evaluation of the ground-shaking hazard is an important element of: (1) vulnerability studies; (2) specification of seismic design parameters for earthquake-resistant design of buildings, lifeline systems, and critical facilities; (3) assessment of risk (chance of loss); and (4) the specifications of appropriate building codes. Although the physics of ground-shaking, a term used to describe the vibration of the ground during an earthquake, is complex, ground-shaking can be explained in terms of body waves (compressional, or P, and shear, or S) and surface waves (Rayleigh and Love) (see Figure 1). Body and surface waves cause the ground and, consequently, a building and its contents and attachments to vibrate in a complex manner. Shear waves, which cause a building to vibrate from side to side, are the most damaging waves because buildings are more susceptible to horizontal vibrations than to vertical vibrations.

The objective of earthquake-resistant design is to construct a building so that it can withstand the vibrations caused by body and surface waves. In earthquake-resistant design, knowledge of the amplitude, frequency composition, and time duration of vibrations is needed. The quantities are determined empirically from strong motion accelerograms recorded in the geographic area or in other areas having similar geologic characteristics.

In addition to ground-shaking, the occurrence of earthquake-induced ground failures, surface faulting, and, for coastal locations, tsunamis also must be considered. Although ground failures induced during earthquakes have caused many thousands of casualties and millions of dollars in property damage throughout the world, the impact in the United States has been limited primarily to economic loss. During the 1969 Prince William Sound, Alaska, earthquake, ground failures caused about 60 percent of the estimated $500 million total loss; landslides, lateral spread failures, and flow failures caused damage to highways, railway grades,

---

Dr. Hays is Deputy for Research Applications, Office of Earthquakes, Volcanoes, and Engineering, U.S. Geological Survey, Reston, Virginia. He prepared this paper as background information for those making presentations at the BSSC meetings in January and February 1985.

bridges, docks, ports, warehouses, and single-family dwellings. In contrast to ground failures, deaths and injuries from surface faulting are unlikely; however, buildings and lifeline systems located in the fault zone can be severely damaged. Tsunamis, long period water waves caused by the sudden vertical movement of a large area of the sea floor during an earthquake, have produced great destruction and loss of life in Hawaii and along the West Coast of the United States. Tsunamis have occurred in the past and are a definite threat in the Caribbean. Historically, tsunamis have not been a threat on the East Coast.

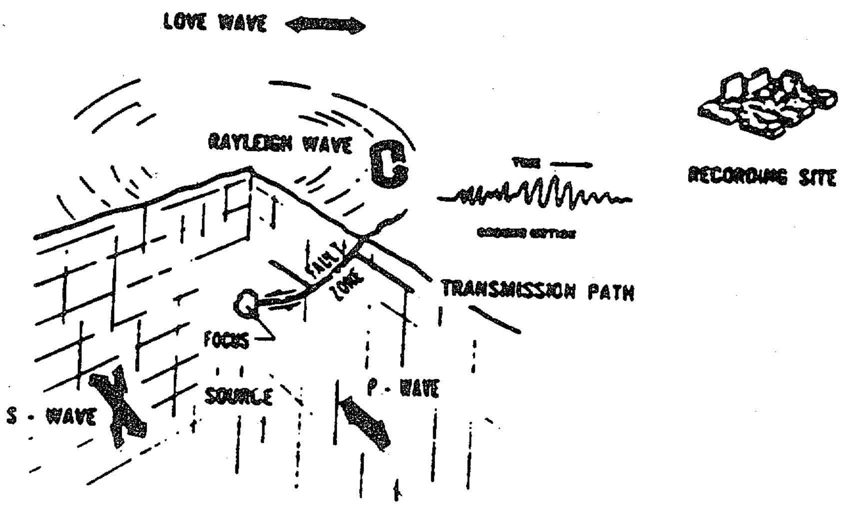


FIGURE 1 Schematic illustration of the directions of vibration caused by body and surface seismic waves generated during an earthquake. When a fault ruptures, seismic waves are propagated in all directions, causing the ground to vibrate as a consequence of the ground-shaking, and damage takes place if the building is not designed to withstand these vibrations. P and S waves mainly cause high-frequency (greater than 1 Hertz) vibrations that are more efficient in causing low buildings to vibrate. Rayleigh and Love waves mainly cause low-frequency vibrations that are more efficient than high-frequency waves in causing tall buildings to vibrate.

EVALUATION OF THE GROUND-SHAKING HAZARD

No standard methodology exists for evaluating the ground-shaking hazard in a region. The methodology that is used (whether deterministic or probabilistic) seeks answers to the following questions:

1. Where have past earthquakes occurred? Where are they occurring now?
2. Why are they occurring?
3. How big are the earthquakes?
4. How often do they occur?
5. What are the physical characteristics (amplitude,frequency composition, duration) of the ground shaking and the physical effects on buildings and other facilities?
6. What are the options for achieving earthquake-resistant design?

The ground-shaking hazard for a community (Figure 2) may be presented in a map format. Such a map displays the special variation and relative severity of a physical parameter such as peak ground acceleration. The map provides a basis for dividing a region into geographic regions or zones, each having a similar relative severity or response throughout its extent to earthquake ground-shaking. Once the potential effects of ground-shaking have been defined for all zones in a region, public policy can be devised to mitigate its effects through appropriate actions such as avoidance, land-use planning, engineering design, and distribution of losses through insurance (Hays, 1981). Each of these mitigation strategies require some sort of zoning (Figure 2). The most familiar earthquake zoning is contained in the <u>Uniform Building Code</u> (UBC) whose aim is to provide a <u>minimum</u> earthquake-resistant design standard that will enable the building to:

1. Resistant minor earthquakes without damage,
2. Resist moderate earthquakes without structural damage but with some nonstructural damage, and
3. Resist major earthquakes with structural and nonstructural damage but without collapse.

HISTORY OF SEISMIC ZONING

Zoning of the earthquake ground-shaking hazard--the division of a region into geographic areas having a similar relative severity or response to ground-shaking--has been a goal in the contiguous United States for about 50 years. During this period, two types of ground-shaking hazard maps have been constructed. The first type (Figure 3) summarizes the empirical observations of past earthquake effects and makes the assumption that, except for scaling differences, approximately the same physical effects will occur in future earthquakes. The second type (Figures 4-6) utilizes probabilistic concepts and extrapolates from regions having past earthquakes as well as from regions having potential earthquake sources, expressing the hazard in terms of either exposure time or return period.

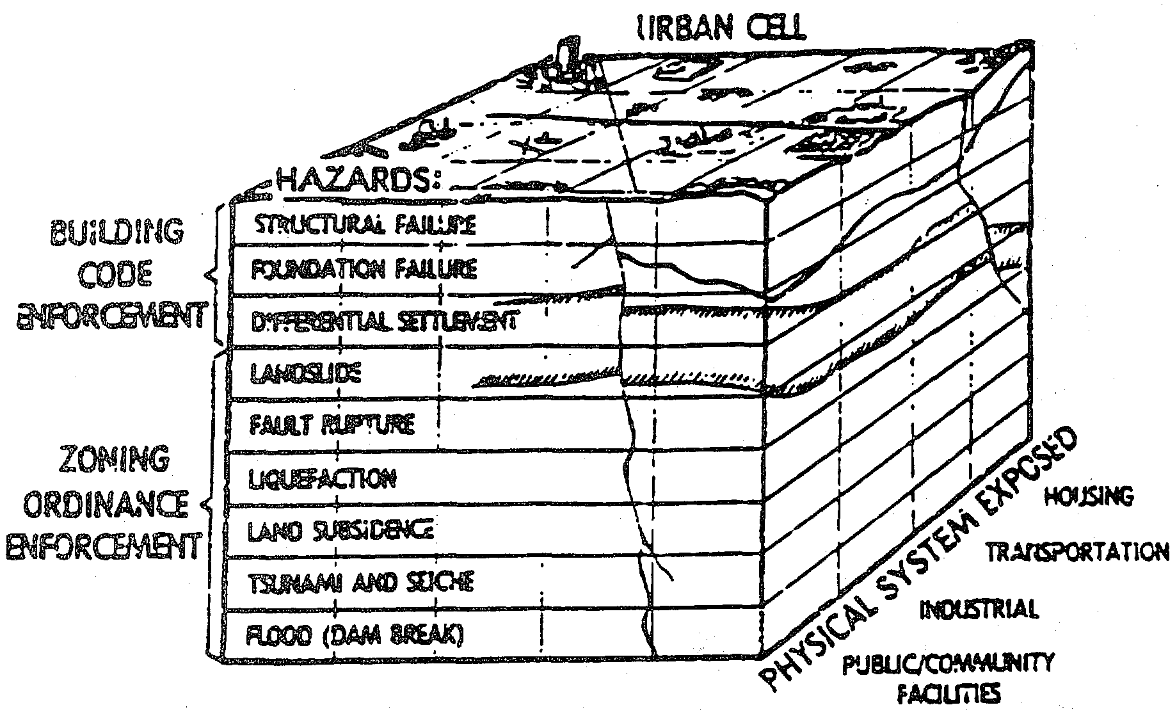


FIGURE 2 Schematic illustration of a typical community having physical systems (public/community facilities, industrial, transportation, and housing) exposed to earthquake hazards. Evaluation of the earthquake hazards provides policymakers with a sound physical basis for choosing mitigation strategies such as avoidance, land-use planning, engineering design, and distribution of losses through insurance. Earthquake zoning maps are used in the implementation of each strategy, especially for building codes.

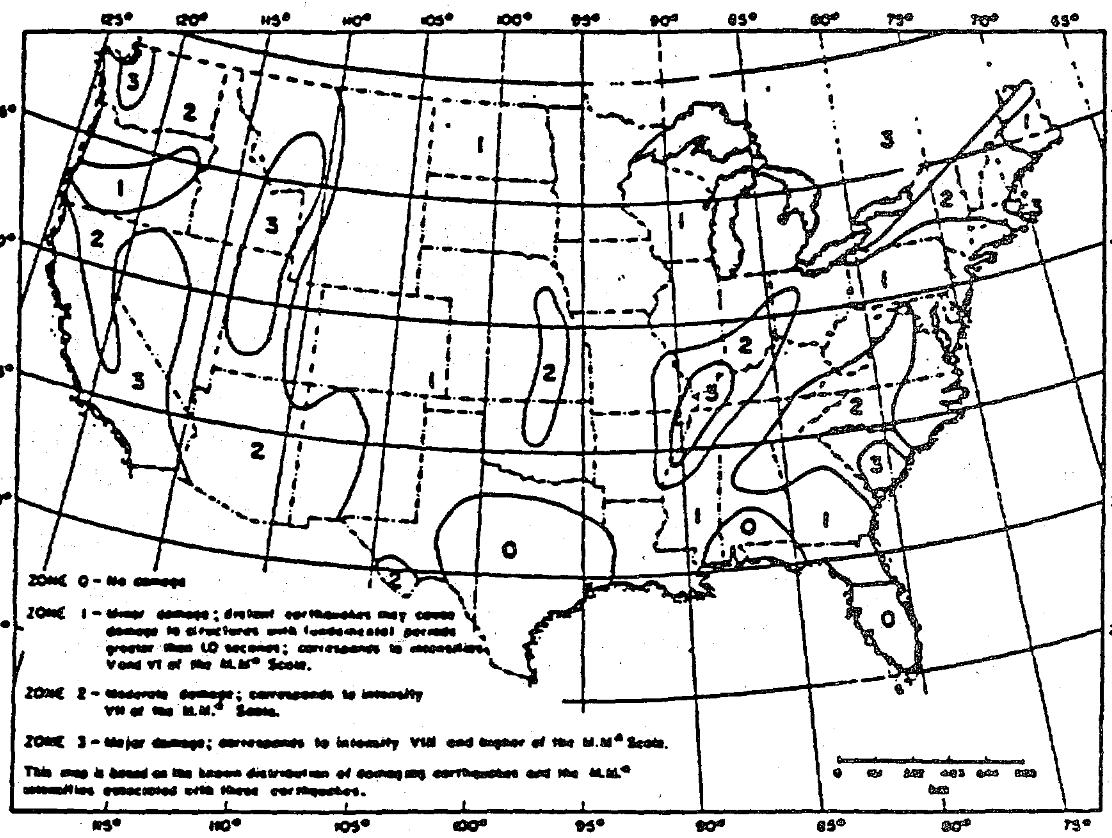


FIGURE 3 Seismic hazard zones based on historical modified Mercalli intensity (MMI) data and the distribution of damaging earthquakes (Algermissen, 1969). This map was adopted in the 1970 edition of the UBC and incorporated, with some modifications, in later editions. Zone 3 depicts the greatest hazard and corresponds to MMI VIII and greater.

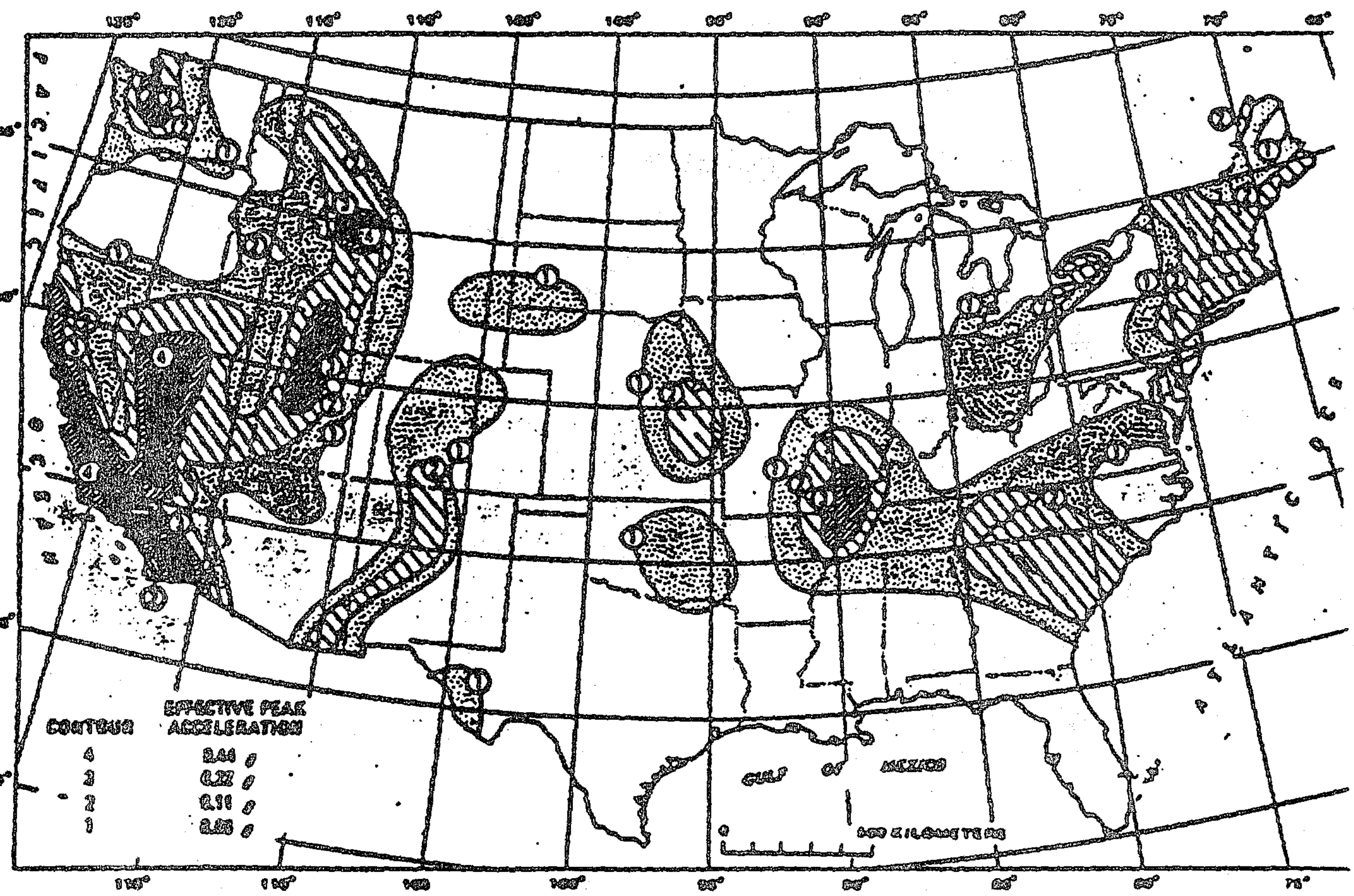


FIGURE 4 Map showing preliminary design regionalization zones for the contiguous United States proposed by the Applied Technology Council (ATC) in 1978. Contours connect areas underlain by rock having equal values of effective peak acceleration. Mapped values have a 90 percent probability of not being exceeded in a 50-year period. Zone 1 represents the lowest hazard (0.06 g). Sites located in Zone 4 require site-specific investigations. This map was based on research by Algermissen and Perkins (1976).

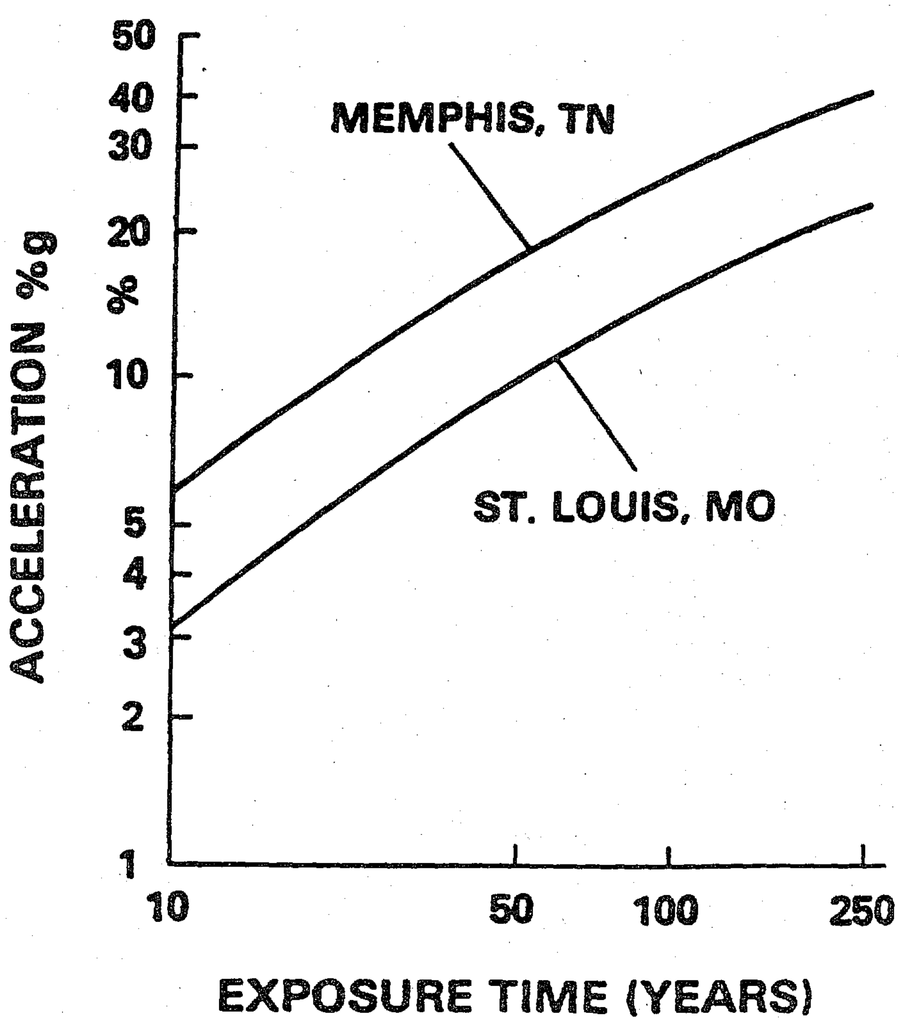


FIGURE 5 Graph showing levels of peak horizontal ground acceleration expected at bedrock sites in the Memphis, Tennessee, and the St. Louis, Missouri, areas in various exposure times. The values of peak acceleration have a 90 percent probability of nonexceedance. An exposure time of 50 years corresponds to the useful life of an ordinary building and is typically used in many building codes.

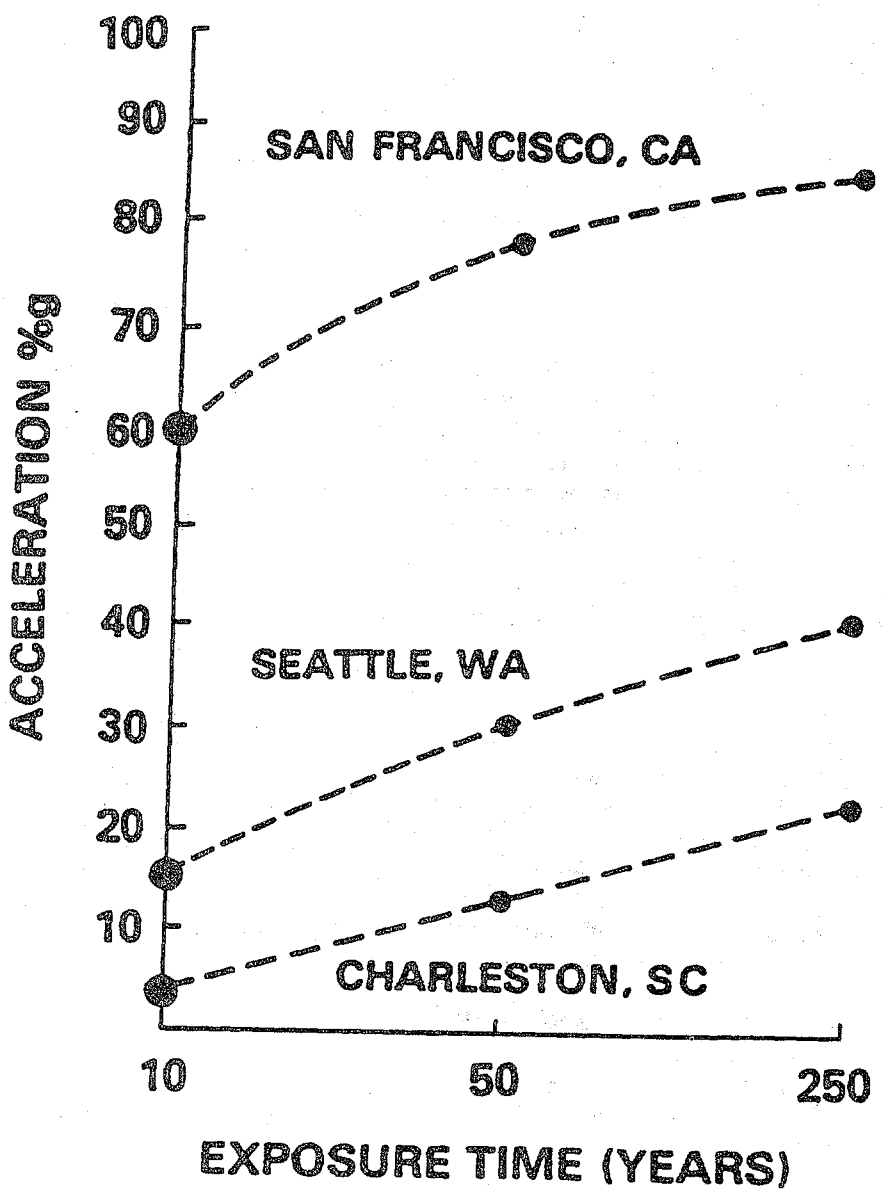


FIGURE 6 Graph showing levels of peak horizontal ground acceleration expected at bedrock sites in the Charleston, South Carolina, and the Seattle, Washington, areas in various exposure times. For comparison, San Francisco, California, also is included. The values of peak acceleration have a 90 percent probability of nonexceedance. An exposure time of 50 years corresponds to the useful life of an ordinary building and is typically used in many building codes.

PROCEDURE FOR EVALUATING THE GROUND-SHAKING HAZARD

Construction of a ground-shaking hazard map requires data on:

1. Seismicity,
2. Earthquake source zones,
3. Attenuation of peak acceleration, and
4. Local ground response.

The procedure for constructing a ground-shaking hazard map is illustrated schematically in Figure 7. Except for probabilistic considerations a deterministic map would follow the same general procedure.

RESEARCH PROBLEMS

A number of complicated research problems are involved in the evaluation of the ground-shaking hazard (Hays, 1980). These problems must be addressed if more accurate specifications of the ground-shaking hazard are desired. The problems can be categorized in four general areas--seismicity, nature of the earthquake source zone, seismic wave attenuation, and local ground response--with each area having a wide range of technical issues. Presented below are representative questions, which generally cannot be answered with a simple "yes" or "no," that illustrate the controversy associated with ground-shaking hazard maps.

Seismicity

- Can catalogs of instrumentally recorded and felt earthquakes (usually representing a regional scale and a short time interval) be used to give a precise specification of the frequency of occurrence of major earthquakes on a local scale?

- Can the seismic cycle of individual fault systems be determined accurately and, if so, can the exact position in the cycle be identified?

- Can the location and magnitude of the largest earthquake that is physically possible on an individual fault system or in a seismotectonic province be specified accurately? Can the recurrence of this event be specified? Can the frequency of occurrence of small earthquakes be specified?

- Can seismic gaps (i.e., locations having a noticeable lack of earthquake activity surrounded by locations having activity) be identified and their earthquake potential evaluated accurately?

- Does the geologic evidence for the occurrence of major tectonic episodes in the geologic past and the evidence provided by current and historic patterns of seismicity in a geographic region agree? If not, can these two sets of data be reconciled?

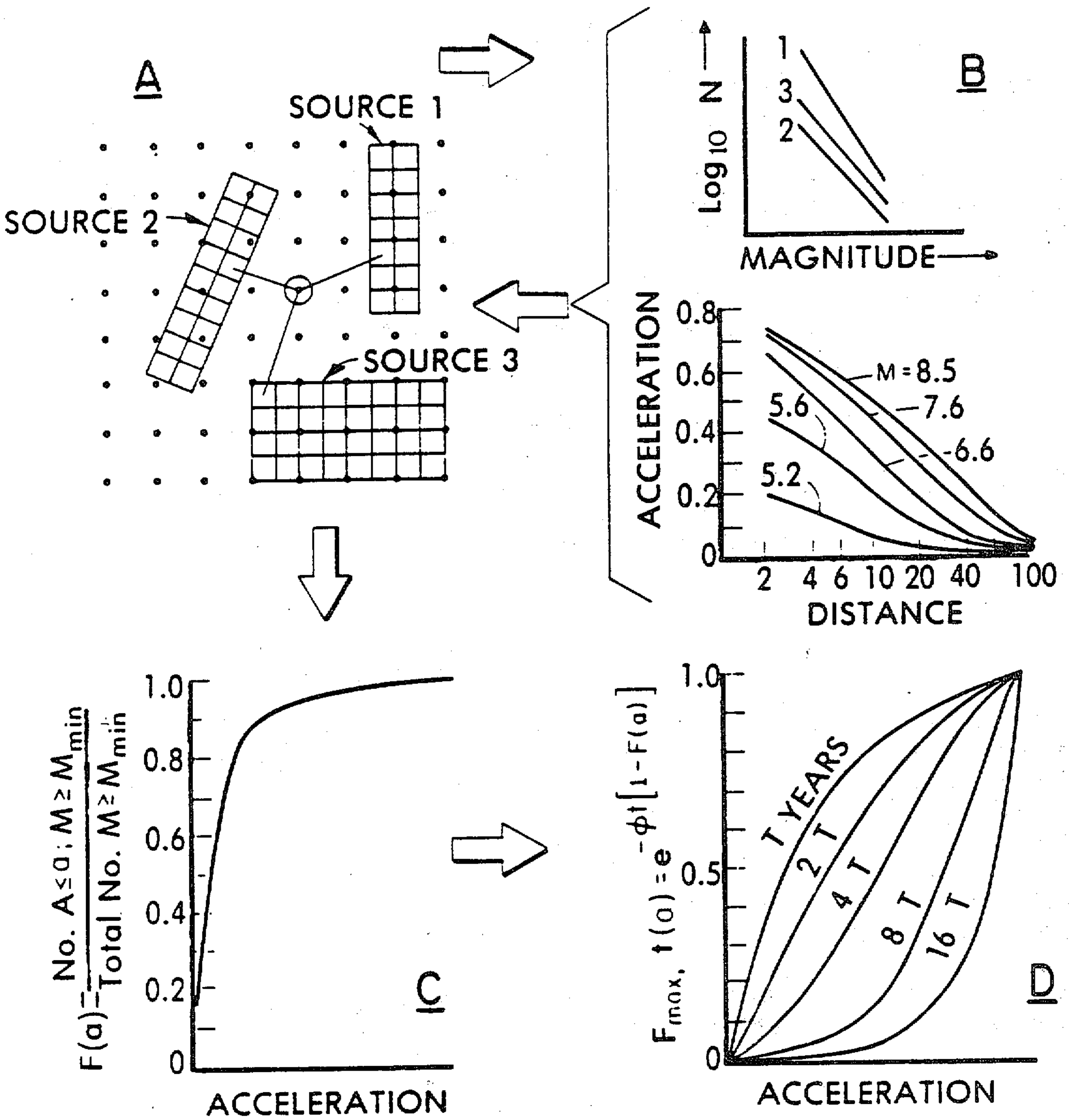


FIGURE 7 Procedure for constructing a grounding-shaking hazard map.

The Nature of the Earthquake Source Zone

- Can seismic source zones be defined accurately on the basis of historic seismicity, on the basis of geology and tectonics, or on the basis of historical seismicity generalized by geologic and tectonic data? Which approach is most accurate for use in deterministic studies? Which approach is most accurate for use in probabilistic studies?

- Can the magnitude of the largest earthquake expected to occur in a given period of time on a particular fault system or in a seismic source zone be estimated correctly?

- Has the region experienced its maximum or upper-bound earthquake?

- Should the physical effects of important earthquake source parameters such as stress drop and seismic moment be quantified and incorporated in earthquake-resistant design even though they are not traditionally used?

Seismic Wave Attenuation

- Can the complex details of the earthquake fault rupture (e.g., rupture dimensions, fault type, fault offset, fault slip velocity) be modeled to give precise estimates of the amplitude and frequency characteristics of ground motion both close to the fault and far from the fault?

- Do peak ground-motion parameters (e.g., peak acceleration) saturate at large magnitudes?

- Are the data bases adequate for defining bedrock attenuation laws? Are they adequate for defining soil attenuation laws?

Local Ground Response

- For specific soil types is there a discrete range of peak ground-motion values and levels of dynamic shear strain for which the ground response is repeatable and essentially linear? Under what in-situ conditions do non-linear effects dominate?

- Can the two- and three-dimensional variations of selected physical properties (e.g., thickness, lithology, geometry, water content, shear-wave velocity, and density) be modelled accurately? Under what physical conditions do one or more of these physical properties control the spatial variations, the duration, and the amplitude and frequency composition of ground response in a geographic region?

- Does the uncertainty associated with the response of a soil and rock column vary with magnitude?

CONCLUSIONS

Improved maps of the earthquake ground-shaking hazard will come as relevant geologic and seismological data are collected and synthesized. The key to progress will be the resolution of the research problems identified above.

REFERENCES

Algermissen, S. T. 1969. "Seismic Risk Studies in the United States." In Proceedings of the 4th Conference on Earthquake Engineering, Vol. 1.

Algermissen, S. T., and D. M. Perkins. 1976. A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States. USGS Open File Report 76-416. Reston, Virginia: U.S. Geological Survey.

Applied Technology Council. 1978. Tentative Provisions for the Development of Seismic Regulations for Buildings. Report ATC 3-06. Palo Alto, California: Applied Technology Council.

Hays, W. W. 1980. Procedures for Estimating Earthquake Ground Motions. USGS Professional Paper 1114. Reston, Virginia: U.S. Geological Survey.

Hays, W. W. 1981. Facing Geologic and Hydrologic Hazards-Earth Science Considerations. USGS Professional Paper 1240. Washington, D.C.: U.S. Government Printing Office.