215 codes
8-3-70

DWR-1484

OAK RIDGE NATIONAL LABORATORY

operated by

UNION CARBIDE CORPORATION

for the

U.S. ATOMIC ENERGY COMMISSION

MASTER

ORNL-NSIC-28
UC-80 — Reactor Technology

EARTHQUAKES AND NUCLEAR POWER PLANT DESIGN

T. F. Lomenick and NSIC Staff

NUCLEAR SAFETY INFORMATION CENTER

NSIC

DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

DISCLAIMER

This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency Thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product, or process disclosed, or represents that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof.

DISCLAIMER

Portions of this document may be illegible in electronic image products. Images are produced from the best available original document.

Printed in the United States of America. Available from Clearinghouse for Federal
Scientific and Technical Information, National Bureau of Standards,
U.S. Department of Commerce, Springfield, Virginia 22151
Price: Printed Copy \$3.00; Microfiche \$0.65

— LEGAL NOTICE —

This report was prepared as an account of Government sponsored work. Neither the United States, nor the Commission, nor any person acting on behalf of the Commission:

- A. Makes any warranty or representation, expressed or implied, with respect to the accuracy, completeness, or usefulness of the information contained in this report, or that the use of any information, apparatus, method, or process disclosed in this report may not infringe privately owned rights; or
- B. Assumes any liabilities with respect to the use of, or for damages resulting from the use of any information, apparatus, method, or process disclosed in this report.

As used in the above, "person acting on behalf of the Commission" includes any employee or contractor of the Commission, or employee of such contractor, to the extent that such employee or contractor of the Commission, or employee of such contractor prepares, disseminates, or provides access to, any information pursuant to his employment or contract with the Commission, or his employment with such contractor.

Contract No. W-7405-eng-26

Nuclear Safety Information Center

EARTHQUAKES AND NUCLEAR POWER PLANT DESIGN

T. F. Lomenick and NSIC Staff

LEGAL NOTICE

This report was prepared as an account of work sponsored by the United States Government. Neither the United States nor the United States Atomic Energy Commission, nor any of their employees, nor any of their contractors, subcontractors, or their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness or usefulness of any information, apparatus, product or process disclosed, or represents that its use would not infringe privately owned rights.

JULY 1970

OAK RIDGE NATIONAL LABORATORY
Oak Ridge, Tennessee
operated by
UNION CARBIDE CORPORATION
for the
U.S. ATOMIC ENERGY COMMISSION

DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

fly

THIS PAGE
WAS INTENTIONALLY
LEFT BLANK

CONTENTS

	<u>Page</u>
FOREWORD	vii
PREFACE	xi
ABSTRACT	xiii
1. INTRODUCTION	1
2. SUMMARY	4
2.1 Shaking	10
2.2 Faulting	16
2.3 Tsunamis	17
References	19
3. GROUND SHAKING	20
3.1 Generation of Ground Motions	21
3.2 Recording of Ground Motions	21
3.2.1 Instruments	22
3.2.2 Records	25
3.3 Intensity of Ground Motion	26
3.4 Effects on Soils	28
3.4.1 Amplification of Ground Oscillations	29
3.4.2 Liquefaction	31
3.4.3 Subsidence	34
3.4.4 Landslides and/or Slope Stability	34
3.4.5 Soil Tests	35
3.5 Intensity, Magnitude, and Acceleration Relations	37
3.6 Evaluation of Ground Motions at Specific Sites	38
3.6.1 Maps	42
3.6.2 Frequency of Occurrence	43
3.6.3 Intensity of Ground Shaking During Large Earthquakes	43
3.6.4 Instrumentation for Site Evaluations	44
3.7 Engineering Formulation of Excitation	47
3.7.1 Selection of Design Basis	47
3.7.2 Modification of Motion in Various Media	48

3.7.3	Spectra of Vibratory Motion	50
3.7.4	Computer Calculations Based on Input of Seismograph Record	59
3.8	Excitation of Reactor Structures	62
3.8.1	Soil-Structure Interaction	62
3.8.2	Piling	64
3.8.3	Excitation Energy and Its Dissipation Through Elastic Damping	65
3.8.4	Inelastic Structural Response	69
3.9	Design of Specific Systems	72
3.9.1	Reactor Core	73
3.9.2	Piping Systems	74
3.9.3	Secondary Containment System	77
3.9.4	Control Features	82
3.9.5	Seismic Instrumentation for Reactor Protection	85
3.9.6	Dams	88
3.10	Dynamic Testing for Earthquake Survival	94
3.11	Costs of Reactor Plants Designed for Earthquake Survival	97
	References	101
4.	FAULTS	113
4.1	Faults and Earthquakes: Cause or Effect?	114
4.2	Fault Occurrence	117
4.3	Types of Faulting	121
4.4	Faults and Earthquake Energy	123
4.5	Fault Zones	128
4.6	Fault Activity	131
4.7	Fault Creep or Slippage	133
4.8	Evaluation of Faulting at Specific Sites	135
4.8.1	Historic and Seismographic Records	136
4.8.2	Geological and Geophysical Investigations	137
4.9	Design Considerations for Fault Movement	141
4.9.1	Testing and Models	142
4.9.2	Effect of Faulting on Water Supply	143

4.10	Design Concepts for Fault Displacements	143
4.10.1	Integral Foundation	144
4.10.2	Use of Caisson Structures	145
4.10.3	Floating Foundations	145
4.10.4	Artificial Offshore Island	146
4.11	Materials for Isolation from Fault Slippage	146
	References	147
5.	TSUNAMIS	154
5.1	Mechanisms and Occurrences	154
5.2	Properties	157
5.3	Evaluation of Tsunami Effects	159
5.4	Tsunami Warning System	162
	References	163
6.	CONCLUSIONS AND RECOMMENDATIONS	165
6.1	Faults	166
6.1.1	Incidence and Activity	166
6.1.2	Fault Motion Characteristics	167
6.2	Design To Accommodate Fault Movement	168
6.2.1	Design Requirements	168
6.2.2	Design Concepts	168
6.3	Ground Shaking	169
6.3.1	Characteristics of Motion	170
6.3.2	Induced Effects	171
6.3.3	Seismicity	171
6.4	Design To Accommodate Shaking	172
6.4.1	Selection of Design Basis	172
6.4.2	Soil-Structure Interaction	172
6.4.3	Damping	173
6.4.4	Structural Components and Materials	173
6.4.5	Motion-Sensitive Control Systems	175
6.4.6	Seismic Instrumentation at Nuclear Power Plants	176
6.4.7	Design Analysis and Computation	176
6.4.8	Dynamic Tests	178

6.5 Tsunamis	179
6.6 Design for Protection from Tsunamis	181
References	181
APPENDIX A. GLOSSARY	185
References	190
APPENDIX B. REACTOR SITE EVALUATIONS	191
Bodega Bay	191
Browns Ferry	197
San Onofre	199
H. B. Robinson No. 2	204
Malibu	206
References	211
APPENDIX C. SOME IMPORTANT EARTHQUAKES	215

FOREWORD

The recent surge in the building of large nuclear power plants, particularly with the projected desirability of using urban sites for such installations, has focused attention on many aspects of the AEC's responsibilities for licensing reactors and insuring the public safety. Since the industry is "young," meaningful long-term operating experience is sparse and the definition of the possible accident spectrum, as well as a set of firm design requirements, is subject to a largely analytical approach that necessarily involves conservative judgments. As plant designs become standardized and operating experience on the newer large reactors is gained, the inevitable process of refinement and of acquiring confidence in the operation of the plants will occur. This relatively slow evolutionary approach to acquiring firm design standards and criteria is not felt to be conducive to achieving the great national benefits of atomic energy within a reasonable time in terms of the conservation of resources, combating pollution, and the multitude of gains resulting from low-cost electricity.

As part of the effort to improve on this approach, the Regulatory Review (Mitchell) Panel recommended the formation by the AEC of a Steering Committee on Reactor Safety Research to coordinate the needs of the Regulatory Program with the direction of the safety research and development programs. This committee, in turn, recommended that several studies be undertaken to provide guidance for the Reactor and Development projects, and this was, in turn, implemented by the AEC Division of Reactor Development and Technology into the series of discussion reports described here. It was intended that these reports provide a comprehensive assessment of the present status of specific aspects of nuclear safety and that by identifying accepted technology and the technology needing further experimental verification, they would enhance the understanding and confidence in this new industry. Therefore a number of the safety aspects of large light-water-cooled power reactors were selected by the AEC* as subjects for detailed study to reveal whether gaps in knowledge exist where Reactor and Development programs could be of benefit. These subjects cover many of the areas where inadequate factual bases exist or where research that duplicates expected conditions is very difficult. In general, the subjects are in areas considered critical in the safety analysis of power reactor installations. Eight subjects were thus identified, and a state-of-technology type of discussion report was prepared on each. The reports, which are directed primarily toward a technical management audience, generally compare existing or planned plant applications with what is capable of being done at this time. Such comparisons have helped to identify inadequacies in assumptions, available data, or general basic knowledge so that, together with the opinions of experts in a particular field, areas of meaningful Reactor and Development have been identified.

*Letter from Milton Shaw, Director, AEC Division of Reactor Development and Technology, to ORNL, March 28, 1966.

This report is one of the series of eight listed below:

<u>Title</u>	<u>Author</u>	<u>ORNL-NSIC No.</u>
Missile Generation and Protection in Light-Water-Cooled Reactor Plants	R. C. Gwaltney	22
Potential Metal-Water Reactions in Light-Water-Cooled Power Reactors	H. A. McLain	23
Emergency Core-Cooling Systems for Light-Water-Cooled Power Reactors	C. G. Lawson	24
Air Cleaning as an Engineered Safety Feature in Light-Water-Cooled Power Reactors	G. W. Keilholtz, C. E. Guthrie, and G. C. Battle, Jr.	25
Testing of Containment Systems Used with Light-Water-Cooled Power Reactors	F. C. Zapp	26
Review of Methods of Mitigating Spread of Radioactivity from a Failed Containment System	NSIC Staff	27
Earthquakes and Nuclear Power Plant Design	T. F. Lomenick and NSIC Staff	28
Protection Instrumentation Systems in Light-Water-Cooled Reactor Plants	H. G. O'Brien and C. S. Walker	29

Although not specifically one of this series, a related discussion report on reactor pressure vessels, ORNL-NSIC-21, edited by G. D. Whitman, G. C. Robinson, and A. W. Savolainen, has also been prepared at ORNL.

The general approach in the preparation of these reports was to select a primary author-investigator knowledgeable in the subject area and to establish committees of experts to review the work at several stages during its preparation. Review groups were formed both from within ORNL and outside. The external review committee members were drawn principally from other national laboratories, universities, and private research institutes. In some cases, part of the material used was developed and/or written by a subcontractor, who is similarly identified. In all cases, correspondence and/or visits were made to many sources of information, particularly to reactor operators, suppliers, architect-engineers, and public utilities, as well as to the appropriate national laboratories. This wide use of acknowledged experts was made in an attempt to include their opinions and knowledge toward the ultimate goal of achieving, through intensive Reactor and Development programs, well-defined design criteria to insure the public health and safety and to maintain a viable nuclear power industry. In most instances the authors have presented information and expressed conclusions and recommendations that reflect their own judgment and not that of any particular individual or group, such as seismic consultants, the AEC, reactor designers, or utilities, although in some instances the report was modified to accommodate AEC views.

In most subject areas more information was developed than it has been possible to include in the body of the reports prepared in this series. In some instances such information has been included in the appendices, and in other instances this information is included in more technically oriented reports that have been published separately. Two such reports were prepared in conjunction with this study: Some Considerations in Earthquake Resistant Design of Nuclear Power Plants, Holmes & Narver report HN-189 (rev.), and Summary of Current Seismic Design Practice for Nuclear Reactor Facilities, USAEC report TID-25021, prepared by John A. Blume & Associates, Engineers. In addition, it is expected that additional discussion reports will be written on some of the many other safety aspects of large water-cooled reactors, as well as other reactor types as they come into wider usage.

The authors of this report received considerable assistance from three subcontractors. Geotech, A Teledyne Company, provided information and services on the earth sciences, and Herbert Robertson of that company prepared early manuscripts of some portions of the report. Holmes & Narver, Inc. and John A. Blume and Associates, Engineers, who prepared the two reports mentioned above, supplied much of the information in the sections dealing with engineering.

The major part of the work in assembling the information for this report and in drafting it was done prior to 1968. While the essence of the problem discussed here has not changed since that time, there have been developments — in both research and licensing considerations — which will significantly affect the way in which present technological developments will be applied.

Wm. B. Cottrell, Director
Nuclear Safety Program

THIS PAGE
WAS INTENTIONALLY
LEFT BLANK

PREFACE

The Nuclear Safety Information Center was established in March 1963 at the Oak Ridge National Laboratory under the sponsorship of the U.S. Atomic Energy Commission to serve as a focal point for the collection, storage, evaluation, and dissemination of nuclear safety information. A system of keywords is used to index the information cataloged by the Center. The title, author, installation, abstract, and keywords for each document reviewed are recorded on magnetic tape at the central computer facility in Oak Ridge. The references are cataloged according to the following categories:

1. General Safety Criteria
2. Siting of Nuclear Facilities
3. Transportation and Handling of Radioactive Materials
4. Aerospace Safety
5. Accident Analysis
6. Reactor Transients, Kinetics, and Stability
7. Fission Product Release, Transport, and Removal
8. Sources of Energy Release Under Accident Conditions
9. Nuclear Instrumentation, Control, and Safety Systems
10. Electrical Power Systems
11. Containment of Nuclear Facilities
12. Plant Safety Features
13. Radiochemical Plant Safety
14. Radionuclide Release and Movement in the Environment
15. Environmental Surveys, Monitoring, and Radiation Exposure of Man
16. Meteorological Considerations
17. Operational Safety and Experience
18. Safety Analysis and Design Reports
19. Radiation Dose to Man from Radioactivity Release to the Environment
20. Effects of Thermal Modifications on Ecological Systems
21. Effects of Radionuclides and Ionizing Radiation on Ecological Systems

Computer programs have been developed which enable NSIC to (1) produce a quarterly indexed bibliography of its accessions (issued with ORNL-NSIC report numbers), (2) operate a routine program of Selective Dissemination of Information (SDI) to individuals according to their particular profile of interest, and (3) make retrospective searches of the references on the tapes.

Other services of the Center include principally (1) preparation of state-of-the-art reports (issued with ORNL-NSIC report numbers), (2) preparation of the quarterly technical progress review, *Nuclear Safety*, (3) answering technical inquiries as time is available, and (4) counsel and guidance on nuclear safety problems.

Services of the NSIC are available without charge to government agencies, research and educational institutions, and the nuclear industry. Under no circumstances do these services include furnishing copies of any documents (except NSIC reports), although all documents may be examined at

the Center by qualified personnel. Inquiries concerning the capabilities and operation of the Center may be addressed to:

J. R. Buchanan, Assistant Director
(Phone 615-483-8611, Ext. 3-7253)
Nuclear Safety Information Center
Oak Ridge National Laboratory
Post Office Box Y
Oak Ridge, Tennessee 37830

ABSTRACT

A survey was made of the state of technology in the design of nuclear power plants to withstand earthquakes to the extent necessary for safety in order to present technical management with information needed in making decisions on safety research. The principal active contributors to the fields of research on earthquakes are identified. The Federal Government supports much of this research through the Departments of Interior and Commerce and the National Science Foundation.

Present activities in the engineering field and related earth sciences reveal substantial evolution of engineering practice for earthquake survival, which has been stimulated by the occasional occurrence of large earthquakes. It was found that engineering for nuclear facilities developed from the preexisting building codes and engineering practices to a point, today, at which the consideration of public safety generally dictates more conservatism in design.

In this survey the bases for siting and design of nuclear power plants in the United States were studied in consultation with both industry and regulatory personnel. It was found that in all geographical areas nuclear plants are currently designed to withstand safely more severe earthquakes than are considered in the design of other modern structures in the same locations.

A substantial body of knowledge on earthquakes has been accumulated and is being utilized in the siting and design of nuclear facilities. While the probabilities of occurrence of earthquakes of various intensities are not known well in the sense that the incidence of other severe natural phenomena is known, it is generally recognized that all nuclear plants can be expected to be shaken by an earthquake of some intensity. Further, there is little doubt that nuclear plants can be designed to withstand the most severe earthquake vibration with a good margin for safety.

A review of engineering practices shows that the site-sensitive factors that determine the transmission of vibrational energy to plant structures or which contribute to secondary effects, such as slides and tsunamis, are generally recognized as important; but methods for making better quantitative assessment of these factors in siting surveys need to be developed. Where the propensity for adverse secondary effects is recognized the problem can usually be resolved with current and common design practice, although the methods commonly used will be refined and improved by future development of survey techniques.

Surface faulting is a common occurrence in many earthquakes in the western United States and has posed difficulties in siting nuclear power plants in that region. Research in both the earth science and engineering aspects of faulting is recommended, although the current practice is to avoid location of a reactor on an active fault.

Precise prediction of future earthquakes in time and place is not likely to be possible in the near future. This situation is not a serious disadvantage in nuclear plant safety provided the safety consequences of all possible effects on the system are considered early in the design. This provision is the subject of recommendations on engineering and design studies.

1. INTRODUCTION

There are several natural phenomena, such as storms, floods, and earthquakes, that must be taken into account in the planning of human activities. This planning is normally accomplished through systematic consideration of the probability with which each event may be expected to occur, the effects of each phenomenon, and the variation in magnitude of these effects over a specified range of conditions. Because of their potential for great destruction over relatively large areas and their unpredictability in terms of location and time of occurrence, earthquake phenomena are among the natural hazards of primary concern in evaluating potential sites for nuclear power plants and in designing their structures.

As with the introduction of any new technology into society, nuclear power superimposes certain man-made benefits and risks upon those presented by the natural environment. The risk of primary concern for nuclear power plants is that of exposing the general public to hazardous levels of radiation. As related to earthquakes, the concern is that of releasing radioactive fission products from the reactor as a secondary result of the damage that could be caused by an earthquake. This problem requires consideration of not only those very large earthquakes, which would be catastrophic in themselves, but also the effects of even moderate nearby earthquakes that might also lead to significant nuclear hazards in some circumstances.

The effects of an earthquake are felt by every feature of a structure. In the case of a nuclear power plant, the possibility of multiple failures leading to fission-product release dictates more extensive evaluation of the seismic properties of the site and of the plant design and much more conservative design features than would be appropriate for a fossil-fueled plant.

Evaluation of reactor sites includes consideration of the seismology and geology of the region in which the plant is located and the properties of the earth material underlying the plant. Earthquake-caused vibratory ground motion, seismically generated water waves, and surface faulting must all be considered both as they affect the plant and the stability of the material underlying it.

The procedures for design of structures to withstand the effects of earthquake phenomena, referred to as earthquake engineering, were developed principally for application to civil structures, such as buildings, bridges, and towers. Seismic evaluations made in accordance with codes such as the Uniform Building Code (UBC) of the International Conference of Building Officials are developed in a manner that makes some indirect provision for the actual dynamic response of a structure without attempting to carry out a rigorous analysis. In the case of a reactor, of course, the functions of various mechanical, electrical, and electronic devices, some of which can be of crucial importance to reactor safety, must be also considered.

Experience in major earthquakes with UBC-designed buildings has shown that although the structures have generally maintained sufficient integrity to minimize loss of life, very severe damage in terms of property loss may occur. For this reason, more sophisticated design methods are frequently appropriate for structures where large capital investments are involved, such as office buildings and all types of power plants. Sophisticated design measures are also employed where property damage can lead in a secondary manner to public hazard, as is the case for nuclear power plants, either from fission-product release or from loss of availability of electric power.

For these reasons the impact of earthquakes directly on nuclear plants and indirectly on public safety must be considered more broadly than in the conventional context of earthquake engineering. And, since nuclear power plants require the consideration of many effects that have had little previous significance in earthquake-related phenomena, it is important that the complete spectrum of these phenomena be correlated with current and feasible technical practices.

This report discusses earthquakes and nuclear plant design principally on the basis of what the earth science and engineering professions were doing about earthquakes prior to 1968. The earthquake-related phenomena of shaking, fault movement, and tsunamis are discussed from two points of view: (1) incidence and quantitative characterization and (2) engineering practices and capabilities with respect to each phenomenon. The

appendices present a glossary of terms and background information, including data on U.S. earthquakes and site evaluations.

2. SUMMARY

Faults, vibrations, and tsunamis are the major earthquake-induced phenomena to consider in reactor siting and design. Thorough geological engineering investigations of potential reactor sites are usually made first, and ground motions, tsunami runups, and displacements are described and defined. Then, when the site has been found to be acceptable, the structures are designed accordingly. There is currently, however, some uncertainty in establishing the expected earthquake effects, as well as in the structural and mechanical design parameters for some sites, and therefore special research and development efforts are required to insure the economic and safe development of nuclear power in all parts of the country.

When an earthquake moves the ground, everything supported by it soon must conform to this movement. Allen, a seismologist, has said that engineers can design for any specified earthquake better than the geologist can tell him what the earthquake is likely to be.¹ The consensus among engineers is that if the excitation and fault slippage are defined, a design can be made to accommodate them, although not necessarily economically. As discussed in this report, detailed calculations are required to design a reactor plant to survive its specified design earthquake safely, and the plant must be carefully constructed and tested.

Until the advent of power reactors, much of the advance in quality of earthquake engineering came from individuals who understood the seriousness of the problem.² In many industries, earthquakes are still not a cause of concern in the design of structures or equipment. No nation has, in fact, given adequate support to the development of strong-motion science and engineering commensurate with the loss that could result from poor siting, design, and construction. Since intense earthquakes are rare, it has been the good fortune of modern urban man that few great cities have suffered this disaster. Although the sizes and numbers of great urban areas are increasing rapidly, the essential problem with respect to earthquakes is the quality of structure,³ not population concentration. Unlike volcanoes, tornadoes, and hurricanes, which are now amenable to warning

systems, earthquakes rarely give any warning that can be heeded. However, both conventional structures and power reactors can be built with excellent earthquake survival capabilities solely by carefully applying the best of both current dynamic design and construction practice.

Original building code criteria for the accommodation of earthquakes were very crude, but the best of them now take into account, although not usually explicitly, dynamics, yielding, and even ductility. Much of the earthquake engineering design of conventional structures throughout the world stems from early work in Japan.² Earthquake engineering in the United States may be said to have begun following the 1925 Santa Barbara earthquake, with the first mandatory earthquake code enacted following the 1933 Long Beach, California, earthquake. The latter was the first earthquake in the U.S. for which accurate measurements of strong motion were made.³

Actual structural designs of reactors to survive earthquakes are based principally on a very few strong-motion accelerograph records collected with much care by the U.S. Coast and Geodetic Survey. In view of this designers are expected to make highly conservative estimates of ground motion to use as the basis for plant design. Data on several strong earthquakes are listed in Table 2.1. Additional future strong-motion measurements, including those almost certain to be made in Japan, will significantly influence design bases. Actual experience with large modern structures in great earthquakes is largely limited to that in San Francisco in 1906, Tokyo-Yokohama in 1923, Alaska in 1964, and Caracas in 1967. Apparently no structure with response characteristics similar to those of a power reactor building has ever been shaken in any intense earthquake; certainly the postearthquake inspection of such a structure has never been reported.

Before World War II seismic design was based almost entirely on empirical information gained from inspection of buildings that had been subjected to earthquakes. Earthquake intensities, that is, the degree of shaking at a specific place, were estimated in a crude fashion from examination of building damage. Such empiricism is presently the main basis for the seismic design of many conventional structures. Although intensity ratings (see Chap. 3, Fig. 3.4) are so subjective that some specialists

Table 2.1. Records of Typical Strong Earthquakes

Earthquake and Date	Richter Magnitude (surface waves)	Approximate Relative Total Energy	Measured Maximum Horizontal Acceleration (g)	Comment
Parkfield, California, in 1966	5.5 (Ref. 4)	1	0.5 ^a	Largest recorded strong motion; it is not known whether these unique records collected near a fault trace are typical ⁵
El Centro, California, in 1940 (Ref. 7)	7.1 (Ref. 6)	210	0.33	Largest magnitude with recorded strong motion
Assam, India, ^b in 1897	~8.7 (est., Ref. 6)	25,000	Not measured	Perhaps the most intense great earthquake for which there is a detailed report
Chile in 1906 and Japan in 1930	8.9 (Ref. 6)	43,000	Not measured	Largest recorded magnitude
Extrapolated maximum magnitude; possibly Lisbon in 1755 (Ref. 8)	9.25	110,000		There is a wide range of expert opinions on what are the maximum possible magnitudes and intensities

^aThe 1949 Imaichi earthquake is estimated to have reached an acceleration of 0.9 g.⁶

^bRichter says that there are ground accelerations equal to the acceleration of gravity or greater in the meizoseismal area of the greatest earthquakes.⁶

would like for them to be abandoned, rating scales such as the Modified-Mercalli (MM) scale, which is the one most often used, are necessitated by the scarcity of strong-motion accelerometers and their records.⁶ Design bases for most localities in the U.S. are dependent on records of Modified-Mercalli intensity and thus are lacking quantitative support. Progress toward greater use of quantitative estimates of intensity is limited by the small number of strong-motion instruments in use. Benioff and Housner have suggested measuring intensity in terms of maximum structural motion rather than ground motion.⁶

One of the principal engineering problems is to determine how to normalize the existing records of strong motion for application to sites where only magnitude and intensity data, at best, are available. There is no widely accepted correlation of measurements of magnitude and intensity, and these are not easily correlated with maximum acceleration. Herschberger collected 589 records from a total of 189 earthquakes and selected 108 of these from 60 earthquakes for a linear regression calculation of the maximum acceleration on the reported MM intensity.⁹ Since there was, for example, a 50-fold range in maximum accelerations just for intensity V, it is apparent that there is poor correlation between maximum acceleration and intensity.

Power reactors must be designed for specific sites, and earthquake considerations must originate with geoseismologic studies that may inherently not allow exact bases. It is important that the geologist and seismologist make detailed, complete reports on the selected site as early as practical so that the design engineers can have the information for application at the outset of design. The paucity of specific information for a particular site often necessitates comparison with other locations with better earthquake records, and from this must come numbers adequate for design. Reasoning by this process may well be the best of the current methods of evaluating sites, but additional complementary methods are needed.

There have been several approaches used in assigning design-basis earthquakes* to reactor sites. The current method for determining the

*See Glossary (App. A) for definition of design-basis earthquake.

maximum ground motion at reactor sites consists, in general, of evaluating strong ground motions originating from small nearby shocks, as well as large earthquakes. It is commonly assumed that the largest earthquake that may affect the area will be no larger than the maximum-magnitude shock, recorded or estimated, that occurred in the general vicinity of the site during historic time, unless the study of nearby geologic structures indicates that to account for an observed Quaternary dislocation related to the structure a larger earthquake must have occurred. If an active fault lies relatively near the proposed site, it is assumed that earthquakes up to a size commensurate with the length of the fault could occur along it. Ground motions from expected small shocks nearby (the magnitudes of which are also based on the historic record) would be dominant in establishing the maximum ground motion at the site only if epicenter locations of past large shocks and active faults were located at such great distances from the site that their potential effect would be relatively small.

Maximum accelerations of the ground motion may be estimated from Modified Mercalli intensity values or by calculating acceleration at the site based on accelerations recorded at other sites and appropriate transmission models. The effects of rocks and soil conditions on acceleration amplification must also be considered. By translating the maximum ground acceleration to equivalent average spectra curves, or perhaps to the area under the spectral curve of a single recorded shock, a design earthquake for establishing a basis for accommodating ground motion can be recommended. In the eastern United States, the problem of establishing a design-basis earthquake is particularly difficult because the large earthquakes that have occurred there have not been clearly related to surficial geology.

In the usual AEC practice for siting nuclear power plants the operating-basis earthquake (the earthquake assumed as the basis of design of features of the plant that are not related to safety) must be equal to at least one-half the design-basis earthquake in acceleration. The design must be investigated for assurance that there would be no loss of safety function in the event of the larger excitation, which has meant in practice that the containment and other important structures and mechanical systems have been designed to survive this maximum design earthquake. The AEC

regulatory staff emphasizes the proper selection of the larger earthquake for which safe shutdown must be assured.

A major difficulty in establishing a rational design basis is that all the data available are far too few to establish practical probability levels for designated observed or estimated intensities. Design values have been generally suggested for what some specialists consider the most hazardous regions of the world to correspond to accelerations as high as 1.0 g, velocities of up to 6 ft/sec, and displacements of up to several feet.¹⁰ Oldham claimed a velocity measurement of 14 ft/sec for the 1897 Assam earthquake and that some stones were hurled through the air to a measured distance of up to 8.5 ft.¹¹ Newmark¹⁰ suggests a "standard" earthquake approximately 50% larger than the 1940 El Centro record; that is, a maximum acceleration of 0.5 g, a maximum velocity of 24 in./sec, and a maximum displacement of 18 in.

It would be difficult to overemphasize that the best practices of earthquake engineering, if given early consideration and preceded by both a proper geophysical investigation of the site and geological investigation of the region, could greatly influence original design choices in a beneficial way at negligible cost. For example, careful component placement that resulted both in symmetrical dynamic loading and minimum rotational displacements could greatly add to a reactor structure's capacity to allow for safe shutdown in an intense earthquake.

Previous engineering of structures of the heights of those for reactors has seldom required sophisticated design approaches. Most reactor structures, except the stacks, have dominant response periods of vibration shorter than those of maximum amplitude in the earthquake spectrum, whereas for high-rise buildings the maximum response and earthquake periods coincide. The fundamental considerations and philosophies of design of the gross structures in the two cases are, despite this difference, closely related. Suspension bridges, arch dams, and nuclear reactors have been considered in a special class by engineers, for until the present very few such structures have undergone significant excitations, and since they all can have several natural periods of vibration that are not well separated, they present special problems in analysis.¹²

American power reactor manufacturers have in some cases imposed more stringent earthquake criteria than required by foreign nations in which reactors are located. There appears to be agreement in the U.S. that power reactors should be designed for the "worst possible" conditions, but specific estimates of what these worst possible conditions are vary greatly among specialists.

Nuclear power plants have many types of structures and devices that are depended upon for adequate performance of normal and safety functions. Many of these features are common to nonnuclear facilities that have been designed for earthquake survival or have experienced earthquakes. Dams, building foundations, building structures, tanks, stacks, and towers are in this category. Other features, such as piping, electrical equipment, and control equipment, have received less attention in ordinary earthquake engineering. These and unique features, such as nuclear fuel, are depended upon for vital safety functions that have no nonnuclear counterpart. The earthquake phenomena of primary interest are ground shaking, faulting, and tsunamis.

2.1 Shaking

When rocks rupture or move suddenly within the subsurface, earthquakes are generated and shock waves spread out in all directions. These waves, upon reaching the earth's surface, may be so small as to be discernible only with sensitive instruments, but they may also range up to catastrophic proportions. Great earthquakes, such as the Alaskan earthquake of 1964, generate violent motions that may be felt over thousands of square miles and may persist for up to several minutes in duration. The larger earthquakes then are of especial interest in the siting and design of power reactors. The instrument commonly used to record the significant properties of severe shaking is the strong-motion accelerometer, which generally records useful data over a range of 50 to 100 km. Since 1932, when these instruments were first put into use, ground accelerations have been recorded for a relatively large number of small shocks, but only a few large earthquakes have been recorded on strong-motion instruments. The characteristics of these ground accelerations have been thoroughly analyzed, and this

information is commonly used, along with other seismic and geologic data, for estimating the amount and nature of the expected strong ground motions at other sites. Because of the general lack of instrument records of strong ground motions and since there is no suitable theoretical basis for predicting these motions, there are no good quantitative ways to determine precisely the accelerations, velocities, and displacements of the ground motions for a given earthquake at most reactor sites. Thus conservative estimates must be made of the important design parameters of ground motion.

Earthquake-induced vibrations in soils are usually stronger than those in hard rock; however, it is not possible to state precisely the magnitude of the difference. Most studies that relate the intensity of shaking in various types of rock and soil have been conducted with small shocks, and there is no satisfactory technical basis for extrapolation from small to large shocks. Recently, through determinations of the dynamic properties of soil sections and theoretical considerations, important relationships have been observed between some soil conditions and ground-motion characteristics; however, additional studies are needed before reliable predictions can be made for the great majority of soil conditions. In addition to their amplification and/or attenuation properties, the behavior (consolidation, differential compaction, sliding, and liquefaction) of soils under earthquake-induced ground oscillation must also be determined. In general, those sites have been avoided in the past where soil failures might be expected to occur. However, since potentially susceptible materials (soft alluvial and unconsolidated materials) underlie many stream and river basins, which are highly desirable locations for reactors, the critical soil properties of future reactor sites must be thoroughly evaluated either to provide assurance that they will not fail or to determine the design parameters for withstanding any expected failures.

The induced effects of shaking, such as slides and liquefaction, require careful attention in design of plants. Slides have been considered in some cases with respect to both on-site disturbances and off-site disruption of power or water supplies, but the record of such studies is sketchy. The potential for liquefaction is recognized for certain types

of noncohesive saturated soils; however, there are many borderline cases for which specialists disagree on whether liquefaction could or could not occur in an intense earthquake. Experience with pilings in earthquakes suggests that their design may also depend critically on the liquefaction phenomenon.

The reactor plant consists of massive structures whose response depends largely on the dynamic properties of the supporting material. The effect of soil-structure interaction is in practice usually assigned as a damping value on structural response and can have a large influence on the design. Some computer modeling of the interaction is now under way, but reliable measurements of massive structures on a variety of supports either in earthquakes or during full-scale tests with low-level induced accelerations may ultimately be needed to verify these analyses.

Methods for designing nuclear plant structures for a given or assumed ground motion include use of equivalent static load factors, response spectra, and time-history input ground motion. Each approach can be used satisfactorily for limited applications in structural analysis. The simpler analyses become inadequate as excitation levels are increased, either from the economic burden of overconservatism or from the neglect of important effects; for example, excitation of high vibrational modes and inelastic behavior of materials. Of practical significance at present is the question of the accuracy with which any method is applied, and this can be evaluated only by examination of details of calculations for the particular cases of each specific structure.

Application of earthquake engineering to reactors has developed around an objective of safe shutdown capability during and after the occurrence of a design-basis earthquake agreed to between the AEC and the designer. Designers have usually specified that structures, such as the containment vessel, shall not be stressed beyond the elastic range in this earthquake. While there is substantial experience in designing structures in this way, many of the functional requirements for safety in reactor plants are not satisfied merely by designing to keep within the limits of elastic response of structures. The proper functioning of instrumentation systems depends upon more than the ability to respond elastically. Therefore conventional

earthquake engineering design practice needs to be supplemented by vibration analysis or testing for items such as piping, machinery, electrical systems, electronic equipment, and electromechanical systems.

There is little doubt that all important components of the reactor plant could be analyzed and designed to function properly during the vibrations induced by earthquakes. Since many mechanical and electrical systems are continually in service in a reactor and are presumed in safety assessments never to fail, at least these should be thoroughly analyzed and provided with components that will function properly under earthquake excitation. Problems of vibration of instrumentation and controls should be considered by specialists who design such devices to assure that characteristics of ground motion important to this field but not necessarily to structural design are included in the design bases for nuclear plants. Furthermore, applying the same design bases to controls as those currently applied to structures does not provide a margin of safety against malfunction of controls.

Since design-basis earthquakes* nominally take some portions of reactor structures to the yield point, behavior of structures and materials undergoing nonelastic vibratory motion must be understood. The practical advantage of specific design for performance in the inelastic range is a matter of controversy in the reactor field. Of the many areas requiring work, the behavior of steel and reinforced concrete, including prestressed-concrete elements and joints, under reversing loads into the yield range must be investigated further. Energy adsorption, structural period changes resulting from yielding, and various ways in which failure can occur are all of much importance. There is presently too little known of strength, behavior, and failure mechanisms of reinforced-concrete members to permit widespread inelastic design. In reinforced-concrete containment vessels the shear capacity of diagonal bars and the shear distribution between vertical or horizontal and diagonal reinforcing need more investigation.

Application of prestressed concrete to nuclear plants requires study of many conditions of seismic loading, internal containment pressure, and

*The larger of the two design levels (see Glossary, App. A).

yield conditions; for example, it may be necessary to design prestressed concrete so that it has additional reinforcement for yielding to prevent local failure. Little field experience is available on modes of earthquake-induced failures of prestressed-concrete structures of the type that might be used for reactors, although there are few portions of large reactor structures that are not being vigorously assessed for application of prestressed concrete.

Damping values can have an important influence on design. Many aspects of reactor plant structural damping require investigation, with the initial objective being to supply designers with urgently needed specific source data. Compared with the need, very little experimental information about damping is available, even for conventional structures. Although information on structural damping is one of the main areas of need, research should not be confined to conventional structures. Equipment, piping (especially long runs), and pipe mounts need to be studied. Furthermore, little is known about damping of prestressed concrete. Likewise little experimental information has been obtained for massive sections of reinforced concrete under reversed loads up to and beyond yield conditions.

Seismic scrams and monitoring systems are usually not proposed for power reactors, since utilities generally consider it preferable to continue operations during earthquakes. The development of specific design limitations for various important parts of the plant may provide a basis for considering both seismic scrams and postincident inspection. Strong-motion accelerometers strategically placed in reactor structures, as well as in the nearby soil and rocks, could provide extremely important information regarding specific characteristics of strong ground motion, as well as the response of the structure to these forces should large earthquakes be felt at reactor installations. Techniques for measuring strain and tilt changes in seismic belts might also be expected to provide data on local seismicity, and records from seismographs and strong-motion instruments located at prospective reactor sites would be useful in designing automatic shutdown systems, since they could provide information on wave amplitudes of nearby earthquakes. However, there are differences of opinion among specialists as to the advantages and disadvantages of seismic instrumentation in nuclear plants, particularly for use in seismic scrams.

Since each mode of vibration of a structure has its particular damping, it would be difficult if not impossible to gain full confirmation of design in any way other than full-scale excitation. Small-amplitude excitation of full-scale plant structures appears feasible and should yield damping values that are conservative. Shaker technology for excitation of large reactor structures to large amplitudes is not and apparently cannot become available without radical advances. A study should be made of dynamic shaking of full-scale reactor plants to determine whether the required huge excitation machines could be built to induce strong-motion response. One possibility is a massive rotary shaker driven by a solid propellant rocket.

Tests of smaller components under vibration are feasible. The Japanese have used this technique, which is the only practical way of indicating the proper designs of some delicate or complex motion-sensitive mechanisms.

Reactors are currently being designed for maximum ground accelerations ranging up to 0.5 g. In some cases the differences from site to site arise from considering the earthquakes of historic record that occurred in the general vicinity of the site or along a nearby geologic structure. In many instances, geologic evidence is strong enough to dictate a higher design earthquake than is indicated by historic records alone. Although the records of seismic events of low magnitude indicate wide differences in general seismicity among various regions, the statistical basis for distinguishing between regions with respect to events of large magnitude is comparatively poor. This situation requires that conservative assumptions be used in making siting judgments.

Two studies are suggested. First, the probability distributions of seismic events of various magnitudes within major geologic regions should be derived from historic records for the purpose of determining whether these records provide an adequate basis for the distinctions being made. Secondly, the comparative costs of designing for various earthquakes in this range of accelerations should be determined.

2.2 Faulting

The exact mechanism or mechanisms for generating earthquakes are not known, but it is generally agreed in this country that faulting whereby there was relative displacement between adjacent rock masses was the cause of most of the large shallow earthquakes in California and in other tectonically similar areas of the world. For the continental United States, historic surface faulting has been largely confined to the area west of the Rocky Mountains, and there is good reason to believe that faulting in the foreseeable future will continue to be restricted primarily to this general region.

Through appropriate investigations, the approximate lengths, widths, and directions of movement of major through-going faults may be ascertained, along with very approximate estimates of the extent and amount of surface displacement that may be expected with a specific-magnitude earthquake along the fault. Because of the scarcity and incompleteness of records for surface breaks and because of the complexities and general lack of understanding between the surface expression of faulting and the state of stress-strain within the rock and soil overburden, it is not generally possible to define and determine with a high degree of confidence such things as the exact location and extent of all breaks within fault zones, the time-history of the movements, and the propagation of the breaks. However, it is not necessarily compulsory that all parameters of faulting be precisely identified in order to construct and operate power reactors safely and economically in regions of the country where surface faulting is likely to occur. Nevertheless, without further knowledge of the important characteristics of surface faulting, it is imperative that in some areas extreme conservatism be used in the siting and design of reactors to circumvent or withstand these forces.

Existing reactor power plants and those presently under construction are not designed with any special features for accommodating fault movement under the plant; the aim has been to avoid locations where it is expected that faulting would occur. However, not all future sites will be necessarily free of the possibility of some faulting.

A few earth-filled dams have been designed for specified fault conditions; however, no record of comparable rigid structures has been found. It is believed by some that reactor plants can be designed to survive some fault movement, and conceptual design studies have been made for the purpose of identifying some of the details of the problem that need further consideration. While it appears likely that nuclear plants of current design could survive in good order a very few inches of movement with little or no modification, large displacements (e.g., those above 10 ft) are so far removed from engineering experience that sites with such potential conditions should be avoided.

Designs proposed and the studies conducted suggest that a plant carrying all important equipment on a single foundation mat would be a feasible approach to accommodating a few feet of displacement. Such a concept would have to be carried through further definition of design conditions, detailed design, and testing of novel features before an evaluation could be made of its economics and safety.

2.3 Tsunamis

Tsunami and tsunami-generated oscillations are potentially dangerous to reactor installations at coastal sites, since they may cause damage to the reactor plant and water intake structures through runups and/or withdrawals. Most tsunamis are thought to be generated by vertical displacement of the subsea bottom. About 60% of all recorded tsunamis originate in the Pacific Ocean, where large earthquakes occur along deep, bordering trenches. Runup heights in Japan, Kamchatka, Chile, Peru, Alaska, and the Hawaiian Islands have often exceeded 30 ft; however, during historic time, the west coast of the United States has not been subjected to runup heights greater than 16 ft, with one possible exception — the reported, but since refuted by some, 50-ft-high tsunami runup at Gaviota, California, associated with the large off-shore-centered earthquake of December 21, 1812. The relatively low runup along the west coast of the United States is attributed to the coastal shelf. In contrast to the experience on the west coast, damaging tsunamis have not been recorded along the Gulf and eastern coasts of the United States during historic time. The chances of

a tsunami being generated by a local disturbance off the west coast are thought to be limited, since there have been no locally generated damaging tsunamis in the past, with the possible exception of the tsunami at Gaviota.

Prediction of maximum runup height on the basis of historical data alone is difficult at many sites, since differences in relatively obscure coastal and submarine features can cause significant differences in runup. The best guideline for prediction along the west coast appears to be previous experience with tsunamis, since some sites have consistently high waves, whereas most locations have consistently low waves, regardless of the direction of approach. With the exception of the March 1964 Alaskan tsunami, which caused runups as high as 16 ft at Crescent City, California, recorded runups along the west coast have not exceeded 10 ft.

Studies of wave records and runup measurements, coupled with geological and seismological studies, are currently used to evaluate the effects of tsunamis near coastlines and the sensitivity of particular sites. To improve on these methods for evaluating the tsunami hazard, theoretical and experimental models of tsunami generation and effects on various coastal configurations are needed, along with more recorded data of actual runup and withdrawal at various sites.

The present approach in reactor design for tsunamis is to assure that the wave defense of the shore or island has the capacity to withstand the design-basis waves. Cooling-water structures located offshore are given particular attention. Inundation of some plant auxiliary systems is sometimes considered, since this problem may arise from floods and wind-driven waves. Equipment essential to safety is necessarily protected from such events. Placement of the entire plant where complete inundation is a consideration has not been attempted. If this were contemplated, careful evaluations based on hydrodynamic calculations and model studies would be required.

References

1. C. R. Allen, Aseismic Design and Testing of Nuclear Facilities, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, June 12-16, 1967 (undocumented).
2. F. Neumann, Engineering Seismology, in *Reviews in Engineering Geology*, edited by T. Fluhr and R. F. Legget, p. 161, Geological Society of America, 1962.
3. N. H. Heck, *Earthquakes*, Princeton University Press, Princeton, New Jersey, 1936.
4. T. V. McEvilly, W. H. Bakun, and K. B. Casaday, The Parkfield, California, Earthquake of 1966, *Bull. Seismol. Soc. Amer.*, 57(6): 1221-1244 (December 1967).
5. S. T. Harding, W. Reinhart, and W. K. Cloud, The Parkfield, California, Earthquake of June 27, 1966, U.S. Government Printing Office, Washington, D.C. 20402, 1966.
6. C. F. Richter, *Elementary Seismology*, W. H. Freeman and Company, San Francisco, 1958.
7. C. R. Allen et al., Relationship Between Seismicity and Geologic Structure in the Southern California Region, *Bull. Seismol. Soc. Amer.*, 55(5): 753-797 (August 1965).
8. G. W. Housner, Reply to Discussion III - 113, Third World Conference on Earthquake Engineering, New Zealand, 1965.
9. J. Herschberger, A Comparison of Earthquake Acceleration with Intensity Ratings, *Bull. Seismol. Soc. Amer.*, 32(3): 171 (June 1942).
10. N. M. Newmark, Design Criteria for Nuclear Reactors Subjected to Earthquake Hazards, Draft of paper for IAEA Meeting, Tokyo, 1967.
11. R. D. Oldham, Report on the Great Earthquake of 12th June 1897, *Mem. Geol. Survey India*, 29: 1-379 (1899).
12. K. V. Steinbrugge and V. R. Bush, Earthquake Experience in North America, 1950-1959, *Proceedings of Second World Conference on Earthquake Engineering, Japan, 1960*, pp. 381-396.

3. GROUND SHAKING

Ground fracturing may be more critical to the siting and design of reactors than earthquake-induced ground shaking (e.g., movement without fracturing) in those areas where active faults are recognized or suspected, since in general shaking can be more easily accommodated in design than faulting. Nevertheless, ground motion can be expected to some degree at every site in the U.S., whereas surface faulting is likely to occur only in a few specific locations. Unlike surface faulting, which is generally associated with shocks having Richter magnitudes of about 6.5 and greater and is restricted to regions of high seismic activity, ground motions are generated by all magnitudes of shocks (including very small ones) that occur over all parts of the earth's crust. Too, various types of soil failures and oscillations of surface bodies of water commonly occur as a result of these ground-shaking motions. In general, only the larger shocks generate ground motions that would require special considerations in reactor design, and thus the areas of large-magnitude seismic activity are of primary concern. However, extremely large areas covering thousands of square miles may be severely shaken during strong earthquakes. The corresponding areas affected by surface faulting for these large shocks are, of course, quite small.

The vibratory effects of earthquakes have been observed, studied, and documented to some extent throughout historic times, but not until 1932 was the first instrument installed within the U.S. that was designed to measure and provide engineering data on earthquake-induced ground motions. Since this rather late beginning of "engineering seismology," many advancements have been made in methods and techniques for recording ground shaking and for the analysis and application of these records for structural design. In addition, through studies of nonearthquake-induced or forced vibrations, various analytical techniques have been developed that are useful in all structural designs to withstand ground motions; nevertheless, more basic data on earthquake-induced ground and structure shaking are needed to attain optimum economy and safety in the siting and design of reactors. This chapter summarizes the state of the art with respect to ground shaking and the engineering design of structures to accommodate it,

and two supplemental documents prepared in conjunction with this study present detailed information on current seismic engineering practice as applied to nuclear power plants.^{1,2}

3.1 Generation of Ground Motions

The cause or causes of earthquakes and, hence, earthquake-induced ground motions are varied and may never be precisely known. However, it is generally believed that tectonic forces in rocks build up elastic strains of such magnitude in adjacent rock masses that the strength of the rocks is exceeded or the frictional resistance along preexisting faults is surpassed, and fracturing or rupturing occurs that releases energy in the form of elastic waves that subsequently propagate to the surface. Thus ground vibrations probably result from rock ruptures at various depths. In general there are three types of earthquake (shock) waves, namely, *P* or sound waves, *S* or shear waves, and *L* or long waves. *P* waves travel at about 3 1/2 miles/sec and are the first to reach the surface, while *S* waves, which are generally considered to be the most destructive because of their shearing motion, move about 2 miles/sec. *L* waves are relatively slow-moving surface waves having long periods that commonly cause the swaying of tall buildings. Since these three general types of waves arrive at the surface at different times, the duration of the shaking motion is greater than that of actual fault movement. At greater distances from the source, the motions may be caused not only by the primary waves but also by reflected and refracted shock waves that tend to add more complexity to the ground shaking.

3.2 Recording of Ground Motions

The distribution in time and space of seismic events is recorded by an extensive worldwide network of standardized seismograph stations.³ The instruments at these stations are extremely sensitive to small vibrations and can detect earthquakes occurring over the entire earth's crust. Thus ground deflections of only a few millionths of an inch from shocks occurring hundreds or thousands of miles away may be recorded. However,

strong motions, which are of primary importance for structural design, cannot be adequately monitored by these instruments. Perhaps the most common strong-motion recording instrument in the U.S. is the strong-motion accelerograph, which records true ground acceleration.

3.2.1 Instruments

The strong-motion accelerograph (Fig. 3.1), which provides information on the manner in which acceleration varies with time, is probably the most important instrument developed for engineering seismology.⁴ This instrument records ground acceleration directly, and from this record the ground velocity and displacement may be calculated (Fig. 3.2). Spectra of ground motion calculated from the record of accelerations are extremely important in engineering design, since they provide information on maximum

Fig. 3.1. Strong-Motion Accelerograph, Model RMT-28, Produced by Earth Sciences, a Teledyne Company. Photograph by J. L. Hawkins, Pasadena, Calif.

Fig. 3.2. Record of Ground Acceleration and Integrated Corresponding Velocity and Displacement, Kern County, California, 1952. (Reprinted with permission of Prentice-Hall.)

acceleration at various frequencies of interest. The importance of the records of these instruments in a seismic structural design is exemplified by the fact that in regions of high seismicity, present building codes, such as the Los Angeles code, require that strong-motion accelerographs be installed in many buildings over six stories high — in the basement, mid-portion, and near the top of the building. Indeed, one of the greatest

problems in a seismic design is that few strong-motion records are available for many recent major earthquakes. For instance, there are no strong-motion records for recent earthquakes in Mexico (1957), Chile (1960), Agadir (1960), Iran (1962), Skopje (1963), Alaska (1964), and Turkey (1966 and 1967).

By installing strong-motion accelerographs on the ground surface at reactor sites, detailed information on the important ground-motion characteristics of any ensuing strong shocks can be obtained. In addition, it would be extremely desirable to install similar instruments at various levels in reactor structures and perhaps on critical systems and equipment to record structural behavior precisely during large earthquakes. These data would be invaluable in aseismic design of future plants.

Commercially available accelerographs employ three-component accelerometers with natural frequencies of about 15 cps (0.066-sec period) for sensing accelerations during strong local seismic disturbances (Fig. 3.1). A seismic shock, if greater than some preset intensity, causes the seismic switch or horizontal pendulum starter to make electrical contact and start the light traces and the drive mechanism for the photographic paper that records the acceleration output of the accelerometers. The recorder is programmed to continue automatically for a period of several seconds following the last contact of the actuating pendulum. Strong-motion accelerographs cost \$1000 to \$4000, and their complex nature requires that they be serviced every few months. A less expensive and more simply constructed device, termed the seismoscope, was developed in the 1950's. This instrument, which records strong earth motions mechanically by means of a scribe on a smoked watch glass, is used mainly in arrays with strong-motion accelerographs, since it provides indications of maximum accelerations.

In today's terminology the word seismograph implies an instrument whose response corresponds more to ground-vibration amplitude or velocity than to acceleration. To this end, the pendulum system of a seismograph has a much longer natural period than that of an accelerograph. Seismometers having natural periods near 1 cps could be used to operate shut-down systems, or they could monitor earth motions continuously at some specific level of sensitivity or at several levels of sensitivity. A

recording system or seismograph includes a seismometer, amplifier, timing system, test and calibration circuits, and recorder. In the last five years, seismograph components and seismic systems engineering have been advanced greatly as a result of the VELA program, a government program directed toward the detection and identification of underground nuclear explosions. Records from seismographs located at prospective reactor sites could be extremely useful in developing reactor protection systems, since they could provide information on wave amplitudes of nearby earthquakes. The seismographs could also be incorporated into reactor control systems.

Seismometers responding to vertical and horizontal motions are commercially available for installation on reactor foundations or in boreholes. According to Press and Bruce, sophisticated systems consisting of an array of sensors in surface vaults or boreholes have been operated in the VELA program.⁵ Short-period triaxial seismometers, which record vertical and horizontal components of motion, are now available for the borehole systems, along with their associated amplifiers, filters, attenuators, recorders, and digitizing equipment, if desired. If continuous operation is not desired at any time, a seismograph can be put into a standby mode; the amplifiers and recording equipment can then be made operational by a seismic switch or circuit.

3.2.2 Records

As of January 1966 there were only 138 strong-motion accelerometers in service in the U.S.⁶ To provide strong ground motion records, strong-motion accelerometers, which record useful data generally only over a radius of about 50 to 100 km, should be located relatively close to surface faults or seismic zones, and a very large number of these units would be required to obtain strong-motion records for the majority of the larger shocks. Ground acceleration has been recorded for many earthquakes, but good records for destructive ground accelerations have been obtained for only four large earthquakes in the U.S.⁷ The characteristics of the ground motions at the sites where these four shocks were recorded have been determined, but there are many situations that have not been evaluated that have various other combinations of earthquake intensities,

soil and rock conditions, distances from the shocks, etc. for which basic data are needed.

Records of ground motions associated with small shocks are much more numerous than those associated with large shocks because of the higher frequency of occurrence of smaller shocks. However, it is difficult to correlate small ground motions with large ones. It is well known that the frequency spectra and other characteristics of large and small shocks are significantly different. Numerous instrumental recordings of small earthquake ground motions have shown that ground accelerations in basement rocks are amplified many times in soft soils; but since much more energy is absorbed by the soil during strong motions, these amplifications in soil would not be expected to be as great during large earthquakes. It is also generally agreed that there is little conclusive evidence to indicate significant correlations, at least on a local scale, between the frequency of occurrence of small shocks and large ones. Indeed, many investigators feel that large shocks are independent of small shocks. Furthermore, based on energy relationships and historic evidence, it is suggested that small earthquakes do not act as "safety valves" to release energy that might otherwise be released in major earthquakes and that small earthquakes only incidentally indicate strain accumulation on a regional scale.⁶ Thus, even though the study of small earthquakes may provide a means for obtaining data on the seismicity of a given area in a rather short period of time, this information may not be useful in forecasting and determining the characteristics of larger earthquakes.

3.3 Intensity of Ground Motion

Since structures must try to follow the rapid changes in ground movement during an earthquake, acceleration is in general the most important characteristic of ground shaking and thus is of primary interest in design considerations. However, maximum acceleration is not in itself a precise measure of the damage that may be caused to a structure. The dynamic characteristics of the structure and the strength of its members must also be known, as well as the frequency characteristic of the ground motion. In a study of the effects of vibrations, it is desirable to

consider displacement, velocity, and acceleration; a good-quality recording of one (displacement, velocity, or acceleration) can be converted to the others by differentiation and/or integration processes. Thus accelerograms of earthquakes, when corrected for all possible sources of error, provide the basic data needed for design purposes.

The ground motion accompanying earthquakes generally begins with small accelerations of 1 to 2 sec. It then builds up rapidly to a more or less steady-state peak acceleration before gradually decreasing (see Fig. 3.2). Motions strong enough to be felt and recorded have been observed to persist over periods of about 60 sec.⁹ According to Housner, the duration of strong motions is related to earthquake magnitude and to the length of break of the causative fault.¹⁰ Housner has thus concluded that the duration of strong shaking near the causative fault during an earthquake probably will not exceed 45 sec. Also, from studies of the signal attenuations with distance from epicenters, he has concluded that the maximum ground acceleration near a fault during a strong earthquake is not likely to exceed about 0.5 g. Others believe that 0.5 g is a reasonable upper limit for acceleration. For instance, according to Sherard, the Consulting Board for Earthquake Analysis for California has stated that for large earthquakes along the San Andreas fault it would be reasonable to expect maximum ground acceleration of 0.5 g.¹¹ Furthermore, Sherard reports that it is accepted by many that the maximum strong-motion shaking which might be experienced in California within about 12 miles of the surface fracture of a large earthquake is not likely to exceed 60 sec duration and have peak accelerations of about 0.5 g at periods within the range of 0.1 to 0.5 sec. On the other hand, there are indications that ground accelerations may exceed 0.5 g during some earthquakes. The Richter magnitude 5.5 earthquake at Parkfield, California, on June 27, 1966 had a peak acceleration of at least 0.5 g. Furthermore, Saint-Amand has predicted peak accelerations on the order of 1 g, with average accelerations of about 0.4 g for several minutes, as a result of a Richter magnitude 8.3 earthquake along the San Andreas fault.¹²

The distance to which earthquakes will produce destructive waves depends on the nature and magnitude of the earthquake, as well as past experience within the area of concern. Focal depth is one of the most

important considerations. In general, if the earthquakes of a given area have very shallow focal depths, high-intensity shaking will occur only within the epicentral area, while little destructive shaking will occur at a distance. On the other hand, for earthquakes with deep focal depths, the intensity of shaking will not be great in any area, although the earthquakes may be felt over a wide area. One important exception to these generalities can occur at distances of perhaps 100 miles or greater where the shorter-period vibrations are damped out and only ground periods greater than about 1 sec remain. In areas where soils or tall buildings show about the same natural periods of vibration, resonant vibrations may be set up and major damage may be sustained. Such was the case for tall buildings in Los Angeles as a result of the Kern County, California, earthquake of 1952 and for Mexico City, where many tall buildings are founded on soft soil and a Richter magnitude 7.5 earthquake occurred in 1957 at a distance of more than 200 miles from the city.

In general, for strong-motion accelerations, it has been found that the vertical component of acceleration is about one-third to one-half the horizontal acceleration when the movement along the fault that generated the ground motion is primarily horizontal; whereas if the faulting is primarily vertical, the vertical component of acceleration may be nearly equal to or slightly greater than the horizontal. It is now standard practice to assume a vertical component about two-thirds that of the horizontal. Since most engineering structures inherently have greater strength in the vertical direction than in the horizontal, there has been generally less concern with these motions than with the more destructive horizontal motions.¹³ Horizontal ground motions apparently have not caused damage preferentially in any given direction. Thus it is generally agreed that there is random motion of the earth during the strong shaking phase of large earthquakes.

3.4 Effects on Soils

Many parts of the country, such as the Atlantic and Gulf Coastal Plains and most river basins, are underlain by thick soil deposits and/or unconsolidated sediments that may behave quite unlike hard rock when

subjected to earthquake-induced ground motions. Thus reactor sites within these areas should be thoroughly evaluated for the possible effects of shaking of the soil column. Most studies of seismic effects in soils have been associated with the amplification of ground oscillations, but soil failure through consolidation, liquefaction, or sliding must also be considered. Vibrations can, of course, cause compaction in soils, especially cohesionless ones, and thus soils are frequently compacted during earthquakes. Ground surfaces over loose sediments often settle during shaking; however, when the sediments are waterlogged, an increase in water pressure occurs that causes an upward flow of water to the surface, which causes sand boils, mud spouts, water spouts, and "quick" conditions to develop.¹⁴

Several dramatic instances of soil liquefaction have occurred during earthquakes. Partially unconfined liquefied soils frequently cause flow slides, which are simply mass movements of soil.¹⁴ In special cases where thin horizontal beds of unconsolidated sand are interbedded with other sediments, liquefaction of the sand may allow the overlying mass to slide and cause graben-type structures. Such was the case in some parts of Anchorage during the 1964 Alaskan shock.¹⁴

Slides and soil failures may also develop on clay soils during earthquakes. It is well known that great landslides and related phenomena may accompany large earthquakes, such as the 1959 Hebgen Lake, Montana, shock, and in particularly susceptible areas, landslides may be triggered by comparatively small shocks of the type that occur over most of the U.S.¹⁵

3.4.1 Amplification of Ground Oscillations

In general, ground motions in soils result in some amplification and/or attenuation of the oscillations in contrast with their effect in hard rock and may also cause failure of the soil by consolidation, sliding, differential compaction, and liquefaction. The role of soil failures in earthquake damage was emphasized recently in the destructiveness of large earthquakes in Chile (1960), Alaska (1964), Niigata, Japan (1964), and Venezuela (1967). It has been substantiated that the softer the underlying unconsolidated sediments or soil the lower will be the predominant frequency of the ground motion in comparison with that on hard rocks, but data are not available to assess quantitatively the effects of soils on

the intensity or severity of the ground motion. Although there are no instrument records to show the amplification of ground motions in soft soils for any major earthquakes, a few measurements of ground motion were made simultaneously on various rocks and soil types during smaller earthquakes. The strong-motion shaking associated with the 1957 San Francisco shock (magnitude 5.3) was recorded at five stations within a 20-mile radius of the epicenter. In general the records showed that ground accelerations in soil exceeded those in rock by a factor of 2.¹⁶ It was also observed that no simple correlation existed between the amount of amplification and the depth of alluvium in the San Francisco Bay area.¹⁶ On the other hand, studies of microtremor motions in soil and rock in the Los Angeles area by Gutenberg showed that the amplitude and duration of shaking increased linearly with soil thickness.¹⁷ Also it was found that the average duration of shaking and the amplitudes of the motions were four to five times greater in soils than on hard rocks.¹⁷ From these studies it is evident that ground motion may be expected to be somewhat less in hard rock than in soft soils; however, it is not possible to state the exact magnitude of the difference. To date most studies have been limited to relatively small shocks, and because of the differences in behavior of soils under small stresses as compared with large stresses and uncertainties as to correlation between ground motions from large and small shocks, it is difficult to extrapolate from small to large shocks. For instance, Seed states that almost all soils amplify motions during microshocks; however, for large shocks there may be attenuation of the ground motions in the same types of soil because dynamic properties of soils change with increasing excitation.¹⁸

Recent studies based on soil-response analysis procedures showed that the predicted subsoil behavior of soft soils at several locations agreed well with accelerograms of earthquake motions recorded at the same sites.^{19,20} Although further verification and improvements are needed, the development of appropriate analytical procedures may eventually provide qualitative, if not quantitative, assessments of the effects of soil conditions on the intensity of ground shaking.

3.4.2 Liquefaction

Perhaps the most dramatic instance of soil liquefaction in recent times occurred in Niigata, Japan, during the June 16, 1964 earthquake, which centered about 35 miles from the city. Major damage occurred to buildings, bridges, highways, utilities, dock areas, oil refineries, and railroads as a result of liquefaction of the underlying soils. One of the most spectacular examples of the effects of soil liquefaction on engineering structures is shown in Fig. 3.3. During the earthquake, liquefaction of the sand beneath the apartment buildings resulted in the settling and tilting of the structures. Extensive damage to engineering structures also occurred as a result of soil liquefaction during recent earthquakes in Alaska, 1964, and Chile, 1961. There is considerable evidence that liquefaction of sands also occurred during the great New Madrid, Missouri, earthquakes of 1811 and 1812, and the great Charleston, South Carolina, earthquake of 1886. These same areas are subject to liquefaction in the future, as well as countless other areas throughout the United States where the potential for liquefaction is present but has not been observed. As may be seen there is ample evidence that soil liquefaction is a phenomenon that can occur over many areas.

Variations in the distance of the site from the shock, the magnitude of the earthquake, its duration, focal depth, etc., all influence the response of soils. For instance a small nearby earthquake of short duration may not cause sand to liquefy; on the other hand a large earthquake of long duration at a greater distance may. The depth to the water table, depth of sand, and the physical characteristics of the soil are also important parameters. The actual mechanism of liquefaction often occurs when an unconsolidated, cohesionless, saturated soil is vibrated sufficiently to induce destruction of the soil structure, with concurrent loss of shear strength between the soil particles that allows a transfer of load to the water, which has negligible shear capacity; there is an accompanying increase in the pore pressure in the soil. If the soil configuration is such that the liquefied soil is confined, the pore pressure cannot be dissipated until a sufficient amount of the water escapes. The depth to the water table is critical, and even small changes of a few

Fig. 3.3. Effects on Structures with Foundations in Soil that Liquefied During Earthquake at Niigata, Japan, in 1964. (Photograph supplied by L. S. Cluff of Woodward-Clyde & Associates.)

feet in the depth to water can change the liquefaction potential of a soil deposit. This was shown at Niigata, where in some cases liquefaction did not occur when the water table was a few feet lower than in adjacent areas where the soil conditions were apparently the same but liquefaction did occur.²¹ Other physical and chemical properties of soils, such as grain size, particle size distribution, and admixtures of clay, can also affect liquefaction.

It is believed that only saturated cohesionless soils will liquefy in the fashion described above. It has been demonstrated by observation during earthquakes, as well as in laboratory tests, that the more dense the sand the less likelihood there is of liquefaction. Many preconsolidated, dense, clayey soils have never shown evidence of liquefaction, but other clays that have a high water content and weak structure can fail in ways described by some as liquefaction. This case would be much less important in reactor foundation problems, however. It is assumed that soils must be saturated before liquefaction can occur; however, there have been references, but not well-documented ones, to liquefaction of soils with air filling the interstitial pore space rather than water. This phenomenon, too, is of little importance in reactor support problems.

In addition to the physical and chemical properties of the soils, including the depth, width, slope, and geometric shape of the deposit, the nature of the earthquake, such as its duration, is extremely important. In general the longer the duration the greater the chance for liquefaction. This has been shown both in the laboratory and in actual earthquakes. Records at Niigata, Japan, show that liquefaction began after about 15 sec, while in Alaska during the 1964 earthquake numerous landslides resulting from liquefaction did not occur until after several minutes of shaking.

The intensity of the ground motion also affects liquefaction; the stronger the earthquake motions, the sooner liquefaction of a sand will occur. Strong earthquakes, of course, occur throughout the world each year, and soil liquefaction is associated with many of these. Many of the larger earthquakes have caused extensive liquefaction. For instance, many sands liquefied in the great New Madrid, Missouri, earthquakes of

1811 and 1812; the Charleston, South Carolina, earthquake of 1880; and the great Alaskan earthquake of 1964. The liquefaction was not confined solely to the epicentral regions of the earthquakes, but as in Anchorage, Alaska, the liquefaction extended to areas as far away as 100 miles. The importance of the intensity of earth motions in causing liquefaction has been studied in the laboratory.²² An increase in the rapidness of liquefaction with an increase in the energy of the wave motion was found. Thus for strong motions, sand will liquefy more quickly than for weaker motions, which may also cause soil to liquefy following longer periods of shaking.

3.4.3 Subsidence

Since vibration is an effective means of compacting loose soil, it is not surprising that ground shaking during earthquakes often causes settling of the ground surface and compaction of dry cohesionless soil deposits. Some of the more dramatic examples of soil subsidence occurred during the 1964 Alaskan earthquake. At Homer, during this earthquake, a decrease of 2.5 ft in the thickness of the soil layer was measured through the projection above ground of a well casing that was anchored in hard rock below the soil deposit.²³ For the same earthquake it is reported that the soil in and around Portage, Alaska, compacted as much as 5 ft as a result of the shaking.²³

Dry cohesionless soils generally account for most of the settling and compacting during earthquakes; however, water-saturated cohesionless soils may also settle. In the former case, settling is virtually instantaneous, but in the latter the subsidence is time-dependent, since the compaction is due to the slow dissipation of any excess hydrostatic pore water pressures induced by the shaking of the ground. Currently, most studies of soil subsidence are being conducted in the laboratory on dry cohesionless sediments in conjunction with nuclear cratering experiments.

3.4.4 Landslides and/or Slope Stability

During the Alaskan earthquake of 1964, several slides occurred in Anchorage that involved great masses of soil. Many of these slides may have been caused by the liquefaction of thin seams or layers of saturated

silts or sands that allowed the overlying soil mass to slide laterally along the liquefied sands that formed the base of the slide. As the soil mass moved laterally, the soil on the rear end of the sliding mass sank into the void areas and resulted in graben structures, or sunken blocks. Landslides may also occur in clays, since these materials can lose some of their strength during an earthquake as a result of the added forces.

In general it is difficult to predict slope stability during earthquakes, especially in areas where past soil failures are not evident. Predictions must take into account such local features as types of strata present, their slopes, and their water content and drainage patterns.

3.4.5 Soil Tests

There is lack of agreement on methods for determining the liquefaction potential of soil. Some prefer to rely mainly on the results of cyclic load tests of representative soil samples in the laboratory, while others prefer to apply past experience with soils of known static properties. In the latter case, the liquefaction potential is commonly based on the relative density of the soil determined by the blow test, since the static, as well as the dynamic, behavior of cohesionless soils is believed to be controlled by the relative density. Thus the measurement of this important variable is of great interest in the evaluation of potential reactor sites.

The blow test is utilized primarily to obtain the bearing capacity of soils for foundation design. It is based on the resistance of soils to the penetration of a standard sampler, which also collects representative but disturbed samples of the soils for identification purposes. Penetration resistance is expressed as the number of blows required to force a split-tube sampler 1 ft into the soil with a 140-lb hammer that falls freely over a distance of 30 in.

In general the test is conducted by first clearing the hole to the depth to be sampled by standard drilling bits or augers. The standard 1 3/8-in.-ID 2-in.-OD split-tube sampler is then lowered into the oversized hole and driven 6 in. into the layer to be tested. The test is then started, and the penetration resistance is recorded as the number of blows required to force the sampler an additional 12 in. into the

soil. Fifty or more blows per foot indicates a very dense or firm material, and thus the test is not carried beyond such a value. After each test, the sampler is removed from the hole, and its contents are used to identify the soils tested. Ground-water levels and other data that might affect the reliability of the test data are also recorded.

For cohesionless soils the recorded blow counts, which are interpretable from a lower limit of 5 to 10 blows per foot to 30 to 50 blows per foot, are used to indicate relative density. Only fairly clean coarse-grained sand and gravel can be evaluated, but these determinations can be made over a range of water contents. The relative consistency of cohesive soils can also be determined with the blow test, but it is reliable only for saturated or nearly saturated fine-grained soils. Because of the nature of the test and the variations that may be experienced due to soil type and soil water content, interpretations of the field data should be made by those experienced in this type of testing.

Correlations between the strengths of sandy soils and their standard penetration resistances (blow counts) have been made. However, apparently similar relationships for the finer grained materials, that is, silt and clay, do not exist, even though their relative firmness can be determined by the field penetration test. A series of curves that relates blow count to relative density for dry and saturated cohesionless sands has been prepared by the U.S. Bureau of Reclamation.²⁴ If sands show a relative density of 70% or more, they are considered to be adequate for most static loadings. For gravel soils, 60% relative density is usually considered to be adequate, while for very fine sands, 80% relative density is considered to be suitable.

There are numerous other techniques for determining relative density in the field, such as the standard plate load test and the use of a calibrated sand for measuring the volume of a test hole and comparing it with the material removed from the hole.²⁴ They do not generally have the same range of application as to depth, etc., as does the blow test. Another method for determining soil density in the field, which has been used extensively in other countries and appears to be gaining acceptance in this country, is the static-cone penetration test.^{25,26}

The laboratory procedure for determining the liquefaction of soils at any given location consists, first, of an assessment of the magnitude and characteristics of the expected base motions in the underlying hard rock at the site. Generally, little is known about the characteristics of these base rock motions at any site, since strong motions have been recorded at few locations. An estimate of the probable magnitude of these motions can be obtained by observing strong-motion records of comparable earthquakes under comparable conditions. The intensity of the recorded motions can then be scaled up or down and the duration of the motions fixed to best fit the specific site.

Once the base motion has been determined, it is necessary to compute the response of the soil to the motion. By assuming that the deflections of the soil result from the vertical propagation of shear waves from the base and determining the nature of the soil deposits (relative density, shear moduli, and damping factors), shear stresses throughout the soil deposit can be computed for the entire time history of the accelerations, displacements, stresses, and strains in the systems.

The number and magnitude of cyclic shear stresses developed at various depths in the deposit are then determined by representing the computed stress-time records by an equivalent number and magnitude of uniform stress cycles. Representative samples of the sand at various depths are tested in the laboratory by triaxial or simple shear devices to determine the cyclic shear stress required to cause liquefaction for the critical number and magnitude of stress cycles. These data are compared with the magnitude of the equivalent cycle stress developed at the various depths to determine whether liquefaction will occur at any or all depths.

3.5 Intensity, Magnitude, and Acceleration Relations

As discussed above, maximum ground acceleration is perhaps the single most important shaking consideration in regard to reactor siting and design, but since so few measurements of strong acceleration have been made and the chances of recording the maximum expected acceleration at any reactor site are so small, it is apparent that this important parameter must be estimated at each reactor site.

Maximum ground accelerations are frequently estimated from the Modified Mercalli intensity rating of earthquakes (see Table 3.1),²⁷ since intensity values are available for most large and moderate earthquakes that have been recorded in the U.S. This measure of earthquake intensity is a purely subjective one, however, that describes structural damage and is not entirely suitable for estimating the true magnitude of ground motions in rock or soil. For instance, postearthquake damage observations generally show that structures built on alluvium are damaged considerably more than those on hard rock. However, this does not necessarily indicate a more intense ground motion of the soil, since significant differential movements of soils may occur and thus may cause disproportionate amounts of damage to structures situated on soil. Housner¹⁰ and Hershberger²⁸ also have illustrated the large errors that may result when attempting to predict ground acceleration from Modified Mercalli intensity numbers, but on the other hand it is remarked by Murphy that if adequate qualifications are made, reasonable estimates of ground accelerations can be obtained from Modified Mercalli intensity numbers.²⁹

The intensity of ground motion in the epicentral region of an earthquake may also be estimated indirectly from the earthquake magnitude. However, since the higher frequency components are attenuated by the travel path, the magnitude determined at one location cannot be used to accurately determine the nature of the shaking at another location. The instruments by which magnitudes are determined are commonly located hundreds or perhaps thousands of miles from the epicenters of many shocks, as was the case for the 1964 Alaskan shock, and thus they may give an indication of the length of fault breakage but not a good quantitative description of the ground motion near the causative fault.³⁰

3.6 Evaluation of Ground Motions at Specific Sites

Estimates of the nature and intensities of maximum probable ground motions at specific sites are generally based on (1) the earthquake histories of the local and regional areas, (2) the local and regional geotectonic patterns, (3) the strong ground motion records of several earthquakes in the western part of the country, and (4) the local soil and

Table 3.1. Approximate Relationships Between Intensity, Acceleration, Magnitude, and Energy Release²⁷

Modified Mercalli Intensity Scale	Description of Effects (Masonry A, B, C, and D Are Defined Below; ^a From Ref. 27)	Maximum Acceleration (g)	Richter Magnitude	Energy Release (ergs)
I	Not felt; marginal and long-period effects of large earthquakes evident		M2	10 ¹⁴
II	Felt by persons at rest, on upper floors, or favorably placed			10 ¹⁵
III	Felt indoors; hanging objects swing; vibration like passing of light trucks occurs; duration estimated; might not be recognized as an earthquake	0.003 to 0.007	M3	10 ¹⁶
IV	Hanging objects swing; vibration occurs that is like passing of heavy trucks, or there is a sensation of a jolt like a heavy ball striking the walls; standing motor cars rock; windows, dishes, and doors rattle; glasses clink; crockery clashes; in the upper range of IV, wooden walls and frame creak	0.007 to 0.015	M4	10 ¹⁷
V	Felt outdoors; duration estimated; sleepers waken; liquids become disturbed, some spill; small unstable objects are displaced or upset; doors swing, close, and open; shutters and pictures move; pendulum clocks stop, start, and change rate	0.015 to 0.03		10 ¹⁸
VI	Felt by all; many are frightened and run outdoors; persons walk unsteadily; windows, dishes, glassware break; knickknacks, books, etc., fall off shelves; pictures fall off walls; furniture moves or overturns; weak plaster and masonry D crack; small bells ring (church, school); trees, bushes shake	0.03 to 0.09	M5	10 ¹⁹
VII	Difficult to stand; noticed by drivers of motor cars; hanging objects quiver; furniture breaks; damage occurs to masonry D, including cracks; weak chimneys break at roof line; plaster, loose bricks, stones, tiles, cornices fall; some cracks appear in masonry C; waves appear on ponds, water turbid with mud; small slides and caveins occur along sand or gravel banks; large bells ring	0.07 to 0.22	M6	10 ²⁰
VIII	Steering of motor cars affected; damage occurs to masonry C, with partial collapse; some damage occurs to masonry B, but none to masonry A; stucco and some masonry walls fall; twisting, fall of chimneys, factory stacks, monuments, towers, and elevated tanks occur; frame houses move on foundations if not bolted down; loose panel walls are thrown out; changes occur in flow or temperature of springs and wells; cracks appear in wet ground and on steep slopes	0.15 to 0.3		10 ²¹
IX	General panic; masonry D is destroyed; masonry C is heavily damaged, sometimes with complete collapse; masonry B is seriously damaged; general damage occurs to foundations; frame structures shift off foundations, if not bolted; frames crack; serious damage occurs to reservoirs; underground pipes break; conspicuous cracks appear in ground; sand and mud ejected in alluviated areas; earthquake fountains and sand craters occur	0.3 to 0.7	M7	10 ²²
X	Most masonry and frame structures are destroyed, with their foundations; some well-built wooden structures and bridges are destroyed; serious damage occurs to dams, dikes, and embankments; large landslides occur; water is thrown on banks of canals, rivers, lakes, etc.; sand and mud shift horizontally on beaches and flat land; rails are bent slightly	0.45 to 1.5		10 ²³
XI	Rails are bent greatly; underground pipelines are completely out of service	0.5 to 3	M8	
XII	Damage nearly total; large rock masses are displaced; lines of sight and level are distorted; objects are thrown into air	3.5 to 7	M9	10 ²⁴

- ^aMasonry 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.

rock conditions. The methods and techniques for presenting and analyzing these fundamental data vary considerably due to limitations such as the geoseismic complexity of the local and regional environments and the availability of data; thus, estimates of the critical parameters of the expected ground motions may be more realistic in some areas than in others.

In some parts of the country, such as the west coast area, where the larger earthquakes generally occur along well-defined throughgoing surface faults, the expected shaking is commonly determined by first analyzing the relationship of the specific site to the location and extent of nearby active faults. As discussed in Section 4.4, the maximum earthquake that would be expected to occur along any given fault is based on the length of the recognized fault. Thus for faults that do not exceed about 100 miles in length, the maximum earthquake that will occur along them will not exceed about Richter magnitude 7, whereas for a fault length exceeding 200 miles the maximum earthquake may approach magnitude 8.0. It is further assumed that the epicentral location of this greatest earthquake will occur at a point along the fault that is nearest to the proposed site. Of course it is recognized that the strongest ground motions could come from either a smaller earthquake along a nearby fault or from a greater earthquake along a fault more distant to the site. In some cases it may also be desirable to consider the possibilities of the occurrence at or near the site of a large aftershock, which may or may not be related to any recognizable surface fault or other structural features of a great earthquake. Smaller earthquakes that have occurred nearby in the past but are not necessarily related to specific surficial geologic structures may also provide the maximum expected ground motions, especially if the larger earthquakes occur at great distances from the site.

In other parts of the country where earthquakes are not clearly related to surface faulting or some other geologic structure, such as in the eastern U.S., the maximum expected motions may be determined from the historic record of shocks that have occurred nearby or from the projected motions associated with great earthquakes that have occurred in historic times in the U.S., such as at Charleston, South Carolina (1886), and New Madrid, Missouri (1811-1812). To illustrate the importance of the latter two earthquakes, it is noted that the maximum expected ground shaking at

2

the Crystal River, Florida, and the Browns Ferry, Alabama,³¹ power plants were based on the reoccurrence of the great historic earthquakes at Charleston and New Madrid, respectively. The Crystal River site is roughly 300 miles from Charleston, while Browns Ferry is about 200 miles from New Madrid.

Once the maximum earthquake is determined and its location with respect to the reactor site is ascertained, the actual shaking intensity at the site is then established from Modified Mercalli intensities of past large earthquakes in that general area or empirically derived equations that relate intensity of strong ground motions with distance from the epicentral regions of earthquakes. Intensity ratings are also commonly related to acceleration through empirically derived equations. Then, based on the periods of shaking, the duration of strong motion, the depth of the expected shock, etc., the maximum acceleration is scaled to a recorded earthquake spectrum such as that of the El Centro, California, earthquake in 1940 or that of the Taft, California, earthquake in 1951 to provide the basic ground-motion input data needed for design purposes.

For those sites not located on hard rock but, rather, underlain by various thicknesses and types of unconsolidated sediments or soils, the ground motions may be amplified or, in some cases, attenuated in relation to those of the underlying or adjacent hard rock. Since these relations cannot be determined with a high degree of accuracy for most situations at the present time, conservative estimates of soil amplification must be made. The dynamic properties of soil that control the transmission of seismic energy, and hence the attenuation and amplification of earthquake-induced ground motions, are currently determined by wave propagation tests (both in the laboratory and in the field) and/or cyclic loading tests, but neither test is entirely satisfactory. In general, wave propagation tests are conducted at stress and strain levels that are well below those of actual earthquake conditions, while cyclic shearing tests, which are limited in their range of test conditions by control requirements, must be conducted on samples of soil that have been removed from their natural environment and presumably disturbed to some extent.

To determine the potentialities of soil liquefaction, consolidation, compaction, and subsidence at specific sites, tests for determining the

physical properties of soils under static as well as dynamic conditions must be made. Cyclic testing is one technique that is being used to determine the potentiality of such liquefaction. Other methods include reliance on past field experience, tests to determine the relative density of soils through laboratory or field determinations, such as the in situ standard penetration test, and critical void ratio determinations in soils.

3.6.1 Maps

The relative seismic activities of various regions within the U.S. are commonly and perhaps most simply illustrated by epicenter maps. However this emphasizes the number of shocks and frequently cannot provide a representative expression of the seismic patterns in regions of interest because of severe cartographic problems. Recently a variety of techniques involving energy and strain releases by earthquakes have been used to give more convenient cartographic and quantitative representations of seismicity.^{32,33} "Seismic zoning maps," which are based mainly on the number and intensity of past earthquakes and are used to indicate the maximum intensity to be expected for each point within the area covered by the map, are commonly used to estimate seismic risk for engineering purposes.³⁴ The reliability of the maps depends, of course, on the size of the sample and the amounts and type of data collected, so the maps tend to be most reliable in areas for which the most data are available. Other basic limitations of these maps stem from the difficulties of extrapolating probabilities of occurrence from small to large shocks and uncertainties in estimating the intensities of shaking in various soils and rocks from observations of small shocks.¹⁸

Another indication of the probable maximum earthquake intensity at a particular site in the U.S. may be obtained from the seismic regionalization maps of Richter,³⁴ but little use has been made of these maps in AEC hearings on reactor applications. These regionalization maps are based on the probable occurrences of earthquakes, the distance of the site from probable epicenters, and the characteristics of the rock and soils at the site.

3.6.2 Frequency of Occurrence

By analyzing the past seismicity of an area and assuming that this provides a statistically representative sample for predicting future seismicity, the probability or statistical expectation of future ground motions at a site can be stated. Housner has presented a table for California showing the expected number of earthquakes during a period of Y years that will have a magnitude greater than M .³⁵ This table shows that (1) an earthquake of magnitude 8.2 (San Francisco, 1906) or greater can be expected to occur about once every 200 years, (2) a magnitude 7 earthquake (El Centro, 1940) or greater can be expected to occur approximately 63 times each 200 years, and (3) a magnitude 6.25 earthquake (Long Beach, 1933) or greater can be expected to occur about 138 times in 200 years. Of course, it is expected that the probabilities of smaller shocks are greater than those for the more infrequent larger shocks. Frequency of occurrence predictions based on seismic history may also be made for small areas or even for particular sites. However, confidence is low for predictions of higher magnitude shocks due to inadequate samples of large shocks in small areas.

3.6.3 Intensity of Ground Shaking During Large Earthquakes

Strong ground motions have been recorded during earthquakes having magnitudes of 7 and 7.7, but strong motions for larger magnitude shocks, such as the 1906 San Francisco earthquake (magnitude 8.2) have not been measured. Magnitude 7 shocks along the San Andreas fault would be expected to result in rock rupture that would extend about 50 miles (this is the approximate extent of the El Centro, 1940, magnitude 7 shock). Also it would be expected that movement along the San Andreas fault in the event of a magnitude 8.2 shock (San Francisco, 1906) would extend perhaps as much as several hundred miles. Since the additional area of the fault break for the larger magnitude shock would be more than 25 miles from a center point of measurement for the smaller shock, it is reasoned by Housner that measurements of ground motion taken near the center points of magnitude 7 and magnitude 8.2 shocks would be about the same in maximum intensity, even though the duration of shaking would, of course, be

greater for the larger shock.³⁰ On the contrary, others contend that the intensities for magnitude 8 earthquakes are considerably more than those for magnitude 7 shocks, since the basic definition of magnitude implies greater intensities.

For sites situated within a mile or so of a causative fault, there is some question as to the intensity of shaking. Based on a hypocentral, or focal, depth of about ten miles, Housner concluded that there is no significant difference in intensity of shaking with distance from the center of the initial rupture area for points out to about five miles from the surface rupture. On the other hand, decreasing intensities of shaking at points nearer to surface faults have been reported for some strong shocks. According to Housner, this phenomenon could be due to the unequal throw on adjacent sides of the fault,³⁵ even though others contend there is no scientific record to support this.¹⁸ For the June 1966, Parkfield, California, earthquake a progressive increase in ground acceleration approaching the ground rupture was recorded by a string of strong-motion accelerometers located approximately normal to the fault trace.³⁶ Since they are the only recorded measurements of this type and the earthquake was of moderate intensity (magnitude 5.5), these results cannot be extrapolated with confidence to other smaller or larger earthquakes.

3.6.4 Instrumentation for Site Evaluations

Instrumental observations of strain and tilt changes in seismic belts may be expected to provide knowledge of the anticipated seismic effects within broad geographic regions and perhaps even at specific reactor sites. For reactor siting, information on local seismic activity is of fundamental importance. The study of very small earthquakes or microearthquakes may be important in reactor siting, since many more small earthquakes than large earthquakes occur in active areas. For example, as many as 200 microearthquakes per day have been recorded in seismic areas of Nevada and Japan.³⁷ Instruments that provide a continuous record of local seismicity include portable, highly sensitive seismographs. These seismographs should have a band pass from 1 to 100 cps to respond to the higher frequencies of microearthquakes. Tiltmeters

and strainmeters (see Fig. 3.4) also have applications in siting, but they are especially sensitive to surface noise. The Japanese have operated water-tube tiltmeters effectively in mines, and strainmeters have been operated effectively in mines and tunnels in the United States.

Preliminary seismic data on a site can be collected by a portable seismograph that consists of a light-weight seismometer and its associated amplifier, filter, timing, test calibration, recording, and power systems. Resulting records may be on paper, which is adequate for preliminary analysis of displacement velocities or accelerations, or on magnetic tape that can then be used directly for analysis by computer.

Highly sensitive portable seismographs that have magnifications up to 100 million at 25 cps have been designed and operated in the field.³⁷ One such seismograph consists of a Dayton 4-cps geophone, a solid-state-amplified Electrotech SPA-1, a Bulova Accutron break-circuit chronometer, and a single-channel Sanborn 229 oscillograph. Power is supplied by storage batteries through an inverter. These systems are capable of detecting local earthquakes having magnitudes far less than can be sensed by standard seismographs.

Triangulation and leveling surveys may be made with transits or geodimeters across faults and seismic belts to sense strain changes that are associated with earthquakes. The principal drawbacks to these types of measurements are the relatively long period of time required to obtain meaningful information and lack of precision, especially over long distances. Laser interferometers, modulated light beams, and microwave phase measurements have been proposed for improving the sensitivity of strain measurements over distances exceeding 1 km. Other instruments that may have application in siting are magnetometers, gravity meters, telluric current sensors, and earth-resistivity meters. These instruments are not generally being used in site surveys; however, they are being considered as possible auxiliary instruments for earthquake-prediction observatories in the United States and have been used to some extent in Japan.

Geophysical techniques developed for petroleum exploration may also be used in evaluating reactor sites, both on land and at sea, if there are uncertainties as to the depth of unconsolidated surface or subsea

Fig. 3.4. North-South Strain Trench at Stone Canyon, California, Geophysical Observatory. (Photograph supplied by Donald Tacher.)

material or where there is possible normal or reverse faulting. Refraction and reflection surveys can be made at land sites, and marine reflection surveys can be made at oceanic sites. Refraction surveys provide travel-time curves from which depths of unconsolidated material can be determined by simple calculations. Reflection surveys can provide information on subsurface structures and deep faults.

Transducers developed for use in seismology can be used to monitor strain, tilt, and shift of foundations, as well as other critical reactor motion components during and after earthquakes. A nuclear facility may be subjected to a large earthquake without apparent damage; however, slight displacements and tilting of the reactor might induce weakness or strain that could later lead to failure. In addition, large aftershocks might cause additional damage to an already weakened structure. After a major earthquake, systems for recording seismic activity, tilt, and strain should be monitored closely to guard against conditions that might lead to failure.

3.7 Engineering Formulation of Excitation*

3.7.1 Selection of Design Basis

When a structure is to be built, specific data must be available on which to base the design. These data must be the best available and should be used in a rational fashion despite their possibly wide confidence limits. Judgments as to the construction of a reactor at a designated site must be made with knowledge of the uncertainties involved in the basis for the design, and it should be recognized by the design engineer that seismologists are not entirely unanimous in their estimates of the uncertainties, as exemplified by views of Richter²⁷ and Heck.³⁸ Richter says that it is unwise to suppose that the highest intensity known to have affected a given locality will never be exceeded there in the future, while Heck has stated that "In any region with sufficiently long record and reasonably frequent shocks it may be safe to assume that the greatest known earthquake will

*The detailed information on which this Section is based is presented in Refs. 1 and 2, which were prepared in conjunction with this overall study.

not be exceeded. It would be reasonably safe to assume that California will not have an earthquake greater than that in 1906."³⁸

In the face of the geological and seismological uncertainties recognized by both the engineering and the earth science professions the designer must consider the added costs of setting higher criteria for design and the adverse consequences of underdesign.³⁹⁻⁴² A rational way of assessing the acceptable hazards to the public that might result from a reactor accident initiated by an earthquake involves comparisons with other reactor accident hazards, with other hazards resulting from man's actions, and with hazards that stem solely from nature. It appears that measurements and assessments of these hazards would result from some sort of product of the probability of each event under consideration and its effect if it were to occur. It also appears reasonable that the hazard resulting from the worst earthquake and the resulting reactor accident might now be calculable, as well as the hazard from lesser events. From this probabilistic conclusions might evolve that would show on a rational basis that while earthquake-initiated accidents, including the extreme accident, are entirely possible, their occurrence would be of such low probability that large reactors could be built even in the center of populated zones. Such assessments are yet to be made.

3.7.2 Modification of Motion in Various Media

Some specialists⁴³ are convinced that each site has a fairly characteristic response to earthquakes and that many sites have peak responses near a 1/3-sec period; while other specialists maintain that the evidence from underground nuclear tests shows that a site may respond differently depending on the magnitude and distance from the focus. No agreed-upon method is available for predicting the overall effects of earthquakes of different magnitudes on the frequency spectra. Some specialists believe that all spectra will show diminished responses below a 1/3-sec period. Another specialist suggests that if, with untutored eye, one compared the response spectra of El Centro, Taft, and Olympia, which have widely different site characteristics, it would be difficult to relate differences in the response spectra with site characteristics. More research needs to be done.

It is thought that only two records giving dominant site periods have been made: one at Mexico City in the bed of an old lake on volcanic clay and one at Olympia on filled ground. Neumann⁴³ reported a 22-fold amplification when earthquake vibrations passed through granite basements and forced the most responsive types of surface formations into vibrations. Ratios of 30-fold between water-soaked alluvium and adjoining outcrops have been reported, and it is known that intensity can vary through 4 or 5 Modified Mercalli grades at the same epicentral distance, depending on the variety of soil and rock types.⁴³ The vast majority of instrument measurements showing high amplification in alluvial materials have been made for extremely small earthquakes,⁴⁴ and extrapolation to significant intensities almost certainly is not warranted. Such measurements, as with those of damping coefficients at low excitation, should give worthwhile maxima and minima, respectively. Some think spectra in rock vary greatly and that those measured in soil are more consistent. Newman⁴³ states that the California State Earthquake Investigation Commission's report on relative damages due to structures on various types of soil noted that if one took damage on serpentine and sundry solid rocks as a reference value of 1, damage on other soils would range as follows:

	<u>Relative Damage</u>
Serpentine and sundry solid rocks	1
Sandstone	1 to 2.4
Sand	2.4
Mission Valley sand	4.4
Made land	4.4 to 11.6
Marshland	12

Measurements of excitation at an electric power station 45 m underground indicated that the intensity on cracked or weathered rock can be twice as large as that on bedrock.⁴⁵ There is some belief that the frequency of peak response of alluvial beds grows longer when they are subjected to successive strong earthquakes. Gutenberg concluded from measurements of very weak ground motions that the intensity of shaking on granite would be perhaps 1/8 to 1/4 that on alluvium.¹⁷ Still other specialists feel that where a deep overburden is present, its capacity

for either impedance or amplification could make the response characteristics of a particular site dominate over variations in the shapes of response spectra in the base rock from various earthquakes. There is some belief that the same site may amplify incoming ground motion from one earthquake and diminish that from another.

3.7.3 Spectra of Vibratory Motion

The Fourier Spectrum. Ground motion recorded during an earthquake can be described by a Fourier spectrum,^{27,46,47} which resolves the ground acceleration time record into an infinite series of simple harmonic functions in the frequency domain. Such spectra are not currently used in reactor structural design, but they do present a precise description of any time-varying function in terms of its frequency content. Shock and other vibratory motions are commonly resolved into the Fourier spectrum for the purpose of identifying important characteristic frequencies and phase relationships. This use could also be applied to the comparison of existing and future strong-motion seismograph records. Accelerometer records made during shaking tests could also be transformed into their Fourier spectra for diagnosis. A nonseismic record with its Fourier spectra is illustrated in Fig. 3.5

The Response Spectrum. The response spectrum technique, invented by Benioff⁴⁸ and Biot⁴⁹ and extensively developed by others,^{46,50} has been found to be one of the most helpful and widely used devices in earthquake and structural shock design. The response spectrum has been considered⁵¹ to be the most useful known strong-motion statistic. It can be developed analytically for very simple structural motion, but it is in current practice carefully determined⁴⁶ from accelerograph records by using analog or digital computers. It characterizes a particular accelerograph record made at one site, in one direction, in a single earthquake. Response spectra from several records have been averaged in various ways, and empirical formulas⁵² have been developed but little used in U.S. practice.

A response spectrum is a resolution of the recorded earth motion into a variable that expresses the maximum amplitude of response of a simple damped oscillator (a single mass on a spring). The physical situation

Fig. 3.5. Fourier Spectra for Typical Random Acceleration-Time History. Not based on an earthquake record. (From Ref. 2)

on which this conversion to response spectrum is based is illustrated in Fig. 3.6. Since Acceleration, velocity, and displacement are always interrelated the response spectrum is usually calculated for only one of these variables. Thus the spectral displacement, S_d , is the maximum

Fig. 3.6. Velocity Spectrum for North-South Component of El Centro, California, Earthquake of May 18, 1940. (Adapted from Ref. 2)

absolute value of $Y(t)$:

$$Y(t) = -\frac{T}{2\pi} \int_0^t \ddot{z}(\tau) \exp \left[-(t - \tau)\omega\zeta \right] \sin \left[\frac{2\pi}{T} (t - \tau) \right] d\tau ,$$

where

$Y(t)$ = displacement of mass relative to ground,

T = damped natural period of vibration

$$= 2\pi/\omega \sqrt{1 - \zeta^2} ,$$

t, τ = time,

$\ddot{z}$ = acceleration of base of oscillator,

ω = undamped natural circular frequency of vibration,

ζ = fraction of critical damping.

The spectral velocity, S_v , and acceleration, S_a , are related to S_d by

$$S_v = \frac{2\pi}{T} S_d ,$$

$$S_a = \left(\frac{2\pi}{T} \right)^2 S_d .$$

The structural significance of a response spectrum is illustrated by a reed gage. A movable horizontal base to which is attached a series of simply cantilevered inverted pendulums of increasing natural periods, as shown schematically at the top of Fig. 3.6, is assumed. Damping is illustrated schematically by dashpots, all of which are set at a specified percentage of critical damping, ranging from 0 to 100%. If the base is subjected to a specific earthquake's ground motion in one direction, each inverted pendulum of the reed gage will at some instant during the earthquake be at its maximum response value in terms of one of the three variables: relative acceleration, velocity, and displacement.

Figure 3.6 shows the spectral velocity, S_v , for one component of the El Centro earthquake. As the period approaches zero the velocity response should also approach zero, and the acceleration response then becomes the acceleration of the ground. It is also evident that damping has a great effect on the response of these simple "structures."

The spectral velocity and the Fourier amplitude spectrum of the Taft earthquake are shown for comparison in Fig. 3.7.

The response spectrum allows a separation in structural calculations of those parts dependent on the particular earthquake from those dependent on the particular structure. A response spectrum derived independently of details of any particular structure can be coupled with a model analysis of the structure to provide approximate values of its deflection and loading. Because of its relative ease of application and its usefulness in certain limited extensions to inelastic behavior, and because it gives sufficient information for many (but not all) earthquake engineering problems, the response spectrum is widely used in practice. Although it is essentially a single-mode approach it has been used often for approximate solutions of multimodal systems.⁵³

Despite their usefulness, response spectra are not applied without difficulties. Some think that accelerograms as now recorded cannot be

Fig. 3.7. Comparison of Fourier and Velocity Response Spectra for Earthquake Ground Motion. (From Ref. 2)

used for accurate determinations of ground displacement and that current accelerometers may attenuate the high-frequency content and distort low frequencies in spectra.⁵⁴ Velocity spectra vary significantly, depending on how they are calculated from the record, and since energy varies as the square of the velocity the variation can be important. One objection to the whole response spectrum approach is that it is not possible to relate the response maximum velocity distribution to the underlying earthquake probability distributions. Others contend⁵¹ that some artificial (as well as the real) accelerograms possess all the known properties of strong-motion earthquakes that are pertinent to structural analysis and that these are the most promising starting points for development.

In the past, response spectra prepared by different workers from the same accelerometer record have differed in computed value by as much as a factor of 2. Not all analyses of short periods have been made with care sufficient for all reactor applications. As previously mentioned, before the advent of reactor power plants, attention of earthquake structural engineers was focused mainly on long-period high-rise buildings, and for that application inaccuracies in the response spectrum curves at short periods seldom present major practical difficulties. However, since it has been shown that important amplification can still occur in reactor plant structures at periods well below 0.2 sec, care must be taken down to periods as low as 0.05 sec, where many conventional structures are considered to respond essentially directly with their support. Even at these short periods the appendages in nuclear plants are important, but it is not clear what are the shortest periods of importance for instruments and controls.

Response spectra of one or more recorded earthquakes, together with damping curves, are often combined to form smoothed curves that are sometimes used in practice. Figure 3.8 illustrates an "average" velocity spectrum. Although the computation of a response spectrum is a detailed procedure, the averaging methods are not always clear. Past practice for shock (response) spectra at the Naval Research Laboratory was to "envelope" a number of spectra. One approach in earthquake engineering has been to compute "spectral intensities" of several records by integrating the velocity response spectrum curves over periods of from 0.1 to 2.5 sec.

Fig. 3.8. Average Velocity Spectrum Curves for Intensity of Ground Motion Recorded at El Centro, California, 1940. (From Ref. 2)

These curves can be used to form the basis of an average acceleration spectrum, as shown in Fig. 3.9. Gluckmann says that in some cases structures are so sensitive to the shape of the earthquake ground acceleration spectra that average earthquake spectra do not give a correct picture of the behavior of the structure.⁵⁵

Since the spectral values S_v and S_a are not (as S_d) derived by finding the maximum values of velocity and acceleration, they are frequently referred to as pseudo velocity and pseudo acceleration. However, the simple relationship among the three parameters allows the practical plotting of tripartite response spectrum curves with various percentages of critical damping, examples of which are shown in Figs. 3.10, 3.11, and 3.12. The basis of the approach given here is that often used in calculations of transient response to arbitrary forces. These tripartite plots simultaneously give the spectral displacement, velocity, and acceleration. It may be noted that if one of the three is defined at a specified

Fig. 3.9. Average Acceleration Spectrum Curves from Four Earthquakes Normalized to El Centro, California, 1940 Earthquake Intensity. (From Ref. 2)

frequency and damping level, the other two are automatically defined. As the response period becomes shorter, the spectral acceleration approaches the maximum ground acceleration for that period, a characteristic which can be proved in general. Note on Fig. 3.12 that both the maximum values of the ground motion and the response spectrum are shown. The latter, of course, represents the response of a highly idealized structure to the ground motion. It may be seen that the maximum ground acceleration is the primary influence on the response of structures having frequencies near 2 cps or higher, that velocity is of primary influence between about 0.3 and 2 cps, and that at frequencies less than about 0.3 cps the maximum ground displacement has a major influence on the response of structures.⁵⁶

Fig. 3.11. Design Spectra. (From Ref. 2)

3.7.4 Computer Calculations Based on Input of Seismograph Record

Instead of a response spectrum, some specialists use one or more digitized earthquake accelerograph records selected as "operating-basis", or "design-basis" earthquakes for selected sites and put these records directly into large computers. In practice time-history calculations have the same objective as those based on the response spectrum: to give maximum response values at all important frequencies for a specified earthquake acting on a given structure.

Fig. 3.12. Smoothed Tripartite Logarithmic Response Spectrum for 0.33-g Earthquake and 2% Critical Damping. (From Ref. 56)

It has been the practice to use some 40 lumped masses for the time-history calculation of the response of a reactor and up to 100 masses for certain pipe systems. This procedure must be modified to provide an assessment of many effects associated with durations: fatigue, reversals, yielding, damping changes, permanent displacement, and the possible resulting failure. In fact some codes require treatment of earthquake cyclic loadings.^{57,58}

The 1940 El Centro record dominates in response through all periods over most other records, except in the higher frequencies, where a somewhat unsatisfactory 1957 San Francisco earthquake record exceeds it, and the recent Parkfield-Chalome record, which was of such an unusually short duration that it may alone be of little direct use as a time-history. There is, however, some dissatisfaction with the El Centro record as a general design basis. In some cases the response spectrum is modified to take into account characteristics of specific sites.

Some specialists who have developed computer programs for using inputs of accelerograph records say that it may be practical to employ a band-limited white noise source that, with a linear filter, could be used to model all the site characteristics to the extent that they can be established in any case. As will be mentioned in the discussion of soil-structure interaction, there are other specialists who are attempting to describe ground input excitation for any site by starting with an assumed time-history in the base rock and computing by finite-element methods the resulting motion at the surface. The geological deposits between the base rock and the structure must be assigned characteristics that may not have been measured with confidence.

When using time-history input, it has been noted that damage comparisons for various durations of shaking at the same level may be more clearly related to ratios of duration than to differences in durations. For example, differences in damage between 0.2 and 1.2 min of shaking would be greater than between 3 and 4 min at the same intensity.

3.8 Excitation of Reactor Structures*

Failures of conventional structures have ordinarily been assumed to occur either as (1) structural collapse, (2) distortion beyond functional limits, or (3) failure to protect occupants.⁵⁹ Safety of building occupants corresponds in reactors to the safety of all off-site persons that might be endangered, and property damage beyond the boundaries of the reactor installation also extends beyond the earthquake considerations for conventional buildings. It is important to note that protection of the economic value of the reactor plant is often a consequence of safety analyses but never a primary factor. It is easy to state a main objective relating reactors and earthquake safety: All safety-related structures, equipment, and instruments must perform as required during and following any earthquake.

3.8.1 Soil-Structure Interaction

All structural support ground material is called "soil." Soil is never completely rigid, and when both the soil and a structure are excited, they move in fashions that neither would have alone. The motion implied by the response spectrum would not actually occur, and the modification of motion from that which would be calculated without consideration of interaction could be significant for both the structure and the soils.

Support materials act, in effect, as additional elements in the vibrating system and are thought to affect significantly the reactor plant structures' periods and mode shapes and to introduce new modes of motion in rotation. In fact some specialists think that the characteristics of the supporting soil would dominate the response interaction. For example, for the first Malibu design the natural period of the reinforced-concrete secondary containment vessel was calculated to be 0.45 sec when soil translation and rocking were considered. The period of the same structure on a fixed base was determined to be only 0.22 sec, which would lead to substantially different earthquake forces and thus to different

*See Refs. 1 and 2 for details on this subject.

resulting structural stresses. Comparable calculations for the Surry Nuclear Power Plant gave values of 0.5 and 0.2 sec, respectively.

Earthquake motion that is transmitted through the earth into a reactor plant structure is, in part, returned to the earth supporting the structure. The cyclic loading of the soil by the foundation of the building would, in some cases, alter the properties of the soil. The number of load reversals during an earthquake that could take soil beyond its yield strength is of much importance to its stability and load-supporting capacity. For Niigata this value was near 10 and for Anchorage near 40, and the number might possibly reach 60 or even 100 in a great earthquake.⁶⁰

In past theoretical studies of soil-structure interactions, real conditions had to be greatly simplified to permit analysis. It has been considered impossible in practice to define accurately the mass of soil interacting.⁶¹ To take the interaction into account in building design, Newmark has proposed modification of response spectra by an "amplification factor," which is determined by the foundation material.⁵⁶ Any mode of vibration of a structure involving interaction with the soil may be assigned a damping significantly above that allowed for modes involving the structure alone. It is debated by specialists whether or not the damping values currently used in reactor structural design reflect appreciable soil-structure interaction, and assigned magnitudes of translation and rotational stiffnesses of the ground and their relationship to those of the structure are not yet agreed upon.

Earthquake engineering practice is almost always aimed at siting on the most sound support material available, although this approach is not universally accepted.^{62,63} In general, structures built on preconsolidated clays have stood up well in earthquakes, and at a large portion of the sites that are also economically practical, medium-stiff clays capable of supporting reactor plants can be found within 100 ft of the surface. Proper modification of support material is an attractive procedure, at least for conventional structures, for the record is overwhelming that the more rigid and solid the foundation the better the structure's survival of earthquake shaking.

There are investigators who feel they may now have sufficient information and techniques to be able to approximate, through the finite-element

computer programs, the ground surface response by starting with a geologic base rock's digitized time-history strong-motion record and computing response to the surface. This approach is now being evolved, and, if its logical extension to include soil-structure interaction is developed and accepted, there are at least three implications: (1) many additional strong-motion records at significantly high intensities are needed, especially those (that do not now exist) which have been simultaneously recorded on geologic base rocks, the earth surface, and on reactor structures, (2) there does not appear to be any way a response spectrum, rather than time-history calculations, can be used directly in the approach, and (3) further extension and incorporation of such calculations to include those for reactor plants and their internals and appendages would seem the next step. Some specialists believe that finite-element time-history calculations now available allow very satisfactory determinations of soil-structure interactions. It is certain that confirming measurements in real cases on prototypes are greatly needed.

3.8.2 Piling

Piling can reduce displacement caused by ground excitation both in the vertical direction and in rocking. However, much care must be taken to avoid pounding of piles against the structures they support. Such hammering could lead to the failure of either or both. It is believed that piles can resist little of an earthquake's horizontal motion and thus must be flexible to maintain their integrity. At Niigata the failure of some structures on 40-ft piles was related to liquefaction.

A large number of laboratory tests have shown that for model piles in dry sand (1) the ultimate bearing capacity of a group of piles may be larger or smaller than that of one pile times the number of piles in the group, (2) the settling of a group of piles can be many times greater than the settling of an isolated pile, and (3) the extrapolation from models to prototypes is difficult.⁶⁴ Even where piles penetrate to and are supported upon sound rock their elastic deformations significantly affect the response periods of the structures they support.^{65,66} In those cases where piling might be driven through sand to rest on support rocks, complete load transfer to the pile tip might take place only in

liquefaction, when vertical settling of an inch or two might then result solely from compression of the pile. In addition to the main plant structure, other safety-related portions should be examined for possible earthquake-precipitated accidents, especially for the radically different support arrangement proposed for some reactors that are to be on sites of poor soil.

Reliable data in the whole field of soil-structure interaction are so scarce that it does not yet appear possible to assign specific engineering values of demonstrable validity for reactor plants, although more rational approaches are being developed on some pile support problems.⁶⁷ Very little is known as to how short, stiff piles modify the soil-structure interaction. Apparently no one has put dynamic instrumentation on pile clusters, and in fact few measurements have been made with static instrumentation; this must be done. Both differential and mass settling of entire mats must be measured, in addition to settling of loaded single piles or loaded plates.

3.8.3 Excitation Energy and Its Dissipation Through Elastic Damping

The energy losses in an excited system, which are collectively called damping, must go into generating heat. In a solid mechanical system the important mechanisms of damping are either internal friction or dry friction. Internal friction is generated by the relative displacement of elements such as the crystals in the solid. These are related to inelastic behavior. Measurements can be made for a "hysteresis" loop of a force-displacement curve. Dry friction can occur when elements are forced to rub over each other, as in bolted or riveted connections. In responding elastically, a structure will dissipate by internal friction only a small amount of the input energy, only 0.1 to 0.3% of critical damping in the "truly elastic" regime, and the remainder goes into dry friction, strain energy, or kinetic energy. Dry friction damping can be large in riveted and bolted joints but will be relatively small in welded steel structures, as well as in all other homogeneous solids, including prestressed concrete and even conventional reinforced concrete before it is cracked.⁵⁶

Energy absorption on the faying surfaces, as in cracked concrete, on the other hand, can be large.

Equivalent viscous damping is almost always the basis of practical evaluation, although it can be a relatively poor approximation in many cases. Damping in structures cannot be verified conveniently because it may be in the form of friction internal to the material, plastic deformation, slippages between elements, or energy imparted to fluids surrounding the structure or component. Proper values are hard to determine, especially since each vibration mode has its own damping factor.⁵⁹ Relative rigidities at short periods of vibration can vary greatly from those indicated by the fundamental mode.⁶⁸ Fortunately, damping has little effect on the response period, for even at damping up to 20% of critical the computed period is changed by less than 2%.⁴¹ It is not known generally how damping varies with amplitude, and it is not known how hysteretic properties vary with excitation duration. Careful measurements on specially prepared samples have indicated that pure, unfractured high-quality concrete damps at near 1% of critical.⁶⁹ In assigning damping levels for reinforced concrete, if it is assumed that the designs for earthquake-induced stresses are so low that cracking and breaking in the concrete are unimportant, strain and thus damping factors based on energy absorption in the concrete must also be quite small.

Reducing the percentage of critical damping from 5 to 2% can, in some cases, double the response of a structure and thus have an important effect on the design. A small percentage change in the damping coefficient in the range 0 to 5% of critical can cause large differences in response, whereas above the 10% level the same percentage change would have little effect on response. A decrease in an already low damping factor has an effect that can be approximately equivalent in design calculations to raising the intensity of the specified earthquake. Several years ago, due mainly to incorrect interpretation of measurements, the generally assumed damping factors were appreciably larger than at present — often by factors of 2 and in some cases by several fold. Table 3.2 gives examples of assigned damping factors for three PWR plants; average values for four BWR plants, Dresden II, Quad Cities, Monticello, and Browns Ferry, are 5% for concrete, 2% for steel, and 0.5% for vital piping. These values

Table 3.2. Damping Factors for Three PWR Reactors

Component	Percentage of Critical Damping		
	San Onofre ^a	Connecticut Yankee ^b	Malibu ^c
Reactor vessel internals and control rod drives			
Welded assemblies	1.0	1.0	1.0
Bolted assemblies	2.0	2.0	2.0
Reinforced-concrete reactor support structure, including reactor vessel	4.0		7.0
Vital piping systems			
Carbon steel	0.5	0.5	0.5
Stainless steel	0.5	1.0	1.0
Containment structure and foundation	4.0	7.0	8.5 to 9.5 ^d
Steel frame structures, including supporting structures and foundations			
Welded	2.5	1.0	1.0
Bolted	2.5	2.5	2.5
Concrete structures above ground			
Shear-wall type	7.0		7.0
Rigid-frame type	5.0	5.0	5.0
Mechanical equipment, including pumps, fans, and similar items		2.0	2.0

^aFrom Ref. 70.

^bFrom Ref. 71.

^cFrom Ref. 72. Lower damping factors will probably be used in the revised design.

^dTentative; subsoil shear modulus of 60 ksi assumed.

represent practice at the time of application, and the bases on which they were established necessarily are much less dependable than desired. Ordinary concrete has appreciable cracking throughout its volume, and some specialists believe damping should be between 2 and 10% of critical. For

prestressed concrete, since it is designed not to yield, the range is thought to be 0.5 to 2%.

Although piping, even important piping, is designed to undergo yielding, it is only allowed in limited amounts in limited regions and, since it involves only a small volume of the total, the overall damping factor is still low. It is thought that for current large PWR containment vessels the mode that includes rocking should be designed for no more than 5% critical damping, and at higher modes damping should not go above 2% critical. They feel that higher values could be realistic only if sufficient yielding took place to result in substantial cracking, and this in turn would mean deterioration in the containment vessel's function. Blume estimates from extrapolation of other work on concrete shear-wall structures that damping in concrete reactor buildings is between 5 and 8%.¹ Newmark states that homogeneous solid structures, such as welded steel or uncracked concrete, will absorb little energy.⁵⁶ For low stress levels before cracking he estimates damping of no more than 0.5% critical; at half yield for a well-reinforced or prestressed-concrete structure he estimates 3 to 5% critical damping. At near yield, damping may be 5% for steel and prestressed concrete that has not completely lost its prestress, and he thinks proper values should be 7 to 10% critical for ordinary concrete and up to 10 to 15% for structures with play in their joints. In the inelastic range he estimates as much as 1% damping for homogeneous structures, whether steel or concrete, and 15 to 20% if there is cracking or play in the joints.

Since most specialists believe containment design must stay in the elastic range, they believe the damping values should be the same for the design-basis as for the operating-basis earthquakes. Others maintain that the larger earthquake would have larger displacements and therefore cause larger plastic deformations in the foundation. Unfortunately, it is difficult to separate the damping solely within the structure from that also involving soil structure interaction. It appears that where these two effects are directly involved (for example, in bases) that the dissipation of energy could be far higher than when only "elastic" damping occurred (for example, between structures and their bases or between appendages

and structures). Data from measurements of damping for actual power reactor systems are not available.

Although rarely if ever used in the reactor industry outside the application of snubbers on certain vital piping, there are many methods of structural isolation and damping that have been developed in other efforts, such as in the design of hardened missile systems, that might be applied. Many reactor architect-engineers consider early improvement of the information on damping in equipment, instruments, and piping, as well as in gross structures, to be among the greatest needs in the power reactor industry.

3.8.4 Inelastic Structural Response

If the net energy input to a structure in excitation exceeds its elastic strain-energy capacity, response will become inelastic, and part of the excess energy will be dissipated through yielding (that is, in permanent distortion). Duration of shaking becomes much more important where repeated yielding can occur. So far, little work has been done on inelastic design of reactor structures, although it is general practice to design piping for limited yielding under operating conditions. Although a power reactor plant is usually stronger than a conventional building up to its yield point, it is more brittle both because it tends to respond to shear rather than flexure and because the reactor plant has few energy-absorbing nonstructural elements.¹ Elastic design has an advantage, in addition to its relative simplicity, because the problems of fatigue are much reduced over those of inelastic design, and damage control features can be included more easily. Recent improved evaluations of measurements made during an earthquake which a conventional structure survived in apparently good order have indicated that if the responses had all been elastic, strains would have been far greater than the materials would have allowed.⁷³⁻⁷⁵ It is not yet possible to explain the exact mechanism by which many conventional structures have survived earthquakes that were, theoretically, far beyond those they were designed to survive elastically. Elastic response analyses show responses far greater than would be surmised from investigation of the buildings. The currently accepted explanation is that the structures absorbed energy through yielding.

However, no arguments have been found for or against explaining the survival through use of base response concepts, as is currently done for equipment studied in U.S. Navy shock tests.

Designs based on plastic or limit concepts generally should result in structures that are no more expensive than those from ACI-basis elastic working designs, and they may be significantly less expensive. Plastic and limit designs are now widely used in conventional structures. The point is reached in practice where, if the design earthquake is to be increased, at least for a conventional structure, say, by a factor of 2, it may be much cheaper to allow for an assured energy dissipation capacity through yielding than to increase the sizes of structural members to undergo the increased excitation elastically. As specified earthquake intensities are increased, these concepts must be applied to power reactors. If the structure could be designed with multiple load paths at critical points, it should be less prone to fail than one with fewer load paths.⁷⁶ It has been found that spectra from elastic responses can be used in some cases to obtain estimates of the deflections in yielding.^{77,78} The validity of such calculations may depend significantly on the response periods.

Where a member is subject to reversal of loading, as in earthquakes, reinforcement must be placed to carry all loading that the concrete cannot carry, and this may not be easy to anticipate in reinforced-concrete structures under accident conditions. There is also a requirement for confining the member. For example, spiral wrapping of columns is commonly used to insure confinement and ductility in compression, although it can present some problems in fabrication and erection. The advantage in earthquakes can be large; inspections of a number of earthquake-damaged buildings have shown that spirally wrapped columns seldom fail. Ductile bands can often be used instead of spiral wrapping and are more convenient in fabrication and construction. At present ductile members apparently are considered by the majority of designers as only supplementary to more conventional members, but one manufacturer says this is now standard design code practice and is being used. The latter opinion may stem from a different definition of "ductile," for all concrete design has as an

objective the yielding of reinforcement bars before the yielding of the concrete. Although it may be difficult to design all members for ductility, it seemed a satisfactory and practical means for insuring against collapse of most elements, at least before the 1967 Caracas earthquake. Questions of the extent of damage of structures designed to be ductile are yet to be clarified.

Ductile concrete may have little application in the main reactor plant structure, where many reactions are dominated by shear, rather than flexure, and where shear walls are usually the practical structural elements. It may thus be preferable in these designs to stay in the elastic range. It is worthwhile to use deflection criteria for both the lower and higher design excitations, since the requirement of no loss of function usually involves, at least for structures, deflection criteria, as well as those for stress. It is not unusual for deflection to be the limiting factor; indeed it can be the dominant factor.

Experimental results are scarce for many important earthquake conditions, including important cyclical reversing loads into the postyield range.⁷⁹⁻⁸¹ Worthwhile dynamic tests require much care to insure proper design and execution, as well as accurate recording and analysis of the data. It is difficult to deal simultaneously with all relevant time-dependent properties, and valid tests become more complicated if composite materials vary in characteristics with the rate of strain. In both research on conventional concrete structures and in the design of such structures that are to be carried beyond yield, emphasis must be placed on preserving ductility. Some of the most difficult problems in ductile concrete involve compression, especially in columns where concrete tends to flow.

It is thought that yielding does not increase the total energy input to a structural system.⁸² However, when yielding causes a shift in response frequency that increases the magnitude of response, more energy may be absorbed. In addition the duration of this excitation and the number of excursions of elements beyond their elastic limits must be considered, as well as the number of reversals. Even major earthquakes seem to exhibit relatively few response lurches sufficiently violent to carry well-designed structures into the inelastic range,⁷³ but there are few strong-motion measurements collected near the fault trace of even modest

earthquakes on which to base such a judgment. The number of such lurches is estimated to have been five or six in the El Centro earthquake, which lasted approximately 1/2 min. The number might vary, perhaps widely, depending on the structure and earthquake.

3.9 Design of Specific Systems*

In an earthquake, the chief difference between a reactor structure and an ordinary building is the reactor's critical equipment. It is essential that detailed evaluation of margins of safety be made on all safety-related systems whose functions might be impaired by earthquakes.⁸³ These margins of safety only apply, of course, if they are properly carried through construction.

From a reactor safety viewpoint, the principal elements of the plant (frequently designated Class I) whose continued function is of particular interest during and following an earthquake are (1) the reactor core, including the structural supports of fuel and control elements, (2) the primary coolant pressure boundary, including pressure vessels, piping, pumps, valves, and heat exchangers, (3) normal and emergency coolant supply systems, including coolant storage and ultimate sources of coolant at the site, (4) control systems for matching rate of energy release in the reactor core with the prevailing capacity of heat removal systems under normal and emergency modes of operation, (5) secondary containment and other effluent control measures, (6) radiation shielding, (7) fuel storage facilities, and (8) normal and emergency power supplies. The importance of these features is in preventing an accident that would release radioactive material to the environment or in confining radioactive materials within the facility in the event any of the set of design-basis accidents occurred. Achievement of both these goals during a design-basis earthquake requires only the usual earthquake engineering practices for most parts of a plant. Some features, however, require special attention because either they are unique to nuclear plants or in nonnuclear applications they are not usually of such importance to safety. Specific features are

*See Refs. 1 and 2 for details on this subject.

discussed here in detail to illustrate the unusual and important technical areas into which earthquake engineering must extend for nuclear facilities.

Since structural elements, items of equipment, and instruments vary so greatly in their functions, in the materials from which they are made, in the ways they respond to excitation, and in the ways their responses and their proper functions are related, any design bases and limitations established for a nuclear plant must at least take into consideration the important functional categories to which individual elements belong. In some cases the design limits for parts of units may be different from those of the unit as a whole.

At present it is reported that the BWR reactor, along with the other Class I items that are a part of the nuclear steam supply equipment, is generally designed for a seismic load of 0.50 g horizontal and an equivalent static load at normal stresses of 0.07 g vertical.^{84,85} Designs allow loads of twice these values without causing deformation large enough to result in functional failures. Other Class I features of this type of nuclear power plant are designed on the bases prescribed for a specific site. Some components in a reactor plant do not have a high level of reserve energy:¹ buckling of lower steel plates often controls BWR drywell design; pipe designs are often controlled by loads on supports and nozzles rather than pipe flexure; shock-suppressing pipe supports have virtually no reserve strength beyond their maximum linear load; and concrete-embedded anchor bolts for equipment support also have very little reserve energy in shear.

3.9.1 Reactor Core

When subjected to earthquake vibrations, many reactor core elements act as beams on end supports.⁸⁶ Those in-core portions of any control system that would not necessarily fail to a safe condition under earthquake loadings must be designed with special care, and it is an overriding objective to provide substantial seismic safety margins for the core elements. One consideration in design is that the support conditions, and therefore the stress distributions, may be quite different in testing, shutdown, accident, and operating modes.

While the costs of initially developing an adequate procedure for dynamic analysis of reactor pressure vessel internals (i.e., the core and associated components) to demonstrate seismic survival are substantial, the costs of subsequent similar analyses should not be large. Dynamic tests have been performed on BWR's cores, and theories have been developed for conditions both with and without fluid. Load tests on PWR fuel elements have been made to obtain their deflections and moments of inertia. The current practice of most manufacturers is to have cores capable of undergoing excitation of at least 0.3 g without change in power. Recent experiments at the Westinghouse Atomic Power Division showed that the maximum summed deflection of control-rod-cluster guide tubes subjected simultaneously to blowdown forces and to twice the "design earthquake"* was 76% of that allowed in the design.⁸⁷

3.9.2 Piping Systems

As all real elastic systems, piping has an unlimited number of vibrating modes and natural frequencies. Both primary coolant piping and piping installed solely for accident conditions play central roles in the earthquake safety of reactors. The response characteristics of piping are radically different from those of most other elements in reactor plants. The mass of the piping system is quite small in comparison with the major portions of the reactor plant, so it responds in many cases as a special multiply attached type of appendage. Proper analyses of piping are quite complex, inasmuch as a three-dimensional lumped-mass model of piping can have from 30 to 150 degrees of freedom. Pipe supports are designed to accommodate other conditions in addition to earthquakes, and there is a tendency for incompatibilities between designs for temperature changes, where flexibility must be insured, and those for earthquakes, where firm support is needed. The best method of dealing both with desired rigidity for earthquakes and flexibility for thermal expansion is to establish initially common mass points for both the dynamic and thermal analyses.

*The lesser of the two design earthquakes; see Glossary, App. A.

For those cases where the response period of all the piping can be made sufficiently short, some large computer static piping design programs, such as the program used at the Mare Island Navy Yard,⁸⁸ have already incorporated capacities for calculations based on seismic coefficients. Response spectra or time-history calculations can be used, but the former, which require less computer time, are thought to be less accurate. It should be kept in mind that there is no way to apply the response-spectrum technique where supports are widely separated in the structure.

Testing studies of power reactors that are no longer to be operated should be initiated, and studies of the piping should be included. The Japanese have run autocorrelation tests on power plant piping,⁸⁹ and their work should be reviewed for extension and application. They obtained much information on the distribution of vibration modes and vibration sources but little information on the damping of the system.

The Japanese practice in designing piping, equipment, and instrumentation is to use (1) a seismic coefficient of 3.6 times that used for rigid items in their code for conventional buildings, or (2) results from time-history calculations, or (3) values 1.2 times the seismic coefficient at the supporting points.⁹⁰ Vertical forces for Class A equipment are assumed to act simultaneously, and Class A items are designed to function during the "maximum-potential" earthquake, which is defined as 1.5 times the "design-basis" earthquake.*

Several approaches have been used for designing equipment and piping in the United States. It was said at the IAEA meeting in 1967 that the primary concern has been to avoid the natural frequency of the supporting structure and to have both ductility and maximum rigidity while allowing for necessary thermal excursions and possible unequal settling.⁹⁰ Dangers from jet forces and whipping of broken pipes were said to have been taken into account. Vibration tests were said to have been used where estimates were difficult to make. The IAEA panel recommended the formulation of qualification procedures.

*The Japanese terminology used here for design bases should be compared with current USAEC terminology (see App. A).

Piping systems generally can yield into the plastic region, where damping is large, and still maintain their integrity. Indeed it is established practice to allow carefully specified limited yielding from secondary stresses even under ordinary operating conditions. Nevertheless, current earthquake design practice is to assume elasticity and to assign piping a damping factor no larger than 0.5%. If the pipe supports are appreciably displaced in an earthquake by the response of the containment system, a design that has been based solely on static simplification can result in an unsafe condition. Conditions are more difficult to analyze if the natural period of the piping system is near the spectral response peak of either the structure or site or if there is a significant contribution to its vibration from higher modes. Pipes connecting two structures on separate foundations will be subjected simultaneously to the motion of both. There are, of course, obvious ways of improving earthquake survival of vital piping, as by building integrated plant structure foundations, by using special piping supports, by installing snubbers, by unusually careful design and construction at points where pipes penetrate barriers, such as the containment vessel, and by providing displacement capacity where needed.

Damping must be considered in setting limits in design, and proper damping values may be difficult to verify under conditions of design. Dynamic designs of elements must consider both masses and stiffnesses and thus be related to strength. The elastic design is not always necessarily conservative nor is the inelastic design necessarily unconservative under one specified condition of excitation. Thus satisfactory failure criteria must be specific and detailed, for generalized criteria can be both impractical and unconservative.

Wherever safety-related units are designed to allow yielding, it must be determined that any displacement will not compromise other necessary qualities, such as the capacity for safe shutdown. For example, the aim in earthquake design of steam generators⁵⁵ is to hold them sufficiently close to their operating position to avoid either rupture of the main piping or collapse of the supporting towers. There is an additional large force on steam generators if a main steam line is ruptured. Uncertainties of steam discharge during and following intense earthquakes

make it difficult to establish design levels for all reactor containment systems, whether primary or secondary. Also the failure criteria are greatly different for different materials of construction. They depend for example, on whether the containment structures are of steel, reinforced concrete, or prestressed concrete.

One manufacturer's statement on the criteria for allowable stresses in vessels and piping was: "In the matter of earthquake excitation, it is generally assumed that the piping is responding at the peak of the response curve as this provides maximum safety margin in the determination of pipe hanger loads. The actual hanger spacing on the piping to provide proper drainage results in very low stress levels in the piping (stiff systems) so that the response to excitation is less than half that assumed."⁹¹ It has been reported that reactors have recently been designed in such a way that the forces associated with two times the operating basis earthquake would cause only slight yielding of the primary system vessels.⁹¹

3.9.3 Secondary Containment System

From the viewpoint of earthquake design, the secondary and primary containment systems are among the least conventional of the major structures in power reactors, in part because they must be designed to survive with integrity either a high internal pressure or seismic excitation, as well as both conditions simultaneously. It is otherwise rare, except in certain tanks and pressure pipes whose failure would have little safety consequence, for reinforced concrete to be used where heavy tension constitutes the primary design load. It is thought that penetrations and seals would be less vulnerable to excitation than to distortion, as from fault slippage. For each case integrity might be demonstrated by non-destructive tests at the location. Although seismic and wind forces cannot be ignored, accident pressure forces usually are the largest, and they often dominate under present design levels. If limit or plastic designs are allowed instead of the presently required elasticity, the enormous simplification of superpositioning will be lost and the already highly involved design problem will be further complicated. Table 3.3

Table 3.3. Examples of Principal Seismic Design Criteria for Secondary Containment Structures^a

	San Onofre	Connecticut Yankee	Malibu ^b
Horizontal ground acceleration of maximum probable earthquake	0.25 g ^c	0.17 g	0.30 g ^d
Stress basis	Working ^c	Working	2/3 yield ^e
Ratio of vertical to horizontal ground acceleration	2/3 ^c	2/3 ^{b,f}	2/3 ^g
Provision for safe shutdown in earthquake and accident conditions	No loss of function at 0.50 g ^c	Stress less than or equal to yield and no loss of function at 0.17 g ^f	Stress less than or equal to yield ^e at 0.45 g
Maximum horizontal ground acceleration estimate by Coast and Geodetic Survey	0.40 g ^h	0.17 g ⁱ	0.30 g ^j

^aSee Appendix B for details on geologic and seismologic evaluation of five reactor sites, including two of the above.

^bCurrent status does not include possible design provisions for fault displacement.

^cFinal Engineering Report and Safety Analysis, Section 11.2, Ref. 70.

^dPreliminary Hazards Summary Report, Amendment 3, Exhibit C, Fig. 1, Ref. 72.

^ePreliminary Hazards Summary Report, Amendment 7, Exhibit F, p. 9, Ref. 72.

^fPreliminary Hazards Summary Report, Amendment 2, Section 2.5 (rev.), Ref. 71.

^gPreliminary Hazards Summary Report, Amendment 8, p. 3, Ref. 72.

^hU.S. Coast and Geodetic Survey Report, Oct. 4, 1963.

ⁱU.S. Coast and Geodetic Survey Report, Jan. 9, 1964.

^jU.S. Coast and Geodetic Survey Report, Nov. 24, 1964.

gives typical examples of principal seismic design levels for existing and proposed PWR secondary containment structures.

Vertical seismic loads could result either from the vertical component of the earthquake motion, from a rocking mode in response to the earthquake,⁹² or from differential movement of the foundation either before or during the earthquake. Spectra for the vertical component are relatively accentuated in the short-period range and reduced in the longer-period range.⁹³ Problems involving responses to vertical components of earthquakes have received relatively little attention until recently, although they can be quite important in concrete containment design. The previous lack of emphasis resulted from the fact that the large safety factors used in the static design of conventional buildings almost always insured ample capacities to survive the design earthquake's motion in the vertical direction.

Since almost all reactor plant structures have short natural periods, they would be disproportionately affected by earthquake forces of high excitation intensity compared with structures of longer period having the same lateral strength.⁹⁴ Maximum stresses induced by the design earthquake on spherical steel containment vessels can be far smaller at many points than those caused either by wind loads or postaccident pressure. Containment shells of the type used for PWR's are highly rigid and have predominant responses that are influenced principally by foundation characteristics and by mass distributions.⁹⁵ Due principally to mass differences, seismic forces on concrete containment shells may be as much as tenfold greater than for steel containment vessels of the same volume. The main vibration of one secondary steel containment structure was estimated to be by "lobaring" at 3 to 4 cps.⁹⁶ For a steel BWR drywell, appreciably different values of 8 cps flooded and 12 cps empty have been found. Steel shells are also subject to buckling and, possibly, to brittle fracturing, which are much more serious possibilities than "lobaring." Apparently a combination of shaking and tilting presents a much more important problem than tilting alone.

In BWR's, under present design levels, direct earthquake forces may give maximum stresses in steel pressure-suppression systems that are

around 1000 psi, which is quite small compared with those from the peak loss-of-coolant accident pressure. However, the design earthquake can dominate some of the shear loading on these structures. The BWR containment system, whether of steel or concrete, is more rigid than the steel building around it and thus would respond with a smaller amplitude. In an intense earthquake the building would tend to pull over the drywell, and, in this case, seismic shear stresses, even in a steel BWR containment vessel, would be large. The forces resulting from the building-drywell interaction under seismic excitation would have a far greater effect on resulting stresses than would inertial forces on the drywell alone. Even the more conventional reactor structures (e.g., a steel BWR reactor building) should be designed with care, for no simple statement even on the effect of bracing of such a relatively conventional structure can be made.⁹⁷ In some BWR buildings, the base, drywell, and the lower portion of the building are of concrete and the upper portion is of steel. Since the upper portion has all the characteristics of an appendage, it must be so designed.⁹⁸

In the "light-bulb" drywell design, the steel drywell may be of hemispherical shape on its bottom, and concrete may be placed on its inside to provide a floor. It is not always clear how shear can be transmitted through this arrangement. The BWR's drywell is usually designed so that it is isolated from the building, except at two points — where it is concreted at the bottom and where it is supported at the top. The drywell periods usually vary from 0.05 to 0.20 sec, and the period of the suppression chamber is about 0.20 sec.

Some who most strongly advocate prestressed secondary containment structures claim that a careful examination of conventional secondary containment structures in either excitation or fault slippage is a strong incentive to accept prestress design. They believe the experience at Anchorage, where most prestressed structures that failed were made of pretensioned, precast elements, to be misleading and that the detailed care expected in reactor construction should insure a superior structure. The statement has been made that all major vibrational damage of prestressed structures at Anchorage could be attributed to construction procedures, but, as Wood indicated,⁹⁹ no damage would imply perfect

standards. It is apparent that a dynamic analysis of a prestressed secondary reactor containment structure has never been published, although such structures have been approved, perhaps on a static basis. It is thought by some advocates of prestressed concrete that while it would be quite superior below the yield point, it would be about equivalent to conventional concrete beyond the yield point. As with reinforced concrete, much less work than needed has been done on prestressed concrete under combined tension and shear.¹⁰⁰

A concrete containment shell is so massive that earthquake-induced stresses may require design for high loading, especially where the containment shell joins the base. Postaccident leakage of fission products from a PWR is prevented by a steel liner, but, in the unlikely event that the liner failed under accident conditions, stresses from internal pressure could far exceed the tensile strength of the concrete alone. Heavy reinforcement is considered by some to be necessary to control cracking widths to keep them below 1/16 in., but other measures have been suggested. Some experts do not believe that estimates of leakage through concrete structures without liners are or can be reliable; they say that even barriers with no detectable cracks still leak prohibitively. Subjecting containment vessels to test pressures tends to form cracks, and such testing must be monitored carefully to insure that the structure is not thereby weakened significantly. If the design is so severe that cracking under excitation is minimized, damping is thereby necessarily low. More fundamental study is needed in the control of cracking.¹⁰¹ Neither the strength nor stiffness properties of reinforced concrete subjected simultaneously to large axial tension, flexure, and shear forces are known precisely from experimental data.

Concrete has high capacity to transmit shearing forces when under compression but has very poor ability to do so when under tension. There is disagreement among engineers on whether fundamental research aimed at increasing the tensile strength of concrete could yield practical results in the foreseeable future. Under earthquake excitation difficulties would develop in containment structures if there were any cross-section area of concrete that was subject to shearing forces while not simultaneously under compression. Even where the shearing forces on concrete under tension are

relatively small, design problems are not easy, for complex and possibly quite unsatisfactory earthquake effects might result. Diagonal reinforcement would help to avoid complete reliance on concrete for shear transmission; Gluckmann says that either diagonal reinforcing or prestressing is necessary.⁵⁵ If yielding occurred the cracked concrete would have to have sufficient integrity to remain bonded to the reinforcement even under the rapidly changing, reversing loads that earthquakes would place on the structure. Following the great 1964 earthquake, an investigation of failed conventional structures at Anchorage led to the suggestion that only shear walls that are completely determinant should be used.⁹⁹

In an earthquake the steel containment vessel liner might interact with the concrete in a fashion not easy to predict, and vertical accelerations could subject the containment structure to some unusual stress situations that in some cases might greatly increase tension in the reinforcement. Careful testing would be worthwhile. The proper design of openings in reinforced-concrete containment shells is complex. In some cases steel framing may be required, especially for the large openings needed for personnel, equipment, and fuel access. As in many other cases, good design must be based on many quite conservative assumptions, some of which must be compounded with others.

3.9.4 Control Features

Instruments in the reactor's control and safety systems, as well as those in many other portions of the plant, are vital to the operation and safe shutdown of the plant, including shutdown in the event of the maximum design earthquake for the site (i.e., the design-basis earthquake). For these instruments it is the effect of vibration on function, rather than damage, that must be considered.

One of the difficulties in reactor control system design on which there is little documented experience is the identification of components of the system that might be caused to malfunction under earthquake excitation. Many specific potential malfunctions arising from other causes have been identified, and these form the design bases for some of the reactor protection systems and engineered safety features currently in use. However, none of these protective features have been designed for a reactor

control malfunction caused by vibration, which is a sufficiently likely event to warrant investigation in the course of plant design. Typically the kind of malfunction that must be considered is one that could lead to the adverse movement of reactor control rods as a group, since this has not been considered in establishing design bases.

If, in the course of design, components are identified whose malfunction would lead to abnormal conditions beyond the capacity of the usual protective features, several alternative courses of action may be considered. One remedy that would generally be technically feasible and economically justifiable would be to design and test the critical component to assure proper functioning while undergoing vibration.

Instrument and control circuits have been built for ships, aircraft, and special military applications that could not only survive strong excitation but would continue to function during earthquakes. The advent of solid-state circuits has allowed unusual improvements in capability for design for many extreme conditions. Since many instruments and controls and their related circuits, connections, and mountings are usually a part of the reactor safety system, license applicants must give assurance that they will survive specified levels of earthquakes. Careful designing, detailing, and construction are necessary to assure seismic survival of connecting wiring. The IEEE Criteria for Nuclear Power Plant Protection System¹⁰² require that the design basis shall document, as a minimum, "the malfunctions, accidents, or other unusual events (e.g., fire, explosion, missiles, lightening, flood, earthquake, wind, etc.) which could physically damage the protection system components or could cause environmental changes leading to function degradation of system performance, and for which provisions must be incorporated to retain necessary protection system action;"

The excitation of each component of the plant's normal control system and the power supplies and controls for engineered safety features, including protection systems, should be determined for the maximum design earthquake. The magnitude of the vibration problem is illustrated by a time-history elastic design based on 6 sec of the El Centro record normalized (i.e., reduced) to maximum recorded acceleration of 0.25 g.

Calculations for a large BWR have shown that the peak response acceleration on the top floor of the structure can reach 20 g (see Fig. 3.13), which is an 80-fold amplification in a narrow frequency band.¹ The same record gave a ground response acceleration of 1.6 g. The structure used in this calculation was of reinforced concrete, and if it had, instead, been steel the responses would have been significantly greater. In either case, it is unlikely that response spectrum techniques, with their necessary approximations, would predict these high values.¹⁰³ Such great amplifications are also known in simplified theoretical models.¹⁰³ Where the mass of an attachment is much smaller than the structure to which it

Fig. 3.13. Response Acceleration Spectra for Floor 1. (From Ref. 1)

is connected and the two have natural frequencies that are quite close, the magnification factor can be very large. Any safety-related instrument might have additional amplification in its support, rack, etc., that must be taken into account once the input values are known, since it would act as an appendage in many cases. No records have been found to show that shaker-table testing of specified central-station reactor power plant instrumentation, controls, or protection systems has been done to validate calculated responses to design earthquakes.

It appears from information presented in safety analysis reports that the designs of all safety-related equipment and instruments, as well as emergency power supplies, are based solely on the specified ground motions. The accelerations that some safety-related instruments might experience in the specified earthquake therefore could be two orders of magnitude greater than the ground acceleration.

3.9.5 Seismic Instrumentation for Reactor Protection

Seismic instruments are being employed for automatic shutdown (scram) systems in several experimental reactors and apparently for all fissile-material production reactors;^{104,105} use of these instruments for power reactors is now being debated. The power and utility companies seem to prefer that reactors should be operable during and immediately after major earthquakes in order to provide electric power during the period of recovery. (It is of interest, though, that the loss of power in Anchorage, Alaska, at the time of the magnitude 8.5 earthquake of March 28, 1964, was considered to be providential in that many fires were probably avoided because of the lack of electric current in the severed wires.¹⁰⁶)

In regard to the safety of operating reactors during large earthquakes, it has been pointed out that all critical components of reactor plants are designed to withstand the strongest earthquake-induced ground motions that might be expected at the site (i.e., the operating-basis earthquake).¹⁰⁷ In addition, for those items which effect the shutdown or assure the containment integrity of the system, further examinations are made to insure their performance during even stronger seismic forces, as in the design-basis earthquake. Shutdown and containment during severe accidents are

normal functions of the automatic protection systems. Thus it is usually concluded that this design procedure provides all the necessary automatic protection even for earthquake-induced failures, such as a loss of coolant. However, this design philosophy is somewhat compromised by the fact that protection systems and other engineered safety features are not (and some cannot be) dynamically tested at the specified excitation of the reactor site.

The standard triggering device used by the U.S. Coast and Geodetic Survey to start recording seismometers is utilized as a seismic switch in several reactors; commercial shock switches, most of which are patterned after the U.S. Coast and Geodetic Survey starter, are employed in other reactors. Figure 3.14 shows one of these devices. Basically these instruments are damped vertical pendulums surrounded by adjustable rings with electrical contacts that are activated by tilting or horizontal movements. By varying the pendulum gap the switch can be set to initiate signals in response to a specified ground motion (usually expressed on the Modified Mercalli scale, which indicates the nature and violence of the ground motion in a particular area) and/or acceleration.¹⁰⁴ Another design involves the use of a brass ball that completes a circuit over two terminal strips. When the ball is dislodged from its normal resting place by lateral forces, the circuit is interrupted. Although all these instruments are perhaps satisfactory for their intended purpose, it has been pointed out that they provide no further information on ground motions after the first impulse, and they may respond to a maximum impulse of a small earthquake as readily as a similar impulse from a larger earthquake.¹⁰⁴

The objections to this type of input to a seismic scram system can be largely overcome by incorporating a triggering circuit in a strong-motion accelerograph or a seismograph to provide a visible record of ground motion, as well as a reliable scram signal based on more significant characteristics of motion. The Japanese Power Demonstration Reactor has an operating system of this type in which eight accelerometers are installed in cells on the head of the caisson wall. The pendulums of these accelerometers are electromagnetically damped to increase the

Fig. 3.14. Seismic Switch, Model SP-210, United Electro Dynamics, Inc. (Photograph supplied by Teledyne Company)

sensitivity of the instrument at frequencies greater than 10 cps. The accelerometers are connected to an electromagnetic oscillograph that operates automatically following excitation of the starter.^{108,109}

Strong-motion accelerographs can also be incorporated into scram systems, as follows. The output of an accelerometer is fed into an integrator, and the resultant dc potential is used to actuate a relay, set at a predetermined threshold, which then activates the scram system. The strong-motion recorder would continue to operate for a preset time after being started by the seismic switch. This approach could be used to eliminate the possibility of activating a scram as a result of a large initial acceleration of a small earthquake (i.e., if such accelerations were, indeed, not damaging to reactor components) and would allow a record to be made of the earthquake for study and spectrum analysis.

3.9.6 Dams

Nuclear plants in many localities will be dependent on dams. Dams are used routinely to provide condenser cooling water for power plants and may provide, as well, sources of emergency cooling water.

Masonry Dams. All masonry dams can be divided according to the ways they are designed to maintain stability. Gravity dams are of approximately triangular cross section, and they achieve stability against overturning and sliding principally by their low center of gravity, great mass, and consequent large foundation bearing force. Arch dams, on the other hand, must be keyed into the walls of a comparatively narrow valley, and these supports must be of unusual stability. Among the many variations and combinations of these two concepts, buttress dams, which are composed of relatively thin water-retaining barriers supported by beam and strut arrangements, are perhaps most important because they are now usually less costly than gravity dams.¹¹⁰ Designs have been projected for buttress dams up to 750 ft high, which is comparable to the 770-ft height of the new soil-embankment Oroville dam in California.¹¹¹

In a general discussion of dam failures at the Ninth International Conference on Large Dams,¹¹² it was stated that

"A long uneventful period of service is no guarantee for the future safety of a dam. Distinctive types of dams can usually be traced back

to an original prototype. The basic conception embodied in this prototype is then applied with increasing skill and refinement to new works until unexpected failures make it evident that the previously accepted theories and design procedures which have proved satisfactory when used for works of moderate scale ... are not suitable for those on a large scale. It is not possible from purely theoretical considerations to determine either the ultimate height for which any specific type of dam can be developed, or the maximum economy that can be achieved from materials of construction.

"The failure of a gravity dam as a result of overtopping will not necessarily occur where the height of the dam is greatest. On the contrary, it occurs at that section which is most subject to excessive strain. These are liable to occur where there is a sudden step in the foundation level.

"Sudden and total failures have occurred to both arch and gravity dams situated in narrow valleys. Progressive failure may overtake earth dams and those standing on erodable foundations.

"In spite of the heavy destruction to life and property which usually follows in the train of an earthquake, there is no evidence, apart from the occurrence of slips on the upstream face of embankment dams, that such seismic shocks have in the past caused the failure of a dam."

Earthquake design of masonry dams has up to the present generally employed¹¹³ some horizontal static equivalent seismic coefficient, together with another horizontal force calculated to result from the interaction of the dam and reservoir in the earthquake.¹¹⁴⁻¹¹⁶ More recently attempts have been made to employ the response spectrum technique in arch dam design. The results of response spectrum analyses compare favorably with numerical evaluations of arch dams.¹¹⁷

As is the case in many other structures, there may be important modes that are not well separated in frequency, and this makes realistic response spectrum calculations difficult. For example, model tests have indicated that the second symmetric and first antisymmetric vibrations can occur at the same period.¹¹⁸ Also, others have found that arch dams

give complex resonance patterns; for example, two normal vibrations were found to have equal periods and quite different modes.¹¹⁹

Arch dams are not designed to undergo significant tensile stresses, and they have little reinforcement steel. Dungar¹²⁰ says that in a strong earthquake "the resultant cracks would be disastrous in a water-retaining structure. It is concluded that the purpose of any earthquake design consideration in an arch dam should be to insure that the structure remains in the elastic region of stress under the earthquake loading," and "It would appear that the displacement and stress responses of about 1.4 times hydrostatic may be expected if a strong-motion earthquake occurs within 50 miles of a dam. This is a tentative finding."

Earthquake criteria are being considered in most nations that lead in dam construction. In 1951 the ASCE established a maximum value of horizontal intensity of 0.10 g,¹²¹ and recently the Japanese have required dynamic analyses and stress and strength studies of foundation materials comparable to those required for the dam body itself, increased studies of stresses in the dam concrete, and additional consideration of hydrodynamic pressure.¹²² The U.S. Bureau of Reclamation periodically reviews the seismicity, geology, and calculations of dams under its jurisdiction in the light of new data and techniques.¹²³

There has been a recent large increase in interest in earthquake design of large dams. The development of finite-element calculation techniques has, for the first time, permitted the possible development of rational detailed analyses.¹²⁴ However, the techniques must be further developed, applied to specific systems, and checked under seismic conditions to insure the high levels of confidence they promise.

Earth Dams. In general the higher the dam and the deeper the water, the greater are the hazards of foundation shear failure, abutment slides, landslides into the reservoir, embankment settling, and overtopping.¹²⁵ Over the last half century, maximum earth dam heights have increased from 100 to 770 ft. During this period there has been little experience with earth dams in intense earthquakes. The Crystal Spring dam, a remarkable, early, first-class concrete structure, survived the 1906 San Francisco earthquake although located less than 300 yards from the rift zone.²⁷ It is true, however, that embankments of compacted rock and earth generally do not

lose their strength either when taken beyond failure strain or when subjected to a number of alternating pulses of strain. Hundreds of large dams and thousands of small ones have given satisfactory performance under ordinary conditions.¹²⁵ This experience suggests that the most reliable earth dam might be of the rockfill type, be located on massive sound rock, have good transition zones, and have a core not susceptible to piping. The most unreliable might be a homogeneous soil dam, which, along with its foundation, would be constructed of materials with either low strength or low piping resistance.

The failure rate of earth dams has, until the present, been low, despite the fact that they are quite frequently built over faults, for rivers have a noticeable tendency to follow fault traces.²⁷ No major earth dam has been damaged by an earthquake in the past 25 years, but few have been subject to a major earthquake.¹²⁶ Recent major dam disasters have been associated primarily with foundation failures, and these can empty a reservoir in a few minutes. Of the 400 dams built since 1945, 55% are of the embankment type, and the proportion of this type is increasing. However, earthquakes are recognized by some as a serious problem,²⁷ at least since the failure of the Sheffield Dam at Santa Barbara in 1925. In this case, a large portion of the entire dam moved; such movement is thought to be one of the most probable failure modes.¹²⁵

There is a diversity of opinion as to what constitutes an adequate basis for seismic design of earth dams, ranging upward from designing only for static conditions.¹²⁷ The pseudo-static ground acceleration range commonly used in the U.S. is 0.05 to 0.15 g,¹²⁸ but no argument is available for these values;¹²⁹ in Japan it is now 0.12 to 0.25 g. Some specialists think large dams of soil are in serious danger when under horizontal acceleration of 0.1 g, and others believe that only with an evaluation of the composite state of stress both near the face and within a dam will it be possible to estimate factors of safety against its sliding.¹³⁰ If an embankment is constructed of dry or partially saturated cohesionless material, failure may result from the sliding of a mass of soil near the surface, with a resulting flattening of the overall slope.¹³¹

It has been recommended by Sherard that all dams in California be designed for the same strong ground shaking, for there is not yet enough

evidence on the prediction of intense earthquakes to allow differentiation between sites.¹¹ It is thought that dams can vary in margin of safety by factors of 100 or 1000, depending solely on their types and foundations, even when all are constructed with equal competence both in engineering judgment and in care for details in construction.¹¹ Sherard says the current practice of providing a safety factor of 1.1 under a seismic coefficient of 0.1 to 0.15 g, using conventional laboratory tests for evaluation of formation strength, gives no assurance of safety in strong earthquakes.¹²⁵ A majority of the important dam safety decisions (materials, zoning, core thickness, freeboard thickness, and details of the top) are made empirically.¹²⁵

Dams have almost always been designed with seismic coefficients that are independent of either the dimensions of the dam or properties of the fill material.¹³² Sliding, overturning, overtopping by seiches, effects of pore water, changes in the effective shear strengths of soil, rock, and masonry under dynamic loading, and liquefaction and piping through dam foundations must all be given attention, since the good record may have resulted from the absence of a good test. A consensus is reported on the fear that shear failure or large shear strains might occur in dam soil foundations in large earthquakes.¹²⁵

Usually, conservatism in design is not varied from dam to dam except for those that are to be located just above populated centers, although it is known that it is important to distinguish such embankments with respect to heights and materials.¹³³ One opinion on current design of earth dams is that such structures are completely safe in earthquakes, and thus it is not necessary to examine the likelihood of failure.¹¹ Contrary to this attitude, the Division of Dams of the State of California recently strengthened its dam safety requirements.

Possible modes of an earth dam in a strong earthquake are (1) decreases in foundation strength, (2) increased leakage through the dam, especially at the top, (3) overtopping, (4) changes in the abutments either in their slope or by sliding, (5) landslides into the reservoir, (6) fault movements through the dam's foundation, (7) reduction in dam height above pool level heights, and (8) reduction or loss of dam drainage. The magnitude of slope displacement during an earthquake should be

the criterion for assessing the degree of stability or instability of a slope. Computed values of surface displacements have been shown to be sensitive to minor changes (1 or 2 degrees) in the soil strength parameters.¹³¹ There are many examples of failure of cohesionless materials, including those that have occurred in earthquakes. The most accurate assessment of the stability of a sloping core embankment is thought to be obtained by a sliding-block analysis that includes the critical direction of the boundary between the blocks, the contact force between the blocks, and a friction angle for the shell material.¹³⁴ Although limit equilibrium methods are not required in the stability analysis of embankments, future computer applications should, through stress analysis, make practical assessment of stresses and strains throughout an entire embankment.¹³⁵ Nonlinear materials response can be considered in incremental finite-element procedures with very little additional computer effort. There remains, of course, a severe shortage of properly verified input information. Since not only frequency and amplitude of excitation, but also its duration, have great effects on the stability of earth dams, undoubtedly many advances will involve improved time-history calculations that take these concepts into account.

Designs of earth dams to survive fault displacements are few in number and are of recent origin.¹¹ A fault slippage passing through the dam and resulting in motion in an upstream direction would probably be the worst fault-slippage condition. Dams should have wide transition or filter zones, well-designed wide tops and abutments, monolithic open-channel spillways, and carefully prepared abutments to survive fault slippages.

Diverse conclusions have been drawn from the survival of the damaged earth-filled Hegben Lake, Montana, dam during the magnitude 7.1 earthquake of August 17, 1959, in which the maximum vertical displacement was about 20 ft. It is one of the few well-documented examples of the behavior of a moderate size earth dam near the epicenter of a strong earthquake.¹²⁵ Some specialists were encouraged that the dam stood while only 1000 ft from a major scarp, while others believe that its fortuitous survival allows the continuation of construction practices that would have been substantially, perhaps radically, changed had the dam failed. Sherard said that although the dam actually required a minimum of repairs, an extremely

fortunate array of circumstances prevented failure: (1) the fault slippage was in the valley wall rather than in the channel under the dam, (2) the dam's rock support sank 9.7 ft as a block, but the reservoir went down 10 ft, (3) none of the landslides around the reservoir developed on the right abutment, (4) the massive rockslide developed 7 miles downstream rather than into the reservoir, as at Vaiont, and (5) the spillway was not needed before it could be repaired.¹²⁵

Inspection. All dams that might have a relationship to the safety of a reactor plant should be reinspected as soon as possible following even relatively small earthquake excitations. Desired measurements on earth embankments include movements, strains, stresses, pore water stresses, seepage losses, in-place properties of materials in the embankment and its foundation, and the monitoring of earthquake effects.¹³⁶ Simple rugged instruments that will permit measurements over long periods of time are preferred.

The Federal Power Commission Order 135 requires a safety inspection of licensed hydroelectric projects every five years.¹³⁷ All important new dams should have strong-motion instruments, and 15 to 20 dams in the western United States now have two or more accelerometers. The Corps of Engineers now instruments all large dams and has under way a study of instrumentation of earth embankments and foundations.^{136,138}

3.10 Dynamic Testing for Earthquake Survival

Many kinds of "tests," including those in actual earthquakes, will occur on all operating reactor plants, even though not deliberately planned. Dynamic tests of complex devices to confirm that they can perform the clearly established functions required by safety criteria, as is done in many cases in the space industry, should be performed wherever practical. The need for proof-testing specific types of nuclear plant equipment was discussed in Section 3.9. Mr. C. F. Luce, Chairman and Chief Executive of Consolidated Edison, puts the correct viewpoint very well: "There is nothing as convincing as a practical demonstration that the nuclear plants are completely safe."¹³⁹

A report on model testing of reactor structures for seismic resistance was prepared by Wyle Laboratories under subcontract to ORNL.¹⁴⁰ This report indicates the variety of uses for experimental testing in designing and developing earthquake-resistant buildings. These include full-scale shaker tests on smaller components, such as instrumentation and control devices; model tests on larger parts of the system, such as the containment vessel; and model tests on the soil-foundation interaction. The latter tests would be complicated by the nonlinearity of soil properties, but it is believed that the properties of both sand and clay soils can be scaled over a considerable range. It is proposed to obtain both the required combination of ground loading and the required inertia under the foundation by lumping the mass at the top and bottom of the model.

Some of the most important test principles and test objectives applicable to power reactors have been known and practiced in other fields for many years. The Rolls-Royce Company stipulated a half century ago that in high-quality fabrication for dynamic loads only the best raw materials in the most efficient form should be used and that each part should then be properly tested until broken, the cause of the breakage found and eliminated, and the process repeated for the assembled system. When used this procedure increased test survival times by some two orders of magnitude. The application was to automobiles, but the principles should apply to the dynamic survival of almost any kind of machinery.

Dynamic tests for structures range from shaking of small models to shaking of components to shaking of full-scale structures either by dynamic machines, mechanical shocks, or explosives.¹⁴¹⁻¹⁴³ Two types of test machines are described by their names: the "trapeze" and the "cellar door." Variations of intensities of horizontal and vertical excitation can be great in tests where explosives are used to excite full-scale structures, and this is particularly the case for close-in nuclear detonations. Extrapolations from tests made on small models can have only limited use in directly obtaining the necessary measurements. There are many applications in which small models could be made to indicate qualitatively a wide range of gross response characteristics in complex systems, and only those well versed in specific fields of dynamic responses, as of pipes, shells, mats, soils, equipment, appendages, or instruments, would be able

to say where they could be practical. Useful measurements probably could not be made on models smaller than one-eighth of full size in linear dimensions. Some specialists set size ratios for dynamic modeling at one-fourth instead of one-eighth, and the dynamic modeling of a reactor could be unusually difficult. Since there are now highly sensitive accelerometers commercially available, and as-measured damping values under excitation at low displacement of full-scale reactor plants would be conservative, a project should be started to develop such tests. They should prove to be more rewarding and much less expensive than model testing.

Reaction-type vibration machines may consist of two unbalanced loads, rotating in opposite directions, that can be varied in frequency. Those large and powerful enough could be placed directly on full-scale structures to induce vibration.¹⁴¹ So far only conventional structures much lighter than power reactor power plants have been excited. Full-scale tests could establish the structure's natural frequencies of vibration at all significant modes, the modal shapes, and the damping in each of these modes. All vibrations that can be accurately measured on real structures at excitation amplitudes that are less than those of the design excitation will give values of percentages of critical damping that are less than those for the design excitation, and thus they are conservative.

Before 1968 no dynamic tests had been done on full-scale reactor structures, although a small student demonstration reactor was excited by shakers to 0.01 g at UCLA.¹⁴⁴ There is pressing need in reactor design for information on structural damping, and this would come, along with other valuable data, from such full-scale tests. There do not seem to have ever been laboratory tests conducted at earthquake frequencies to yield stresses in massive structural elements of the types found in reactors. There have been practically no measurements of the effects of simulated earthquake on any reactors and none on power reactors undergoing earthquakes, except those on the JPDR where, so far, only very small excitations have occurred.¹⁴⁴ The small earthquakes were, however, large enough to allow checking of calculated response of the containment structure at low intensities. Calculated periods gave comparatively good agreement. Such measurements, including those on vital piping, instruments, and

the containment structure, would simulate additional analysis. Dynamic test approaches could be borrowed and, with modification, applied from the dam, aircraft, space, weapons, and shipbuilding fields.¹⁴⁵

3.11 Costs of Reactor Plants Designed for Earthquake Survival*

The seismologic and geologic information available today does not include the possibility that reactors currently under construction will experience earthquakes substantially in excess of those for which their safety features were designed and tested. Thus conservative design is certainly called for in the foreseeable future despite the substantial efforts this requires.

Such considerations make it difficult to establish safety-cost relationships that are generally acceptable, and undoubtedly any realistic cost-risk-benefit relationship is unusually complex. Nevertheless, improvements in design for safety will undoubtedly be made whenever it is clear that the cost of improvement is reasonable. Conversely, a full appraisal of the cost of present earthquake design practice will make it possible to identify factors of conservatism that should be attacked through research to diminish costs.

Those who make cost analyses, such as architecture-engineering firms, usually neither publish cost breakdowns nor reveal their costing procedures, in part because they are considered proprietary information and in part because each firm has its own estimation system. The cost breakdown suggested by the AEC, as given in TID-7025 (rev.), is usually ignored. If commonly used the AEC breakdown would give advantages in intercomparison, reference, and much convenience in realistic evaluations. In general the earthquake design and construction cost estimates that are published do not allow conclusive judgments for power reactors. Indeed, this is a field almost devoid of reliable published information. The cost statements presented here, largely from the literature, have been read with skepticism by most of the qualified reviewers, and it cannot be established

*See Ref. 2 for additional discussion of this subject.

whether some of the preliminary cost estimates for reactor survival of intense shaking and fault slippage made by architecture-engineering firms are optimistic.

The literature on earthquake design and construction costs is quite limited, even for conventional structures, despite the wide agreement that in many cases a structure's capacity to survive very severe earthquakes can be provided in design with little increase in the structure's total cost.¹⁴⁶ Cost considerations are, of course, involved in almost every design and lie near the heart of building codes; yet there is little that is explicit. Housner says that although ultimately earthquake design reduces to a problem of cost versus benefit, the available information is not adequate for solving cost-benefit problems even for ordinary structures.¹⁴⁷

Cost increases for augmented seismic survival capacities in structures can be divided into those for design and those for construction. For both conventional structures and nuclear plants, as seismic loading is increased, the burden of work on the structural design engineer is greatly increased. It has been estimated that for the recent code changes in Los Angeles to permit high-rise reinforced concrete structures, if ductility requirements are met, the increase in the structural engineering costs will be 50% over those under the previous code.¹⁴⁸ Other estimates range from 30 to 100% to prepare working drawings.⁶⁸ Still other specialists feel that experience will prove this cost increase to be much lower, and the question is now a controversial one in the profession in the Los Angeles area. It seems agreed that the proportional change in costs of materials and construction will be much less than the change in engineering costs.

It has been estimated that a conventional building 12 stories high would cost some 4 to 6% more when designed for 0.1 g than if earthquake loadings were ignored,¹⁴⁹ and it was estimated a number of years ago in Japan that conventional buildings less than 100 ft high could be built using earthquake factors in "any kind" of ground for an increased cost of 10 to 15%.³⁸ It is thought at the U.S. Naval Research Laboratory that in time great improvements can be effected in the survival of machinery and equipment under strong excitation without sharp rises in cost.¹⁵⁰

For a large fossil-fueled power plant, it is estimated that \$1.25/kw(e) must be added to the capital cost for construction in zone 3 over that for zones 0, 1, or 2 (Ref. 151). In practice in "seismic" regions of the U.S. (for example, in California), fossil plants are often designed for 0.2 g rather than the 0.08 to 0.10 g required by the code for zone 3 because the added cost is not prohibitive in comparison with the advantages.

On one large U.S. power reactor, when the design ground acceleration was raised to 0.11 g, a cost increase of \$75,000 was found necessary for two items alone: the stack and reinforcement. One architecture-engineering firm of long experience is of the opinion that although any design for earthquakes costs, there is not much difference between costs for those designed for 0.1 and 0.2 g. They are of the opinion that spectrum analysis (i.e., response spectrum design) adds \$0.1 million in engineering costs. The Japanese 350-Mw(e) JAPC is thought to have a cost of \$2/kw(e) for all earthquake provisions.

Present engineering costs, with overhead, are thought to be 5% of the capital cost of the reactor power plant. Some fear that the doubling of this cost, which they see as coming with requirements for a substantially increased earthquake survival capacity, would make fossil-fueled plants more attractive than reactors in many cases.

In California and under the Uniform Building Code it is thought that the percentage increase in engineering costs for designs to survive a specified earthquake are less for a piping system than for a reactor building. Of course, only piping that is unusually firmly supported can be properly designed under coefficient design. The first Malibu reactor design provides a rough estimate of the added cost of steel for earthquake reinforcement of the containment structure, and it comes to no more than 10%, which makes this increase in total plant cost less than 1%.¹⁵²⁻¹⁵⁴

Rough figures were received from one design group on the increased cost of construction when one of the first \$100 million light-water reactor plants was redesigned for location at a zone 3 site instead of a zone 2 site; that is, for seismic force increases by a factor of somewhat less than 2.5. The increment in cost was estimated to be near \$0.3 million. Other workers consider the charges for even the best reactor

earthquake work at present design levels to be an almost negligible portion of the overall cost. So far, apparently, it has been rare that any standard item of equipment or any instrument in a central-station power reactor has been redesigned because of an anticipated earthquake loading.

If good light-water-cooled power reactor capital cost data were available for a series of designs, with the only variable being the design earthquake intensity, it is believed that they would show that there would be no important additional materials and construction costs up to 0.1 g and relatively little up to 0.2 g. Beyond 0.3 g, materials and construction costs would increase at an increasing rate. When the design ground acceleration reached 0.6 g, stresses from earthquakes would undoubtedly have a large and perhaps dominant part in the design of many of the structural components.

Earthquake engineering, as applied to nuclear plants, is sufficiently new that alternatives to the current procedure for defining design earthquakes for reactor sites should be considered. The historic record and the lack of evidence of prehistoric earthquakes make this task difficult. In addition, the wide variations in adjacent population numbers for different sites and the potential for large improvements in earthquake design of reactors should be considered in the future. The questions that need detailed study are:

1. Are the costs of design and construction for resistance to damage by various levels of earthquakes highly dependent on the design earthquake intensity?
2. What are the benefits of designing to higher levels compared with the estimated penalties at lower levels?

It is clearly feasible to investigate the first question with respect to present methods of design and construction. The second question involves the probability of earthquake occurrence, since the benefits of changing the design basis are entirely dependent on occurrences that must be considered random, within rather wide limits, in time and location. Nevertheless, the question is subject to quantitative study based on the data used currently in setting design conditions.

References

1. J. A. Blume and Associates, Engineers, Summary of Current Seismic Design Criteria for Nuclear Reactor Facilities, USAEC Report TID-25021, September 1967. (Prepared under subcontract to Oak Ridge National Laboratory.)
2. R. A. Williamson, Some Considerations in Earthquake Resistant Design of Nuclear Power Plants, USAEC Report HN-189 (rev.), Holmes & Narver, Inc., March 1967. (Prepared under subcontract to Oak Ridge National Laboratory.)
3. L. M. Murphy, Worldwide Seismic Network, paper presented at ESSA Symposium on Earthquake Prediction, February 1966.
4. H. T. Halverson, The Strong-Motion Accelerograph, in *Proceedings of the Third World Conference on Earthquake Engineering*, Vol. I, pp. 75-93, New Zealand, 1965.
5. F. Press and W. F. Bruce, Earthquake Prediction, *Science*, 152(3729): 1575-1584 (June 17, 1966).
6. W. K. Cloud, Strong-Motion Seismology, paper presented at ESSA Symposium on Earthquake Prediction, February 1966.
7. G. W. Housner and D. E. Hudson, Vibration of Structures Produced by Seismic Waves, in *Shock and Vibration Handbook*, edited by C. M. Harris and C. E. Crede, McGraw-Hill, New York, 1961.
8. L. S. Cluff, Definitions and Basic Concepts Important in Earthquake Engineering, in *Symposium on Earthquakes and the Practice of Soil and Geological Engineering*, December 12-13, 1964, pp. 5-25.
9. R. F. Scott and P. C. Jennings, Forces and Displacements in Soil, paper presented at Canadian Symposium on Engineering Design for Earthquakes, 1966.
10. G. W. Housner, Intensity of Earthquake Ground Shaking Near the Causative Fault, in *Proceedings of the Third World Conference on Earthquake Engineering*, Vol. 1, pp. 94-115, New Zealand, 1965.
11. J. L. Sherard, A Study of the Influence of the Earthquake Hazard on the Design of Embankment Dams, A report prepared for the State of California Department of Water Resources, July 1966.
12. P. Saint-Amand, Preliminary Hazards Summary on Bodega Head: Memorandum to Assistant Secretary of Interior, Mineral Resources, from Director, Geological Survey, 1965.

13. G. W. Housner, Behavior of Structures During Earthquakes, *J. Eng. Mech. Div., Proc. Amer. Soc. Civil Engrs.*, 85(EM4): 109-129 (October 1959).
14. H. B. Seed, Damage to Embankment and Foundations Caused by Earthquakes, in *Symposium on Earthquakes and the Practice of Soil and Geological Engineering*, December 12-13, 1964.
15. W. G. Owens, Recent Seismic Activity in the Rocky Mountain Area and Its Influence on Building Code Requirements and Some Thoughts on Triggering Effects of Small Tremors, in *Symposium on Earthquakes and the Practice of Soil and Geological Engineering*, December 12-13, 1964.
16. D. E. Hudson and G. W. Housner, An Analysis of Strong-Motion Accelerometer Data from the San Francisco Earthquake of March 22, 1957, *Bull. Seismol. Soc. Amer.*, 48(3): 253-268 (July 1958).
17. B. Gutenberg, Effects of Ground on Earthquake Motion, *Bull. Seismol. Soc. Amer.*, 47: 221 (1957).
18. H. B. Seed, Department of Civil Engineering, University of California, Berkeley, personal communication to T. F. Lomenick, Oak Ridge National Laboratory, July 1967.
19. I. M. Idriss and H. B. Seed, Seismic Response of Horizontal Soil Layers, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 94(SM4): 1003-1031 (1968).
20. N. C. Donovan and R. B. Matthiesen, Effect of Site Conditions on Ground Motions During Earthquakes, State-of-the-Art Symposium, Earthquake Engineering of Buildings, San Francisco, 1967.
21. I. M. Idriss and H. B. Seed, Analysis of Soil Liquefaction: Niigata Earthquake, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 93(SM3): 83-108 (May 1967).
22. H. B. Seed, H. Bolton, and K. L. Lee, Liquefaction of Saturated Sands During Cyclic Loading, *J. Soil Mech. Found. Div., Amer. Soc. Civil Engrs.*, Vol. 92, No. SM6, November 1966.
23. A. Grantz, G. Plafker, and R. Kachadoorian, Alaska's Good Friday Earthquake, March 27, 1964, Geological Survey Circular 491, United States Department of the Interior, Washington, D.C., 1964.
24. U.S. Department of the Interior, Reclamation Bureau, *Earth Manual*, 1st ed., Washington, D.C., 1963.
25. G. Sanglerat, *El Penetrometro*, Dunod, Paris, 1966.
26. J. H. Schmertmann, Static Cone Penetrometers for Soil Exploration, *Civil Eng.*, June 1967.

27. C. F. Richter, *Elementary Seismology*, W. H. Freeman and Company, San Francisco, 1958.
28. J. A. Hershberger, A Comparison of Earthquake Accelerations with Intensity Ratings, *Bull. Seismol. Soc. Amer.*, 46: 317-320 (1956).
29. L. M. Murphy, U.S. Coast and Geodetic Survey, Rockville, Maryland, personal communication to T. F. Lomenick, Oak Ridge National Laboratory, 1967.
30. G. W. Housner, The Groundshock Problems of Earthquakes and Explosions, *Noise Control*, 7(6): 36-42 (Nov.-Dec. 1961).
31. Tennessee Valley Authority, Browns Ferry Nuclear Power Station Design and Analysis Report, Vol. I, Docket No. 50-259 and 50-260, July 7, 1966.
32. A. Ryall, D. B. Slemmons, and L. D. Gedney, Seismicity, Tectonism, and Surface Faulting in the Western United States During Historic Time, *Bull. Seismol. Soc. Amer.*, 56(5): 1105-1135 (October 1966).
33. C. R. Allen, P. Saint-Amand, C. F. Richter, and J. M. Nordquist, Relationship Between Seismicity and Geologic Structure in the Southern California Region, *Bull. Seismol. Soc. Amer.*, 55(4): 753-797 (August 1965).
34. C. F. Richter, Seismic Regionalization, *Bull. Seismol. Soc. Amer.*, 49(2): 123-162 (April 1959).
35. G. W. Housner, Earthquake Hazards and Earthquake Resistant Design, Bodega Bay Power Plant Site, Pacific Gas and Electric Company, January 1961.
36. J. L. Coffman (Ed.), The Parkfield California, Earthquake of June 27, 1966, U.S. Department of Commerce, ESSA, Coast and Geodetic Survey, U.S. Government Printing Office, 1966.
37. J. Oliver et al., Microearthquake Activity Recorded by Portable Seismographs of High Sensitivity, *Bull. Seismol. Soc. Amer.*, 56(4): 899-924 (August 1966).
38. N. H. Heck, *Earthquakes*, Princeton University Press, 1936.
39. L. D. Leet and F. Leet, *Earthquakes - Discoveries in Seismology*, Dell, New York, 1964.
40. Geological-Seismological Factors Pertaining to the Proposed Construction of a Nuclear Power Desalting Plant at Bolsa Island, California, A Report to S. L. Udall, Secretary of the Interior, from an Advisory Committee chaired by H. B. Seed, October 1967.

41. J. A. Blume, N. M. Newmark, and L. H. Corning, *Design of Multistory Reinforced Concrete Building for Earthquake Motions*, Portland Cement Association, Chicago, 1961.
42. A. E. Scheidegger, *Principles of Geodynamics*, 2nd ed., Springer-Verlag, New York, 1963.
43. F. Neumann, Engineering Seismology, in *Reviews in Engineering Geology*, p. 168, edited by T. Fluhr and R. F. Legget, Geological Society of America, 1962.
44. H. B. Seed, Seismic Problems in the Use of Fills in the San Francisco Bay, Prepared for the San Francisco Bay Conservation and Development Commission, May 1967.
45. S. Okamoto and T. Mizakoshi, Earthquake Ground Motions Observed on Rock Foundations, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, 1967 (draft).
46. D. E. Hudson, Some Problems in the Application of Spectrum Techniques to Strong-Motion Earthquake Analysis, *Bull. Seismol. Soc. Amer.*, 52: 417-430 (April 1962).
47. C. M. Harris and C. E. Crede, *Shock and Vibration Handbook*, Vol. 2, Data Analysis, Testing, and Methods of Control, McGraw-Hill, New York, 1961.
48. H. Benioff, The Physical Evaluation of Seismic Destructiveness, *Bull. Seismol. Soc. Amer.*, 24: 398 (1934).
49. M. A. Biot, Analytical and Experimental Methods in Engineering Seismology, *Trans. Amer. Soc. Civil Engrs.*, 108: 365-385 (January 1943).
50. J. L. Alford, G. W. Housner, and R. R. Martel, Spectrum Analysis of Strong Motion Earthquakes, California Institute of Technology, Earthquake Research Laboratory, Pasadena, California, 1951.
51. P. C. Jennings, Response of Yielding Structures to Statistically Generated Ground Motion, in *Proceedings of Third World Conference on Earthquake Engineering*, Vol. 2, pp. 237-259, New Zealand, 1965.
52. K. Kanai, An Empirical Formula for the Spectrum of Strong Earthquake Motion, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. II, pp. 1541-1551, Japan, 1960.
53. H. C. Marchant and D. E. Hudson, Mode Superposition in Multi-Degrees of Freedom Systems Using Earthquake Response Spectrum Data, *Bull. Seismol. Soc. Amer.*, 52(2): 403 (April 1962).
54. A. Schiff and J. L. Bogdanoff, Analysis of Current Methods of Interpreting Strong Motion Accelerograms, *Bull. Seismol. Soc. Amer.*, 57(5): 857-874 (October 1967).

55. A. L. Gluckmann, Some Notes on Dynamic Structural Problems in the Design of Nuclear Power Stations, *Nucl. Struct. Eng.*, 2: 419-437 (1965).
56. N. M. Newmark, Design Criteria for Nuclear Reactors Subjected to Earthquake Hazards, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, 1967 (draft).
57. American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section III. Rules for Construction of Nuclear Vessels, 1968 ed. and addenda.
58. American National Standards Institute, Draft Standard B31.7, Nuclear Power Piping.
59. G. W. Housner, Analysis of Structural Response and Instruments, Reporter's Summary, in *Proceedings of Third World Conference on Earthquake Engineering*, New Zealand, 1965.
60. H. B. Seed, Soil Strength During Earthquakes, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. I, pp. 183-195, Japan, 1960.
61. D. L. Lycan and N. M. Newmark, Effect of Structure and Foundation Intersection, *J. Eng. Mech. Div., Proc. Amer. Soc. Civil Engrs.*, 87: 1-31 (October 1961).
62. H. Q. Golder, State of the Art of Floating Foundations, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 91: 81-88 (March 1965).
63. J. H. Wiggins, Jr., Effect of Site Conditions on Earthquake Intensity, *J. Struct. Div., Proc. Amer. Soc. Civil Engrs.*, 90: 279-313 (April 1964).
64. T. H. Hanna, Model Studies of Foundation Groups in Sand, *Geotechnique*, 13(4): 334-351 (December 1963).
65. R. V. Whittman, Analysis of Foundation Vibrations, in *Vibrations in Civil Engineering*, pp. 159-1279, Butterworth, 1965.
66. R. V. Whittman and F. E. Richart, Design Procedures for Dynamically Loaded Foundation, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 93: 169-193 (September 1967).
67. J. Penzien et al., Seismic Analysis of Bridges on Long Piles, *J. Eng. Mech. Div., Proc. Amer. Soc. Civil Engrs.*, 90: 223-254 (June 1964).
68. C. R. Allen, Aseismic Design and Testing of Nuclear Facilities, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, June 12-16, 1967 (draft).

69. C. F. Muckenhoupt, Study of Concrete as a Material for Turbine Foundations, Nov. 17, 1929 (unpublished).
70. Southern California Edison Company and San Diego Gas and Electric Company, San Onofre Nuclear Generating Station, Unit 1: Final Engineering Report and Safety Analysis, Docket 50-206, 1965.
71. Connecticut Yankee Atomic Power Company, Facility Description and Safety Analysis - Haddam Neck, Docket 50-213, 1966.
72. Department of Water and Power, City of Los Angeles, Preliminary Hazards Summary Report, Malibu Nuclear Plant Unit No. 1, Docket 50-215, Amendment 2, Sect. 2.5, revised.
73. J. A. Blume, A Reserve Energy Technique for the Earthquake Design and Rating of Structures in the Inelastic Range, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. II, pp. 1061-1083, Japan, 1960.
74. R. W. Clough, Dynamic Effects of Earthquakes, *J. Struct. Div., Proc. Amer. Soc. Civil Engrs.*, 86: 49-65 (April 1960).
75. S. L. Lee et al., Earthquake Response of Inelastic Shear Buildings, *Bull. Seismol. Soc. Amer.*, 56(2): 393 (April 1962).
76. A. M. Freudenthal, Safety, Reliability, and Structural Design, *J. Struct. Div., Proc. Amer. Soc. Civil Engrs.*, 87: 1-6 (March 1961).
77. A. S. Veletsos, N. M. Newmark, and C. V. Chelapati, Deformation Spectra for Elastic and Elastoplastic Systems Subjected to Ground Shock and Earthquake Motions, in *Proceedings of the Third World Conference on Earthquake Engineering*, Vol. II, pp. 663-682, New Zealand, 1965.
78. A. S. Veletsos and N. M. Newmark, Effect of Inelastic Behavior on the Response of Simple Systems to Earthquake Motions, in *Proceedings of the Second World Conference on Earthquake Engineering*, Vol. II, pp. 895-912, Japan, 1960.
79. B. P. Sinha et al., Stress-Strain Relations for Concrete Under Cyclic Loading, *Proc. Amer. Concrete Inst.*, 61: 195-211 (1964).
80. B. P. Sinha et al., Response of Singly Reinforced Beams to Cyclic Loading, *Proc. Amer. Concrete Inst.*, 61: 1021-1038 (1964).
81. G. L. Agrawal et al., Response of Doubly Reinforced Concrete Beams to Cyclic Loading, *Proc. Amer. Concrete Inst.*, 62: 823-835 (1965).
82. G. V. Berg and S. S. Thomaidis, Energy Consumption by Structures in Strong Motion Earthquakes, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. 2, pp. 681-691, Japan, 1960.

83. N. M. Newmark and W. J. Hall, Special Topics for Considerations in Design of Nuclear Power Stations Subjected to Seismic Motions, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, 1967 (draft).
84. Burns & Roe, Inc., Conceptual Design of a Boiling-Water Reactor Plant to Resist Ground Fault Displacement, USAEC Report TID-24530, March 1968.
85. General Electric BWR Plant Requirements, Nuclear System Model 251R 764, 218T 548, 183T 368, Document 22A 1300, Sept. 25, 1967.
86. G. J. Bohm, Natural Vibration of Reactor Internals, *Nucl. Sci. Eng.*, 22: 143-152 (1965).
87. Pacific Gas & Electric Company, Preliminary Safety Analysis Report, Suppl. 5, Docket 50-275.
88. Pipe Flexibility Analysis Program MEL 21, San Francisco Bay Naval Shipyard, Mare Island Division, Vallejo, California.
89. S. Fujii et al., Experimental Work in Japan for Earthquake Research in Piping Systems in Power Plants, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, 1967 (draft).
90. IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, 1967 (draft).
91. R. A. Weisman, R. E. Tome, and R. Salvatori, Ultimate Strength Criteria to Ensure No Loss of Function of Piping and Vessel Under Earthquake Loading, USAEC Report WCAP-5890 (Rev. 1), Westinghouse Electric Corporation, October 1967.
92. H. Tajimi, M. Ohmura, T. Uchida, and K. Akino, Observed Vibrations of a Nuclear Reactor Building During Some Weak Earthquakes, in *Proceedings of the Third World Conference on Earthquake Engineering*, Vol. II, pp. 619-630, New Zealand, 1965.
93. A. K. Chopra, The Importance of Vertical Components of Earthquake Motions, *Bull. Seismol. Soc. Amer.*, 56: 1163-1175 (1966).
94. G. R. Walker, Earthquake Resistant Design - The Pulse Method, in *Proceedings of the Third World Conference on Earthquake Engineering*, Vol. II, pp. 683-694, New Zealand, 1965.
95. K. Akino and H. Tajimi, Aseismic Design and Dynamic Analysis of Nuclear Power Plants, Parts I and II, *Nucl. Struct. Eng.*, 1(2): 232-238 (February 1965) and 2(1): 120-125 (July 1965).
96. I. Toriumi, Model Analysis of Aseismic Design of a Nuclear Reactor Building, *Nucl. Struct. Eng.*, 2: 301-305 (1965).

97. R. W. Clough and V. A. Jenschke, The Effect of Diagonal Bracing on the Earthquake Performance on a Steel Frame Building, *Bull. Seismol. Soc. Amer.*, 53(2): 389-401 (February 1963).
98. H. J. Sexton, H. J. Sexton and Associates, Engineers, personal communication to C. E. Bell, Oak Ridge National Laboratory, 1968.
99. F. J. Wood (Ed.), *The Prince William Sound, Alaska, Earthquakes and Aftershocks*, U.S. Coast and Geodetic Survey, Washington, D.C.
100. Report of ACI-ASCE Committee 326, Shear and Diagonal Tension, *Proc. Amer. Concrete Inst.*, Vol. 59, Pt. 1, pp. 3-30; Pt. 2, pp. 277-333; Pt. 3, pp. 353-395, 1962. [See also ACI Building Code 63, Requirements for Reinforced Concrete, *Proc. Amer. Concrete Inst.*, 60: 809-815 (1963).]
101. Research Needs in Structural Engineering for the Decade 1966-75, *J. Struct. Div., Proc. Amer. Soc. Civil Engrs.*, p. 287, October 1966.
102. Institute of Electrical and Electronics Engineers, Criteria for Nuclear Power Plant Protection System, IEEE Specification No. 279, Aug. 30, 1968.
103. E. J. Keith, Seismic Design of Critical Equipment in Nuclear Reactor Plants, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, 1967 (draft).
104. T. H. Thomas et al., Chap. 8, Protection Systems, pp. 233-245 and Appendix D, Earthquake Experience in Existing Reactor Plants, pp. 351-352, in *Nuclear Reactors and Earthquakes*, USAEC Report TID-7024, Lockheed Aircraft Corporation and Holmes & Narver, Inc., 1969.
105. F. Bevilacqua, Boiling Nuclear Superheater (BONUS) Power Station Technical Specifications, USAEC Report GNEC-214, General Nuclear Engineering Corporation, Oct. 26, 1962.
106. W. R. Hansen, Effects of the Earthquake of March 27, 1964, at Anchorage, Alaska, Geological Survey Professional Paper 542-A, pp. A4-A8, U.S. Government Printing Office, Washington, D.C., 1965.
107. J. C. Russ, General Electric Company, Atomic Power Equipment Department, personal communication, to T. F. Lomenick, Oak Ridge National Laboratory, March 1967.
108. E. Shima, T. Tanaka, and N. Den, Some New Instruments Used in Earthquake Engineering in Japan, in *Proceedings of the Second World Conference on Earthquake Engineering*, Vol. II, pp. 761-780, Japan, 1960.

109. H. Tajimi, M. Ohmura, T. Ucheda, and K. Akino, Observed Vibrations of a Nuclear Reactor Building During Some Weak Earthquakes, in *Proceedings of the Third World Conference on Earthquake Engineering*, Vol. II, pp. 619-630, New Zealand, 1965.
110. J. M. Raphael, The Effect of Lateral Earthquakes on High Buttress Dams, in *Proceedings of the Second World Conference on Earthquake Engineering*, Vol. III, pp. 1791-1801, Japan, 1960.
111. A. R. Golze et al., Earthquake Resistant Design of the Oroville Dam, *Ninth International Congress on Large Dams*, Vol. 4, pp. 443-470, Paris, 1967.
112. E. Gruner, The Mechanism of Dam Failures, *Ninth International Congress on Large Dams*, Vol. 3, pp. 197-206, Paris, 1967.
113. C. V. Davis and K. E. Sorenson (Eds.), *Handbook of Applied Hydraulics*, 3rd ed., McGraw-Hill, New York, 1969.
114. H. M. Westergaard, Water Pressure on Dams During Earthquakes, *Trans. Amer. Soc. Civil Engrs.*, 98: 418-433 (1933).
115. T. M. vonKarman, Discussion of Pressure on Dams During Earthquakes, *Trans. Amer. Soc. Civil Engrs.*, 98: 434-436 (1933).
116. S. Kotsubo, Dynamic Water Pressure on Dams During Earthquakes, in *Proceedings of the Second World Conference on Earthquake Engineering*, Vol. II, pp. 799-814, Japan, 1960.
117. H. C. Merchant and D. E. Hudson, Mode Superposition in Multi-Degree of Freedom Systems Using Earthquake Response Spectrum Data, *Bull. Seismol. Soc. Amer.*, 52(2): 405-416 (April 1952).
118. T. Yoshida and K. Baba, Dynamic Response of Dams, in *Proceedings of the Third World Conference on Earthquake Engineering*, Vol. II, pp. 748-764, New Zealand, 1965.
119. S. Okamoto and T. Takahashi, On Behavior of an Arch Dam During Earthquakes, in *Proceedings of the Second World Conference on Earthquake Engineering*, Vol. II, pp. 1401-1412, Japan, 1960.
120. R. Dungar et al., The Effect of Earthquakes on Arch Dams, *Ninth International Congress on Large Dams*, Vol. 4, pp. 123-147, Paris, 1967.
121. Discussion of Lateral Forces of Earthquakes and Wind, *J. Struct. Div., Proc. Amer. Soc. Civil Engrs.*, 78: 1-26 (1951).
122. T. Hatano, Aseismic Criteria for Arch Dams in Japan, *Ninth International Congress on Large Dams*, Vol. 4, pp. 1-17, Paris, 1967.

123. F. E. Dominy, The Safety of Dams, Program of the Bureau of Reclamation, *Ninth International Congress on Large Dams*, Vol. 2, pp. 57-72, Paris, 1957.
124. O. C. Zienkiewicz et al., Buttress Dam Analysis for Earthquake Loads, *Water Power*, 19(9): 359-362 (September 1967).
125. J. L. Sherard, Earthquake Consideration in Earth Dam Design, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 93: 377-401 (July 1967).
126. N. M. Newmark, Effects of Earthquakes on Dams and Embankments, *Geotechnique*, 15(2): 139-160 (June 1965).
127. A. Casagrande, Role of the "Calculated Risk" in Earthquake and Foundation Engineering, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 91: 1-40 (July 1965).
128. H. B. Seed, Slope Stability During Earthquakes, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 93: 299-323 (July 1967).
129. H. B. Seed, Earthquake Resistant Design of Earth Dams, Civil Engineering Department, University of California, Berkeley.
130. R. W. Clough and A. K. Chopra, Earthquake Stress Analyses in Earth Dams, *J. Eng. Mech. Div., Proc. Amer. Soc. Civil Engrs.*, 92: 197-211 (April 1966).
131. R. E. Goodman and H. B. Seed, Earthquake Induced Displacements in Sand Embankments, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 92: 125-146 (March 1966).
132. N. N. Ambraseys, The Seismic Stability of Earth Dams, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. II, pp. 1345-1364, Japan, 1960.
133. H. B. Seed and G. R. Martin, The Seismic Coefficient of Earth Dam Design, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 92: 25-58 (May 1966).
134. D. J. D'Appolonia and E. D'Appolonia, Determination of Maximum Density of Cohesionless Soils, in *Proceedings of the Third Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Haifa, Israel, September 25-28, 1967.
135. J. Lowe III, Stability Analysis of Embankments, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 93: 1-33 (July 1967).
136. S. D. Wilson, Investigation of Embankment Performance, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 93: 135-156 (July 1967).

137. F. S. Brown, Dam Safety — An Objective of Federal Licensing, *J. Power Div., Proc. Amer. Soc. Civil Engrs.*, 93(P01): 67-72 (March 1967).
138. H. B. Seed and H. A. Sultan, Stability Analysis for a Sloping Core Embankment, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 93: 69-83 (July 1967).
139. C. F. Luce in *Nucleonics Week*, Vol. 9, No. 4, Jan. 25, 1968.
140. R. H. White, Preliminary Investigation of Criteria for Earthquake Simulation Tests on Nuclear Power Stations, Wyle Laboratories, Huntsville, Alabama, June 1968.
141. D. E. Hudson, Resonance Testing of Full-Scale Structures, *J. Eng. Mech. Div., Proc. Amer. Soc. Civil Engrs.*, 90: 1-19 (June 1964).
142. T. Hisada and K. Nakagawa, Vibration Tests on Various Types of Building Structures up to Failure, in *Proceedings of the World Conference on Earthquake Engineering*, Berkeley, California, 1956.
143. J. A. Blume, The Building Vibratory, Earthquake Investigations in the Western United States 1931-1964, Publication 41-2, U.S. Coast and Geodetic Survey, 1964.
144. R. B. Matthiesen and C. B. Smith, A Simulation of Earthquake Effects on the UCLA Reactor Using Structural Vibrations, UCLA-NEL-105 Report No. 66-65, University of California at Los Angeles, October 1966.
145. R. E. Freeland, Mariner Venus 67 Structural Development and Qualification Vibration Test Report, JRL Technical Report 32-1161, Nov. 1, 1967.
146. J. A. Blume, Earthquake Ground Motion and Engineering Procedures for Important Installations Near Active Faults, in *Proceedings of Third World Conference on Earthquake Engineering*, Vol. III, pp. 53-71, New Zealand, 1965.
147. G. W. Housner, Seismic Problems in Nuclear Power Reactor Plants, IAEA Panel on Aseismic Design and Testing of Nuclear Facilities, Tokyo, June 12-16, 1967 (draft).
148. J. J. Friskell, Designing to Meet Ductility Requirements — 1966 Recommended Lateral Force Requirements, paper presented at the 1966 Annual Convention Structural Engineers Association of California.
149. Recommendations, Earthquake Resistant Design of Buildings, Structures, and Tank Towers, Pacific Fire Rating Bureau, San Francisco, Revised 1950.
150. ASME Comments on Pressure Vessel Criteria — Challenge AEC Philosophy, *Nucleonics Week*, 9(10): 2 (Mar. 7, 1968).

151. Jackson and Moreland, Cost of Large Fossil Fuel Fired Power Plants, Report No. 636A, July 15, 1966.
152. Department of Water and Power and the City of Los Angeles, Preliminary Hazards Summary Report, Part B, Technical Information in Support of Application for Constructive Permit and License, Malibu Nuclear Plant, Unit No. 1, Docket 50-215, 1963.
153. Seventh Amendment to Application for Construction Permit and Facility License for Malibu Nuclear Plant, Unit No. 1, Docket 50-215.
154. Eighth Amendment to Application for Construction Permit and Facility License for Malibu Nuclear Plant, Unit No. 1, Docket 50-215.

4. FAULTS

The potential location of power reactors in highly seismically active regions of the country has brought about considerable interest in the siting and design of structures to circumvent or withstand differential ground displacements. Much general information about the physical phenomena of faulting* has been documented through the study of historic breaks; however, in the past structures have not, in general, been designed to withstand this type of movement, and much less information is available on the influence of faulting on structural design. In the discussion below, a brief review of the ground-movement aspects of faulting is given, and this is followed by an evaluation of current designs for withstanding these movements.

The possibility of permanent ground displacement or faulting has been the subject of controversy with respect to several reactor sites and is one of the greatest concerns with regard to earthquake effects on nuclear reactors. This is due largely to present inability to state precisely enough whether faulting will occur at a selected site, and if it is likely to occur, what will be the amount and nature of the expected displacements. Furthermore, even if the expected displacements could be adequately characterized and specified, the reactor designs that have been proposed for withstanding substantial displacements have not yet been proved.

Strictly speaking, faults are ruptures in soil or rock along which there has been differential shear movement of the opposing faces. Almost all rocks are fractured to some extent, especially those in regions that have undergone strong tectonic activity; however, most major faults and fault zones, whether they are active or inactive, generally exhibit, among other characteristics, unique topographical expressions, extensive rock deformation, and anomalous patterns of ground-water circulation. These features of faults are of great importance in the location, design, and construction of most all structures. In addition the possibility of renewed movement along preexisting breaks and the likelihood of new breaks

*See Glossary (App. A) for types of faulting.

occurring within existing zones are of concern, especially for nuclear reactors. In the U.S., only a relatively small number of faults, practically all of which are located in the western part of the contiguous states and Alaska, have experienced any surface movement in recorded history. Those faults which have exhibited historical breakage and those that show evidence of movement in the Quaternary period of geologic time (the last 1 million years) are given some consideration in reactor siting and design.

The engineering problems currently posed by fault slippage result from the large displacements known to have occurred, the difficulty in determining specifically what to design for at a given site, and the inexperience in designing large structures for faulting. The tasks of insuring the survival of reactors both under shaking alone and in combination with fault slippage is difficult. Such strong controversy has focused on faulting problems in some reactor licensing hearings that they have dominated the other questions. As brought out in the Bodega licensing proceedings (see App. B), the applicant confidently believed that economically practical power reactor plants could be built to survive differential fault movement of from 2 to 5 ft. In addition to the Bodega design, there are several other designs discussed here that appear to be capable of surviving substantial fault movements.

4.1 Faults and Earthquakes: Cause or Effect?

Faults are related to earthquakes, and there is good evidence to support the belief that faulting is the cause of many earthquakes. This mechanism for the generation of earthquakes, which is called the elastic-rebound theory,¹ suggests that strain accumulates in the rock column until the strength of the rocks is exceeded, and then rupture occurs, with ensuing shock waves (see Fig. 4.1). On the other hand, some investigators suggest that earthquakes may result from other more obscure causes, such as phase changes in rocks at depth or plastic rupture in the mantle, and that surface faulting may be regarded as a secondary effect.^{2,3} According to Hodgson most seismologists accept the elastic-rebound theory.⁴ By recording the directions of the first motion at a number of points near

Fig. 4.1. Illustration of Elastic-Rebound Theory of Earthquakes: (a) Unstrained Crustal Rocks, (b) Crustal Rocks Strained and Distorted, (c) Strain Relieved in Crustal Rocks Through Fault Movement.

the epicentral locations of earthquakes, seismologists have found that most shocks give the quadrant distribution of compressions and dilations shown in Fig. 4.2. Straight lines (or nodal planes) are then used to separate the compressions from the dilations. If the elastic-rebound fault model were the mechanism of earthquakes, one of these lines would represent the fault, regardless of whether it was on the surface or at depth (see Fig. 4.2a); however, an alternate hypothesis of earthquake mechanism, that is, sudden release of regional forces without faulting, is also possible with the observed pattern of compressions and dilations (see Fig. 4.2b) and is supported by some seismologists. Thus, even though most seismologists believe that the fault-plane solution explains earthquake mechanisms best, it is not possible to prove with the patterns of dilations and compressions which model is the correct one. The complexity of the earthquake mechanism is illustrated in parts of Japan for which it is reported that there is no obvious correlation between surficial faulting and the subterranean points of origin of earthquakes. This suggests a more complex earthquake mechanism than simple elastic rebound. Nevertheless it is reported that there are at least 45 known instances throughout the world where there is unequivocal evidence of faulting in relation to earthquakes,¹ and recent observations indicate that surface

(a) Fault Plane Solution for Earthquake Mechanism.

(b) Sudden Release of Regional Forces, Without Faulting, for Earthquake Mechanism.

Fig. 4.2. Quadrant Distribution of Compressions (+) and Dilations (-) Around an Earthquake Epicenter. (From Ref. 4; reprinted with permission of Prentice-Hall.)

faulting accompanying earthquakes is probably a more common phenomena than has been recognized heretofore.⁵

In California and tectonically related areas, such as New Zealand, almost all large shocks are believed to be clearly related to faults,⁵ but for large numbers of earthquakes that occur within these and other areas and for the deep-focused shocks, correlations with existing surface faults cannot usually be made. Some of the larger and more shallow shocks apparently do not exhibit visible surface faulting, but this may be due in part to the inaccessible nature of the faulting, such as that in submarine environments or remote areas. In other cases unconsolidated surface materials, such as alluvium, may disguise or complicate the surface expression of faulting so much that the faulting is imperfectly known. Such was the case in the faulting associated with the 1811-1812 New Madrid earthquakes,^{6,7} if, indeed, surface faulting accompanied these earthquakes.

In addition to the ground ruptures accompanying earthquakes, surface faulting may also be associated with volcanic eruptions and other natural geologic processes, as well as with nontectonic events such as the withdrawal of fluids from the earth's crust and nuclear test detonations. In some cases it is difficult to determine whether faulting is tectonically

or nontectonically induced; and even though both types are of concern in reactor siting, nontectonic faulting is outside the scope of this report.

4.2 Fault Occurrence

Faulting that is associated with earthquakes occurs most frequently within the two major earthquake zones of the earth; namely, the Circum-Pacific belt and the Alpide belt, which extends from southern Europe through Asia (see Fig. 4.3). The west coast of the continental U.S. and Alaska lie within the Circum-Pacific belt. The San Andreas system of faults in California is one of the active areas of the belt; however, this area accounts for only a small percentage of the total earthquake activity in the entire belt.

For the contiguous U.S., about 35 historic earthquakes are known to have been accompanied by surface faulting (see Fig. 4.4).^{8,9} It is worthy of note that all these faults occurred west of the Rocky Mountains, with the possible exception of the New Madrid, Missouri, earthquakes of 1811-1812. There, as mentioned above, a thick alluvium cover complicated the surface expression of the faulting so much that if faulting did occur, it

Fig. 4.3. Major Earthquake Zones of the Earth.

Fig. 4.4. Historic Surface Faulting in the Contiguous U.S.

cannot be accurately defined.⁶ The Charleston, South Carolina, earthquake of 1886 was probably one of the greatest shocks that has occurred in the U.S. during historic time; however, no known surface faulting accompanied the shock.¹⁰

In parts of Texas, Louisiana, and California, several additional instances of local and generally slow movement along faults have been reported,¹¹⁻¹⁶ but most of these movements are believed to be related to the withdrawal of fluids from the underlying rocks, and therefore the faulting is nontectonic in origin. Thus, from the historic record, it can be concluded that future tectonically induced surface fault movements will be restricted, with few exceptions, to that part of the U.S. west of the Rocky Mountains.

In addition to the historic record of faulting, there are other indications, although they are by no means conclusive, that future tectonic faulting will occur only in those areas that have experienced historic breaks. First, it is recognized that most, if not all, of the earth's crust is constantly undergoing tectonic changes, but the areas of greatest change, and therefore the areas of greatest and most frequent earthquakes, are those in the western part of the U.S. that are presently experiencing the most active stages of the mountain building processes. The eastern part of the U.S. is, in general, in a relatively quiescent or late stage of the mountain building cycle and therefore less active. As for possible explanations of activity of the type experienced at New Madrid and Charleston, it has been suggested that New Madrid lies in an area of contrasting crustal movements where sediments of the subsiding Mississippi embayment lap over much older rocks that, toward the north, are undergoing uplift as a result of the unloading of the recent ice cap.^{17,18} It has also been suggested that the activity in the Charleston area is related to the nearby Cape Fear arch structural feature, which has been undergoing uplift since the Tertiary period of geologic time (70 million years ago).^{17,18} Further, it is known that approximately 90% of all historic earthquake activity in the U.S., exclusive of Alaska, has occurred in California and Nevada;¹ that large earthquakes do not occur frequently in the eastern part of the U.S. — the New Madrid and Charleston shocks were exceptions — and that surface faulting that results in large displacements is more frequently

associated with large shocks ($M > 6.5$)* than with small ones.¹⁹ Finally, there is no evidence to indicate that movement has occurred along surface faults in the eastern U.S. in recent geologic time (the last 10,000 years) nor has creep movement or strain attributed to tectonic forces been associated with such movement. It should be mentioned, however, that few measurements to determine these strain phenomena have been made along or near faults in the eastern U.S. In contrast, there are many faults in the west that show geologic evidence of recent movement and that exhibit strain and creep movement. It is reasonable then to conclude, with a high degree of confidence, that future surface movements associated with earthquakes should not occur along faults in the U.S. east of the Rocky Mountains, with the possible exception of the New Madrid, Missouri, area.

The San Andreas fault system, which extends from northern California to the Gulf of California, is generally regarded as the most important zone of active faulting in the U.S., excluding Alaska (see Fig. 4.5). Several breaks have been recorded along this network of fractures in historic time. However, historic surface ruptures have also occurred along other fault systems in California, Nevada, and Montana. The genetic relationship of the San Andreas fault to other active faults, such as Newport-Inglewood, Ft. Sage, Sierra, etc., in the far west area is not clear. Perhaps, as suggested by some investigators,^{2,20-23} all faulting in the western part of the country is related to a single system of tectonic activity. On the other hand, it is argued by some that there are several large but independent zones of tectonic activity that account for the localization of earthquakes and faulting in the western part of the U.S.^{1,17} In any case, it should be recognized that recent surface faulting has occurred in areas other than those along the San Andreas fault, but the exact relationship of these ruptures to the San Andreas system is not known.

*Magnitude M was originally defined by Richter as the logarithm of the maximum amplitude on a seismogram written by an instrument of a specified standard type at a distance of 100 km from the epicenter. The zero of the scale was set arbitrarily to fit the smallest recorded earthquake at that time; the largest known earthquake magnitudes are near 8.75 (Ref. 1).

Fig. 4.5. Known Active Faults in California.

4.3 Types of Faulting*

Rift faulting (strike-slip faulting in which the principal displacement is horizontal and parallel to the regional structure) is the principal tectonic type of faulting in the Circum-Pacific seismic zone of California and also in parts of New Zealand and the Philippines; whereas thrust faulting (faulting in which the principal displacement is along low-angle thrust planes) is the principal type of faulting in the remaining part of the Circum-Pacific zone.¹ In California, as in New Zealand and the Philippines, the faulting is characterized by single master faults, which are primarily strike-slip in character and which generally are found with a series of branch or associated faults that usually tend to be parallel to the master

*See Glossary (App. A).

faults but may have any direction. The San Andreas fault and its associated faults, such as the Hayward, Garlock, and San Jacinto, typify rift faulting in California¹ (see Fig. 4.5). Most instances of surface faulting in the U.S. have been associated with the San Andreas family of faults in California and with block faulting in the Basin and Range province to the east. Recent faulting along the San Andreas zone is mostly horizontal, while the principal movement in eastern California, Nevada, Utah, and Montana involves a greater vertical component.¹

Diagrams of the three types of faults that are of most concern in reactor siting and design are shown in Fig. 4.6. In diagram (a), it may be observed that the fault movement is essentially horizontal and in the direction of the bearing of the fault. This is commonly referred to as strike-slip or rift-type faulting. Transverse, transcurrent, or tear faulting are other terms used to refer to horizontal movement on faults. In diagram (b) the movement is primarily vertical. When the overhanging block moves down in relation to the foot block, as illustrated, the

Fig. 4.6. Diagrams Showing Various Fault Movements: (a) Strike Slip, (b) Dip Slip, (c) Combination Strike Slip and Dip Slip; Also Designated Are Main, Branch, and Secondary Faults.

fracturing is termed normal or gravity faulting. However, if the hanging block moves up in relation to the foot wall, the fracturing may be referred to as reverse faulting and is usually designated as thrust faulting in the case of a low angle of inclination of the fault plane. If no distinction is made between the relative movements of the blocks shown in diagram (b), dip-slip may be used to describe the type of faulting. Main (primary), branch (auxiliary), and secondary faulting are distinguished in diagram (c). In general there is much less uniformity in the professional terminology used to describe this faulting than the types illustrated in diagrams (a) and (b). As may be seen in diagram (c), the principal or main through-going fault is designated the main fault, while discontinuous but connecting fractures are designated branch faults. Minor secondary breaks are the small faults that presumably relieve local stress conditions that may be set up as a result of movements on the principal fault but are not directly coupled to it. Photographs of active faults that vividly exhibit strike-slip and dip-slip movements are shown in Figs. 4.7 and 4.8.

4.4 Faults and Earthquake Energy

Although the number of known instances of surface faulting associated with earthquakes is relatively small it is noteworthy that most shocks above magnitude 6.5 that have occurred in the western U.S. since 1933 have been accompanied by some form of surface faulting.¹⁹ Generally, for those earthquakes below magnitude 6, surface faulting has not been observed.¹⁹ There are, however, some notable exceptions. The Olympia, Washington, earthquake (magnitude 7.0) of 1949 and the 1954 Eureka, California, shock (magnitude 6.6) apparently occurred without surface faulting.^{24,25} On the other hand, some smaller earthquakes, such as the 1950 Ft. Sage, California, shock (magnitude 5.6), the Imperial, California, shock (magnitude 3.6) of March 1966, and the Parkfield-Cholame, California, shock of June 1966 (magnitude 5.5), were accompanied by faulting of the ground surface.²⁶⁻²⁸ The latter earthquake produced a surprising amount of ground breakage (about 5 in. maximum displacement over a length of about 23 miles) for such a relatively small shock (magnitude 5.5). On the basis of the

Fig. 4.7. Strike-Slip Movement Along San Andreas Fault in Northern California, 1906. (Photograph supplied by L. S. Cluff of Woodward-Clyde & Associates.)

Fig. 4.8. Dip-Slip Fault Movement That Accompanied 1954 Earthquake Along Fairview Peak in Nevada. (Photograph supplied by L. S. Cluff of Woodward-Clyde & Associates.)

historic record, therefore, it is highly probable that some surface faulting will accompany any large earthquake in the western U.S. In addition, ground breakage may also occur with some of the smaller shocks, especially with extremely shallow focused ones. The higher incidence of surface faulting with the shallower earthquakes is due principally to the relatively shorter distance that the break must propagate to reach the surface than would be the case for intermediate-focused earthquakes (60 to 300 km) and deep-focused earthquakes (300 to 720 km).

Several investigators have observed general relationships between earthquake magnitudes, fault lengths, and amounts of displacement.^{8,19,29} In general, all these studies have shown a general increase in the length of faulting and the amount of surface displacement with earthquake magnitude. Based on data from all historic earthquakes in the U.S. and Alaska in which there was undisputed evidence of surface faulting, Bonilla has expressed graphic relationships between earthquake magnitude and fault length (Fig. 4.9) and between length of surface rupture and maximum displacement (Fig. 4.10).⁸ The relationships established by Bonilla,⁷ along with those found by others,^{19,29} may be used to estimate the expected lengths of surface faulting and the amount of fault displacement for given earthquake magnitudes. These empirically derived guidelines refer only

Fig. 4.9. Relationship Between Length of Fault Rupture and Earthquake Magnitude; Line Drawn To Bound Extreme Upper Limit Data Points. (From Ref. 8.)

Fig. 4.10. Relationship Between Length of Surface Rupture and Maximum Displacement; Line Drawn To Fit Data by Method of Least Squares. (From Ref. 8.)

to breakage along the main traces of the faults and may not be expected to define branch or secondary faulting.

Tocher³⁰ has postulated that the lengths of fault breaks and their associated displacements may also be related to the energy of earthquakes through the empirical equation

$$E = 3.4 \times 10^{17} lD,$$

where E is energy in ergs, l is fault length in kilometers, and D is maximum ground displacement in centimeters. If the energy associated with an earthquake varies directly as the volume of rock over which the energy is stored, it follows that the width of the strained rock zone is roughly proportional to the displacement along the fault, while the total area of the strained rock is roughly proportional to the length of faulting multiplied by the displacement of the faulting.³⁰

Measured rupture velocities of faults or the rate of propagation of slip along faults generally varies from about 3 to 4 km/sec;³¹ however, the actual rate of the displacement of adjacent rocks is apparently not known. It has generally been assumed that the velocity of displacement

of the rocks at the fault will not exceed that of the measured and predicted peak ground motion velocities, and even though this has not been refuted, some recent measurements of fault movements associated with underground nuclear tests suggest that the displacement velocities may be greater than heretofore recognized.³²

4.5 Fault Zones

Major faults, such as the San Andreas fault in California, are generally not single fractures but, instead, are wide zones that have experienced breakage over long periods of geologic time. In general, however, the break or breaks that accompany individual earthquakes occur only over a small segment of the zone, usually that part which shows the most recent fracturing. Since there are relatively few examples of historic surface faulting and since many of the accounts of fracturing associated with these historic earthquakes are incomplete, it is difficult to establish good quantitative relationships between the various fault types and the widths of zones over which faulting accompanying individual earthquakes has occurred and the amounts and nature of the faulting in various parts of the fault zones. Nevertheless, some qualitative guidelines for these important parameters may be deduced from a recent study of historic surface faulting in the U.S. by Bonilla,⁸ who summarized and evaluated published accounts of historic surface faulting, and in some cases, supplemented the documented accounts by additional field investigations. Data are summarized in Fig. 4.11 that show the observed widths of zones of main, branch, and secondary faults. As may be seen, the zone of faulting for strike-slip type movements along the main trace of the indicated faults (not counting branches and measured from center of zone) was less than or equal to approximately 0.12 mile in width, while the upper limit of the width of the zones for other types lies between 1.0 and 1.2 miles. It may also be seen that the maximum widths of branch and secondary faulting are considerably less for strike-slip type movements than for other types. Figure 4.12 shows relationships between the amounts of reported displacements on branch and secondary faults at various distances from the main faults for the different fault types. The measured displacements along

Fig. 4.11. Relationship Between Earthquake Magnitude and Fault Zone Widths. (From Ref. 8.)

Fig. 4.12. Relationship Between Amount of Displacement and Lateral Distance from Main Fault. (From Ref. 8.)

secondary breaks may be seen to vary considerably; however, it is most significant that displacements exceeding 1 ft at distances of 1 to 8 miles from the main fault traces have occurred in several instances.

Although it is sometimes difficult to determine precisely the trace of the main or primary faulting on the surface, it is usually more difficult to define the extent and location of branch faults and, especially, the numerous secondary or minor faults that might accompany a major break. Apparently many secondary faults are associated with preexisting cracks, faults, joints, and fissures in the rocks, and also many new secondary displacements and cracks apparently occur during major breaks. These secondary breaks may also extend over wide areas. For instance, in the earthquake of February 1899, at Yakutat Bay, Alaska, secondary faulting may have occurred as far as 21 miles from the main fault trace.^{8,33,34} Thus because of the relatively large number of secondary faults that may accompany a major earthquake, the wide areas over which they may occur, the difficulties in predicting their expected occurrence, and the nature and extent of their movement, branch and secondary breaks are probably of greater concern in reactor siting and design than movements along the main traces of ruptures, which are more likely to be restricted to a single fracture or an easily identified narrow zone.

As faults propagate to the surface from their subterranean points of origin, a great variety of rock types, ranging from hard consolidated rocks to soft unconsolidated sediments, may be displaced. Because of differences in strength, wide variations may be expected to occur in the response of these materials to the shearing motions of fault movements. For example, the surface displacement of faults is more likely to be of the straight-line single-fracture type in hard massive rocks than in soft alluvial materials. This is due mainly to wide variations in the response of unconsolidated deposits to earthquake-caused stresses, which commonly result in ridge, trough, and en echelon fault patterns. En echelon fault patterns, or a series of relatively short overlapping faults, in alluvium or other soft materials usually cover a much wider area than do fractures in hard rocks. For example, the report of the 1906 San Francisco earthquake states that, in many areas, en echelon faulting in soft materials occurred in bands from 200 to 1500 ft wide, whereas most of the faulting

in hard rocks was restricted to bands that were less than 20 ft wide.³⁵ Also it is reported that the maximum width of faulting in hard rock at Montague Island for the 1964 Alaskan earthquake was only about 5 ft, but where the faulting crossed alluvial planes the zones of disturbance were about 100 ft in width.³⁶

It has frequently been observed that single fractures in hard rocks are not propagated to the ground surface in areas containing alluvium covers. In some cases the single fracture in hard rock is converted to a series of surface fractures, with each having a smaller displacement than the single hard rock fracture; whereas, in other instances, apparently the displacement is completely absorbed in the alluvium and no faulting extends to the surface. Furthermore, Cluff has found in some bedrock areas along the Dixie Valley fault zone that the fracture is not a single line, but in alluvium it is a single line.³⁷ However, in spite of the apparent effectiveness of soft materials such as alluvium in reducing, at least in some cases, the amount of displacement associated with faults, there is much uncertainty in determining the nature and extent of faulting in areas where alluvium and other soft materials occur.

4.6 Fault Activity

Another extremely important and controversial consideration of reactor siting in relation to earthquakes is the degree of activity of faults. There are many faults in the western part of the U.S. that are clearly active; for instance, those that have exhibited movement in historic times. There are also numerous faults about which it can be stated unequivocally that movement has not occurred for millions of years; and it is generally assumed that if movement has not occurred along a principal fault in the last few million years, the forces responsible for its activity are no longer present and the fault is inactive or dead. In these extreme cases, faults may be classified as simply active or inactive, even though there is no guarantee that either dead faults, especially in seismically active regions, will not be reactivated or that an active fault will ever produce another break in the future. These generally accepted ideas obviously do

not imply that faults that may have moved in intermediate times are necessarily still active. The difficulty then lies in the inability to define more explicitly the elapsed period of time since the last movement for assurance that the fault is indeed dead, which is the basis for assuming that no further movement will occur in the foreseeable future. At present because of the general lack of understanding of faulting and the earthquake mechanism, the probability of future movement along a particular fault that has not moved in historic time is considered to be greater the more recent its latest movement, although many other factors may be involved. It is also recognized that some faults show much more recent activity along certain parts of their traces than along other sections. Thus specific sections of some faults may be considered more active than other sections.

Although there is no general agreement as to what constitutes an active fault, many investigators define an active fault as one that shows clearcut evidence of movement in Recent Geologic Time; that is, during the last 10,000 years. Certainly, according to this definition, faults that have exhibited movement in historic time would be classified as active, as would those faults that clearly displace Recent sediments. It is generally assumed that the tectonic environment of fault zones in California during Recent time has been similar to the present environment.³⁸ Presumably faults that show seismological evidence of current activity (i.e., earthquakes occur along them), even though Recent surface faulting is not clearly demonstrated, would also be classified as active. It does not hold, however, that a fault which does not show seismologic evidence of current activity is not active; geologic evidence over a longer time span must be taken into consideration. Based on movements in historic time, activity in Recent Geologic Time, and/or seismological evidence of present activity, several faults in the western part of the U.S. have been classified as active^{1,39-41} (see Fig. 4.5). The current listing of active faults does not specify, however, that these are the only active faults in the area, but the available information indicates that these have been found to be active. It is likely that many more active faults are present in the area, but geologic and seismological investigations are needed to determine their activity.

Although inactive faults are not likely to move further as a result of tectonic activity associated with the fault itself, it has been suggested that slippage may occur along fault planes that may be either active or inactive as a result of activity on an adjacent or nearby fault.⁴² An example of this type of induced movement may have occurred along the Garlock, California, fault during the 1952 earthquake on the adjacent White Wolf fault.⁴³ More recently, in the Borrego Mountain, California, earthquake of April 9, 1968, which was caused by breakage along a 33-km section of the Coyote Creek fault in southern California, minor displacements, or creep, apparently occurred simultaneously on the Superstition Hills, Imperial, and Banning-Mission Creek faults that were located at distance of 45, 70, and 50 km from the epicenter of the shock.⁴⁴

4.7 Fault Creep or Slippage

In addition to rapid fault movements that generate seismic waves and ground shaking, slow nonseismic motions along faults were recently recognized. This phenomenon, commonly referred to as fault creep, or fault slippage, was noted at the following locations in California: across an active overthrust fault in the Buena Vista Hills Oil Field in Kern County, along the trace of the San Andreas fault near Hollister, on at least two faults within the Inglewood fault zone, and along four sections of the Hayward fault stretching from Richmond to Irvington.⁴⁵⁻⁵⁵ Evidence of the creep along the Hayward fault is shown in Fig. 4.13.

Even though the number of recorded instances is small, it is generally acknowledged that creep along faults may be rather common. Since slow creep along faults generally does not emit seismic signals, this movement is usually detected by observations of existing engineering structures situated astride faults or by the installation of measuring devices on the surface to record differential movement. Measured average rates of movement range from 0.1 to 0.8 in. per year. In some cases, the slippage appears to be concentrated in spasms of short duration (perhaps a week or two) followed by periods (perhaps a month or so) of little or no activity. In other instances, the slippage apparently proceeds at a more uniform rate. Generally the slippages do not coincide with local small

Fig. 4.13. Creep Along Hayward Fault Shown by 6-in. Offset of Boards Aligned Along Construction Joint in Warehouse That Straddles the Fault. (Photograph supplied by L. S. Cluff of Woodward-Clyde & Associates.)

earthquakes, at least those greater than magnitude 3; however, since strains in rocks may be relieved by this mechanism, slippage or creep along a fault probably affects its future behavior. It can be argued that slow movement along a fault relieves strain and effectively prevents strain accumulations of sufficient magnitude to cause an earthquake. This argument is hard to accept considering the fact that large earthquakes have occurred along sections of faults that now show slow creep movements. Perhaps another explanation is that suggested by Bolt and Marion, who reasoned that slippage is an indicator of strain energy in rocks at depth but does not relieve that strain.⁵² It is well recognized that rocks which bound the fault near the surface commonly are crushed and exhibit fractures filled with gouge.* Thus it is likely that the surface rocks may be deformed by slippage due to forces originating at greater depths, while the rocks adjacent to the fault at depth are likely to be deformed by elastic strain. If, when an earthquake occurs, the rocks at depth are indeed responding to tectonic forces by elastic strain and those at the surface by slippage, it follows, according to Bolt and Marion, that the greater the amount of slippage on a fault, the less will be the displacement on the surface.⁵²

In addition to creep movement along faults in seismically active parts of the country, slow slippage and/or subsidence along faults has been reported in relatively aseismic regions. Most of these movements, however, have been vertical and presumably related to the withdrawal of liquid (hydrocarbons, water, etc.) from the earth (see Sect. 4.2, Fault Occurrence). Small-scale geodetic triangulation nets are commonly used to measure slow movement on faults; however, when the fault motion is confined to a small zone perhaps a few inches in width, displacement meters (differential transformers) probably give more precise measurements.⁵²

4.8 Evaluation of Faulting at Specific Sites

The evaluation of the amount and nature of the expectable permanent ground displacement at reactor sites usually entails local, as well as

*See Glossary (App. A) for definition.

regional, geological and geophysical investigations, seismological studies, and a review of the historic evidence of breaks. In general the activity of nearby faults is established, and then, depending on the location of the site with respect to the fault or fault zone and the geological structure of the fault zone and site, the nature and extent of any expected ruptures are estimated.

4.8.1 Historic and Seismographic Records

The best indicator of recent fault activity is the observation of permanent ground displacement in historic time, since a fault that has moved in recorded history must be considered active. The 35 known occurrences of fault movement in the U.S. in historic time definitely indicate active faults. Most faults have not moved in historic time, but from geological evidence it can be determined that many faults that may or may not have ruptured in historic time have experienced some movement in Recent Geologic Time (less than 10,000 years). In addition, seismological evidence suggests that the tectonic forces which generate earthquakes are still active along some of these faults. Thus, the evaluation of the activity of faults by geological and seismological evidence can, in many cases, define accurately a part of the past and present state of fault activity.

By plotting the locations of epicenters of observed and instrumentally recorded shocks, the tectonic activity of some faults and fault zones can be ascertained. However, it is difficult in many cases to relate a specific earthquake, especially a small one, to a specific fault or some other local structural feature. Obviously, most of the deep-focus earthquakes, as well as many of the shallow shocks, are not clearly related to any specific surface fault. Of course this lack of correlation stems largely from the lack of precision in locating epicenters (in general, epicenter determinations are accurate only to within about 20 miles) and the lack of knowledge concerning the trends of faults from the surface to the depth of the origin of the earthquake. Most earthquakes of magnitudes less than 6 do not show any surface expression of faulting. Either the faulting is dissipated before it reaches the ground surface or it is too small to be noticed. In addition to the difficulty of relating seismic activity

to specific faults, it is recognized that the history of seismic recording is extremely short and that records are needed over much longer periods of time to adequately evaluate the seismicity of any given area.⁵

4.8.2 Geological and Geophysical Investigations

Geological and geophysical investigations of faults and fault zones are concerned primarily with determining the location, distribution, and extent of existing surface and near surface ruptures and with ascertaining the degree of activity of the faults in the vicinity. Estimates of the extent of expected ruptures are then based on the data obtained. Geological investigations of fault systems include both regional and local studies. Generally faults are not isolated local structural features but are intricately related to regional patterns of deformation. Thus to define the characteristics of faulting at specific sites, it is normally necessary that regional aspects of faulting, as well as local considerations, be evaluated. Physiographic features are often important indicators of fault activity. For instance, typical fault features, such as sag ponds, shutter ridges, triangular facets, offset streams (see Fig. 4.14), side hill ridges, etc., generally indicate fault movement, although the recency of movement is often misinterpreted due to variations in climatic conditions from one area to another.⁵⁶ Detailed stratigraphic studies, coupled with isotopic dating of truncated or faulted materials, may also be useful in defining fault movements; however, materials of significantly different ages frequently do not occur on adjacent sides of faults.

Maps and aerial photographs are valuable aids for determining the presence of active faults in some areas. For instance, 50-year-old fault scarps less than 1 ft high have been detected in Nevada by aerial photographs.⁴¹ New remote sensing techniques of some value in defining faults include multispectral photography, radar imagery, and infrared imagery.^{57, 58} Seismic refraction, gravity studies, magnetic surveys, and electrical resistivity methods of geophysical investigation are used for determining the nature and extent of faulting at depth.^{59, 60} In general, seismic refraction studies used to determine the seismic velocities and geometry of rock units supply useful information on the structure and physical properties of faults and fault zones. Seismic reflection techniques also provide

Fig. 4.14. Off-Set Streams Along San Andreas Fault, Southern California.
(Photograph supplied by L. S. Cluff of Woodward-Clyde & Associates.)

information on subsurface faulting. Gravity studies, which are much less expensive than seismic investigations, may be used for determining the distribution of rocks of contrasting densities and possibly for locating and defining faults. Also they are frequently used for determining gravity anomalies that may indicate zones of crustal instability along or near areas of high seismic activities. Magnetic surveys are used to define and determine the extent of rocks of differing magnetic properties and thus are useful in studying fault trends. Electrical methods of geophysical investigation also provide information on the physical properties of faults, such as zones of crushed rock, ground-water circulation, etc.³⁸ In areas where faults are covered by alluvium, long-line resistivity has proved to be useful in defining the location and width of fault zones.⁵⁹

For determining the likelihood of renewed movements along faults, sensitive seismographs, tiltmeters, and precise surveying techniques may be employed. The presence of creep or strain buildup in rocks may be measured by geodimeter surveys along triangulation nets that cross faults, by the installation of strainmeters in rocks, and perhaps by laboratory investigations of rock properties. Tiltmeters, tide gages, and precise leveling provide useful information on tectonic uplift and/or subsidence in the vicinity of faults. However, measurements over periods of several months to several years are generally necessary to evaluate possible movements along faults with these techniques.

Anomalous strain changes were detected some 9 hr before the Niigata, Japan, earthquake of 1964 by vertical strainmeters in shallow holes.³¹ However, in general, strainmeters and tiltmeters are limited greatly by the accuracy of measurements and/or by ground noises associated with atmospheric conditions. Electromagnetic-wave distance-measuring techniques may provide more accurate measurements of horizontal deformations, and deep hole emplacement of strainmeters may reduce ground noises significantly; however, such instrumental and observational improvements are only in the research and development stage at the present time.

Changes in local magnetic, electric, and gravitational fields may be related to changes in the stress or physical state of rocks, and measurements of such changes may also provide data on expected fault movements. Studies of microearthquakes can be used to help evaluate the

tectonism of some areas.^{61,62} It has been reported that in some relatively aseismic areas it may be possible to estimate the probability of occurrence of large shocks through the monitoring of microearthquakes over a relatively short period of time.³⁸ Microearthquake studies are sometimes used to locate and define the subsurface configuration of faults and to indicate fault activity in some areas.⁶³

The location and characteristics of faults at shallow depths are usually explored by drilling, geophysical surveys, and shallow trench excavations. Apparently the latter technique is one of the most useful and positive means for determining the location and characteristics of secondary faults that do not exhibit distinct surface expressions. It is noteworthy that much of the minor but controversial faulting associated with the Bodega Bay and Malibu sites was discovered through shallow trench excavations.^{64,65}

In general, surface deformation accompanying earthquake faulting is studied theoretically by dislocation models and/or through fault-plane solutions. In many cases the surface expression of faulting coincides well with the deep structure controlling faults; however, in other cases, such as the 1964 Alaskan earthquake, the 1954 Fairview Peak earthquake, and the 1959 Hebgen Lake earthquake, it is suggested by some that the major surface ruptures accompanying these shocks are not direct extensions of the deep-seated earthquake-generating faults but are secondary features of the deep structure-controlling faults that do not break to the surface.^{66,67} From these observations, it is noted that in cases where the surface faulting does not coincide with the deep-seated earthquake-generating faults, further uncertainty is introduced into the evaluation of surface faulting associated with historic shocks and into the prediction of possible future shocks. Several marine geophysical profiling systems that were developed mainly to aid in petroleum exploration are available to identify and define faulting at offshore sites.

In summary, there are many geological, geophysical, and geochemical tools presently available for use in evaluating faulting at potential reactor sites. In addition to those that have proved to be useful in past site evaluation and to those that are currently acceptable in other types

of geological evaluations, there are many promising methods, such as new remote-sensing devices, strain sensors, microearthquake studies, etc., that may eventually be extremely useful. However, before these can be utilized effectively, investigations must first be initiated to define their limitations and adaptability to reactor site evaluations.

4.9 Design Considerations for Fault Movement

Some specialists believe that after experts in the earth sciences have given assurance that no fault slippage will occur at a specified site, structural design should be concentrated on well-developed conservative engineering practice rather than novel concepts intended to insure safe survival in the event of fault slippage. The most obvious extrapolation of current engineering practice that is employed to provide structural resistance to faults is to reinforce the foundation mat of the structure. However, in so doing, if the turbine and reactor are on separate foundation mats, either the connecting primary piping and cables are designed to be sacrificed or all are carefully designed to withstand a number of specified linear and rotational displacements. Vertical slippage under a diametral plane would cause maximum tension near the top of a flat base slab, and some vertical fault slippage near an edge would require critical tension steel in the bottom of the slab. A less severe base shear case would occur with horizontal faulting through a diametral plane. In the Bodega BWR it is likely the concrete containment structure would fail before any appreciable deformation of the steel drywell would begin. In some older PWR plants, a part of the safety injection system piping is underground, and any pipes or cables that must be so placed generally have much better chances of surviving moderate fault slippage if they are encased in large corrugated pipes to allow room for deflection. Also, PWR fuel transfer tubes present special problems in fault slippage design.

If foundations can safely survive fault slippages of up to 1 ft, confirming analyses should show that any reactor not deeply embedded and not in the zone of a significant fault would have an unusually small chance of being subjected to a fault slippage large enough to cause a major accident. However, if the reactor structure extends 10 to 50 ft below

ground, a common case under present design, it would have a very limited capacity to survive fault slippage because of loads on the walls of the structure. Some increase in fault survival capacity of reactor structure might come from application of the new ductile reinforced concrete, which is designed to allow greatly increased yielding in the reinforcement and thus prevent crushing of compression concrete. It has, in theory, capacity to survive far more strain than ordinary, more brittle concrete. However, this type of construction needs further development before application to fault-resistant reactor designs. As is the case with shaking, experience with prestressed concrete undergoing simulated fault slippage is too limited to give firm conclusions. However, as with excitation, prestressed concrete should be much superior under combination of tension and shear for intensities up to the design limits. The primary system should be much more tolerant of modest tilting if there is no associated differential movement. In design for fault slippage, as with excitation, it should be advantageous to have much of the reactor plant, together with its many necessary safety devices, constructed on an integral foundation pad.

4.9.1 Testing and Models

Few of the elements of any reactor have been thoroughly tested for fault slippage survival or for shaking, most have not been tested at all, and there are many parts for which no excitation tests have been made even on comparable elements. It is thought that refueling of the BWR should still be practical following a $1/2^\circ$ tilt, the reactor should still be operable following a 5° tilt, and it should not suffer mechanical damage until tilted more than 30° .⁶⁸ However, many suppliers have no estimates of levels of tilting and of distortion under which reactor equipment can still function. Since testing of a full-scale plant under simulated displacements is obviously out of the question, consideration has been given to the use of models.

Modeling for fault slippage appears to have some attractive features. However, techniques for the successful physical modeling of soil are not well developed, especially if the soil is fine grained and contains water, and most especially if it is of clay, for these materials are nonhomogeneous and undergo nonlinear plastic deformation in fashions difficult

to predict. Models of from 5 to 8% size or larger would be required in almost any case. However, it might be possible instead to program a number of carefully controlled hydraulic shakers simultaneously supporting a simple model of a reactor plant structure. The model might then be subjected to a wide range of vibrations, displacements, and simulated reactions of supporting soils covering most situations that might be experienced by a prototype. On the other hand, a study using only a properly programmed large computer⁶⁹ might give ample conclusions over wide ranges much more quickly and inexpensively. Physical models, other than those for gross checks should not be built unless their need and unique advantages are clearly stated and most critically examined.

4.9.2 Effect of Faulting on Water Supply

Movement or warping of crustal blocks in earthquakes can directly affect surface-water sources, such as those that may be needed for post-accident cooling. Floods can be caused by earthquake disturbance of ground waters, and lurching can cause fissures in alluvium that remain open to widths known to reach 30 ft. The effects of the latter might be comparable to fault slippage, other than perhaps being slower, and must be considered in siting.

In the 1906 earthquake, San Francisco did not have water to fight the resulting fires, which caused the major financial losses, because the water mains near the fault rift were almost completely destroyed and shaking immediately put the storage and distribution systems inside the city out of action.

4.10 Design Concepts for Fault Displacements

Linear faulting under a structure can cause rotation that may have effects on some of the safety-related features of the plant in excess of those of the initial fault slippage. Even if a plant and its safety-related auxiliaries were located in an area where only strike-slip faults were anticipated, it would be necessary to consider the associated vertical slippages.

4.10.1 Integral Foundation

In the design of the Bodega reactor plant it was advocated by some that for the specified limited fault slippage a flat-bottomed integral foundation mat be supported by a layer of a few feet of carefully selected, well-drained sand, and it was not anticipated that such a design would require any radically new technology. However, confined sand under even 40- to 50-psi pressure has a shear resistance approaching that of some sandstones and thus might not yield as anticipated. Loose sand would be impractical because of (1) its possible liquefaction and subgrade failure if submerged, (2) its settling under vibration from operating machinery, and (3) its changes both in density and passive pressure with time. As discussed in Section 4.11, a frangible material, such as foamed concrete, might be quite superior in these respects.

A possible solution seems to be a dry moat gap as free space between the foundation mat and the surrounding soil, with the latter held either by a retaining wall of malleable sheet piling or by filling the spaces with frangible material. Such a gap, or even a large moat surrounding the entire plant,^{70,71} might also attenuate certain incoming seismic waves. Reducing the depth to which the containment vessel is bedded would reduce the passive pressures that would develop on the walls during a fault slip whose plane passed through the containment vessel and, hence, reduce the required wall thicknesses and the amount of reinforcing steel needed.

A design for a nuclear plant with an integral foundation was developed by Holmes and Narver.⁷² In this design the containment vessel is bedded to depths up to 50 ft on sand of known internal-friction angle. The advantages of increased depth include better strength of the soil to withstand the pressures under the foundation pad, greater lateral support during shaking, and less height to lift cooling water and feedwater.

Another version discussed in the Holmes and Narver report would extend the base pad beyond the containment walls. It could then be used as the base of a retaining wall, and there could be enough space between the two to permit the retaining wall to fail without endangering the containment structure. As noted above, crushable material between the retaining

wall and the containment wall would also increase shaking stability if that were needed.

A study by Kaiser Engineers suggests increasing the strength of the building so that it can be located on any type of stable soil or rock and making the contained reactor system self-sufficient in shutdown for at least four days.⁷³

4.10.2 Use of Caisson Structures

Another form of reactor support and containment that might be practical for surviving fault slippage is of the caisson type.⁷⁴ If the design were to require that the caisson survive a fault slippage at most of only a very few inches, no provisions might be necessary other than a comparison of maximum calculated earth pressures with the strength of the caisson's wall and base. It may be possible to design artificial slip surfaces in soil by using special materials of low shear strength. For a large design displacement, the caisson could be built to function in a water-filled pool extending to the base of the caisson and having vertical walls retained by properly supported malleable sheet piling. A peripheral moat 10 to 15 ft wide could provide space for fault displacement and perhaps also substantially reduce seismic excitation to the reactor both by the spatial gap and damping by the moat water.⁷⁵ In this case the umbilicals might be subject to sacrifice, with containment cooling then being supplied by an always operative, passive exchanger that would dump heat through the moat to a body of water. An underwater caisson designed to provide superior containment for the reactor might be constructed to be neutrally buoyant.

4.10.3 Floating Foundations

"Floating" foundations on sites where structures would be subjected to differential settling of low shear strength are judged to be entirely practical for conventional buildings.⁷⁶ They are employed principally for vertical loads and are substantially more costly than the usual supports. A design concept evaluated by Burns & Roe bedded the containment vessel in a bentonite slurry in which the vessel could actually be floated.⁶⁸

Floating a nuclear plant on water was considered by Daniel, Mann, Johnson, and Mendenhall.⁷⁷ In this concept the entire plant, including fuel storage, turbine-generators, switch gear, and transformers, is floated on a single barge that draws 30 ft. The bottom of the barge is equipped with open-air cushion cells designed to reduce the intensity of vertical pressure waves by a factor of at least 7. A substantial 30-ft-high break-water surrounds the barge on all four sides to protect it against wave action. The anchoring method provides for protection against tides and tsunamis. It is assumed that tsunamis do not pose a serious threat in water that is more than 30 ft deep.

4.10.4 Artificial Offshore Island

Construction of an artificial island, as has been proposed for some reactor sites, provides an opportunity to develop a foundation specifically to accommodate fault movements. As is the case with any limited number of specified reactor sites, however, there is a very small chance that fault slippage effects would be observed on an artificial island. It appears that a fault slippage of a very few feet through the base of such an island would cause a comparable slippage at the surface without the loss of the island. It might be practical to build the offshore island with an impervious layer between the wave defense and fill material and to employ permanent well points in the island to insure against liquefaction, although there are other, and perhaps more practical, methods available for preventing this. For example, the island fill could be of material not subject to liquefaction.

4.11 Materials for Isolation from Fault Slippage

The frangible materials applied in some nuclear-blast-resistant structure could be used in design for earthquake survival and, in some cases, to disassociate from the structure both vibratory and displacement forces transmitted from soil. For most applications, an ideal isolation material exhibits elastoplastic characteristics; that is, it behaves elastically with strain up to a yield point and then enters a long plateau where stress is nearly unchanged with increasing strain. At the end of this plateau

there is a limit, called the locking point, beyond which stress again increases with strain up to a crushing failure. A second group of frangible materials, the so-called "hardening" type, can also be used in some applications. They do not exhibit the plateau but, rather, increase slowly in stress with applied strain until they reach points where their stress with increased strain builds rapidly to failure. In some cases, isolation materials may be crushed to half their volume before locking.

In general, in designing for fault slippage the reactor must be placed in a support medium having known characteristics. The horizontal stiffness must be described both by a horizontal passive force and a horizontal friction force acting on the bottom of the mat, and the latter in turn depends on the shear modulus of the foundation material. The rotational stiffness must be described by both a vertical couple expressed as a shear modulus on the foundation mat and a horizontal couple due to the passive pressure. In addition, a rocking stiffness and a tributary mass having damping characteristics must all be defined. Without proper definitions of the forces exerted and without a knowledge of the soil displacements that are related, it would not be possible to determine the displacement an embedded or partially embedded structure could survive.

References

1. C. F. Richter, *Elementary Seismology*, W. H. Freeman and Company, San Francisco, 1958.
2. F. F. Evison, Earthquakes and Faults, *Bull. Seismol. Soc. Amer.*, 53(5): 873-891 (October 1963).
3. L. D. Leet and F. J. Leet, The Earth's Mantle, *Bull. Seismol. Soc. Amer.*, 55(3): 619-625 (June 1965).
4. J. H. Hodgson, *Earthquakes and Earth Structure*, Prentice-Hall, New Jersey, 1964.
5. C. R. Allen et al., Relationship Between Seismicity and Geologic Structure in the Southern California Region, *Bull. Seismol. Soc. Amer.*, 55(4): 753-797 (August 1965).
6. M. L. Fuller, The New Madrid Earthquake, Bulletin 494, U.S. Geological Survey, 1912.

7. N. Chryssaopoulos, Earthquake History and Seismic Design Practice in the Midwest, in *Symposium on Earthquakes and the Practice of Soil and Geological Engineering, December 12-13, 1964*, pp. 97-106.
8. M. G. Bonilla, Historic Surface Faulting in Continental United States and Adjacent Parts of Mexico, USAEC Report TID-24124, U.S. Department of the Interior, 1967.
9. D. B. Slemmons, Pliocene and Quaternary Crustal Movements of the Basin-and-Range Province, U.S.A., *J. Geosci.*, 10(1-11): 91-103 (March 1967).
10. C. E. Dutton, The Charleston Earthquake of August 31, 1886, 9th Annual Report 1887-1888, pp. 203-528, U.S. Geological Survey.
11. F. Bryan, Recent Movements on a Fault of the Balcones System, McLennan County, Texas, *Amer. Assoc. Petrol. Geol. Bull.*, 17(4): 439-442 (1933).
12. H. N. Fisk, Geological Investigation of the Alluvial Valley of the Lower Mississippi River, Vicksburg, Mississippi River Commission, 1944.
13. D. E. Bell and V. A. Brill, Active Faulting in Lavaca County, Texas, *Amer. Assoc. Petrol. Geol. Bull.*, 22(1): 104-106 (1938).
14. M. M. Sheets, Diastrophism During Historic Time in the Gulf Coastal Plain, *Amer. Assoc. Petrol. Geol. Bull.*, 31(2): 201-226 (1947).
15. P. Weaver and M. M. Sheets, Active Faults, Subsidence, and Foundation Problems in the Houston, Texas, Area, in *Geological Society of America Guidebook for 1962 Field Trips*, p. 254.
16. P. N. Wiggins III, Geology of Ham Gossett Oil Fields, Kaufman County, Texas, *Amer. Assoc. Petrol. Geol. Bull.*, 38(2): 306-318 (1954).
17. G. P. Wollard, Areas of Tectonic Activity in the United States as Indicated by Earthquake Epicenters, *Trans. Amer. Geophys. Union*, 39(6): 1135-1150 (1958).
18. J. C. Stepp, Seismic Probability Maps, in *Proceedings of ESSA Symposium on Earthquake Prediction, Feb. 7-9, 1966*, pp. 106-109.
19. D. Tocher, Earthquake and Ground Breakage, *Bull. Seismol. Soc. Amer.*, 48: 147-153 (April 1958).
20. A. Ryall, D. B. Slemmons, and L. D. Gedney, Seismicity, Tectonism, and Surface Faulting in the Western United States During Historic Time, *Bull. Seismol. Soc. Amer.*, 56(5): 1105-1135 (October 1966).
21. P. Saint-Amant, Two Proposed Measures of Seismicity, *Bull. Seismol. Soc. Amer.*, 46: 41-45 (1965).

22. S. D. Townley and M. W. Allen, Descriptive Catalog of Earthquakes of the Pacific Coast of the United States, 1769-1928, *Bull. Seismol. Soc. Amer.*, 29: 1-252 (1939).
23. F. Press, Displacements, Strains, and Tilts at Teleseismic Distances, *J. Geophys. Res.*, 70: 2395-2412 (1965).
24. Coast and Geodetic Survey, Earthquake History of the United States, Part I - Stronger Earthquakes of the United States (Exclusive of California and Western Nevada), Special Publication No. 41-1, rev. (through 1963).
25. Coast and Geodetic Survey, Earthquake History of the United States, Part II - Stronger Earthquakes of California and Western Nevada, Special Publication No. 41-1, rev. (through 1963).
26. V. P. Gianella, Earthquake and Faulting, Fort Sage Mountains, California, December 1950, *Bull. Seismol. Soc. Amer.*, 47(3): 173-177 (1957).
27. J. N. Brune and C. R. Allen, A Low-Stress-Drop, Low-Magnitude Earthquake with Surface Faulting: The Imperial, California, Earthquake of March 1966, *Bull. Seismol. Soc. Amer.*, 57(3): 501-514 (1967).
28. R. D. Brown, Jr., and J. G. Vedder, Surface Tectonic Fractures Along the San Andreas Fault, California, in The Parkfield-Cholame, California, Earthquakes of June-August 1966 - Surface Geologic Effects and Preliminary Seismic Data, U.S. Geol. Survey Prof. Paper 579, pp. 2-23, 1967.
29. A. L. Albee and J. L. Smith, Geologic Site Criteria for Nuclear Power Plant Location, *Trans. Soc. Mining Engrs.*, 238: 430-434 (December 1967).
30. D. Tocher, Movement on Faults, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. I, pp. 551-564, Japan, 1960.
31. F. Press and W. F. Bruce, Earthquake Prediction, *Science*, 152(3729): 1575-1584 (1966).
32. F. A. McKeown, D. D. Dickey, and W. L. Ellis, Maps and Classification of Explosion-Produced Fractures in Yucca Flat, Nevada Test Site, U.S. Dept. of the Interior Geological Survey, Special Projects Branch, Tech-Letter NTS-195, Suppl. 1, Dec. 11, 1967.
33. R. S. Tarr and L. Martin, The Earthquakes at Yakutat Bay, Alaska, in September 1899, U.S. Geol. Survey Prof. Paper 135, 1912.
34. P. Saint-Amand, Geological and Geophysical Synthesis of the Tectonics of Portions of British Columbia, the Yukon Territory, and Alaska, *Geol. Soc. Amer. Bull.*, 68(10): 1343-1370 (1957).

35. A. C. Lawson et al., The California Earthquake of April 18, 1906, Report of the State Earthquake Investigation Commission, Carnegie Institute, Washington, Pub. 87, Vol. 1, 1908.
36. G. Plafker, Surface Faults on Montague Island Associated with the 1964 Alaska Earthquake, U.S. Geol. Survey Prof. Paper 543-G, 1967.
37. L. S. Cluff of Woodward-Clyde and Associates, Consulting Engineering and Geologists, San Francisco, personal communication to T. F. Lomenick, Oak Ridge National Laboratory, Aug. 14, 1967.
38. A Report of the Ad Hoc Panel on Earthquake Prediction to the Office of Science and Technology, Earthquake Prediction: A Proposal for a Ten-Year Program of Research, Washington, D.C., May 1965.
39. The Resources Agency of California Department of Water Resources, Crustal Strain and Fault Movement Investigation, Progress Report, Bulletin No. 116-1, May 1963.
40. The Resources Agency of California Department of Water Resources, Crustal Strain and Fault Movement Investigation, Faults and Earthquake Epicenters in California, Bulletin No. 116-2, January 1964.
41. L. S. Cluff, Active Fault Problems, in *Symposium on Earthquakes and the Practice of Soil and Geological Engineering, December 12-13, 1964*, pp. 43-78.
42. J. L. Sherard, A Study of the Influence of the Earthquake Hazard on the Design of Embankment Dams, A report prepared for the State of California Department of Water Resources, July 1966.
43. Earthquakes in Kern County, California During 1952, California Department of Natural Resources, Division of Mines, Bulletin 171, 1955.
44. C. R. Allen et al., The Borrego Mountain, California, Earthquake of April, 1968, *Bull. Seismol. Soc. Amer.*, 58(3): 1183-1187 (June 1968).
45. T. W. Koch, Analysis and Effects of Current Movement on an Active Thrust Fault in Buena Vista Hills Oil Field, Kern County, California, *Bull. Amer. Assoc. Pet. Geol.*, 17: 694-712 (1933).
46. J. W. Wilt, Measured Movement Along the Surface Trace of an Active Thrust Fault in the Buena Vista Hills, Kern County, California, *Bull. Seismol. Soc. Amer.*, 48: 169 (1958).
47. D. E. Hudson and R. F. Scott, Fault Motions at the Baldwin Hills Reservoir Site, *Bull. Seismol. Soc. Amer.*, 55(1): 165-180 (February 1965).

48. L. S. Cluff and K. V. Steinbrugge, Hayward Fault Slippage in the Irvington-Niles Districts of Fremont, California, *Bull. Seismol. Soc. Amer.*, 56(2): 257-279 (April 1966).
49. M. G. Bonilla, Deformation of Railroad Tracks by Slippage on the Hayward Fault in the Niles District of Fremont, California, *Bull. Seismol. Soc. Amer.*, 56(2): 281-289 (April 1966).
50. F. B. Blanchard and G. L. Laverty, Displacements in the Claremont Water Tunnel at the Intersection with the Hayward Fault, *Bull. Seismol. Soc. Amer.*, 56(2): 291-294 (April 1966).
51. D. H. Radbruch and B. J. Lennert, Damage to Culvert Under Memorial Stadium, University of California, Berkeley, Caused by Slippage in the Hayward Fault Zone, *Bull. Seismol. Soc. Amer.*, 56(2): 295-304 (April 1966).
52. B. A. Bolt and W. C. Marion, Instrumental Measurements of Slippage on the Hayward Fault, *Bull. Seismol. Soc. Amer.*, 56(2): 305-316 (April 1966).
53. A. J. Pope, J. L. Stearn, and C. A. Whitten, Surveys for Crustal Movement Along the Hayward Fault, *Bull. Seismol. Soc. Amer.*, 56(2): 317-323 (April 1966).
54. K. V. Steinbrugge and E. G. Zacher, Fault Creep and Property Damage, *Bull. Seismol. Soc. Amer.*, 50(3): 389-415 (July 1960).
55. D. Tocher, Creep Rate and Related Measurements at Vineyard, California, *Bull. Seismol. Soc. Amer.*, 50(3): 389-415 (July 1960).
56. M. P. Billings, *Structural Geology*, Prentice-Hall, New Jersey, 1942.
57. P. C. Badgley, L. F. Childs, and W. L. Vest, The Application of Remote Sensing Instruments in Earth Resource Survey, Paper G-123, presented at the 36th Annual Meeting of the Society of Exploration Geophysicists, October 18, 1966, Houston, Texas.
58. D. C. Parker and M. F. Wolff, Remote Sensing, *Intern. Sci. Technol.*, 43: 20-31 (July 1965).
59. E. B. Waggoner, Geophysical Methods for Fault Location: A Case History and Evaluation of Methods, in *Symposium on Earthquakes and the Practice of Soil and Geological Engineering*, December 12-13, 1964, pp. 153-159.
60. M. B. Dobrin, *Introduction to Geophysical Prospecting*, McGraw-Hill, New York, 1960.

61. E. Bullard, The Detection of Underground Explosions, *Sci. Amer.*, 215(1): 19-29 (1966).
62. Jack Oliver et al., Microearthquake Activity Recorded by Portable Seismographs of High Sensitivity, *Bull. Seismol. Soc. Amer.*, 56(4): 899-924 (1966).
63. W. Stauder and A. Ryall, Spatial Distribution and Source Mechanisms of Microearthquakes in Central Nevada, *Bull. Seismol. Soc. Amer.*, 57(6): 1317-1345 (December 1967).
64. Pacific Gas and Electric Company, Bodega Bay Atomic Park, Amend. No. 5, Docket No. 50-205.
65. Preliminary Subsurface Investigation Proposed Nuclear Power Plant No. 1, Corral Canyon, Los Angeles County, California, in Technical Information in Support of Application for Construction Permit and License, Malibu Nuclear Plant Unit No. 1, Safeguards Report, Docket No. 50-214, November 1963.
66. J. C. Savage and L. M. Hastie, Surface Deformation Associated with Dip-Slip Faulting, *J. Geophys. Res.*, 71(2): 4897-4904 (Oct. 15, 1966).
67. I. J. Witkind et al., Geologic Features of the Earthquake at Hebgen Lake, Montana, August 17, 1959, *Bull. Seismol. Soc. Amer.*, 52(2): 163-180 (April 1962).
68. Burns & Roe, Inc., Conceptual Design of a BWR Plant to Resist Ground Fault Displacement, Report WO 2582-01 (USAEC Report TID-24530), May 1968.
69. D. Morris, Interaction of Continuous Frames and Soil Media, *J. Struct. Div., Proc. Amer. Soc. Civil Engrs.*, p. 13, October 1966.
70. R. N. Joshi, Striking Behavior of Structures in Assam Earthquakes, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. III, pp. 2143-2158, Japan, 1960.
71. A. G. Nazarov et al., The Method of Direct Determination of Reduced Spectra of Seismic Accelerations, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. II, pp. 1285-1296, Japan, 1960.
72. J. M. Duncan et al., Antiseismic Design of a PWR Nuclear Power Station, USAEC Report HN-192, Holmes & Narver, Inc., April 1968.
73. Kaiser Engineers, Design Study for an Earthquake Resistant Power Plant, USAEC Report KE-68-4, January 1968.

74. O. H. Klepper and C. G. Bell, Underwater Caisson Containment of Large Power Reactors, USAEC Report ORNL-4073, Oak Ridge National Laboratory, June 1967.
75. R. W. Clough, Effects of Earthquakes on Underwater Structures, in *Proceedings of Second World Conference on Earthquake Engineering*, Vol. II, pp. 815-831, Japan, 1960.
76. H. Q. Golder, State-of-the-art of Floating Foundations, *J. Soil Mech. Found. Div., Proc. Amer. Soc. Civil Engrs.*, 91: 81-88 (March 1965).
77. Daniel, Mann, Johnson, and Mendenhall, A Floating Earthquake-Resistant Nuclear Power Station, Final Report, USAEC Report TID-24558, April 1968.

5. TSUNAMIS

Tsunamis and tsunami-generated oscillations, such as might occur in harbors, are potentially dangerous to all coastal structures because of their considerable force, scouring action, and ability to carry drift material (see Fig. 5.1). Thus effects of tsunamis must be considered in the siting and design of reactors along coastal areas. Not only must the facility be protected against maximum expected uprushes but also the water intake structures must be located in a protected bay or at a depth in the ocean sufficient to assure continued operation during periods of maximum withdrawal.¹ The prediction of maximum uprush and withdrawal at particular sites is usually quite difficult because of local coastal features that tend to amplify or attenuate tsunamis. However, some localities (Hilo, Hawaii, and Crescent City, California) have consistently high tsunami waves, whereas other localities (Los Angeles and La Jolla, California) have consistently low tsunamis. This may indicate that the height of the maximum uprush for a particular site could be estimated if tsunami data were available from wave records.

5.1 Mechanisms and Occurrences

The Circum-Pacific belt is the major tsunami-generating area in the world due to the high-magnitude shallow-focus earthquakes that occur in the deep trenches bordering the Pacific Ocean. This belt accounts for about 80% of the mean annual seismic release for the world and about 60% of all recorded tsunamis.² Generating mechanisms are considered to be vertical movements (i.e., faulting) of the sea floor or the occurrence of landslides down steeply dipping shelves, although tsunamis can also be generated by nonseismic events, such as volcanic activity and submarine slides. The faulting with which tsunamis are associated is generally thought to be normal, reverse, or thrust; whereas wave motion from strike-slip faults would be much smaller or negligible. Data reported by Wiegel also suggests that the largest tsunamis are generated in the deepest water.³

Fig. 5.1. Tsunamis Damage Along Waterfront at Kodiak, Alaska, 1964.
(Photograph supplied by U.S. Geodetic Survey.)

Tsunamis have caused much property damage and loss of life in Japan, Alaska, Peru, Chile, and the Hawaiian Islands; runup heights have often exceeded 33 ft in these areas. Except for the Alaskan tsunami of March 28, 1964, which attained a height of 16 ft at Crescent City, California, the west coast of the U.S. has not been affected by tsunamis as much as the localities listed above. The U.S. Coast and Geodetic Survey reported that accounts of the California tsunami of December 21, 1812, indicated a height of 50 ft at Gaviota, 30 to 35 ft at Santa Barbara, and 15 ft at Ventura;⁴ however, Marine Advisors, after evaluating all available reports of the 1812 tsunami, concluded that reports of very large runups were not credible.⁵ Most runup heights at west coast sites rarely exceed the tidal range of about 6 ft.³ This is believed to be due to several factors including (1) the relatively low frequency of occurrence of large offshore earthquakes in Alaska in which strikes of the associated faults are situated so that the maximum tsunami wave is beamed toward the west coast (one occurred in 1946 and another in 1964), (2) the attenuation of tsunamis from sources more distant than Alaska due to long travel paths and further attenuation by the continental shelf off the west coast, and (3) the low frequency of occurrence of large earthquakes off the coast of California.

Locally generated tsunamis are not considered to be likely, since there is some doubt regarding the authenticity of the high runup attributed to the California tsunami of December 1812, and there have been no other historic offshore California earthquakes with magnitudes great enough to cause disastrous tsunamis. Shallow water depths, the gently sloping coastal shelf, and the predominately strike-slip nature of the offshore faulting are cited as not being conducive to the generation of large tsunamis along the west coast.⁶ However, the U.S. Coast and Geodetic Survey recommended that both the Bodega Bay and Corral Canyon reactor sites be protected to a vertical height of 50 ft above mean lower low water for tsunami runup from nearby severe marine earthquakes.⁴ As for the distant tsunamis, Alaskan tsunamis appear to pose the greatest danger to the west coast. However, much of the coast of Southern California is situated at glancing incidence to waves from Alaskan tsunamis, and thereby the danger in that area is reduced.³

Tsunamis of significance have not occurred along the Gulf and Atlantic coasts of the U.S. during historic time. Presumably, this is because the West Indian and Atlantic earthquakes are not as large as those in the Pacific, the water is not as deep as in the Pacific, and the very broad shelf along the Gulf and Atlantic coasts attenuates the tsunami waves that are developed. Surges from violent storms and hurricanes are more of a problem along the Atlantic and Gulf coasts than tsunamis. For example, there is no evidence of tsunamis resulting from the great Lisbon earthquake of November 1, 1755 (magnitude thought to be between 8.5 and 8.75) along the coast of America,⁷ whereas storms and hurricanes have caused considerable damage and loss of life from Texas to Maine.⁸

5.2 Properties

The Japanese have analyzed much of the data on tsunamis because the frequency of occurrence in Japan is higher than elsewhere.² From their data it appears that the magnitude of the earthquake must be greater than $6 + 0.01H$, where H is the focal depth in kilometers, for a noticeable tsunami (runup elevation of 2 ft or more) to occur. Also, it appears that the magnitude must be greater than $7.75 + 0.008H$ for disastrous tsunamis (runup elevations of 33 to 66 ft) to occur.^{2,3} One notable exception was the tsunami generated by the magnitude 7.4 Aleutian earthquake of April 1, 1946, which caused considerable damage to Hilo, Hawaii, because of local topography.

Tsunami waves in the open ocean have speeds as high as 600 mph, amplitudes of a few feet, periods of about an hour, and wave lengths of hundreds of miles.⁸ Speeds of tsunami waves are a function of depth of the ocean traversed ($C = \sqrt{gd}$, where C is speed in ft/sec, g is gravitational acceleration in ft/sec, and d is depth in feet);³ for example, speeds may be as high as 385 mph in 10,000 ft of water and 545 mph in 20,000 ft.² However, when tsunami waves approach coastlines, speed, amplitude, and periods are modified — wave speeds may decrease, wave amplitudes may increase to tens of feet, and wave periods may decrease to minutes. In addition, coastal waters are set into oscillation at some nearly constant

frequency, which is attributable to the local topography. In most areas, local oscillations rarely exceed normal tidal range.

The elevation reached by tsunamis on shores depends largely on steepness of the coastal shelf and coastal physiography. Features that favor maximum wave weight include headlands, triangular-shaped bays, and cliffs. Features that favor minimum wave heights include wide fringing reefs, protected shorelines, and low-lying coastlines.^{1,3} Tsunamis are often magnified by cliffs. For example, when the primary wave of the April 1, 1946, tsunami that originated in the Aleutian Islands approached Hilo, Hawaii, a Mach-stem reflected wave was developed well outside the harbor by the piling up of the incident waves along the steep coastal cliffs to the west. This effect resulted in 40-ft waves along these cliffs and a bore that inundated Hilo.¹ Coastal irregularities may cause wave magnification by converting fast open-sea waves to slow edge waves whose crests are normal to coast lines. These edge waves are thought to be caused by the trapping of energy on the coastal shelf.⁹ Also, waves tend to converge at headlands due to refraction, and amplitudes are thereby increased.³

Even in insensitive areas, such as Southern California, where the offshore topography is fairly regular, low-amplitude tsunamis may produce strong and unpredictable currents in bays and harbors.¹ Wiegell cites an occasion in Los Angeles harbor when moored ships surged or swayed up to 5 ft or more while wind waves were small and the swell was less than 1 ft high.³ Under such conditions, reactor intake structures could be damaged if moored vessels or floating debris surged against them.

Amplitudes of wave motion are also dependent upon the azimuth of the generating fault; that is, amplitudes of wave motion are much larger along a line normal to the strike of the generating fault than along one which is on-line with the strike, a beaming effect. For example, a maximum water elevation of 16 ft was recorded at Crescent City, California, which was on a great-circle azimuth normal to the strike of the fault; whereas a maximum change of 1 ft was recorded at the end of the Aleutian chain, on-line with the strike, at about the same distance from the epicenter as Crescent City.¹

Landslides along steep coasts are also sources of local tsunamis. The earthquake-induced landslide at the head of Lituya Bay, Alaska, a

T-shaped bay, caused a wave that destroyed trees at an elevation of 1720 ft at the head of the bay and reached a height of about 30 ft at the mouth of the bay. The tsunami resulting from the inland Atachama, Chile, earthquake of November 11, 1922, originated from a submarine slide near a relatively feebly shaken coast, where the surface sloped steeply to a considerable depth.¹⁰ Landslides along gently sloping coasts, such as California, however, do not appear to be capable of generating tsunamis.

Tsunamis along the west coast of the U.S. are fairly rare, but runup in some areas has been significant, such as at Crescent City and Avila Beach, California. Large tsunamis recorded at sites along the west coast have been generated in the following areas: (1) Aleutians, on April 1, 1946; (2) Kamchatka, on November 4, 1952; (3) Aleutians, on March 9, 1957; (4) Chile, on May 22, 1960; and (5) Alaska, on March 28, 1964. Tsunami runup data indicate that Crescent City and Avila Beach, California, are more sensitive than the other stations along the west coast, regardless of the direction of approach¹¹ (see Table 5.1). In general, tsunami runup and local effects are related to magnitude; however, there was considerably more runup for the magnitude 7.4 Aleutian earthquake of April 1, 1946, than for the magnitude 8.0 Aleutian earthquake of March 9, 1957. This was probably due to the fact that the tsunami from the 1946 earthquake was beamed toward the west coast, whereas the tsunami from the 1957 earthquake was beamed toward the Central Pacific. It is to be noted in Table 5.1 that the runup of the Chilean tsunami was greater at Crescent City than at the other stations on the west coast closer to the source. The runup of 9.1+ ft at Santa Monica, California, as a result of the Chilean tsunami, was probably due to the fact that the coastline in that area changes to an east-west direction and is thereby more nearly normal to the incident waves than the areas to the north or the south.

5.3 Evaluation of Tsunami Effects

Information on tsunamis may be obtained from wave records, runup, and model studies. In addition the occurrence of a barometric wave in the atmosphere following the Alaskan earthquake indicated the possibility of earlier tsunami warning by weather stations.¹ Wave records give the

Table 5.1. Maximum Tsunami Rise or Fall Heights at Selected Stations
for Recent Large Earthquakes^a

Date of earthquake	April 1, 1946	November 4, 1952	March 9, 1957	May 23, 1960	March 28, 1964
Location	Aleutians	Kamchatka	Aleutians	Chile	Alaska
Magnitude	7.4	8.3	8.0	8.5	8.4
Maximum tsunami elevation, ft					
Hilo, Hawaii	26 ^b	7.9	8.9	9.6+	12.5+
Honolulu, Hawaii	4.1	4.4	3.2+	5.5+	2.7
Sitka, Alaska	2.6	1.5	2.6	3.0	14.3
Neah Bay, Washington	1.2	1.5	1.0	2.4	4.7
Crescent City, California	5.9	6.8	4.3	10.9	16.4 ^c
San Francisco, California	1.7	3.5	1.7	2.9	7.4
Avila Beach, California	8.5	9.5+	3.5		10.4
Santa Monica, California		3.6	3.0	9.1	6.5
Los Angeles (Berth 60), California	2.5	2.0	2.1	5.0	3.2
La Jolla, California	1.4	0.8	2.0	3.3	2.2
San Diego, California	1.2	2.3	1.5	4.6	3.7
Antofagasta, Chile	5.9	4.7	3.0	4.6	3.3
Valparaiso, Chile	5.0	5.9+	6.7	5.6	6.2

^aFrom Ref. 11.

^bHeight cited by Richter in Ref. 8.

^cHeight cited by Van Dorn in Ref. 1.

heights and periods of waves from which wave spectra can be obtained. Runup measurements are made by surveying affected areas soon after a tsunami. From measurements made in a particular area, histograms can then be used to estimate maximum runup heights at the sites in question. Although physical models are large and expensive, they can reproduce the important features of tsunamis, such as generation, deep-water propagation, and runup.¹ Studies with these techniques and mathematical models can help in evaluating the effects of tsunamis at coastal sites, the sensitivity of sites, and the effects of tsunamis at offshore sites. There is a need for techniques to evaluate specific coastal sites in California and for developing a local early-warning system.

Recent advances in mathematical theory and computer technology suggest that it may be possible to model effects of tsunamis with computers. For example, Welch has simulated waves crashing against breakwaters and water from a broken dam colliding with downstream obstacles,¹² and the U.S. Coast and Geodetic Survey is developing a computer program that describes tsunami propagation in the Pacific.⁴ This program will calculate the relative convergence of rays of energy for different coastal zones from various source areas and thereby permit some degree of differentiation in the required calculation for protection of specific sites. Recently, Carrier developed a new mathematical theory to explain how tsunami waves can damage one harbor and yet leave a nearby harbor undamaged.¹³ He found that tsunami energy can be channeled by ridges on the ocean floor. When waves travel parallel to the ridges, a high amount of energy is channeled along a relatively narrow path.

Sites such as Hilo, Hawaii, that have consistently high tsunamis are considered to be "sensitive." Model studies have shown that Hilo is sensitive because of the Mach-stem reflected waves that are developed along cliffs to the west and then sweep into the city as bores. Reasons for the sensitivity of other sites are not as clear; however, sensitivity is usually attributed to steep offshore slopes or other features of coastal topography. Steep offshore slopes are believed to contribute to the sensitivity of sites in Japan, Peru, Chile, Hawaii, and Alaska. Even where offshore slopes are gentle, sites along cliffs, headlands, and triangularly

shaped bays are considered to be sensitive because tsunami energy is concentrated by these features.³ However, triangularly shaped bays may become insensitive if the period of the tsunami is much greater than the natural period of the bay.³ In addition, data by Carrier suggest that submarine ridges aligned parallel with the direction of propagation of tsunami waves contribute to the sensitivity of a site.¹³ Except for a few sites located on or near topographic features that have been associated with high runup, little is known about the sensitivities of various shoreline configurations because of insufficient instrumentation, knowledge of submarine topography, theory, and model studies.

Tsunamis do not attain destructive heights in the open ocean, and thus it would appear that sites on small islands might be less susceptible to tsunami waves than mainland locations. For example, the highest wave recorded at Wake Island on the graph of March 9, 1957, was 1.3 ft, whereas most heights recorded at west coast stations were greater. Although this appears to be a general rule, it may not apply to the nearshore island sites being considered for nuclear facilities, since oscillations caused by coastal irregularities may cause higher runups than anticipated on islands near the mainland.

5.4 Tsunami Warning System

The present tsunami warning system is, in general, capable of giving the west coast several hours advance notice of incoming tsunamis generated by distant events, and plans are under way to improve the system.¹¹ However, adequate warning cannot be given for tsunamis generated locally. For example, the destructive tsunami wave from the Prince William Sound earthquake of March 28, 1964, reached the southern coast of Alaska and Kodiak Island in 25 min, and the present tsunami warning system requires 30 to 45 min to locate an epicenter and to determine the magnitude of the event.¹ Therefore tsunami waves caused considerable damage along the Alaskan coast long before an alarm could be given. In order to speed up the determination of epicenters off the west coast, the U.S. Coast and Geodetic Survey has incorporated the Berkeley and Pasadena seismograph nets into their Pacific tsunami early-warning system.¹⁴ On-line digital

array processors that use seismographs, wave recorders, and microbarographs as input devices may speed up the location-magnitude-alarm sequence considerably.

References

1. W. G. Van Dorn, Tsunamis, in *Advances in Hydrosience*, Vol. 2, pp. 1-48, Academic Press, New York, 1965.
2. B. W. Wilson, L. M. Webb, and V. A. Hendrickson, The Nature of Tsunamis; Their Generation and Dispersion in Water of Finite Depth, Report No. SN-57-2, p. 150, National Engineering Science Company, 1962.
3. R. L. Wiegel, *Oceanographical Engineering*, p. 532, Prentice-Hall, New Jersey, 1964.
4. U.S. Coast and Geodetic Survey, Report on the Seismicity of Malibu, California, Area, Nov. 24, 1964.
5. Marine Advisors, Inc., Examination of Tsunami Potential at the San Onofre Nuclear Generating Station, unpublished report to Southern California Edison Company, submitted to AEC Reactor Licensing Division, 1965.
6. Southern California Edison Company and San Diego Gas and Electric Company, Final Engineering Report and Safety Analysis, Vol. 1, San Onofre Nuclear Generating Station, Unit 1, Docket 50-206.
7. H. F. Reid, The Lisbon Earthquake of November 1, 1755, *Bull. Seismol. Soc. Amer.*, 4: 53-80 (1914).
8. C. F. Richter, *Elementary Seismology*, edited by J. Gilluly and A. O. Woodford, W. H. Freeman and Company, San Francisco, 1958.
9. W. H. Munk, F. E. Snodgrass, and G. Carrier, Edge Waves on the Continental Shelf, *Science*, 123: 127-132 (1956).
10. B. Gutenberg, Tsunamis and Earthquakes, *Bull. Seismol. Soc. Amer.*, 29(4): 517-526 (1939).
11. M. G. Spaeth and S. C. Berkman, The Tsunami of March 28, 1964, as Recorded at Tide Stations, U.S. Dept. of Commerce, Coast and Geodetic Survey, 1965.
12. T. H. Thomas et al., Nuclear Reactors and Earthquakes, USAEC Report TID-7024, Lockheed Aircraft Corporation and Holmes & Narver, Inc., August 1963.

13. Sea Floor Guides Tidal Waves, *Science News*; 91(6): 131 (1967).
14. B. Bolt, Seismological Investigation in California, in *Proceedings of Earthquakes and Geologic Hazards Conference, December 7-8, 1964, San Francisco, California*, State of California, The Resources Agency, 1964.

6. CONCLUSIONS AND RECOMMENDATIONS

The recent rapid growth in the number of power reactors has focused attention on those unknown or imperfectly understood aspects of earthquakes that are related to reactor siting and design. In general this lack of knowledge has made it imperative that conservative bases be set on the critical parameters. In many areas of the country it appears that earthquake-induced phenomena can be adequately considered through current geoseismological technology. However, in some regions, conservatism may require that new design concepts be considered.

With regard to the needed improvements in predicting faulting, shaking, and tsunami effects at potential reactor sites, it is emphasized that, since there is no quantitative way to predict earthquakes, empirical and somewhat indirect approaches to the problems must be made. One of the principal means for studying earthquake phenomena is thus through the observation of earthquake events. Since large earthquakes occur rather infrequently and there are currently few, if any, positive correlations between the occurrence in place and time of earthquakes and measurable changes in the physical and/or chemical properties of rocks that comprise the earth's crust, major improvements in defining and determining the geoseismological factors pertinent to reactor siting and design will require intense and concerted efforts in the geological, seismological, and engineering disciplines.

Structural designs for accommodating moderate amounts of differential ground displacement and for assuring plant survival for all measured strong ground motions appear to be feasible; however, demonstrable proof of these designs may be needed, as well as tests to insure the functioning of all components and systems that are related to preventing the release of radioactivity.

The application of seismic information to power reactor engineering is a development of the last decade, with the result that few individuals have a broad technical competence of geoseismology, earthquake engineering, and reactor safety. Information on current practice in design of existing and planned nuclear power plants is primarily that contained in

applications for licenses, which reveal current practice in site selection, site exploration, and engineering criteria.

It is clear that work is needed to improve or confirm future siting and design decisions. Short-term work recommended in a coordinated program consists of (1) definition of more earthquake-related reactor criteria where sufficient knowledge exists, (2) compilation of geoseismological data suitable for design and analyses of reactors relative to earthquakes, and (3) intensive study of the designs of present-day reactors for the purpose of obtaining a better understanding of the problems of attaining safe reactor functioning during earthquakes. Specific technical areas of concern, organized by topic, are discussed in the following sections.

6.1 Faults

The determination of the activities of major faults and the incidence of subsidiary faulting in earthquake regions is one of the principal problems in reactor siting and design. Faulting is generally recognized as the cause of most large earthquakes in the western part of the U.S.; however, the exact mechanism of earthquakes and the physical and chemical conditions under which faulting occurs are imperfectly known. At the present time, it is not possible for some parts of the country to state the probability of earthquake occurrence at some proposed reactor sites and in the event of an earthquake the amount, type, and characteristics of the breaks. Areas of investigation that offer the best hope for alleviating some of the present uncertainties are outlined below.

6.1.1 Incidence and Activity

Aerial photography has proved to be an extremely convenient method for studying crustal movements, especially in arid undeveloped areas. In addition to the obvious advantage in rapidness of coverage, detailed fault patterns can often be observed by this method that may not be detectable by an investigator on the ground. This method of study should shed considerable light on the width of fault zones and perhaps on the nature of faulting that has been associated with historic earthquakes, as well as with those that have occurred in the recent geologic past.

Another way to add significantly to the knowledge of the nature and extent of branch or subsidiary faulting may be to examine in detail the faulting and local geology associated with the several large and destructive earthquakes that occur each year throughout the world. It is extremely important that the fracture patterns accompanying these shocks be mapped immediately after they occur and correlated with local geologic factors, because weathering tends to cover and disguise these breaks within a relatively short period of time.

Since coastal California is the most desirable area for the location of power reactors in that state, a base map of coastal California showing the location of all faults, onshore as well as offshore, that might be considered to be critical in reactor siting and design would be extremely beneficial for planning purposes. With the aid of this map, those areas where critical data on the location and activity of faults are needed would be specified, and then the appropriate studies to alleviate those uncertainties could be initiated.

Observations of stress-strain relationships in the rock column (made with tiltmeters, leveling surveys, tide gages, and gravimeters), magnetic surveys, and measurements of changes in the physical properties of rocks (e.g., elasticity, electrical conductivity, thermal conductivity, and magnetization) that may be related to tectonics should also be thoroughly evaluated to determine their usefulness in site surveying and evaluation for power reactors. Studies of earth currents and heat flow, in addition to well-seismograph studies of the volume strain in rocks, should be investigated for their applicability to reactor siting problems.

6.1.2 Fault Motion Characteristics

At the present time it is not known whether the intensity of ground motion is amplified or attenuated at the associated fault nor is the time-history of fault-break known. For example, about 50% of the total ground displacement associated with the Parkfield, California, earthquake of June 1966 was observed to occur over a period of several weeks after the earthquake; however, it is not practical to determine the time history of the first few minutes or hours of breakage for natural breaks, such

as the one at Parkfield. Since it is now possible to determine, to a large degree, the faults that will be activated as a result of nearby nuclear detonations, the time history of the fault break could conceivably be ascertained by instrumenting faults near future nuclear test shots. In addition to determining the characteristics of the early phase of faulting through instrumental observations, detailed characteristics of ground motions near faults could be measured by the installation of portable accelerographs adjacent to the faults.

6.2 Design To Accommodate Fault Movement

6.2.1 Design Requirements

The growing need for power reactors in regions having historic fault movement may make the development of fault-proof designs imperative. Although conceptual designs for initiating work on the problem are now completed, long-term projects of analysis and development will be necessary before design for movement of more than a few inches can be confirmed. The few structures that have been designed to survive faulting include earth dams, which are constructed to function safely following movement of a few feet.

Since there apparently is no record of a rigid man-made structure surviving a substantial fault break, a careful analysis should be made of the measures necessary for proper engineering design for small fault displacement. The absence of prototype tests must be taken into account. Until recently, surface faulting was thought not to occur rapidly, and thus static design simplifications were allowed. If this is the case, although fundamental research and development could lead to improvements, the structural principles needed for this study are already well known, and little of practical value could be expected from full-scale tests solely to demonstrate survival of a complete structure. Many carefully prepared development tests will certainly be necessary.

6.2.2 Design Concepts

Initial emphasis in reactor plant design development for faulting should be on integral foundation mats, containment structures, connecting

pipings and cables, and their terminals and penetrations. Structures with flat-bottomed thick-mat supports should be examined first, since they appear to be the conventional system most amenable to modification for displacement survival.

Study is needed of ways of circumventing the complications of umbilical pipes and cables between parts of the facility subject to relative movement. This problem could be minimized, for example, by having sensitive portions of the safety system rest on and be integral with the containment foundation mat or by careful placement of snubbers and supports so as to minimize responses. Such integral reactor foundation designs are not common, however. Much improvement within current technology is possible, inasmuch as ways of providing increased vibration and displacement survival capacity in important pipes are already known sufficiently for some applications. Recently completed conceptual designs (discussed in Chap. 4) indicate that if displacements do not exceed 6 ft and ground accelerations do not exceed 1 g, the cost increases for such designs should be moderate.

If conceptual designs and subsequent development tests generally show that central station reactor plants can be improved with realistic costs to survive foundation displacements that can occur in earthquakes, such designs might be considered starting points for detailed design of individual nuclear plants for local conditions.

6.3 Ground Shaking

Since there are relatively few instrumental records of strong ground motion, and since theoretical solutions are not generally available for predicting the important design parameters of earthquake-induced ground motions, it is difficult to define accurately the quantitative factors that characterize shaking at most sites located at specified distances from the epicenters of earthquakes of given magnitude. It is also apparent that the past seismic record is too short to make more than general predictions as to the frequency of occurrence of seismic activity, especially for earthquakes of large magnitude.

6.3.1 Characteristics of Motion

Maximum ground accelerations and ground motions are frequently estimated from the Modified Mercalli Intensity rating of earthquakes, which is based on the degree of shaking at a specified place; furthermore, the correlations between intensity and more quantitative formulations of ground motion are empirical and subject to large uncertainties. By recording a substantial number of events on strong-motion accelerometers, some of the dependence on intensity data could be relieved. To provide these records, a large number of instruments would be needed over relatively large geographical areas, as well as under a variety of geoseismological conditions. Since the frequency of the larger earthquakes is low in most parts of the contiguous U.S., it is realized that the accumulation of a significant amount of instrumentally recorded data of this type will require several years or more, even with a large number of installed instruments. Possibly this data-collection period could be shortened by installing strong-motion accelerometers in highly seismologically active areas of the world, such as in the Aleutians, where the larger earthquakes occur more frequently.

Strong-motion accelerometers could also be strategically placed in reactor structures, as well as in the nearby soil and rock, to provide data on the specific characteristics of strong ground motion and the response of the structure to these forces if a large earthquake occurred near it. In addition the installation of instruments to determine the interaction between structures in the underlying soils would provide extremely useful information in the event of strong ground motions.

Studies that relate the characteristics of small earthquakes to large ones may also provide more quantitative means for predicting the nature and extent of strong ground motion at reactor sites. For instance, micro-earthquake studies in areas that have recently experienced moderate or large earthquakes, such as at Caracas, Venezuela, in 1967, where soils and rocks show contrasting amplification or attenuation of the seismic waves, may be used to provide information on the usefulness of data on small earthquakes for determining the intensity of strong ground motions in various geologic media.

6.3.2 Induced Effects

Earthquake-induced ground motion may also affect soils through consolidation, liquefaction, differential compaction, and sliding. River basins, which can be highly desirable locations for reactors, are commonly underlaid by soft, alluvial, or unconsolidated soils that may be susceptible to failure in one or more of the modes mentioned above. Much can be learned from detailed investigations of previous earthquake-induced soil failures. However, new and improved techniques, equipment, and methods, such as new soil penetration tests that will establish the relative densities of soils at shallow depths, are needed to analyze the critical soil properties of various types of soils and provide assurance that they will not fail or to determine the critical design parameters for withstanding any expected failures. In-situ and laboratory measurements of the static and dynamic properties of soils (perhaps a wide range of soil types should be studied to determine their responses during earthquakes for generalizing soil behavior) and rocks, coupled with theoretical considerations of seismic wave motions, may also be helpful, as would small-scale models of soil behavior, in relating soil conditions and ground-motion characteristics. Finally, a larger field testing program could possibly be initiated through utilization of underground nuclear explosion tests. Comparative data on soil properties obtained in laboratory and field tests are greatly needed.

6.3.3 Seismicity

Seismic records are needed over much longer periods of time than are now available to predict accurately the seismic activity of most potential reactor sites and even of larger geographical areas. Some measure of the frequency of various seismic events may also be obtained by crustal deformation investigations with, for example, strainmeters, tiltmeters, geodetic surveys, and measurements of the physical and magnetic properties of rocks. All these measurements may also provide data pertinent to faulting, as discussed previously.

6.4 Design To Accommodate Shaking

6.4.1 Selection of Design Basis

Precise explanation or prediction of the occurrence of earthquakes in time and place has not been possible. For great earthquakes, recorded seismic history is much too brief and geologic-seismologic relationships too qualitative to support an argument that the probabilities of occurrences at different localities differ sufficiently to warrant selection of different seismic design bases for different sites. At the same time, greater differentiation in categorizing the safety importance of various systems within nuclear plants for application of various design bases should be made. It is recommended therefore that early effort be made to relate explicitly the changes in costs for design and construction of both typical large PWR and BWR reactor plants for survival of earthquakes ranging in intensities from those where seismic problems are minor to levels several times the highest currently used. Study of a requirement of no hazardous fission-product discharge to the environment under earthquake excitations and durations twice those recorded should be undertaken to determine the effects on power costs.

6.4.2 Soil-Structure Interaction

The reactor plant is composed of massive structures whose response to shaking depends largely on the characteristics of the supporting foundation. Seismic interaction of soil and structure is accordingly of much interest in reactor design, and yet the amount of available useful data is still quite small. Some modeling of the interaction is now being done; but reliable measurements, especially with carefully placed accelerometers on massive short-period structures supported by a variety of foundation materials, are needed either from earthquake measurements or from full-scale tests. This work should be supported, since field verification of many current design bases is needed.

Certain requirements, such as the locations of populations, load centers, and cooling-water sources, will sometimes force the siting of power reactor plants on foundation materials that are less than desirable.

Studies of ways of insuring the impossibility of liquefaction, the proper earthquake response of soils, and the limiting of settling to acceptable levels all need support. The whole problem of permanent displacement due to shaking must be considered in the design of auxiliary safety-related structures, such as those for emergency power and emergency cooling water.

6.4.3 Damping

Many aspects of reactor plant structural damping require investigation, with the initial objective being to supply designers with urgently needed specific source data. Compared with the amount of useful information needed, very little is known about damping, even in conventional structures. Assigned damping values can have an important influence on design. Little is known about damping of prestressed concrete, especially for such massive sections as would be used in reactors, and there is little information on damping even of conventional concrete structures when under reversed loads up to and beyond yield conditions. Damping phenomena are highly complex, and encouragement should be given to long-range fundamental studies, preferably in universities that already have groups with experience in the field.

Although information on structural damping is one of the main areas of need, equipment, pipe mounts, and piping, especially long runs, need study, as well as other appendages, instruments, and their supports. Basic studies should be made of damping by the coolant of all elements within the pressure vessel, both with and without coolant flow, with the objective of establishing a systematic approach to design of these components for earthquake survival. The study may indicate a need for reactor core element tests under induced excitation.

6.4.4 Structural Components and Materials

Yield Conditions. The usual aim is to design the entire power reactor plant to survive the design earthquake elastically. Even piping is allowed to yield only in a restricted fashion. Some specialists feel that if any reactor power plant ever undergoes the designated "hypothetical" or "maximum" earthquake, the survival of the structure will not be due to the

completeness of the elastic design to the last detail but, rather, to the fact that all components called upon to do so were able to go into the yield condition and still function adequately. Safety margins in structural design are usually based on the assumption that materials in the yield condition retain the requisite strength to carry design loads. Therefore sufficient knowledge of yielding under dynamic loading to conclude that systems important to safety can survive an earthquake beyond the design earthquake would be of great value in establishing and understanding safety margins in structures.

The practical advantages of specific design for routine performance of structures in the inelastic range is a matter of controversy in the reactor field. If general use of such designs becomes attractive, improvement would be needed in the understanding of strength, behavior, and failure mechanisms of even conventional reinforced-concrete members. Especially lacking is an understanding of deep concrete members, particularly those subjected to combined tension and shear. Fundamental long-range studies that lead ultimately to utilizing much more of the tensile capacity of concrete are justified.

Thus studies on reactor structures should be extended for nonlinear behavior. Of the many areas requiring work, the behavior of steel and reinforced concrete, especially of prestressed-concrete elements and joints under reversing loads into the yield range, should be investigated further. Energy absorption, structural period changes resulting from yielding, and various ways in which failure can occur are all of much importance.

Pressurized Components. There is disagreement as to whether joints, bends, connections, elbows, and valves in piping and nozzles of pressure vessels are particularly sensitive to earthquake forces and need more study, especially for nonlinear behavior in earthquake excitation and fault slippage. No authoritative information showing that they are insensitive could be located, and a special study of such components might therefore be undertaken. Penetrations and access ports also warrant clear, explicit analyses.

Concrete. In reinforced-concrete containment structures, the shear capacity of diagonal bars and the shear distribution between vertical or

horizontal and diagonal bars needs more investigation. Ordinary concrete can be quite brittle; and if a structure is to survive extreme earthquakes, increases in its ductility, as yet not employed in reactors, might be the most reasonable choice in many cases. Failures of structures having limit or ductile designs in the recent Caracas earthquake call for improved design, construction, inspection, and, especially, quality control rather than the abandonment of these concepts. The same can be said for the failures of prestressed-concrete structures in Anchorage.

Prestressed concrete is now being used extensively in reactor containment structures, and its use is being considered for other parts of the plant. Studies are needed of prestressed concrete under many conditions of seismic loading, internal containment pressure, and yield conditions. The work might best be done by certain highly qualified university groups. Studies should be made of ways that prestressed concrete might maintain, even beyond its yield point, sufficient integrity for reactor safety applications. Study is needed of joints for precast members and the energy absorption capacity of columns under combined high moment and shear. The advantages and disadvantages of grouting of posttensioned steel do not yet appear to be resolved and should be studied further.

6.4.5 Motion-Sensitive Control Systems

Up to the present time, reactor engineering for earthquake survival has been strongly influenced by those oriented toward a civil engineering and structural viewpoint. Designers of mechanical equipment have had much less influence and designers of electrical equipment, controls, and safety systems almost none. Many electronic, electrical, and electro-mechanical devices are depended upon for reactor safety, either as engineered safety features or as portions of the control systems. These are usually designated class 1 by designers, but they are normally purchased without analysis of their motion during earthquakes and the effects of that motion on the functioning of the plant. They are seldom tested at even the specified ground input motion. The fact that functional failure of a control or protection system is not necessarily caused by antecedent failure of the structure in which it is located is almost never taken into

account. Improvements could come quickly upon request, for some of the large reactor manufacturers have long been involved in shock testing of equipment and instruments in space, marine, and hardened-systems work.

Spectra of ground motions should be designated for application to motion-sensitive control devices. A design study for such devices should be made for the purpose of determining how adequate margins-to-failure levels can be established. This work might be done most efficiently in a national laboratory.

6.4.6 Seismic Instrumentation at Nuclear Power Plants

Proper accelerometer recording of the history of a central station power reactor near the epicenter of a great earthquake would give important experimental data in reactor engineering for earthquake survival. If a reactor disaster were to be caused by an earthquake at present, however, the resulting useful information might well be extremely limited. Although most earlier central station power reactors did not have strong-motion instrumentation, such instrumentation has been strongly advocated¹ for some time, and most new plants are committed to install strong-motion instruments.² Such instruments will provide essential information by which the operator will decide whether it will be advisable, after an earthquake, to shut the plant down for inspection.

Accelerometers rugged enough to survive a maximum reactor accident may have to be developed, for it is not clear that current instruments could function during and following such an accident. Strong-motion instruments capable of surviving in accident environments should be developed that could be integrated with the reactor's control and safety systems. These would be analogous to the flight recorders required on commercial aircraft and would allow postincident assessments of considerable value.

6.4.7 Design Analysis and Computation

Computational Techniques. There are two general methods used in calculating a reactor plant's dynamic responses to earthquakes: (1) a composite response spectrum approach and (2) direct input of digitized accelerometer records into a large computer programmed for time-history

calculations. There are clear advantages and disadvantages to each procedure. Both approaches may be inadequate if based on only one seismograph, for earthquakes have widely varying characteristics. It is clearer how response spectra should be combined into conservative envelopes, but this is not always done either for conventional or nuclear structures. In the future, a conservative digital time-history calculation procedure might be developed that would be preferable, with the response spectrum approach used as a high-quality check.

Simplified calculational techniques would be useful for making conceptual designs and checking results of machine computations, and these should follow procedures already developed by competent reactor engineers to insure their reliability and conservatism. Detailed and complex programming in large computers is not economically practical for preliminary design surveys and, in all probability, is not needed.

Calculations of Shock Loading. There has been work which strongly indicates that shock loadings consisting of a single strong pulse, as from weapons blasts, and earthquake loadings, which may have a duration of a few minutes, can be analytically related. This work should be verified by other groups and tested experimentally for its validity and limitations. Much could be gained in earthquake reactor work from the vast shock literature.

Earthquake Analysis Center. Costs of development of the highest quality matrix programs and of training specialists for their use are so great that it may be said that no architect-engineer has a set of programs that is entirely satisfactory. It may be practical to gather earthquake computer programs, the development of which was financed by the U.S. Government but which have not been released or disseminated, at a designated analysis center. These could be used in development of new programs, together with detailed programming and checking instructions. The center could also be a repository of earthquake information for reactor application. Once the programs were available, courses of instruction for qualification would seem reasonable, and qualification tests would be in order. The resulting uniform high-quality analytical capability in the industry and the possibility of competent checking should pay large benefits in

both competition and regulatory control. According to some specialists of long experience in the field, once high-level competency is established throughout the industry, design costs even for large earthquake excitations probably would be almost negligible, and the additional costs for construction might not be large.

Long-Range Statistical Studies. Long-range statistical studies interrelating fundamental dynamics, reactor safety, earthquake phenomena, and reactor structures and equipment could ultimately pay unusual dividends by placing the entire problem on a more rational analytical basis. A small group of specialists should initially be assembled and supported.

6.4.8 Dynamic Tests

Structural response measurements have been made during underground nuclear tests in Nevada, and these are now being reported. It is hoped that the geologic considerations and soil-structure relationships will be emphasized and that in the future special attention can be given to massive, short-period structures so that results can be applied to power reactors. Future underground nuclear tests in the Aleutians or at the Nevada Test Site could give important information for reactor design. A study of the use of these tests should be made for defining a program of research. Such a study may justify the construction of massive short-period test structures at these sites.

Specific calculations of excitation levels of all safety-related elements in reactors should be made. Where economically feasible, these components should be tested at above the calculated excitation. Sensing devices, relays, electronic equipment, control rods and their drives, and control valves are examples. Since the maximum acceleration on a reactor's upper floor can be calculated to be many times that of the designated ground input motion, care must be taken in specifying and calculating the motion from the ground through the structure to the appendage and on to all safety-related instruments and equipment. Not only must acceleration, velocity, and displacement be put in their proper perspective in each case, but the duration of excitation must be taken into consideration. Then the safety instrumentation and equipment must be tested at least to that excitation and certified.

Many safety-related items, such as instruments, actuators, and emergency power generators and their control relays, may survive to function quite easily following short-duration shocks and still fail to function during sustained excitation. Since earthquakes may last for several minutes and be followed almost immediately by other earthquakes, specific consideration of these problems must be made in their testing.

Feasibility studies should be initiated for excitation of large, and even full-scale, structures by shakers, rocket power, and explosives. Such work might be considered for LOFT, BONUS, and the EBWR. Such huge tests would have as their prime objectives the understanding of particular responses and the generation of useful source data, rather than the proof of any particular plant. Since so many responses are nonlinear, especially in damping and soil-structure interaction, such tests are well justified. Soil-structure-umbilical interactions and autocorrelation work should be evaluated. All such activities should be initiated only after careful study and preparation.

Model tests may be desired in specific cases where detailed justification can be given. These may be helpful in specific cases to give gross checks on computer programs and design concepts. However, it is felt that they are unlikely to play a major part in reactor safety research.

The Navy has had vast experience in testing equipment and instruments under shock loading, including those cases where the structure (i.e., the ship) survived while the internals failed. They also have gone through large programs of backfitting, including high-priority assignments. A review of their work for implications for land-based power reactors is in order.

6.5 Tsunamis

Tsunami waves pose some hazard to nuclear facilities at coastal or offshore sites along the west coast of the U.S. In general, tsunamis from distant sources have not exceeded the tidal range of about 6 ft along the west coast. However, Crescent City, California, experienced a runup of about 16 ft that caused structural damage and loss of life during the Alaskan earthquake of March 28, 1964, and the California earthquake of

October 21, 1812, reportedly caused a tsunami as high as 50 ft in the Santa Barbara area. Although there is some question as to the credibility of this extreme height, it was recommended that the Bodega Bay and Corral Canyon reactors be protected to a height of 50 ft above mean lower low water.

To alleviate some of the uncertainties associated with predicting the occurrences and nature of tsunamis along the west coast of the U.S., further studies are needed in the following areas: (1) model studies of sites having high runup, (2) statistical studies of wave heights and runup in selected coastal areas, perhaps even at offshore (island) sites, and (3) more intense geological investigations of coastal features that may cause localized tsunamis. Because theory becomes so complicated for tsunami effects near coastlines, model studies appear to hold the most promise for practical answers on problems of runup. Model studies could demonstrate tsunami effects for complex topography, offshore islands, and strike-slip movement. Considering the expense of modeling a number of areas, it may be feasible to model tsunami effects by using a computer. Work of this nature has been done for breakwaters and a broken dam. Tsunami runup could probably be simulated with some simple topographic models in present large-memory computers.

Statistical studies of runup heights appear to be helpful in predictions, because runup heights of different tsunamis recorded at a single coastal site are virtually constant, whereas runup heights from a single tsunami recorded at a number of coastal sites are quite variable. Therefore, if information is available on past runups at a specific site, a prediction of expected heights can be made; however, if no runup information is available, it is difficult to predict runup heights. Therefore, wave recorders should be installed in coastal areas considered for reactors if little or no runup information is currently available. Although wave heights are expected to be low at prospective offshore sites, especially off the coast of Southern California, actual wave height measurements should be made at representative sites.

6.6 Design for Protection from Tsunamis

The present approach in reactor design for protection from tsunamis is to assure that the wave defense of the shore or island has the capacity to provide plant protection during and following the design-basis waves. On the west coast protection has been provided for runup of as much as 50 ft. When sea walls are provided they must be designed for seismic loading. Foundation stability, anchorage, and corrosion must be given special consideration. Those localities with a potential for locally generated waves should be avoided. Large tsunamis that are generated by distant earthquakes should allow adequate warning not only for safe shutdown but even primary system depressurization. However, not all warnings are followed by tsunamis, so such a shutdown procedure might not be considered economically practical. In some cases the withdrawal of the ocean preceding large tsunami waves might present problems in supplying cooling water by the loss of required head on cooling-water pumps.

References

1. S. E. Moore, Seismic Considerations in Nuclear Power Plants, *Nucl. Safety*, 3(4): 58 (June 1962).
2. A. J. Pressesky, AEC-RDT, personal communication to W. B. Cottrell, Oak Ridge National Laboratory, May 5, 1967.

THIS PAGE
WAS INTENTIONALLY
LEFT BLANK

APPENDICES

THIS PAGE
WAS INTENTIONALLY
LEFT BLANK

Appendix A

GLOSSARY

Activity. Of or pertaining to earthquakes.

Beaming. As applied to tsunamis, the propagation of the highest ocean waves in a direction perpendicular to the strike of the generating fault.

Bore. As applied to tsunamis, a wave that approaches coastlines with a high abrupt front.

Breakage. Fractured rock caused by faulting; of or pertaining to faulting.

Cultural Noise. Unwanted oscillations recorded by seismographs that are caused by cars, trucks, trains, pumps, heavy machinery, and other human activities.

Design Earthquake. The earthquake chosen on the basis of evidence of historic occurrences to define the ground shaking for which the plant will be designed. "Design-basis earthquake" is a term currently employed in conjunction with "operating-basis earthquake" to distinguish between design bases for different features of a plant. From the time the nuclear power plant at Bodega was proposed until the present time, applications for construction permits have specified two levels of shaking for which various portions of the plant would be designed. The AEC seismic criteria use the current terminology, but various names have been used in the past, and design criteria in license applications vary somewhat from one case to another. In any case, structures and systems that are vital to safety are generally designed to function properly in the larger of the specified design earthquakes, while the plant is usually expected to be capable of operating normally during the lesser. This report sometimes uses "maximum design earthquake" in the general sense of the larger of the design earthquakes.

Dip. The angle at which a stratum or any planar feature is inclined from the horizontal.

Dislocation Models. The application of static-elastic dislocation theory to the study of surface deformation accompanying faulting.

Displacement. The relative movement of the two sides of a fault (Glossary of Geology and Related Sciences);¹ the difference in positions of ground particles as recorded by a seismograph; the integral of the velocity of ground particles as recorded by a seismograph.

Displacement Velocity (of Faults). The rate of displacement of adjacent rock masses on either side of a fault, expressed either as a relative

velocity of one side with respect to the other or as the velocities of both sides with respect to a stationary observer; cf. Rupture Velocity.

Draping. The structural folding of unconsolidated submarine sediments over a faulted surface.

Epicenter. The point on the earth's surface directly above the focus of an earthquake. (Glossary of Geology and Related Sciences)

Fault. A fracture or fracture zone along which there has been displacement of the two sides relative to one another parallel to the fracture. The displacement may be a few inches or many miles. (Glossary of Geology and Related Sciences)

Active Fault. A fracture that exhibits geological, seismological, and historical evidence of differential displacement. (Cluff)²
A fracture along which differential displacement has occurred in Recent geologic time; that is, in the last 10,000 years. (Cluff)

Block Fault. A fracture along which vertical differential displacement has occurred that forms a boundary between two bodies of rock.

Dead Fault. A fracture that has exhibited no evidence of movement in Recent geologic time.

Dextral-Normal Fault. A normal fault in which the net slip is such that to an observer moving toward the fault the opposite block appears to have moved to the right.

Dip-Slip Fault. A fault in which the net slip is practically in the line of the fault dip.

Earthquake-Generating Fault. A zone of dislocated or faulted rock that is a source of elastic waves.

Inactive Fault. A fault that has exhibited no evidence of movement in Recent geologic time.

Master Fault. Zone of disturbance peripheral to a continent extending to a depth of several hundred miles and including the foci of deep seated earthquakes; great faults or fault systems generally involving much lateral movement that have played a prominent part in the development of earth structure. (Supplement to the Glossary of Geology and Related Sciences)

Minor Fault. See Secondary Fault.

Normal Fault. A fault at which the mass of rock above the fault plane has been depressed with respect to the mass of rock below the fault plane.

Reverse Fault. A fault along which the mass of rock above the fault plane has been raised with respect to the mass of rock below the fault plane.

Secondary Fault. A fracture that arises as a direct result of movement on a master fault.

Strike-Slip Fault. A fault in which the component of shift is parallel to the strike of the fault. (Glossary of Geology and Related Sciences)

Structure-Controlling Fault. See Master Fault, second meaning.

Subsidiary Fault. See Secondary Fault.

Thrust Fault. A reverse fault that is characterized by a low angle of inclination with reference to a horizontal plane. (Glossary of Geology and Related Sciences)

Transcurrent Fault. Same as Strike-Slip Fault.

Transverse Fault. A fault whose strike is transverse to the general structure.

Fault-Plane Solutions. The determination of the type of fault movement by examining the distribution of compressions and dilations in the recording of a given shock that is generated by the fault in question.

Fault Scarp. The cliff formed by a fault. (Glossary of Geology and Related Sciences)

Fault Zone. A fault, instead of being a single clean fracture, may be a zone hundreds or thousands of feet wide; the fault zone consists of numerous interlacing small faults or a confused zone of gouge, breccia, or mylonite. (Glossary of Geology and Related Sciences)

Flow Slides. Mass downgrade movement of liquefied soil.

Fracture. Breaks in rocks due to faulting. (Glossary of Geology and Related Sciences)

Geologic Evidence of Faulting. Structural, stratigraphic, and physiographic phenomena indicating differential displacement (Cluff); phenomena include fault scarps, triangular facets, offset streams, side-hill ridges, shutter ridges, springs, etc.

Gouge. Soft clayey material often present between a vein and a wall or along a fault — also called *selvage*. (Webster's Third New International Dictionary)

Graben. A block, generally long compared with its width, that has been thrown down along a fault relative to the rocks on either side. (Billings)³

Hypocenter. The subterranean point believed to represent the position of the initial rupture causing an earthquake; the focus. (Richter)⁴

Insensitive. As applied to tsunamis, coastal areas that are not subjected to high waves.

Intensity. An index that refers to the degree of shaking at a specific place; a rating assigned by an experienced observer using a descriptive scale with grades indicated by Roman numerals from I to XII (Richter); cf., Magnitude.

Lurches. Cracks and fissures parallel to streams, gulches, etc., caused by earthquake motion at right angle to these original features as a result of yielding of soils in the directions in which support is lacking.

Mach-Stem. As applied to tsunamis, a type of reflected wave that has a height greater than that of the incident wave.

Magnitude. The logarithm of the maximum amplitude on a seismograph written by an instrument of specific standard type at a distance of 100 km from the epicenter of an earthquake (Richter); cf., Intensity.

Major Break. See Master Fault.

Marine Geophysical Profiling System. Systems making use of nondynamite acoustical sources and seismic reflection techniques to determine subsea structural geology by use of recorded visual cross sections.

Maximum Credible. As applied to earthquake magnitude, ground acceleration, tsunami runup, etc., that which might occur with a finite probability and be triggered by understandable mechanisms. (Thomas)⁵

Minor Break. See Secondary Fault.

Quaternary. The last one to two million years of geologic time. This period is subdivided into Pleistocene and Recent Epochs.

Recent Geologic Time. When used with "Recent" capitalized this term means the last 10,000 years of geologic time.

Rift. A large strike-slip fault parallel to the regional structure; specifically applied to the San Andreas rift in California. (Glossary of Geology and Related Sciences)

Runup. The rush of water up a structure on the breaking of a wave. Also uprush. The amount of runup is the vertical height above still water level that the rush of water reaches. (Glossary of Geology and Related Sciences)

Rupture. Same as Fracture.

Rupture Velocity (of a Fault). The velocity with which the leading edge of a fracture progresses into the region where fault movement has not yet begun; cf., Displacement Velocity.

Scram. As applied to power reactors, shutdown of the reactor by automatic insertion of control rods.

Seismic. Pertaining to characteristic of, or produced by earthquakes or earth vibrations. (Webster's Third New International Dictionary)

Seismic Factor. The ratio of actual ground particle acceleration to the acceleration due to gravity.

Seismic Regionalization Maps. Small-scale maps showing the great differences in seismicity between widely separated regions; large-scale maps showing the variation in local effects due to differences in ground, as well as consequences of varying distance from earthquake sources. (Richter)

Seismograph. Usually an electrical system that records seismic waves. Such a system usually includes a seismometer as a sensor and amplifier and systems for filtering, timing, testing, calibrating, recording, and power.

Seismological Evidence of Faulting. Tectonic phenomena, including earthquakes, microearthquakes, creep, and crustal distortion.

Seismometer. Detecting device that receives seismic impulses. (Glossary of Geology and Related Sciences)

Sensitive. As applied to tsunamis, coastal areas that are invariably subjected to high waves; often regardless of direction of approach.

Strike. The course or bearing of the outcrop of an inclined bed or structure on a level surface. (Glossary of Geology and Related Sciences)

Tear. See Strike-Slip Fault.

Tectonic. Of, pertaining to, or designating the rock structure and external forms resulting from the deformation of the earth's crust. As applied to earthquakes, it is used to describe shocks not due to volcanic action or to collapse of caverns or landslides. (Glossary of Geology and Related Sciences)

Transducer. Any device for transmitting energy from one form to another, such as mechanical to electrical, as performed by a seismometer.

Tsunami. Sea waves produced by a submarine earthquake or volcanic eruption that sometime attain heights of 30 ft or greater upon approaching coastlines.

Tsunami-Generated Oscillation. Resonant waves generated by tsunamis along coastlines, in bays, and in harbors.

Uprush. See Runup.

Variable-Density Cross Sections. Visual representations of subsurface structures obtained by a method of recording seismic impulses wherein the density of a photosensitive film or paper is proportional to the intensity of the seismic impulse.

References

1. *Glossary of Geology and Related Sciences*, 2nd ed., American Geological Institute, National Academy of Sciences, National Research Council, Washington, D.C., 1960.
2. L. R. Cluff, Active Fault Problems, in *Symposium on Earthquakes and the Practice of Soil and Geological Engineering, December 12-13, 1964*, pp. 43-78.
3. M. P. Billings, *Structural Geology*, 2nd ed., Prentice-Hall, New Jersey, 1954.
4. C. F. Richter, *Elementary Seismology*, W. H. Freeman and Company, San Francisco, 1958.
5. T. H. Thomas et al., Nuclear Engineering and Earthquakes, USAEC Report TID-7024, 1969.

Appendix B

REACTOR SITE EVALUATIONS

Brief reviews of the geologic and seismologic evaluations of five reactor sites (Malibu, Bodega Bay, San Onofre, H. B. Robinson No. 2, and Browns Ferry) are presented below to illustrate the principal effect of earthquake considerations on reactor siting. The sites are significant in the fact that they were subjected to evaluations that ranged from those requiring the most involved geologic and seismological investigation (Malibu, Bodega Bay) to those that required only the minimum effort (H. B. Robinson No. 2, Browns Ferry).

Bodega Bay

The Pacific Gas and Electric Company's proposed Bodega Bay plant site is situated on the California coast approximately 50 miles northwest of San Francisco. This installation was to consist initially of a single boiling-water reactor of 1008-Mw(th) and 325-Mw(e) capacity. It was to be supplied by the General Electric Company.¹

In general the principal problem at the Bodega Bay plant site was a determination of the nature and amount of expected faulting. However, there was some disagreement among the several investigators as to the maximum intensities of ground motion that might be experienced. The potential danger of tsunami waves at this coastal site was also controversial.

Throughout the course of the preliminary hazards evaluation, several investigators were employed by the applicant to evaluate the geologic and seismologic conditions at the site. In addition studies and comments as to the suitability of the site were made for the USAEC by the U.S. Geological Survey, the Bureau of Mines, the U.S. Coast and Geodetic Survey, and the Research Geologist of the National Park Service. A personal geologic and seismologic study of the site was also made by Pierre Saint-Amand for the Northern California Association to Preserve Bodega Head and Harbor. A preliminary reconnaissance of the site was first made for the applicant in 1958 by Clark E. McHuron, a Consulting Engineering Geologist. Later Dames and Moore, Soil Mechanics Engineers, were employed to do a

geophysical seismic survey of the site and to conduct preliminary subsurface exploration. Detailed geologic and seismologic studies were made by consultants Don Tocher and William Quaide, while George Housner of the California Institute of Technology recommended structural designs based on a seismological evaluation of the plant area. Hugo Benioff and E. C. Marliane also acted as consultants for the applicant. Robert Wiegel of the University of California, Berkeley, supplied information on tsunami hazards at the site.

The Bodega Bay Plant site is situated about 1000 ft west of the generally recognized western limit of the San Andreas fault zone (see Fig. B.1). The uncertainty in this case was not, of course, the activity of the main trace of the San Andreas fault, which is clearly recognized as active, but it was concerned with the probability of minor or subsidiary fracturing at the site resulting from the probable principal movement along the last break (1906) of the San Andreas fault located approximately one mile to the east. Even though it is recognized that in general the number of subsidiary or minor fractures and the amounts of displacement associated with them decrease away from the major breaks, too little is known about these breaks to be able to state precisely what movements might be expected at various distances from the major active faults. For the Bodega Bay site, the applicant's consultants argued that since the plant area was located outside the recognized zone of active faulting associated with the San Andreas fault and since no clear-cut evidence of recent faulting (within the last 40,000 years and possibly as long ago as 100,000 years) was found within the area, it would be extremely unlikely that any differential displacements of the ground would occur during the lifetime of the proposed plant.² On the other hand, the U.S. Geological Survey, acting as consultants for the Division of Reactor Licensing, contended that the probability of faulting beneath the plant was moderate to high, at least for displacements of 2 in. or less.² Their conclusions were based on the discovery of offsets in the bedrock and younger sediments in the shaft excavation for the reactor vessel (see Fig. B.2). These displacements, which may have affected sediments less than 40,000 years old, were interpreted as tectonic-induced disturbances associated with major movements on the main trace of the San Andreas fault, which lies

Fig. B.1. Excavation Pit for Reactor Building at Bodega Bay. The site lies more than 1000 ft west of western limit of the San Andreas fault zone, which strikes approximately parallel to the ridges in the background. Here the San Andreas fault zone extends eastward to the base of these ridges and occupies the lowland between the reactor site and the ridges in the background.

Fig. B.2. Closeup Photograph Showing Minor Offsets in Wall of Excavation for Reactor Building. This faulting led eventually to the abandonment of the Bodega Bay site.

to the east of the site. It is of interest that the applicant's consultants interpreted these same structures in the sediments to be sedimentary features (i.e., nontectonic).³

The U.S. Geological Survey also contended that even though the plant was to be located approximately 1000 ft beyond the generally recognized western limit of the San Andreas fault, not all faulting associated with major earthquakes along the fault is restricted to this zone. For instance it was pointed out that sympathetic faulting was observed to have occurred during the 1906 earthquake in bedrock at Point Reyes Peninsula, which is located about 30 miles to the south at a site that is geologically similar to the Bodega Bay area and is approximately the same distance from the San Andreas fault.⁴

Perhaps the most severe shaking at proposed reactor sites to date would be most likely to occur at the Bodega Bay power plant site. As mentioned above, this site is only about 1 mile from the 1906 break of the San Andreas fault and only about 30 miles from the epicenter of this shock. Based on the known history of seismic activity in the immediate vicinity of the site, small to moderate size earthquakes would not be expected to occur. It was thus generally concluded that the most intense ground motion at the site would occur as a result of a larger earthquake (magnitude 8.2) originating along the San Andreas fault. There were, however, considerable differences in the estimated maximum shaking intensities that might be expected at the site by various investigators. Originally, Tocher and Quaide suggested that the power plant, which was to have its foundation in quartz diorite bedrock, be designed to withstand an earthquake of Modified Mercalli intensity VIII. Furthermore, to provide some margin of safety, a maximum intensity of IX was recommended.¹ (Using Gutenberg and Richter's relationship between intensity and ground acceleration the intensities given above may be expressed as 0.15 and 0.32 g, respectively.) Due to gross misinterpretation of these expected earthquake intensities, Tocher later suggested that the most precise estimate of the maximum expected ground shaking at the site be made according to the response spectrum technique, which had been used by Housner for the site (the strong-motion accelerograms for the 1940 El Centro shock were scaled to apply to the Bodega Bay site), and not to intensity scales (Modified Mercalli).³

With consideration of the intensity of shaking associated with a magnitude 8.2 earthquake, the strongest so far recorded (El Centro, magnitude 7), and the differences in the expected intensities in alluvium (El Centro) and the hard rock at Bodega Bay, Housner concluded that the maximum shaking intensity at the site should be about 0.75% as great as that at El Centro (0.33 g). To allow a safe margin for error, a maximum ground acceleration for the site was estimated by Housner to be 0.33 g, or the same as that at El Centro.¹

Housner assumed that there is no appreciable difference in the intensity of shaking associated with a magnitude 7 earthquake and a magnitude 8.2 earthquake. He reasoned that the principal difference in these two shocks is an additional length of fault break in the case of the larger shock.^{1,5} This generalization was not agreed to by Saint-Amand and Eaton, who contended that peak accelerations as high as 1 g (Saint-Amand) and 0.7 g (Eaton) were more realistic. Saint-Amand pointed out that larger earthquakes are commonly observed to have greater intensities than smaller ones, while Eaton argued that the definition of magnitude itself suggests that the larger the earthquake the greater the shaking intensity.⁴

In regard to tsunamis, it was concluded by the applicant and his consultants that the maximum rise in water levels in Bodega Bay for all recorded tsunamis along the California Coast has not exceeded 5 ft. It was also mentioned that most of these waves originated in the Aleutians, Japan, Chile, etc., and in general destructive tsunami waves would not be expected to be generated along offshore California, since this area does not appear to be conducive to vertical faulting and submarine landslides, which are believed to be the principal means for generation of tsunamis. In summary, the applicant and his consultants contended that the maximum water level rise to be expected at the site as a result of tsunamis would be 15 ft. This is considerably less than the maximum wave heights of 30 ft for distant-generated tsunamis and 50 ft for locally generated waves that were suggested as appropriate for the site by the U.S. Coast and Geodetic Survey.² The 30-ft level was presumably based on experience with historic tsunamis along the California Coast, while the 50-ft value apparently stemmed from the reported 50-ft wave at Gaviota, California,

in 1812, which if it did indeed occur resulted from a local offshore earthquake.

Although there were major differences in the expected amounts of differential displacement, intensities of shaking and tsunami-induced water level rises, as expressed by the applicant's consultants and other interested parties, it should be mentioned that the more conservative evaluations were accepted by the applicant and designs were proposed to withstand these forces. Since there was doubt as to the safety of the proposed plant, the application for a license to construct the plant was withdrawn.⁶

Browns Ferry

The proposed site for the Browns Ferry Nuclear Power Station is on the shore of Wheeler Reservoir, approximately 30 miles west of Huntsville, Alabama. Initially the station will consist of two boiling-water reactor units, each having a gross thermal output of 3293 Mw (1098 Mw electrical).⁷

The site lies within the Highland Rim section of the Interior Low Plateaus. The rocks in this physiographic subdivision were subjected to some slight deformation during the Appalachian Revolution toward the close of the Paleozoic Era (225 million years ago); however, major folding and faulting did not occur within the region, as was the case in the Appalachian geosyncline to the east. Since there is apparently no evidence to indicate that any faulting has occurred within the region since the Appalachian Revolution, it was concluded that no provisions for differential displacement of the ground due to tectonic faulting need be made in the design of the plant.⁷

To determine the intensity of earthquake-induced shaking that might be expected at the plant site, the seismic histories of the Southeastern U.S. and the Tennessee River Valley, in particular, were reviewed. This study revealed, through epicentral plots of several hundred earthquakes, four principal areas of activity in the southeast. These areas were (1) the New Madrid region of the Mississippi River Valley, which includes parts of Missouri, Tennessee, Kentucky, and Arkansas, (2) the Lower Wabash Valley of southern Illinois and Indiana, (3) the Charleston, South Carolina, area, and (4) the Eastern Tennessee and Western North Carolina

regions of the Appalachians. Several light- to moderate-intensity shocks were observed to have occurred within 35 to 45 miles of the plant site; however, it was concluded that the greatest intensities of past shaking at the site probably resulted from the great New Madrid, Missouri, shocks of 1811 and 1812 (Modified Mercalli intensity of VII in nearby Decatur, Alabama) and the Charleston earthquake of 1886 (Modified Mercalli intensity of VI in the Decatur area). It was further concluded that because of the persistence of seismic events in the New Madrid area, it is possible that a great shock could originate in the future in that area that might result in an intensity as high as VII on the Modified Mercalli scale at the plant site. Based on the rough relationship between earthquake intensity I and acceleration a , as expressed by Gutenberg and Richter in the equation

$$I = 3 \log a + 1.5 ,$$

it was determined that the design earthquake at the plant site would be based on a ground acceleration of 0.10 g and that a ground acceleration of 0.20 g would be assumed for the maximum potential earthquake. Since the foundation for the structure is massive limestone, no amplification of ground motion is anticipated.⁷ The U.S. Geological Survey in their comments on the geology of the area agree with the applicant that the minor faulting at the site occurred several hundred million years ago, and there is no evidence of tectonic related movements along any faults since that time. The U.S. Coast and Geodetic Survey in their report on the seismicity of the plant site conclude, as did the applicant, that the greatest historic earthquake intensity (Modified Mercalli intensity of VII) felt at the site was due to the New Madrid series of shocks in 1811 and 1812. The U.S. Coast and Geodetic survey also agreed with the applicant that the facility might be subjected to ground motion of Modified Mercalli intensity VII and a corresponding ground acceleration of 0.10 g. A ground acceleration of 0.20 g was also suggested for the maximum potential earthquake at the site.*

*These two design levels correspond to the "operating basis earthquake" and "design-basis earthquake," respectively, in present Regulatory terminology. See Glossary, Appendix A.

San Onofre

The Southern California Edison Company's San Onofre Nuclear Generating Station is located on the Camp Pendleton Marine Reservation on the shoreline of the Pacific Ocean. The site of the 450-Mw(e) pressurized-water reactor is about 62 miles southeast of Los Angeles and 51 miles northwest of San Diego. The Preliminary Hazards Summary Report, which was submitted to the AEC Division of Licensing and Regulation in February 1963, discussed preliminary studies of geology, seismology, and oceanography that were carried out by Dames and Moore, Soil Mechanics Engineers.⁸

These preliminary studies revealed that the plant would be situated on the San Mateo formation of Middle Pliocene age. This formation, which consists of pebbly, coarse-grained sand is about 300 ft deep at the site. It was also determined that the San Mateo sand had sufficient strength to support the large structures required, and the offshore sediments could adequately support an intake structure for cooling water.⁸

In order to estimate the seismological hazard, seismic regionalization mapping⁹ was used to determine the probable maximum earthquake intensity and hence the design earthquake. Richter's relationships for seismic regionalization mapping are based on the premise that the shaking varies primarily with the age of the foundation material (that is, the density or bearing strength of the material) and secondarily with the location of active faults. Based on these relationships, the San Mateo formation of Tertiary age corresponds to an earthquake intensity of VII.⁸ In addition, the magnitude 6.25 earthquake of March 11, 1933, which was thought to be associated with strain release along the Newport-Inglewood fault about 30 miles north of the site, was determined to have an intensity of VIII at its epicenter. Using Gutenberg and Richter's relationships between intensity and acceleration,¹⁰ it was determined that an intensity of VIII would yield an acceleration of 0.15 g, and an intensity of VII would yield an acceleration of about 0.07 g. Furthermore, since Southern California Edison Company's coastal steam plants are designed to withstand an acceleration of 0.2 g, which corresponds to an intensity of VIII+, it was assumed that there would be an adequate margin of safety if the San Onofre reactor were designed to withstand an acceleration of 0.2 g.⁸

In order to estimate the tsunami hazard, it was noted that the maximum high tide at the site was +7.5 ft, while the maximum breaker height was 13 ft. Maximum tsunami heights at nearby La Jolla and San Diego were found not to have exceeded 5 ft. Based on these preliminary data, a 10-ft-high seal wall is proposed to assure a sufficient margin of safety at the site, which lies 20 ft above mean sea level.⁸

Upon review of the Preliminary Hazards Summary Report, several questions were posed regarding (1) bases for concluding there were no active faults at the site, (2) geological and seismological information upon which development of spectra were based, (3) method of analysis by which the spectra were developed, and (4) the tsunami hazard.¹¹ The only faults that could extend to the vicinity of the reactor site are the Christianitos fault and the Rose Canyon fault. The Christianitos fault trace intersects the coastline about one mile southeast of the site and trends due north from the point of intersection. As a result of examinations by Dames and Moore and consulting geologists, it was concluded that the fault had not moved in the last 10,000 years, since the fault had not displaced the terrace deposits along the shoreline.¹¹ Therefore, the Christianitos fault was considered to be inactive. With respect to the Rose Canyon fault, which is located north of San Diego, Dames and Moore and the geologists could find no field evidence of a major fault. In addition, they could find neither evidence of recent movement at Rose Canyon nor evidence to suggest a connection with a southern extension of the Newport-Inglewood fault.¹¹

Geological and seismological information needed to develop spectra included considerations of strength of surface rock and underlying formations, faults in the area, seismic history, epicenters of nearby shocks, and intensities of known earthquakes.¹² Studies showed that (1) the San Mateo formation has sufficient strength to support a reactor, (2) faults in the vicinity of the site are inactive, and (3) the site does not include earthquake epicenters.¹² However, Dames and Moore determined that the site is about four or five times as active as the average for Southern California based on the analysis of intensities to which the site has been subjected in the past.¹² For example, the average expectancy per 100 years of an intensity VI motion is 1.1, whereas there have been six intensity

VI motions at the site as a result of earthquakes whose epicenters were outside of the site area.¹¹ Based on these historical studies and studies of site characteristics, a "once-per-100-year" earthquake was considered as the design earthquake used to develop spectra. Such a shock would cause intensity VIII motions at the site that would correspond to an acceleration of 0.12 g.¹¹ In addition, a once-per-600-year earthquake was considered. Such a shock would cause intensity IX motions at the site that would correspond to an acceleration of 0.25 g.¹¹

In order to determine spectra, Dames and Moore used a method of analysis that was developed by Kanai.¹² This method consists of the following: (1) selection of a maximum earthquake magnitude and an epicentral distance from the site, (2) an estimate of the amplitudes of earthquake wave motion in basement rock at the site based on a formula developed from empirical studies, (3) calculation of the amplitude of wave motion of the ground surface by determining the effects of the shear-wave velocity of the surficial material, density of surficial material, and the number and thicknesses of the strata between the basement rock and the ground surface, and (4) measurement of frequency of occurrence of wave periods versus period for microearthquakes recorded at the site.¹²

In order to select the once-per-600-year earthquake, the following magnitude shock and faults at the indicated distances from the site were considered:

1. a magnitude 6.5 shock on the Elsinore (25 mi), San Clemente (50 mi), or Newport-Inglewood (25 mi) faults, or
2. a magnitude 7.5 shock on the San Jacinto (45 mi) fault, or
3. a magnitude 8 shock on the San Andreas (65 mi) fault no further south than Cajon Pass, since the historic record indicates no magnitude 8 shocks south of Cajon Pass.

It was concluded by Dames and Moore that the magnitude 8 and 6.5 shocks would be the more critical and would cause accelerations of 0.24 and 0.27 g, respectively. These accelerations would correspond to an intensity IX motion at the site, which would be equivalent to the once-per-600-year intensity.¹²

To select the once-per-100-year earthquake, which was believed to be more reasonable with regard to the lifetime of the plant, the following

magnitudes and faults were considered:

1. a magnitude 6.5 shock on the San Jacinto (45 mi) fault, and
2. a magnitude 6 shock on the Elsinore (25 mi) fault.

It was concluded that these shocks would result in accelerations of 0.1 to 0.13 g and would be equivalent to intensity VIII motions at the site.¹²

Based on the once-per-600-year earthquake, response accelerations vary from 0.25 to 0.62 g for a damping range of 0.5 to 7% at a period of 0.1 sec. The U.S. Coast and Geodetic Survey, on the other hand, concluded that 0.17 g should be considered as a safe design factor for earthquakes that would affect the site and that all important elements of the reactor should be designed to withstand 0.4 g and continue to function.¹³

In order to evaluate the tsunami hazard, Dames and Moore retained Marine Advisors, Inc., of La Jolla, California, to make extensive investigations of historic tsunamis, subsea faulting, and potential slide areas. Perhaps the biggest question to be resolved was that of the offshore California earthquake and resulting tsunami of December 21, 1812, which one source quoted by both the U.S. Coast and Geodetic Survey¹⁴ and Richter¹⁰ reported as attaining a height of 50 ft at Gaviota, 30 to 35 ft at Santa Barbara, and 15 ft at Ventura. However, extensive investigation of historic documents, mission records, private letters, and Dr. Louderback's private papers failed to reveal any reports of wave effects as a result of the tsunami of 1812 considered credible by Marine Advisors.¹⁵ As a result of these studies, it was concluded by Marine Advisors that the wave height may have been approximately that of the tidal range,¹⁵ which is about 6 ft.¹⁶

Also, Marine Advisors considered in detail tsunamis of remote origin, site topography, tsunamis of local origin, and potential landslide areas. Their conclusions were the following: (1) the superposition of a 50-year tsunami of 6 ft from a distant source, a maximum tidal stand of 7 ft, and a 50-year storm surge of 1 ft, amounting to 14 ft, would occur once every 25,000 years, (2) the area of the site is insensitive to tsunamis of remote origin because of the regular subsea topography, (3) there are no indications that the largest likely local tsunamis would exceed runups

attained by distant tsunamis from large sources, and (4) the potential for large submarine landslides is limited.¹¹

At a meeting on August 26, 1965, attended by the applicant and consultants, the AEC staff, after receiving the tsunami report from Marine Advisors, raised questions regarding (1) dip-slip faulting as a source of local offshore tsunamis, (2) the relative importance of warping of the sea bottom in regions adjacent to the San Andreas fault, (3) any writing of the Padre Gil that mentioned the tsunami of 1812, and (4) the height of the resulting tsunami using Iida's relationships if the magnitude 7.25 earthquake of November 4, 1927, was raised to magnitude 8.25. Hugo Benioff assisted in providing answers to these questions, as follows: (1) the offshore faults that exhibit dip-slip components are not considered long enough to produce earthquakes having magnitudes much larger than 7.3, such as recorded on November 4, 1927, and thus they are not considered likely to produce destructive tsunamis (see Chap. 5, which gives relationships between tsunami runup and earthquake magnitude); (2) there is no evidence of folding off the coast of California of sufficient magnitude or rate to be significant as a source of tsunamis; (3) Padre Gil mentioned the earthquake of 1812 but did not mention any sea effects, and the many second-hand accounts of sea disturbances were not found credible by applicant's consultant; and (4) Iida associates a maximum runup of 26 to 39 ft with an earthquake of magnitude 8.25; however, Iida's relationships do not appear to be applicable to California because of the more gentle offshore topography and the history of relatively low tsunamis.¹¹

Based on these answers, it was concluded by the applicant that the plant would not be subjected to tsunami wave heights exceeding 13 ft; however, a seawall having an elevation of 28 ft was to be provided.¹¹ The U.S. Coast and Geodetic Survey concurred that no flooding by combinations of the highest recorded tides, breakers, or tsunamis would occur if a seawall of this height were constructed.¹³

In order to provide information on accelerations at reactor sites, a strong-motion recorder, Model AR-240, was installed in the station Warehouse Building on the site in January 1965 by U.S. Coast and Geodetic Survey personnel, who will also maintain the instrument and collect and

analyze recorded data. The unit was set to record ground motions larger than the threshold acceleration of 0.1 g.¹¹

H. B. Robinson No. 2

Preliminary evaluations of the shaking to be expected at the Carolina Power and Light Company's H. G. Robinson No. 2 plant near Hartsville, South Carolina, were made by Dames and Moore.¹⁷ The site for the 760-Mw(e) plant is on silty clayey sand of Cretaceous age having a depth of about 400 ft that overlies crystalline rock. The evaluation program consisted of the following: (1) a complete study of all damaging shocks in the vicinity of the site and all significant damaging shocks within a radius of 200 miles, (2) an evaluation of the frequency of occurrence of earthquakes of various intensities, (3) a correlation of the geologic structure of the region with the historic pattern of earthquakes, (4) a review of the results of laboratory and field tests to determine the effects of earthquake motion on the foundation material, and (5) the selection of the necessary response spectra.¹⁷

Results of the seismological investigations showed that (1) the McBee shock of October 26, 1959, which had an intensity of V to VI 15 miles from the site, was the only earthquake that had occurred in the immediate vicinity of the site, and (2) the Charleston shock of 1886, which had an intensity of IX 120 miles from the site, probably had an intensity of about V at the site. Frequency of occurrence data led Dames and Moore to expect a shock on the order of the McBee shock, which had an intensity of V at the site, once during the life of the plant. Correlations of geologic structure of the region with earthquake patterns indicated that (1) the McBee earthquake of 1959 may have been associated with buried faulting along the local Deep River basin, and (2) that the Charleston earthquake may have been associated with a buried Triassic graben.¹⁷ Active faults are unknown in the vicinity of the site, and therefore no considerations for differential displacement were made.

Dynamic soil tests indicated that the sandy soils were more susceptible to compressional and shearing stresses than the clayey soils. Also, the sandy soils settled when subjected to an oscillating load of short duration.

Thus it was decided that all critical components and heavy structures would be supported by piles that, in turn, would be supported by stiff clayey soils at a depth of about 50 ft.

Because it was estimated that the site could expect a shock on the order of the McBee shock, which had an intensity of V to VI, once during the life of the plant, a magnitude 4.5 earthquake with an epicentral distance of less than 10 miles was selected as the design earthquake. A maximum ground acceleration of 0.07 to 0.09 g was estimated for this site, with some possible amplification due to soil conditions, since an intensity of about VII rather than V to VI would yield an acceleration of from 0.07 to 0.09 g based on the Gutenberg and Richter relationship (an intensity of 4.5 yields 0.01 g and an intensity of 7.5 yields 0.1 g).¹⁰ A value of 0.1 g was chosen as the design earthquake and a value of 0.2 g was chosen to give an adequate margin of safety for a hypothetical earthquake, such as the Charleston shock, occurring within 35 miles of the site. Because the Robinson site and certain areas in California are similar with regard to soil type and depth, Housner¹⁸ recommended that his average California response spectra be used to obtain horizontal and vertical ground motion spectra for the site.

The earth dam that impounds Lake Robinson, the source of cooling water, was also analyzed by Dames and Moore to determine its strength with respect to acceleration. Data showed that no displacement or yielding of the embankment would occur until the vector resultant of the horizontal and vertical components of earthquake accelerations equaled or exceeded 0.28 g. This strength is considered to be sufficient because the vector resultant of the horizontal and vertical components of the hypothetical earthquake of 0.2 g is only 0.24 g.¹⁷

Additional data to supplement original findings by Dames and Moore included (1) a review and comparison of the effects of the 1913 earthquake on Union, South Carolina, and the Imperial Valley earthquake of May 18, 1940, on El Centro, California, (2) another comparison of effects at Union and El Centro by Leeds, a seismologist with Dames and Moore, (3) a geological analysis of the site for the H. B. Robinson Unit No. 2 by Stuckey and Smith, consulting geologists, (4) a study of the earthquake hazard at the site of the H. B. Robinson Unit No. 2 by Byerly, consulting

seismologist, and (5) a study of the general geology and seismicity of South Carolina by the AEC.¹⁹

These studies provided the following additional information: (1) the intensity of the 1913 earthquake at Union, South Carolina, about 120 miles west of the H. B. Robinson site was about VI rather than VIII as stated by Taber in a report written in 1912,¹⁹ and thus the intensity at the H. B. Robinson site would have been much less than VI; (2) the soil and geologic conditions at the H. B. Robinson site are better than soil and geologic conditions at El Centro, California, and thus the El Centro spectrum may not be applicable to the H. B. Robinson site; (3) the intensity of the Charleston earthquake at the H. B. Robinson site was VI or VII and not V as formerly thought; however, Dames and Moore originally chose a design earthquake of 0.1 g that corresponds to an intensity of 7.5 according to Richter's relationship; (4) the Charleston earthquake of 1886, which had an intensity of VI to VII at the H. B. Robinson site, should be used as the design earthquake rather than the nearby McBee earthquake, which had an intensity of about V; and (5) a maximum intensity of VI or VII as a result of an earthquake similar in intensity to the 1886 earthquake at Charleston is not expected within the next 100 years.¹⁹ Thus, the design earthquake of 0.1 g and the hypothetical earthquake of 0.2 g as originally chosen by Dames and Moore should provide ample margins of safety for the H. B. Robinson Unit No. 2 during the plant's expected lifetime of 40 years.¹⁹

Malibu

The proposed site of the Malibu Nuclear Reactor Plant is in Corral Canyon about 500 ft from the shoreline of the Pacific Ocean. At this site, which is about 30 miles west of Los Angeles, the Department of Water and Power of the City of Los Angeles proposes to build a 500-Mw(e) pressurized-water reactor. The Preliminary Hazards Summary Report,²⁰ a joint report prepared by the Department of Water and Power of the City of Los Angeles, Westinghouse Electric Corporation, and Stone and Webster Corporation, was submitted to the AEC Division of Licensing and Regulation in November 1963. Preliminary studies of geology and seismicity discussed

in the Preliminary Hazards Report were performed by Converse Foundation Engineers²¹ and G. W. Housner,²² respectively.

Even at the time of the submission of the Preliminary Hazards Report, considerable controversy had arisen regarding a zoning hearing on the Department of Water and Power's application to build a reactor at Malibu.²³ Also, persons who owned property in the Malibu area joined the general public of the Malibu area in objecting to the proposed nuclear reactor.²⁴ The main objection was that a fault was known to run through the site, and therefore there was general fear that differential displacement as a result of an earthquake on this fault would destroy the reactor and lethal radioactivity would escape into the environment.²³

Preliminary geological studies conducted by Converse Foundation Engineers and Schoellhamer of the U.S. Geological Survey were based on test borings and visual surface mapping.²⁰ At the site, Quaternary terrace deposits consisting of clay, sand, cobbles, and boulders overlay the Monterey formation, which constitutes the foundation rock. The Monterey formation is of Tertiary age and consists of massive siltstone and shale. Although it has been highly fractured, it is thought to have sufficient bearing strength for the reactor.²⁰ Subtle local variations and reversals of bedding are thought to be due to local minor faulting and the proximity of the Malibu Coast fault, which forms the northern boundary of the site, about 1500 ft inland from the Pacific. Borings indicated that the Monterey formation was highly fractured and sheared. It was thought that this deformation was due to either landsliding or earthquake activity, since landslides have occurred in Corral Canyon and several small earthquakes have occurred in the Pacific Ocean to the west of the site, according to Richter.²¹

Preliminary seismological studies were performed by Housner in order to recommend the design earthquake. For such a recommendation, Housner considered the general seismicity of the site, the magnitudes of past earthquakes associated with known faults in the general area, and the influence of local geology. The resulting information indicated that (1) the Malibu site is within a well-known seismic region that has a long earthquake history, (2) the site would be shaken strongly either by a magnitude 8+ shock on the San Andreas fault or magnitude 5.5 to 6.5 shocks on the

Inglewood fault (15 mi) and possibly even closer to the site, and (3) anticipated ground accelerations would be less than for a site on alluvium, and no permanent displacements of the bedrock (Monterey formation) would be expected.²² Based on the above factors and the uncertainties involved in predicting the occurrences of earthquakes, Housner recommended a design earthquake of 0.3 g. Acting as consultants for the AEC, the U.S. Coast and Geodetic Survey believed that an acceleration of 0.23 g at periods from 0.3 to 0.6 sec should be expected from an earthquake on the San Andreas fault and that an acceleration as high as 0.3 g in the same period range could occur and should be taken into account in the design.²⁵

After the application was filed, several questions arose regarding earth-science aspects, such as maximum tsunami height and criteria for protection against landslides.²⁶ In regard to maximum tsunami height, there was a wide difference of opinion between Marine Advisors,²⁶ consultants on oceanography for the applicant, Saint-Amand,²⁷ and the U.S. Coast and Geodetic Survey.²⁵ Since the Malibu reactor plant was to be built at an elevation of about 38 ft above mean lower low water, Marine Advisors thought the site would be adequately protected from the maximum local tsunami of 10 ft (see tsunami discussion above in section on San Onofre) and the maximum distant tsunami of from 2 to 9 ft.²⁶ Saint-Amand²⁷ believed that local tsunamis might easily reach 30 ft above sea level, and thus the sensitive parts of the plant should be located at elevations above 30 ft. The U.S. Coast and Geodetic Survey²⁵ stated that the site should be protected to a vertical height of 50 ft above mean lower low water for tsunamis from severe, local, marine earthquakes and the site should be protected to a vertical height of 30 ft for tsunamis from distant sources. Because inundation to a height of 50 ft above sea level would not impair the containment structure,²⁶ no provisions were made for a sea wall. In order to reduce the landslide hazard, the applicant planned to remove potentially harmful slumps or slides and to carefully grade and develop the remainder of the site to prevent any further sliding and direct damage to the plant. Suitable preparation such as that described above and construction of closed conduit and open drainage channels through the plant area were thought to be sufficient to reduce the possibility of flood damage induced by a landslide.²⁶

In addition to the application for a construction permit,²⁰ eight amendments were submitted to the Regulatory Staff and the Advisory Committee on Reactor Safeguards by January 8, 1965. After review of the application and the amendments, both the Regulatory Staff and the Advisory Committee concluded that there was reasonable assurance that the reactor could be constructed and operated at the proposed site without undue risk to the health and safety of the public.²⁸

On February 9, 1965, a Notice of Hearing was published in the Federal Register and a prehearing conference was held on February 26, 1965. This conference was followed by 41 days of hearings that began March 23, 1965. Afterward, a posthearing conference was held on May 19, 1966. In these proceedings, Marblehead Land Company, the Malibu citizens for Conservation, Inc., and Lester T. (Bob) Hope intervened and opposed construction of the facility. Also the County of Los Angeles intervened and opposed construction of the proposed reactor, and the State of California introduced two geological witnesses but did not oppose the facility. In addition to the intervenors, there were 51 limited appearances opposing the application and 11 limited appearances favoring the application.²⁸

Although the board determined that the applicant's design criteria for withstanding earthquake shaking met requirements, much of the discussion concerned faults in the general area of the site, and especially a fault that was found to lay directly under the site of the proposed reactor. This fault was uncovered as a result of trenching carried out during the hearings.

Faults at the site were thought to be related to the Malibu Coast fault zone that is included in the Santa Monica fault system and which is considered to be tectonically active at depth.²⁹ Moreover the intensely deformed bedrock at the plant site was considered by the U.S. Geological Survey as a possible locus for future displacement along the Malibu Coast fault zone.²⁸

Although it was established to the AEC Atomic Safety and Licensing Board's satisfaction that the last known displacement at the reactor site most probably occurred more than 10,000 years ago and possibly as long as 180,000 years ago, and even though the applicant submitted arguments based on recurrence theory to the effect that no differential displacement

as a result of an earthquake would take place during the lifetime of the reactor, and the U.S. Geological Survey agreed that the probability of differential displacement at the site during the next 50 years was very low, the Board thought that the probability of differential displacement at the site during the next 50 years was high and could not dismiss the possibility of some risk in regard to differential displacement beneath the proposed site of the reactor.^{28,30} Therefore, the Department of Water and Power of the City of Los Angeles was authorized in the initial decision of the board²⁹ to construct the facility on the condition that the design criteria be modified and be supplemented to include adequate provision for permanent ground displacement at the reactor site from earthquake activity and that resulting modifications and supplements to design criteria should be made available for review by the Commission.²⁹ Also the Director of the Division of Reactor Licensing was directed by the Board to issue a provisional construction permit with the specified conditions that the reactor be designed and modified to accommodate differential displacement of the foundation material. In contrast the Regulatory Staff of the AEC believes that the initial decision²⁹ should be modified and a provisional construction permit should be issued to the applicant without the conditions requiring design against ground rupture.³⁰

The Commission upheld the determination of the Board that the design criteria must be modified and supplemented to include adequate provisions for ground displacement from earthquake activity.²⁸ This was disputed both by the Staff and the intervenors because there were no design criteria in the regulations regarding differential displacement. There were, however, requirements that the applicant "describe the proposed design of the facility, including, but not limited to, the principal architectural and engineering criteria for the design."²⁸ Because the Commission believes it is clear that the criteria for differential displacement should be included in the principal architectural and engineering criteria for the design, such criteria should be appropriately described by the applicant and should be reviewed before a construction permit may be issued. As a result, the Commission has ordered that (1) the order of the Board authorizing issuance of a provisional construction permit be set aside, (2) the applicant must determine the amount of differential ground

displacement the facility must be designed to withstand and must determine the adequacy of the design criteria, (3) the applicant must submit the data on differential displacement and design criteria to the board, and (4) the board must conduct further proceedings to determine the amount of differential displacement and the adequacy of the design criteria after the amendment is reviewed by the Regulatory Staff and the Advisory Committee on Reactor Safeguards.²⁸ As far as is known at this time, the applicant is proceeding to comply with the Commission's requirements.

References

1. Pacific Gas and Electric Company, Bodega Bay Atomic Park Unit Number 1, Preliminary Hazards Summary Report, USAEC Report NP-12476, Docket 50-205, Dec. 28, 1962.
2. *Atomic Energy Clearing House*, Vol. 10, No. 44, Nov. 2, 1964.
3. Pacific Gas and Electric Company, Bodega Bay Atomic Park, Amendment No. 5, Geologic and Seismic Investigation of the Site for a Nuclear Electric Power Plant on Bodega Head, California, Docket 50-205, January 1964.
4. J. Schlocker et al., Geologic and Seismic Investigations of a Proposed Nuclear Power Plant Site on Bodega Head, Sonoma County, California, Part I, Geologic Investigations, and Part II, Seismic Hazards Evaluations, Report TEI-837, Docket 50-205, September 1963.
5. Pacific Gas and Electric Company, Bodega Bay Atomic Park, Amendment No. 9, Docket 50-205.
6. Pacific Gas and Electric Company, Notice of Withdrawal of Application for Utilization Facility License, Docket 50-205, Dec. 3, 1964.
7. Tennessee Valley Authority, Browns Ferry Nuclear Power Station Design and Analysis Report, Vol. I, Dockets 50-259 and 50-260, July 7, 1966.
8. Southern California Edison Company, Preliminary Hazards Summary Report, Nuclear Station at Camp Pendleton, California, Unit No. I, Docket 50-206, 1963.
9. C. F. Richter, Seismic Regionalization, *Bull. Seismol. Soc. Amer.*, 49(2): 123-162 (1967).

10. C. F. Richter, *Elementary Seismology*, W. H. Freeman and Company, San Francisco, 1958.
11. Southern California Edison Company and San Diego Gas and Electric Company, San Onofre Nuclear Generating Station, Unit 1: Final Engineering Report and Safety Analysis, Vol. 1, Pt. 2, Docket 50-206, 1965.
12. Southern California Edison Company, San Diego Gas and Electric Company, Bechtel Corporation and Westinghouse Electric Corporation, 1963, Amendment No. 3 to Application for Construction Permit and for License, San Onofre Nuclear Generating Station Unit No. 1, Docket 50-206.
13. L. Murphy, Report on the Seismicity of San Onofre, California, Area, U.S. Coast and Geodetic Survey, report submitted to AEC, 1963.
14. U.S. Coast and Geodetic Survey, Report on the Seismicity of the Malibu, California, Area, report submitted to AEC, Docket 50-214, 1964.
15. Marine Advisors, Inc., Examination of Tsunami Potential at the San Onofre Nuclear Generating Station, unpublished report to Southern California Edison Company, Docket 50-214, 1965.
16. R. L. Wiegel, *Oceanographic Engineering*, p. 532, Prentice-Hall, New Jersey, 1964.
17. Carolina Power and Light Company, H. B. Robinson Unit No. 2, Preliminary Facility Description and Safety Analysis Report, Vol. 1, Pt. 2, Docket 50-261, 1966.
18. G. W. Housner, Recommended Earthquake Design Spectrum in H. B. Robinson Unit No. 2, Preliminary Description and Safety Analysis Report, Vol. 3, App. B, Docket 50-261.
19. Carolina Power and Light Company, H. B. Robinson Unit No. 2: Fifth Supplement to Preliminary Facility Description and Safety Analysis Report, Docket 50-261, 1966.
20. Department of Water and Power, City of Los Angeles, Preliminary Hazards Summary Report, Malibu Nuclear Plant Unit No. 1, Docket 50-215, 1963.
21. Converse Foundation Engineers, Preliminary Subsurface Investigations, in Department of Water and Power, City of Los Angeles, Preliminary

- Hazards Summary Report, Malibu Nuclear Plant Unit No. 1, Docket 50-214, 1963.
22. G. W. Housner, Seismicity of Malibu Site, in Department of Water and Power, City of Los Angeles, Preliminary Hazards Summary Report, Malibu Nuclear Plant Unit No. 1, Docket 50-214, 1963.
 23. Los Angeles Times, Notice of a Zoning Hearing on the L. A. Dept. of Water and Power Application to Build a Nuclear Reactor Plant in Malibu, p. 2, Oct. 31, 1963.
 24. Los Angeles Times, Malibu Residents Tell A-Power Plant Fears, p. 18, Nov. 5, 1963.
 25. United States Coast and Geodetic Survey, Report on the Seismicity of the Malibu, California, Area, in Hazards Analysis, Malibu Nuclear Plant Unit No. 1, App. D, Docket 50-214.
 26. Department of Water and Power, City of Los Angeles, Fifth Amendment to Application for Construction Permit and Facility License for Malibu Nuclear Plant Unit No. 1, Docket 50-214, 1964.
 27. P. Saint-Amand, Notes on the Seismology and Geology of the Malibu Reactor Site, Statement to the AEC, Docket 50-214, 1965.
 28. Order for Further Proceedings, *Atomic Energy Clearing House*, 13(14): 4-11 (Apr. 3, 1967).
 29. Atomic Energy Commission, Atomic Safety and Licensing Board, Initial Decision in the Matter of Department of Water and Power of the City of Los Angeles, Malibu Nuclear Plant Unit No. 1, Docket 50-214, 1966.
 30. Exceptions to an Initial Decision, *Atomic Energy Clearing House*, 12(36): 20-22 (1966).

THIS PAGE
WAS INTENTIONALLY
LEFT BLANK

APPENDIX C. SOME IMPORTANT EARTHQUAKES

Table C.1. List of Some Important Earthquakes

Year	Date	Name and/or Locality	Richter Magnitude	Modified Mercalli Intensity	Remarks
1755	Nov. 1	Lisbon, Portugal	8.5-8.75		Spectacular effects on ground and surface water; 60,000 deaths
1811-1812	Dec. 16, Jan. 23, and Feb. 7	New Madrid, Missouri		XII	Greatest series of earthquakes in U.S. history; extensive changes in surface features
1812	Dec. 21	Off Coast of Southern California		X	Associated with largest tsunami ever reported to originate in U.S. (50 ft at Gaviota)
1857	Jan. 9	Southern California		X-XI	200-mile length of surface ruptured along San Andreas fault
1872	Mar. 26	Owens Valley, California	8.3	X-XI	Extensive secondary faulting observed away from main fault scarp
1886	Aug. 31	Charleston, South Carolina		X	Felt over area of 2,000,000 square miles
1897	June 12	Assam, India	8.7		Maximum acceleration may have exceeded gravity
1899	Sept. 10	Yukatat Bay, Alaska	8.5-8.6	XI	Caused extensive topographic changes
1906	Apr. 18	San Francisco, California	8.3	XI	Classic example of surface faulting along 250 miles of San Andreas fault
1915	Oct. 2	Pleasant Valley, Nevada	7.6	X	Vertical faulting at surface along 22-mile length
1923	Sept. 1	Tokyo, Yokohama, Japan	8.3		Great change in depth of floor of Sagami Bay
1925	June 29	Santa Barbara, California	6.3	VIII-IX	Much property damage
1932	Dec. 20	Cedar Mountain, Nevada	7.3	X	Extensive and complicated faulting accompanied the earthquake
1933	Mar. 10	Long Beach, California	6.3	IX	Much destruction in proportion to size of earthquake
1934	Jan. 30	Excelsior Mountains, Nevada	6.5	VIII-IX	Vertical fault movement occurred
1940	May 18	El Centro, California	7.1	X	Horizontal displacement of 15 ft occurred along San Andreas fault
1947	Apr. 10	Manix, California	6.4	VII	Faulting not aligned with epicenters of earthquake and after-shocks
1949	Apr. 13	Olympia, Washington	7	VIII	Heavy property damage over wide area but no visible surface faulting
1950	Dec. 14	Fort Sage, California	5.6	VII	Surface fault displacement associated with rather small earthquake
1952	July 21	Taft, California	7.7	XI	Largest earthquake in U.S. (exclusive of Alaska) since 1906; rupture occurred along a relatively small fault

Table C.1 (continued)

Year	Date	Name and/or Locality	Richter Magnitude	Modified Mercalli Intensity	Remarks
1954	July 6	Rainbow Mountain, Nevada	6.6	IX	Surface fault movement occurred
1954	Aug. 23	Rainbow Mountain, Nevada	6.8	IX	Vertical fault displacement occurred
1954	Dec. 16	Fairview Peak, Nevada	7.1	X	Extensive faulting associated with the shock
1954	Dec. 21	Eureka, California	6.6	VII	Relatively large earthquake without visible surface faulting
1957		Mexico City	7.5		Many tall buildings suffered damage from combination of incoming long-period waves and long natural period of vibration of soil
1957	Mar. 22	San Francisco, California	5.3	VII	Flow slides and other soil failures
1958	July 9	Fairweather, Alaska	7.9	XI	Massive rockslide caused water to surge 1740 ft in Lituya Bay
1959	Aug. 17	Hebgen Lake, Montana	7.1	X	Destruction of ranch structures located astride fault that ruptured at surface; great slides
1960	Feb. 29	Agadir, Morocco	5.8		Great amount of structural damage from very shallow earthquake less than 2 miles deep
1964	Mar. 28	Prince William Sound, Alaska	8.4-8.6		Widespread soil failures and tsunami effects
1964	June 16	Niigata, Japan	7.3		Classic example of soil liquefaction
1966	March	Imperial, California	3.6		Surface faulting associated with unusually small earthquake
1966	June 27	Parkfield, California	5.5		Surface rupture associated with relatively small earthquake
1967	July 29	Caracas, Venezuela	6.6		Amplification of ground motions probably occurred in parts of Caracas
1968	Apr. 8	Borrego Mountain, California	6.6		Fault movements observed along nearby faults not associated with earthquakes