231237

UCRL-MA-125085

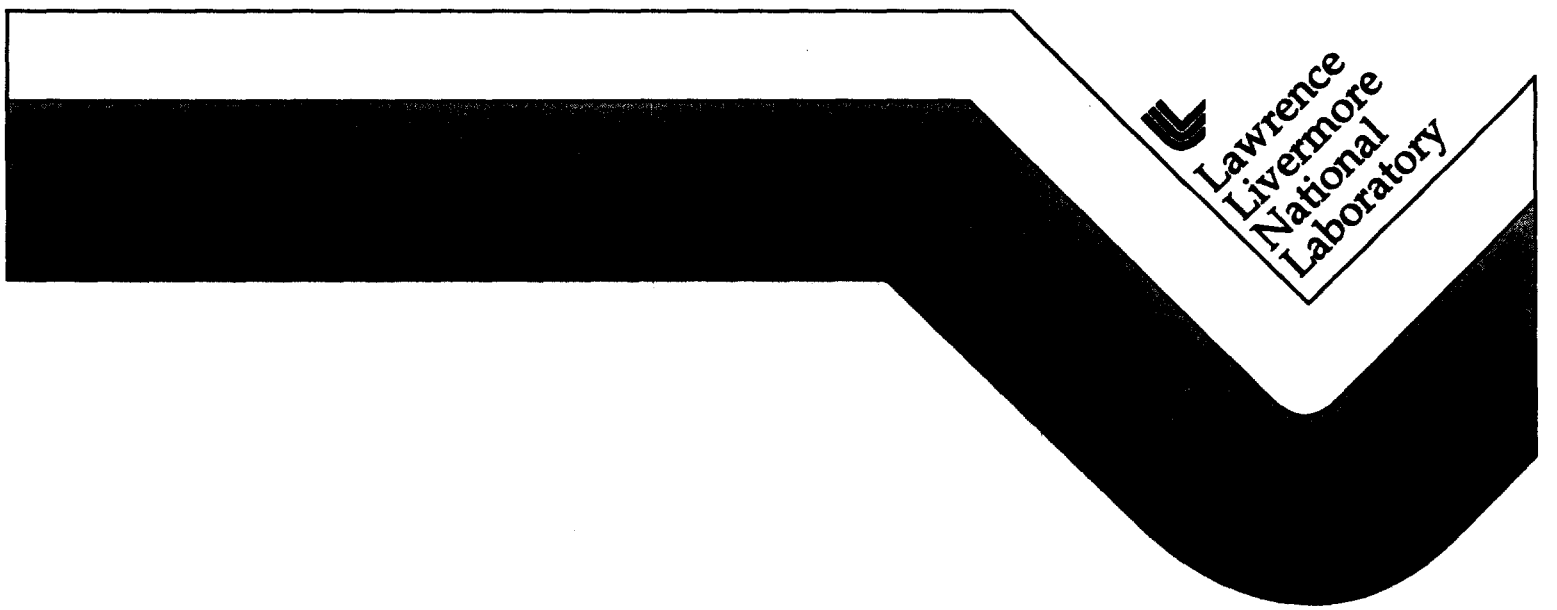
Seismic Safety Manual
A Practical Guide for Facility Managers and Earthquake Engineers

Compiled and Edited by
Donald G. Eagling

A contribution to the
International Decade of Earthquake Hazards Reduction

Prepared for the U.S. Department of Energy
Office of Environment, Safety and Health
Office of Nuclear and Facility Safety

September 1996



DISCLAIMER

This document was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor the University of California 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 the University of California. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or the University of California, and shall not be used for advertising or product endorsement purposes.

This report has been reproduced directly from the best available copy.

Available to DOE and DOE contractors from the Office of Scientific and Technical Information P.O. Box 62, Oak Ridge, TN 37831 Prices available from (615) 576-8401, FTS 626-8401

Available to the public from the National Technical Information Service U.S. Department of Commerce 5285 Port Royal Rd., Springfield, VA 22161

Work performed under the auspices of the U.S. Department of Energy by Lawrence Livermore National Laboratory under Contract W-7405-Eng-48.

SEISMIC SAFETY MANUAL

A practical guide for Facility Managers and Earthquake Engineers

Compiled and Edited by
Donald G. Eagling
Facilities Manager (Retired)
Lawrence Berkeley National Laboratory

Contributing Authors

Donald G. Eagling

Jack R. Benjamin
John J. Earle
John M. Eidinger
Eric Elsesser
Harold M. Engle, Jr.
Ronald P. Gallagher
Terence P. Haney
James R. Hill
James J. Johnson
Frank E. McClure
Robert C. Murray
Maurice S. Power
Daniel Shapiro
Roland L. Sharpe
James L. Stratta
Karl V. Steinbrugge
Sam W. Swan
Douglas W. Thornburg
Diana R. Todd
Peter I. Yanev

Lawrence Livermore National Laboratory
University of California
Livermore, California 94550

September 1996

Prepared for the U. S. Department of Energy
Office of Environment, Safety and Health
Office of Nuclear and Facility Safety

SEISMIC SAFETY MANUAL

Table of Contents

Preface — **Public Policy and Acceptable Risk** vii
Karl V. Steinbrugge

Foreword — **Contributors and Reviewers** x
Donald G. Eagling

Chapter 1 — **Introduction** 1-1
Donald G. Eagling
Motivation for the Seismic Safety Manual, its objectives, format, and intended audience.

Chapter 2 — **The Facility Manager's Role in Earthquake Safety** 2-1
Donald G. Eagling
Practical advice for implementing an effective, comprehensive seismic safety program.

Chapter 3 **Foreword** — **Earthquake Damage**
The Facility Manager's Perspective 3-1
Donald G. Eagling
Lessons from Loma Prieta (1989), Northridge (1994), and Kobe (1995). A solution to brittle moment-resisting steel frame joints.

Chapter 3a — **Types of Damage** 3a-1
James L. Stratta
An overview of earthquake damage; effects on buildings, contents, operations and lifelines.

Chapter 3b — **Assessment of Damage** 3b-1
Ronald P. Gallagher
How to organize a damage-assessment program and implement its essential components.

Chapter 4 **Foreword** — **Seismic Design and Evaluation Requirements for DOE Facilities**
The Facility Manager's Perspective 4-1
Donald G. Eagling
An overview of the philosophy, concepts, and criteria underlying DOE's seismic hazards mitigation program.

Chapter 4a — **Guidelines for Using DOE Orders and Standards for Earthquake Safety** 4a-1
Frank E. McClure
A *road map* to presidential Executive Orders, DOE regulations, and technical documents affecting seismic resistant design.

Chapter 4b **DOE Natural Phenomena Hazards Mitigation Requirements and Standards** **4b-1**
James R. Hill
Robert C. Murray
A summary explanation of DOE criteria for a graded approach to seismic safety.

Chapter 5 Foreword **Site-Use Planning for Earthquake Safety**
The Facility Manager's Perspective **5-1**
Donald G. Eagling
Philosophy for site investigation including risk management, scope, budget, and schedule priorities.

Chapter 5a **Site Evaluation** **5a-1**
Maurice S. Power
An overview of earthquakes and seismic hazards coupled with guidance for evaluating ground shaking and other seismic effects on site facilities.

Chapter 6 Foreword **Seismic Design of New Facilities** **6-1**
The Facility Manager's Perspective
Donald G. Eagling
Practical advice to ensure that new structures are properly and economically designed and constructed for earthquakes.

Chapter 6a **Conventional Analysis and Design** **6a-1**
Roland L. Sharpe
Analysis and design procedures for new buildings and lifelines based on building code provisions coupled with an overview of earthquake basics, construction materials, structural systems, and design criteria.

Chapter 6b **Dynamic Analysis Applications** **6b-1**
Roland L. Sharpe
A comparison of seismic analysis procedures, and a summary explanation of dynamic analysis and mathematical modeling including software programs and their major features.

Chapter 6c **Soil-Structure Interaction** **6c-1**
James J. Johnson
The soil-structure interaction phenomenon, its elements and effects on structures, and procedures for analysis.

Chapter 6d **Seismic Isolation and Passive Energy-Dissipation Systems** **6d-1**
Eric Elsesser
Roland L. Sharpe
Seismic isolation concepts, and supplemental damping and passive energy dissipation systems including pros and cons, critical issues, analysis procedures, and code requirements.

Preface

Public Policy and Acceptable Risk

Karl V. Steinbrugge

It has been over twelve years since the first edition of the Seismic Safety Guide was published in 1983. Since then, a significant number of destructive earthquakes have taken place where structures were designed to resist them and where extensive strong motion data were available for correlation with damage. New methods of analysis combined with marked advances in computer technology have provided seismologists, geotechnical and structural engineers with improved tools for investigation and evaluation. Consequently, a wealth of new information has become available to architects, engineers, building officials, emergency response organizations and the general public with respect to seismic safety. Much which has been learned has been translated into building codes, state and federal regulations, emergency plans and related reference documents.

During this period has come an increased emphasis on probability based estimates for hazard assessments and more specifically to set earthquake design criteria, or in other words, to specify the strength a building must have to resist earthquakes. This development has brought with it a graded approach for setting seismic performance standards appropriate to the degree of hazard and the consequences of failure. Although probability provides a logical and practical methodology for setting performance standards in keeping with the calculated risk hazards represent, it is important to understand that probability cannot accurately predict future earthquake forces. It merely sets a standard for the strength that structures should probably have.

Too often recent history has shown that actual earthquakes are apt to bring exceptions to the rules of probability. Most professionals who have contributed to the development of probability based criteria recognize its limitations, but they know these limitations can be overcome by careful detailing of the structural design to ensure that overloads will not result in catastrophic failure. They are also aware that probability estimates should not be used as an excuse to duck responsibility.

The flood of recent information and the proliferation of new, more sophisticated regulations have inundated the earthquake engineering field during the last decade and the potential for change in the aftermath of the Northridge, California earthquake is great. It's a challenge for practicing engineers to keep abreast of new developments. For those facility managers who are not in the field of earthquake engineering, the task is intimidating. Consequently, a primary objective of the 1996 edition of the *Seismic Safety Manual* is to incorporate and interpret these new developments so that operators and managers of DOE sites can achieve a comfortable level of understanding for the decisions they must make relative to seismic risk management.

A corollary is the question of acceptable risk which, to one degree or another, may be the responsibility of the facility manager. A manager may not fully understand the technical terms which may be used by a seismic design team. Assuming that risk responsibility has been discussed, did all parties understand that social, political, and economic risks were also at stake? In these increasingly litigious times, this subject should be discussed in detail by all parties involved in the design, including the facility manager, and the implications understood. *Risk avoidance* in discussions leaves a residue of *acceptable risk*; will this residue be acceptable after the event?

The U.S. Department of Energy (DOE) has made a great deal of progress with its seismic

safety program during the last twenty years. Damaging earthquakes during the last decade have fueled even more rapid progress, particularly in the western United States.

Generally, progress has been greatest on the front end of the process, i.e., site studies, criteria development, and the design of new facilities. Progress with the retrofit of existing unsafe structures has been slower (with notable exceptions) due to the perceived priority of front end activities.

Improvements in seismic design practices imply a lesser standard of safety for some existing structures and equipment. Retrofit standards applicable to conventional commercial and industrial structures normally do not bring these structures to the equivalent of current standards. Subsequent earthquakes have required upgrading retrofit standards. The problem of *acceptable risk* is again faced by facility managers.

The lessons learned from the Loma Prieta, California earthquake (1989), the Northridge earthquake (1994), and the Hyogo-Ken Nanbu, Japan earthquake (Kobe 1995), have clearly shown that the most serious seismic hazards (and the simplest to diagnose) lie in the details of existing poor construction. Does the building have an elementary lateral force resisting system? Are there missing or brittle links in the system? Does the needed cross-brace actually exist? Is the rod bracing tight? Are there nonstructural elements which are hazardous to life safety or to the integrity of the building structure per se? These are questions of highest priority.

A caveat applies to design. Seismic design, including the mathematics which supports the design, may be viewed as the *concept*; the construction drawings, specifications, and field inspections are the *implementation*. The articulation of the concepts via drawings, specifications, and field inspections must be clear and simple for workers who implement them under non-office conditions. For example, a construction drawing showing a *section of a section* continued on several drawings has been the cause of misunderstanding and failure.

It is vital that existing facilities be evaluated as early as possible in the overall priority of needs. It is not necessary to complete other *front end* elements of the program in advance. With this admonition in mind it is worthwhile to again read the preface to the 1983 edition of the *Seismic Safety Guide* which is reprinted hereafter. I believe it is as valid for 1996 as it was in 1983.

Karl V. Steinbrugge
March 1996

Chapter 7 Foreword **Evaluation of Existing Buildings** **7-1**
The Facility Manager's Perspective
Donald G. Eagling
Practical advice for reviewing existing buildings and descriptions of potentially hazardous building types.

Chapter 7a **Evaluation Guidelines** **7a-1**
Frank E. McClure
Evaluation requirements for DOE facilities, ATC evaluation techniques, and recommended evaluation procedures.

Chapter 7b **Evaluation Methodologies** **7b-1**
Harold M. Engle, Jr.
Brief descriptions of selected building types, their earthquake resistance, and recommendations for investigating them.

Chapter 8 Foreword **Seismic Retrofit of Buildings**
The Facility Manager's Perspective **8-1**
Donald G. Eagling
Practical advice for the retrofit of hazardous buildings including examples of retrofit projects at DOE sites.

Chapter 8a **Retrofit Guidelines** **8a-1**
Frank P. McClure
Recommendations for seismic retrofit design for existing structural systems, nonstructural elements, and life-support systems.

Chapter 8b **Retrofit Methodologies** **8b-1**
John J. Earle
Daniel Shapiro
Rehabilitation techniques for various types of structural systems and construction materials to graded levels of resistance.

Chapter 8c **Seismic Isolation and Passive Energy Dissipation for Building Retrofit** **8c-1**
Eric Elsesser
Applications for the protection of existing buildings including limitations and economic factors.

Chapter 9 Foreword **Seismic Evaluation and Design Considerations for Operations and Contents** **9-1**
The Facility Manager's Perspective
Donald G. Eagling
Practical advice for implementing a program for operational seismic safety.

Chapter 9a **Overview** **9a-1**
James L. Stratta
An overview of the effects of earthquakes on contents, building systems, and occupants including practical means of mitigation.

Chapter 9b **Evaluation of Nonstructural Hazards** **9b-1**
Ronald P. Gallagher
Seismic effects on specific architectural elements, mechanical and electrical equipment, and various building contents including vulnerability surveys for existing construction and preventing nonstructural hazards in new construction.

Chapter 9c **Data-Processing Facilities and Systems** **9c-1**
Roland L. Sharpe
Seismic vulnerability and protection of data-processing facilities and support systems including computer processing, telecommunications, and other electronic facilities.

Chapter 9d **Seismic Experience Data and Equipment Damage** **9d-1**
Sam W. Swan
Peter I. Yanev
The use of seismic experience performance data for effective walkdown surveys of equipment installations to detect and correct earthquake vulnerabilities.

Chapter 10 Foreword **Seismic Design of Concrete Shielding Block Assemblies** **10-1**
The Facility Manager's Perspective
Donald G. Eagling
An overview of the unique seismic hazards posed by massive concrete blocks used for shielding radiation.

Chapter 10a **Planning and Design Methodologies for Blocks** **10a-1**
Donald G. Eagling
John J. Earle
Daniel Shapiro
Earthquake effects, basic block configurations, seismic design criteria, and recommended design and construction techniques for shielding assemblies.

Chapter 11 Foreword **Emergency Planning for Earthquake Safety** **11-1**
The Facility Manager's Perspective
Donald G. Eagling
Practical advice for developing and implementing a comprehensive earthquake preparedness program.

Chapter 11a **Lifeline Considerations and Fire Potential** **11a-1**
John M. Eidinger
An overview of the earthquake performance and design of lifelines with emphasis on the high vulnerability of water systems coupled with the potential for fire following earthquake.

Chapter 11b **The Multihazard Emergency-Response Plan** **11b-1**
Terence P. Haney
An examination of disaster preparedness, emergency response, and operations recovery including risk assessment, hazard mitigation, and a framework for developing or evaluating preparedness plans.

Chapter 12 Foreword **Risk Management** **12-1**
The Facility Manager's Perspective
Donald G. Eagling
Practical guidance for managing the increasing potential for seismic liability, and providing optimum safety and cost benefits with limited resources.

Chapter 12a **Quality Assurance by Peer Review** **12a-1**
Frank E. McClure
A summary of peer review requirements, professional guidelines, example experience, problem areas, and benefits.

Chapter 12b **Risk Management Analysis** **12b-1**
Jack R. Benjamin
The technical process of obtaining the best allocation and expenditure of scarce resources through use of decision tree analysis and probability.

Chapter 13 Foreword **Model Code Related Services** **13-1**
The Facility Manager's Perspective
Donald G. Eagling
A discussion of the use and benefits of model building code membership including fees, technical resources, plan checking services, and the code development process.

Chapter 13a **The World of Building Codes** **13a-1**
Diana R. Todd
An overview of model building codes used in the United States and their relationship to national standards, seismic resource documents, and related professional and industry organizations.

Chapter 13b **The International Conference of Building Officials** **13b-1**
Douglas W. Thornburg
A descriptive summary of the ICBO, publishers of the Uniform Building Code, including its objectives, eligibility requirements, services, publications, and the code change process.

Appendix A **Biographical Sketches for Authors of the Text and the Preface** **A-1**

Appendix B **Bibliography of Documents Produced for the Department of Energy Natural Phenomena Hazards Safety Program** **B-1**

PREFACE

1983 Edition

Author Donald Eagling writes: "*Often the process of studying the seismology of an area, selecting design earthquakes, and developing priorities and analysis techniques becomes so complex and bound up with sophistication that the (seismic) program's practical objectives are lost in the cracks between experts.*" How true! As a person who has been part of the earthquake engineering profession for over 40 years I have observed the rapid growth of sophisticated earthquake engineering analysis and design practices. As a former educator I applaud the great progress brought about by this thrust. Sophisticated *state of the art* analyses accomplished *with understanding* have brought about better earthquake resistive construction and have the potential to continue to do so. However, it is my personal opinion that the complexities of today's most advanced analytical techniques have outstripped the capabilities of the majority of structural engineering's practitioners. While many can manipulate the mathematics, most do not understand the results in physical terms. Over the last few decades public debate about the safety of nuclear facilities has intensified this problem. In the eyes of many, the potential intervenor is *demon god* and to appease this *god* an ever increasing complexity of investigations, analyses and design practices have been served to it in the name of increased safety. Various proposals for appeasement have been to no avail. Opposing arguments have always favored more sophisticated and costly engineering practices and usually more studies have been required. Too often the result has been to put off relatively simple solutions to seismic problems. Where new construction is involved, costs increase with time, but the hazard does not. Where existing poor construction is involved, hazards as well as costs grow with time. When the mitigation of serious seismic hazards is delayed by overly sophisticated reviews or studies, the practical objectives of seismic safety are simply not realized in a timely way.

During these times when socio-political issues often dominate public discussion of seismic safety, it is more important than ever to move ahead with practical and corrective action where the consequences of damaging earthquakes can be serious.

The authors of this *Seismic Safety Guide* represent a cross section of the earthquake engineering profession, from state of the art to practitioner. I recommend their counsel in the chapters that follow for a practical course to seismic safety.

Karl V. Steinbrugge
September 1983

Foreword

Contributors and Reviewers

Donald G. Eagling

This 1996 *Seismic Safety Manual* is a second generation publication that evolved from the *Seismic Safety Guide* first published in 1983. The stimulus for the 1983 publication came from the 1971 San Fernando, California earthquake, the subsequent seismic evaluation and rehabilitation program at the Lawrence Berkeley National Laboratory (LBNL), and James R. Hill who saw the need to bring the lessons learned from the LBNL experiences to the Department of Energy (DOE) community. An important objective of the 1983 publication as well as this one is to provide managers and operators of DOE facilities with practical advice so that they can achieve a comfortable level of understanding for the decisions they must make relative to seismic risk management.

The 1983 publication included the work of nine authors and a few reviewers. Nineteen authors and 14 reviewers contributed topics to this publication which gives one some sense of the rapid growth and increased sophistication that the field of earthquake engineering has experienced during the last decade. It also provides some measure of the challenge it was for the authors to make their subjects readable and practical for managers and engineers who are not specialized in earthquake engineering.

Each author has extensive practical experience in his or her field and has studied first hand the lessons learned in the aftermath of damaging earthquakes. Biographical sketches for authors of the text and the Preface are included in Appendix A.

Special acknowledgments are due the reviewers who were undoubtedly challenged by the comprehensive nature of the subject matter as well as the frailities of unpolished manuscript. However, their efforts were very influential and valuable to this publication and greatly appreciated by the editor.

These professionals whose critique and comments are incorporated throughout the publication are listed hereafter in alphabetical order.

Khawaja A. Akhtar	Staff Geologist, Engineering/Design Support Group, Office of Engineering, Operations, Security and Transition Support, Defense Programs, U.S. Department of Energy, Washington, D.C.
Ann S. Bieniawski	Research Structural Engineer National Institute of Standards and Technology U.S. Department of Commerce Gaithersburg, Maryland.
David W. Coats	Division Leader, Plant Engineering Design and Construction, University of California, Lawrence Livermore National Laboratory, Livermore, California.

Michael D. Davister	Staff Structural Engineer, Engineering/Design Support Group, Office of Engineering, Operations, Security and Transition Support, Defense Programs, U.S. Department of Energy, Washington, D.C.
James R. Hill	Manager (Retired), Natural Phenomena Hazards Safety Programs, Office of Engineering and Site Interface, U.S. Department of Energy, Washington, D.C.
Robert P. Kennedy	President, RPK Structural Mechanics Consulting, Inc., Yorba Linda, California.
Jeffrey K. Kimball	Seismologist, Group Leader, Engineering/Design Support Group, Office of Engineering, Operations, Security and Transition Support, Defense Programs, U.S. Department of Energy, Washington, D.C.
Frank E. McClure	Consulting Structural Engineer, Orinda, California.
Robert C. Murray	Geologic and Atmospheric Hazards Project Leader, University of California, Lawrence Livermore National Laboratory, Livermore, California.
Thomas A. Nelson	Group Leader, Structural Mechanics, New Technologies Engineering Division, University of California, Lawrence Livermore National Laboratory, Livermore, California.
Roland P. Sharpe	Consulting Structural Engineer, Cupertino, California.
Stephen A. Short	Senior Consultant, EQE International, Irvine, California.
Stanley C. Sommer	Structural Mechanics Engineer, Mechanical Engineering Department, University of California, Lawrence Livermore National Laboratory, Livermore, California.
Karl V. Steinbrugge	Structural Engineer, El Cerrito, California.

Editors whose contributions made the text readable were:

Penny M. Sanford	University of California, Lawrence Livermore National Laboratory, Livermore, California.
Barbara B. Johnson	Geomatrix Consultants, Inc., San Francisco, California.

Production and format were the work of:

Dawn S. Matz	University of California, Lawrence Livermore National Laboratory, Livermore, California.
Betty M. Irving	University of California, Lawrence Livermore National Laboratory, Livermore, California.
Dennis Chan	Senior Technical Illustrator, University of California, Lawrence Livermore National Laboratory, Livermore, California

Additional acknowledgements for those who were instrumental in the conceptual development, planning and production of this publication are found at the end of Chapter 1, Introduction.

Finally I would like to acknowledge the U.S. Department of Energy, Office of Environment, Safety and Health, Office of Nuclear and Facility Safety whose support has made the Seismic Safety Manual possible.

Donald G. Eagling
Facilities Manager (Retired)
University of California
Lawrence Berkeley National Laboratory
Berkeley, California.

CAUTIONARY NOTICE

This document was completed in October 1995. The Department of Energy was, at that time, involved in an intense and accelerated effort for reduction of directives. As a part of this effort, DOE Order 5480.28 on Seismic Hazards Mitigation was cancelled and the seismic policy significantly revised and consolidated in October 1995 into DOE Order 420.1, "Facility Safety," and its associated implementation guides. DOE Order 6430.1 was also cancelled. In addition, the DOE Standards referenced in this manual have already been revised to reflect the changes in DOE Order 420.1 issued after completion of this manual.

This document is oriented to standard occupancy structures and essential facilities and includes much guidance from commercial sources. Therefore, it may have limited applicability to DOE nuclear facilities. Hence users are advised to use this document with suitable caution taking these points into account.

Chapter

1

Introduction

Donald G. Eagling

Overview

This *Seismic Safety Manual* provides managers of U.S. Department of Energy (DOE) facilities with practical guidelines for administering a comprehensive earthquake safety program. Such a guide is needed because of the complexity of new seismic and safety regulations, increased public awareness, and the heightened potential for personal and institutional liability.

Often the approach to reviewing existing facilities for seismic safety is so overly sophisticated that the actual abatement of obvious deficiencies is delayed, costly, and often legalistic rather than objective. Furthermore, most buildings are being constructed without benefit of a seismic plan check, a simple process that has proved so effective in experience with earthquakes in California.

Significantly, structural engineers who have observed and studied damaged buildings in the aftermath of earthquakes are generally able to diagnose hazardous deficiencies in existing buildings rather easily and efficiently. It is seldom necessary to carry out elaborate analyses to evaluate the seismic resistance of structures. The process of review does not have to be expensive or complex. Often, the problems found in construction and design are simply the result of failure to implement what has been known and observed about earthquake engineering for many years.

The objective of this *Seismic Safety Manual* is to provide practical advice about earthquake safety and engineering to managers of DOE facilities so that they can get the job done without falling into common pitfalls and prolonged diagnosis. It provides managers with basic guidelines and methodology, but is intended neither as a textbook nor as a substitute for the use of competent personnel.

The *Manual* is comprehensive with respect to earthquakes in that it covers natural hazards, site planning, evaluation and rehabilitation of existing buildings, design of new facilities, operational safety, emergency planning, special considerations related to shielding blocks, nonstructural elements, lifelines, fire protection, and emergency facilities. Management of risk and liability also is covered. Although it is written primarily for managers of DOE facilities, it is also meant as a contribution by DOE to the *International Decade of Natural Disaster Reduction*. Consequently, it is not limited to discussion of DOE requirements per se, but covers the broader principles and concerns for earthquake safety of interest to managers, owners, architects, and engineers in the private sector as well.

Generally, the *Manual* is oriented to facilities that fall into building occupancies defined as *Standard Occupancy Structures* and *Essential Facilities* under the *Uniform Building Code*, (Ref. 1) which correspond to Performance Categories 1 and 2, respectively, as defined under DOE-STD-1020, *Natural Phenomena*

Hazards Design and Evaluation Criteria for DOE Facilities (Ref. 2). DOE facilities in Performance Categories 3 and 4, which have more stringent requirements, and nuclear facilities in general are not dealt with per se. Although the broad philosophy and principles discussed herein should be of interest to all managers of DOE facilities, for the design and construction of such special structures the reader is referred to the standards, regulations, and guidelines that pertain to them specifically.

Each section of the *Manual* is written by a professional with solid design and field experience in his or her subject. Comments and advice from the facility manager's perspective also are provided in the Foreword to each chapter to bridge the gap between the seismic specialist and operational practicalities.

Seismic Review of LBL Facilities

Lawrence Berkeley Laboratory (LBL) is a multipurpose DOE facility operated by the University of California, engaged in large-scale fundamental research and applied science. It is located in the San Francisco Bay Area in *earthquake country*. In February 1971, following the destructive San Fernando earthquake in Southern California, LBL initiated a comprehensive review of its existing facilities and operations to improve earthquake safety.

The experience was enlightening. At LBL, all except a few very old buildings were designed by professional architects and engineers, licensed in California, using the latest edition of the *Uniform Building Code*. *All* structures had received construction inspection. In spite of these procedures, the earthquake safety survey revealed that significant structural deficiencies, stemming from many sources, were present in more than 50% of the buildings reviewed. A few old buildings had no formal lateral-force-resisting system. In other buildings, deficiencies were related to modifications after construction that inadvertently altered the lateral-force-resisting system. Design deficiencies, relatively few in number, usually resulted from the lack of a clear, comprehensive design philosophy rather than design error. Generally, not enough consideration for nonstructural elements in buildings and lifeline services was given by designers. A third-party seismic plan check was not utilized prior to 1971.

Significantly, most hazardous deficiencies in existing buildings were relatively simple to diagnose. They were quickly found by practical techniques used by structural engineers specializing in earthquake safety. Sophisticated analysis techniques were not required and in fact would have complicated and slowed the entire process of detection and, consequently, correction.

All sixty-two buildings that then existed, as well as vital site-utilities and emergency facilities, were inspected. Nonstructural elements and operational conditions also were inspected and analyzed to minimize unnecessary hazards. The physical site was studied to identify, delineate, and evaluate natural seismic hazards such as possible fault displacement and the potential for earthquake-triggered subsidence and landslides. Special facilities such as shielding blocks, storage for hazardous materials, communications centers, medical facilities, fire stations, and emergency generators received careful attention. The order of inspection was based on a priority system that considered life safety, emergency recovery capacity, off-site consequences, program continuity, and property value. The order of subsequent projects to abate hazards and improve earthquake safety was based on a separate priority system. This included consideration of the probability of earthquake occurrence, the structure's probable response, the severity of human exposure and property damage, and the possibility of off-site consequences. The system also evaluated the relative priority of projects to abate other types of risk such as fire, pollution, industrial hazards, and radiation.

These priority systems were simplistic and judgmental. Although due process was followed, the level of sophistication and complexity was minimized in favor of decisiveness and practicality. Structural deficiencies and operational hazards that could be easily corrected were promptly abated. When more complex hazards were identified that were life threatening, interim action was undertaken to reduce risks until the process of full abatement could take place. Usually, this meant taking temporary or expedient measures to reduce the hazard until a permanent solution could be developed and funded. Thirty-four buildings at LBL were strengthened; four were evacuated and later demolished. Projects to repair or strengthen structural systems, nonstructural

elements, and lifelines have been carried out on a priority basis since 1971.

Throughout the earthquake safety survey and subsequently during the design and construction of projects for strengthening buildings, LBL used the services of consultants specializing in all phases of earthquake science and engineering, including seismology, geology, soils, dynamics, earthquake risk analysis, and structural design. LBL also conducted shaking-table research with concrete shielding blocks in conjunction with the University of California's Earthquake Engineering Research Center.

LBL completed the inventory, screening, and evaluations of the sixty-two buildings in approximately two years. Contents and experimental facilities within buildings were not initially evaluated except where obvious hazards involved life safety or could potentially lead to off-site consequences. The idea was that building collapse was the greatest hazard for contents as well as people; thus, building retrofit was very high in priority. Most building deficiencies were corrected during the first ten years, but the upgrading program has continued. A project to upgrade the last of the original thirty-four buildings found deficient was completed in 1994.

Because all buildings at LBL were originally designed to resist earthquakes, most deficiencies were relatively easy to correct. However, eight buildings had fundamental and pervasive design deficiencies that required more complex and costly evaluations and corrective measures. Criteria for retrofit were 0.2g base shear for static analysis and 0.7g peak ground acceleration for dynamic analysis.

Inventory Screening and Evaluation Costs

Although the initial review of LBL's physical plant facilities during the 1971-1973 period was predominantly confined to buildings, the program was expanded to include all aspects of earthquake safety during the years that followed. The review was conducted by the firm of Engle and Engle on an hourly fee basis and included inventory, screening, and evaluation. The LBL staff had previously screened and prioritized buildings for review.

Escalating original fees to 1994 values, the cost of the Engle and Engle review ($180,000) and evaluation of the sixty-two buildings that then existed averaged about $2,900 per building or $0.14 per gross square foot (GSF). At LBL, where the majority of buildings are laboratories and shops and the hillside site is geotechnically complex, the average replacement cost in 1994 is approximately $430 per square foot including design, construction, and project management. Therefore, the cost of the inventory, screening, and evaluation is approximately 0.03% of replacement value. These figures do not include the evaluation of contents, process, or experimental facilities.

Upgrading Costs

The total cost of structurally upgrading thirty-four buildings at LBL for seismic safety was approximately $11 million, based upon 1994 dollars. Of this amount, approximately $7.3M was spent to correct the eight buildings that required the most costly and complex structural upgrades (Table 1-1). The cost to upgrade these buildings, in terms of replacement value, ranged from 2.9% to 14.3%. Construction types included wood frame, "Stress Crete" block (a short-lived building system utilized in the 1950s), reinforced concrete, and structural steel. The average unit cost for upgrading them in 1994 dollars is about $24/GSF or about 6.5% of the 1994 replacement cost.

Upgrading costs shown in Table 1-1 are construction contract costs except for Building 90, in which case the upgrade cost includes only those construction costs required to strengthen the building structure. The entire Building 90 project also included extensive hillside stabilization measures to protect the facility from earthquake-triggered landslides. The cost of this work is not included.

Building retrofit costs at LBL were primarily structural; very little was spent for electrical, mechanical, or operational disruption by virtue of external strengthening whenever possible. Also, it should be recognized that structurally deficient buildings that were initially designed with earthquakes in mind are usually less costly to upgrade for lateral force resistance than those that were initially designed with no thought for earthquakes.

Table 1-1. Costs of upgrading eight buildings requiring the most complex improvements.

Building Number	Const. Type	1971 Area GSF	1994 Bldg. Repl. Value $M	1994 Upgrade Cost $M	% of Bldg. Repl. Value
7	Wood Fr.	28,000	8.40	0.60	7.1
10	Wood Fr.	16,038	6.90	0.36	5.2
25	"Stress Crete"	20,113	7.04	0.39	5.5
29	Wood Fr.	10,576	3.17	0.39	12.3
30	Wood Fr.	10,576	3.17	0.11	3.5
50A	Rein. Conc.	67,475	29.01	0.83	2.9
50B	Rein. Conc.	64,367	27.68	0.83	3.0
90[1]	Str. Steel	88,301	26.49	3.78	14.3
		305,446	111.86	7.29	6.5

[1]Construction cost for structural bracing only.

Cost Benefits

If these eight buildings *were not* upgraded and the LBL site were subjected to the design earthquake (M = 7.0) on the nearby Hayward fault, it is estimated that all eight would be so extensively damaged that they would have to be replaced. In addition, three of them, Buildings 25, 50A, and 50B, totaling about 152,500 GSF with 393 occupants, would suffer catastrophic collapse under this scenario. Typically, in R&D laboratories, the value of laboratory equipment and experimental apparatus is roughly equal to the cost of the building structure. Thus, the replacement cost of all eight buildings ($112M) coupled with the replacement cost of contents for three collapsed buildings ($64M) is about $176M if no allowance for loss of life is included. Costs to provide leased space offsite, moving charges, modifications to leased space, and lost time for the occupants of the eight buildings would total another $60M. If the remaining twenty-six buildings, which had also been upgraded, had not been strengthened, an additional $15M in repairs would be anticipated (not including damaged contents, personal injury, loss of life, or programmatic impact). Thus, for the big earthquake, if the thirty-four buildings were not upgraded, the total monetary loss would be about $250M. For avoidance of this scenario, the benefit-cost ratio is approximately 23 (250 divided by 11) with no allowance for personal injury or loss of life.

In a similar scenario, but based upon ground accelerations measured at LBL during the 1989 Loma Prieta, California earthquake, the Laboratory would have suffered a financial loss of approximately $50M if the three particularly susceptible buildings *had not* been strengthened. The cost to strengthen these three buildings is $2.05M in 1994 dollars. The damage to these buildings would have clearly shown them to be collapse hazards in a major earthquake. Thus, as in the case of the Oakland City Hall, which was damaged in the 1989 Loma Prieta, California earthquake, it would have been mandatory to evacuate them and find off-site leased space for at least three years before they could be strengthened and reoccupied. The benefit-cost ratio for this case is approximately 24. (The Oakland City Hall is scheduled for reoccupancy in 1995, more than five years after the earthquake.)

The earthquake safety survey and improvement program at LBL has been a comprehensive experience in practical risk management. From this perspective, it has been observed that most earthquake safety programs elsewhere become too sophisticated, complex, and expensive to expeditiously achieve the desired results. Often the process of studying the seismology of an area, selecting *design* earthquakes, and developing priorities and analysis techniques becomes so complex and bound up with sophistication that the program's practical objectives are lost in the cracks between experts.

Fortunately, the consultants and specialists who assisted LBL with its earthquake safety program counseled a practical course that achieved early results and minimized costs. It

was with their advice and support that the concept for the first edition of the *Seismic Safety Manual (1983)* was developed. Its emphasis then and now is on the practical application of earthquake safety.

Acknowledgments

The LBL Seismic Safety Program has come a long way since early 1971 when the San Fernando earthquake provided stimulus for the Laboratory to embark on a complete review of its facilities. During the year before the 1971 earthquake, Karl V. Steinbrugge, then Chief Structural Engineer for the Pacific Fire Rating Bureau (predecessor to Insurance Services Office), volunteered to visit LBL and make an informal, cursory assessment of the nature of damage that might take place as the result of an intense earthquake. Steinbrugge, who was also Professor of Structural Design, Department of Architecture (now Professor Emeritus) at the University of California, Berkeley, is one of the most respected earthquake engineers in the profession. His wealth of experience in reviewing earthquake damage throughout the world led LBL to seek his advice. His informal assessment in 1970 of the probable damage to LBL facilities if an intense earthquake should occur was $90 to $120M (in 1994 dollars). This news was rather shocking and prompted facilities management, with Steinbrugge's help, to plan and outline a program for a complete seismic review of LBL facilities by independent earthquake engineers. This proposal was under review by LBL management when the San Fernando earthquake took place. The proposal was accepted one day later.

Much of Steinbrugge's energy over the years has been directed to improving public policy and safety in the face of the inevitable great earthquake. His contributions have been enormous. They include chairmanship of the President's Task Force on Earthquake Hazard Reduction, the Seismic Safety Commission of the State of California, and the U.S. National Committee on Earthquake Engineering. He is past president of the Earthquake Engineering Research Institute and the Seismological Society of America. As a consultant, he served the Nuclear Regulatory Commission Advisory Committee on Reactor Safeguards, the National Oceanic and Atmospheric Administration, and the U.S. Geological Survey. Steinbrugge's book, *Earthquakes, Volcanoes, and Tsunamis*, is a very valuable reference document for both engineers and lay people interested in the subject. Karl Steinbrugge's voluntary contribution to the seismic safety program at LBL is deeply appreciated.

With Steinbrugge's counsel, LBL selected Engle and Engle, Structural Engineers, to carry out an independent review of LBL facilities. The late Harold M. Engle, Sr., was a true pioneer in earthquake engineering, as well as a rugged individualist with very high professional standards. He also possessed a rare practical aptitude for confronting basic issues in a way that enabled LBL to get the most seismic safety for the money available. Great strides were made in spite of tight budgets. Engle's sage advice and support kept LBL from being entangled in the endless and expensive studies and controversies that have plagued some seismic safety programs. Engle and his son, Harold Engle, Jr., who now heads the firm, were extremely effective in getting the job done efficiently and expeditiously, providing timely advice for protecting people and property and avoiding serious pitfalls in risk management.

LBL is indebted to the University of California, Berkeley (UCB) for the contributions of many of its professional staff. Professor Bruce Bolt, seismologist and then Director of the Seismographic Station at UCB, provided site specific design earthquake data for dynamic analysis where required. Professor Joseph Penzein, then Director of the UCB Earthquake Engineering Research Center, also provided counsel for dynamic analyses and made recommendations with respect to complex seismic engineering problems. Professor William Godden, of the UCB Civil Engineering Department, collaborated with LBL's Dr. Theodore Scalice to carry out original research with concrete shielding blocks on UC Berkeley's 90-ton shaking table facility, and provided consultation to LBL's Seismic Safety Committee for special design reviews.

Frank McClure joined the LBL Plant Engineering staff in 1978 as Chief Structural Engineer, a position he held until he retired in 1991. He is an Honorary Member of the Earthquake Engineering Research Institute (EERI) and the Structural Engineers Association of Northern California (SEAONC) and past President of EERI. In 1990 he received the U.S. Department of Energy Distinguished Associate

Award for his efforts to achieve earthquake hazard mitigation. In 1993 he received the Alfred E. Alquist Award for achievement in earthquake safety and sustained leadership in the earthquake field.

His professional career includes long public service with the State of California Seismic Safety Commission Advisory Committee, the State of California Field Act Advisory Board, the Committee on Earthquake Engineering, National Research Council, the Building Seismic Safety Council, the Federal Interagency Committee on Seismic Safety in Construction, the National Institute of Standards and Technology, the Seismology Code Development Committee of the International Conference of Building Officials and many DOE Working Groups on Natural Phenomena Hazards.

McClure provided key support and guidance for the LBL seismic safety program. His encouragement and technical consultation in support of the development of the 1983 edition of the *Seismic Safety Manual* and the current 1996 edition is greatly appreciated by the author.

The Seismic Safety Program at LBL is carefully monitored by an in-house *Seismic Safety Committee* that recommends policy and participates in design reviews when state-of-the-art projects are involved and the design carries an assumption of risk that should be an institutional responsibility rather than that of an individual. Dr. Theodore Scalice, Chairman of the Seismic Safety Committee for many years, contributed greatly to LBL's Seismic Safety Program. In addition to this leadership role, he carried out original research in collaboration with UCB Professor William Godden. Their work significantly contributed to the development of analysis and design procedures used at LBL for the seismic restraint of shielding block systems (see Chapter 10).

Daniel Shapiro and John Earle of Shapiro, Okino, Hom, and Associates, Structural Engineers; Harold Engle, Jr., of Engle and Engle, Structural Engineers; and James Stratta, Consulting Engineer, all contributed to the LBL seismic safety program as well as to the original *Seismic Safety Guide.* Each gave the author strong support for the concept that a seismic safety guide with practical advice would be a valuable reference for facility managers.

Roland Sharpe, Structural Engineer, a leading authority in seismic safety, participated as an author-editor for both the 1983 *Guide* and this *Manual.* His counsel during the conceptual stages of these projects has been inspirational. His strong editorial support has been essential to the successful completion of each edition.

Major support for the DOE Natural Phenomena Hazards Mitigation Program has been provided by Lawrence Livermore National Laboratory (LLNL) under the management of Dr. Robert C. Murray, Geologic and Atmospheric Hazards Project Leader. The 1996 edition of the *Seismic Safety Manual* is just one element among an impressive number of major contributions to the DOE system of standards, guidance documents, and design manuals for the mitigation of natural phenomena hazards that have been produced under his effective leadership.

James R. Hill, Manager (retired), Natural Phenomena Hazards Safety Programs for the DOE Office of Engineering Assistance and Site Interface, Washington, D.C., sponsored both 1983 and 1996 editions of the *Manual.* As well, he wrote the landmark policy embodied in DOE's new regulations for *Natural Phenomena Hazards Mitigation.* Hill, who was appointed DOE's Seismic Safety Coordinator by the Secretary of Energy, provided vital leadership and dedication that made this publication feasible. Without his strong continuous support, the *Seismic Safety Manual* would not exist.

References

1. *Uniform Building Code,* 1994 Edition, International Conference of Building Officials, Whittier, California, 1994.

2. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities,* DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.

3. *Earthquakes, Volcanoes, and Tsunamis,* K.S. Steinbrugge, Scandia America Group, New York, New York, 1982.

Chapter

2

The Facility Manager's Role in Earthquake Safety

Donald G. Eagling

Introduction

Although the field of seismic safety is the domain of the engineering profession, funding to carry out a seismic safety program is normally derived from the budgets that managers (or owners) need to carry out the overall missions of their organizations. Consequently, it is generally managers rather than engineers who must make most of the key decisions that are needed to carry out an effective seismic safety program. It is also managers who must resolve issues of risk management that accompany seismic hazards and involve life safety, damage control, potential evacuation and relocation costs, and potential liabilities, as well as cost-benefit considerations. It is therefore very important that facility managers have a broad understanding of the comprehensive nature of an effective earthquake safety program as well as an awareness of the socio-political, legal, and economic risks that often impede the progress and success of such programs.

This chapter provides practical advice for owners or managers who must carry out earthquake safety programs. It is written from the perspective of an engineer who has had such an experience as a facility manager.

The scope, depth, and focus required to carry out a satisfactory program vary considerably with the age of a facility, the risk involved, and the quality of design that was applied during its construction history. For a new and growing facility, the focus is on design and construction. For an older facility, the need to evaluate existing conditions and prioritize projects for abatement of seismic hazards receives most attention. For the majority of sites, however, a balanced program is most effective in preventing further development of new hazards while reducing the backlog of old ones.

Structural engineers who are experienced in earthquake engineering and have reviewed a number of facilities, both in government and private enterprise, have found a wide variety of serious seismic deficiencies that owners or managers were unaware existed. This is not unusual even in areas of the country where seismic design provisions have been part of the building code for many years.

East of California, few conventional buildings in the United States have been designed for earthquakes, even where there has been a history of earthquakes of sufficient intensity to damage buildings. In locations

where the potential for seismic destruction exists along with a legacy of hazardous buildings and contents, the prospect of carrying out a comprehensive earthquake safety program is indeed challenging.

Generally, managers responsible for operating major facilities are unfamiliar with earthquake engineering and tend to look for answers in techniques that are more sophisticated than those often required to solve the actual problems in earthquake safety. The approach to solving these problems often becomes so academic, ponderous, and expensive that abatement of the seismic hazards simply doesn't get done in a timely manner.

In recent years, the state-of-the-art in seismology, geotechnical theory, and dynamic analysis of structures has progressed tremendously. Spurred on by the need to resolve questions in seismic safety for nuclear power plants, the field has become very compartmentalized and specialized. The great strides made in these specialties have contributed significantly to the field of earthquake engineering and public safety. Unfortunately, it is easy for responsible managers or facilities engineers to *fall into a crack* between these experts who quite naturally tend to resolve seismic questions based on their own specialties.

More time and money can be expended analyzing the problems in earthquake safety than would be needed to design practical solutions to these same problems. To gain a better perspective, it is important to understand that most problems found in existing construction are a result of not implementing what has been known and observed about earthquake-resistant construction for a long time. Structural engineers who have observed and studied earthquake-damaged buildings are able to diagnose hazardous deficiencies in existing buildings rather easily and efficiently, often without complex calculations. They understand which building types are hazardous, and they know cost effective methods for rehabilitating them.

Selecting An Earthquake Safety Consultant

The most important thing managers can do to initiate an effective and economical earthquake safety program is to hire as consultants experienced earthquake engineers who are strong on design and tend to keep analysis straightforward and understandable. Occasionally, there is good reason to apply sophisticated techniques to provide a better understanding of a complex problem, but not very often. Earthquake engineers, working closely with managers, should advise on the selection of specialized consultants such as geotechnical engineers or seismologists, define their scope of work, coordinate their work with the overall program, and ensure that owners are not victimized by unnecessary or impractical studies.

During a seismic safety conference about fifteen years ago, a geotechnical expert was expounding on the sophisticated techniques his firm had used to predict site-specific earthquake ground motion for his client. His study had been the most recent of a series of analyses by various consultants and agencies covering the site of a major facility that included many unsafe buildings that housed hazardous materials. At that point in time, these analyses had absorbed almost ten years during which time a damaging earthquake had not taken place. A well-known earthquake engineer asked, "Haven't we analyzed this site enough? Isn't it time to design corrective measures to upgrade the seismic resistance of the unsafe buildings at this site." The consultant's reply was, "Well, no, not really, because the state-of-the-art is changing all the time." Obviously, the specialist was more interested in analysis for its own sake than he was in mitigating the seismic safety hazards that existed there.

When submitting a proposal or being interviewed for potential selection to review existing buildings for seismic safety, most firms are compelled to choose highly specialized consultants to help them compete for the appointment. Normally, requests for proposals

stimulate this response, thus setting the stage for the most sophisticated approach rather than the most effective and efficient. As a consequence, the leader for the appointed firm, usually the project manager, is likely to follow the profitable path of least resistance, letting the client-manager fall prey to more complex and time-consuming methodology, analysis, and reporting than is necessary to do the job. Unfortunately, this scenario is a wide spread and costly problem that is very detrimental to achieving timely seismic safety. Fortunately, it doesn't have to be this way. Reputable firms that are experienced in earthquake engineering know what is required and what is not required. With the proper direction and encouragement from clients, these firms can steer a more practical path that saves clients from wasteful expense and mitigates the hazards much more expeditiously. However, to achieve this end, the request for proposal and the criteria for selection must be written with this objective in mind.

Selecting the right structural engineering firm to counsel a practical way through the complexities and pitfalls that can befall earthquake safety programs is the most important decision managers can make.

This is also true when selecting geotechnical consultants. The level of sophistication in state-of-the-art techniques for predicting the intensity of ground shaking is intimidating. There is a strong tendency for both consultants and clients to believe the predictions to be more accurate than history shows they are. This tendency may lead participants to spend more money and time than the exercise is worth. The illusion of security thus developed is apt to be in direct proportion to the degree of sophistication applied.

The watchword is to keep the site investigation straightforward and simple, and rely more heavily on design than prediction.

Even when structural dynamics is to be employed, selection of an effective ground-motion input can be a relatively simple matter. There is little to be gained by exhaustive site studies because history shows that the prediction of ground motion is indeed an inaccurate science. The inaccuracies of input can generally be accommodated in good structural design. It is important to select geotechnical specialists who understand this.

Potential earthquake engineering and geotechnical consultants should be asked to explain in understandable terms how they expect to approach the project, what techniques will be applied, and what they expect to find. They should provide examples of previous work, names of clients, and cost histories. Clients should be sure that the lead consultants selected have strong design experience with comparable projects and have made field investigations of earthquake damage.

The Balanced Program

An effective earthquake safety program is analogous to an effective lateral-force-resisting system; it should have no weak links. Several basic precautions should be taken in establishing a program; of primary importance are the following six:

First, someone has to be responsible for code and regulation enforcement to ensure that facilities are designed and constructed to meet DOE orders, standards, and designated building-code provisions. Code provisions often require interpretation for specific applications. The best way to ensure that these provisions and regulations are properly interpreted and enforced is to formally appoint one individual as the *Building Official* for the site. This person should be knowledgeable about building and fire codes and a licensed engineer or architect. This appointment should be properly delegated from the manager or director of the site. *The Building Official should be given authority to review and approve all facilities design before construction can begin.* If this is not done, chances are that code enforcement will be diffused and ineffective. Worse yet, enforcement may be more susceptible to the pressures of the situation than to the intent of the code.

Second, make certain that planned new buildings are not being inadequately designed while the process of reviewing existing buildings for seismic safety is underway. This seems like a profound admonition, but the probability that it will happen is very real. It can be avoided by using a third-party plan-check or

peer review prior to construction to ensure that designs for new structures and retrofit projects will adequately resist earthquakes. It is embarrassing for managers to find that a newly designed and constructed building is worse than an old one, but it happens.

Design criteria should be clearly defined and readily usable. Applicable standards and model codes should be used for buildings that do not require higher criteria by regulation. Complex approaches or criteria should not be applied unless the need is clearly established. Criteria should be practical. At most DOE sites, minor building modifications, experimental setups, equipment installations, cabinetry, nonstructural components, and miscellaneous other projects are usually designed by architects, mechanical engineers, designers, or other nonstructural engineers who have little background in seismic analysis and design. Nevertheless, if the criteria are understandable and straightforward, these minor subprojects will be designed and built with adequate earthquake resistance. The more significant structural problems should be handled by licensed structural engineers. Also, research or production facilities, particularly those using hazardous materials, should be carefully reviewed for seismic safety by structural engineers. Facilities using hazardous materials should be reviewed with the assistance of hazardous materials experts and the designated *Fire Marshal* (or *Fire Chief*) for the site. *A professional engineer's stamp and signature as well as a third-party review should be required whenever life safety is involved.*

Third, review the site for geologic and seismic hazards. Potential conditions that are inherently hazardous in ground shaking should be identified. These may include the following:

- Unstable slopes and existing landslides
- Areas of low-density granular soils subject to densification and subsidence
- Areas of low-density granular soils subject to liquefaction
- Areas where sensitive clays may be subject to strength loss under heavy ground shaking
- Areas where flooding would occur if an up-slope levee or dam failed.

Active faults should be identified and a geologic map of the site developed.

The site review need not be rigorous in detail unless potential hazards pose a high risk for a specific existing building or lifeline. If a new building or improvement is planned, the specific siting should be examined in more detail. *The main point is to recognize potential hazards and take them into account.* For example, it would be folly to permit the typical one-third increase in allowable soil bearing capacity for seismic loading in an area of sensitive clays subject to strength loss under ground shaking.

The initial review should be quite broad and general in character. It is important that it be carried out by geologists or geotechnical engineers who understand the nature of soil dynamics, preferably people who have experience with earthquakes. Generally, except for fault rupture, each of the potential hazards that may exist can be mitigated through standard stabilization practices or by simply avoiding them in the case of new construction. Sometimes fault movement can be accommodated, or the effect of fault movement mitigated, if it is known where surface ruptures are likely to occur.

Fourth, survey and evaluate all existing buildings and structures to determine their earthquake safety ratings. Structural engineers experienced in earthquake damage investigation should do the job. The assessment should be kept simple. Experienced earthquake engineers know which types of buildings are hazardous. A basic requirement is to find out what has to be done to ensure that each building has a predictable lateral-force-resisting system. Establish a plan to carry out needed retrofit projects on a priority basis, but don't delay mitigation of high hazard conditions in order to see the whole picture. Start one step at a time, reducing liability on a priority basis.

Given a limited budget, it is important to determine which building projects will provide the greatest benefits for the money spent for improvements in life safety and property damage. For life safety, the procedures found in

Evaluating the Seismic Hazard of State Owned Buildings, California Seismic Safety Commission, SSC79-01, by McClure, Degenkolb, Steinbrugge, and Olson, are recommended. For property damage, refer to Fig. 10 and its supporting text in the report entitled *Estimation of Earthquake Losses to Buildings (Except Single Family Dwellings)* by Algermissen, Steinbrugge, and Lagorio, U.S. Geological Survey (USGS), Open-File Report 78-441. These references provide a practical rationale for a general approach to cost-effectiveness. Of course, practical risk management must also address socio-political issues that encompass and plague earthquake safety programs and pose questions of public and personal liability. These aspects of risk management are discussed in the Foreword to Chapter 12.

Fifth, make an earthquake-hazards survey of the contents of each building including operational equipment, hazardous materials storage, and nonstructural building elements and systems. Obvious hazards, such as loose-item overhead storage, should be immediately corrected by building managers or operations supervisors. Most operational hazards are obvious by simply observing the scene and imagining an earthquake taking place. Tipping hazards, such as storage cabinets, tall files, library shelving, and similar furnishings, should be braced or anchored. Tie-downs or restraints should be installed on plant equipment such as transformers, emergency generators, tanks, elevator drives, fans, motors, and similar units. Apply a simple and judgmental priority system to use limited resources economically.

Sixth, develop an emergency plan to recover from a destructive earthquake. Apply the scenario technique to develop a probable model for the aftermath of an intense earthquake. Use department heads who will have to handle the recovery to lead the planning effort. Reduce obstacles to recovery by eliminating obvious hazards and ensuring that the supplies and equipment that will be needed in an emergency will *in fact* be available. Lifelines, such as water supply lines, power systems, storm and sanitary sewers, transportation, and communications systems also should be surveyed with earthquakes (and seismically induced fires and landslides) in mind. *Be sure to take into account the potential for loss of service from offsite utilities suppliers.* The consequences of utilities losses can be mitigated by careful emergency planning, and the potential for loss of a given facility reduced by *hardening* the lifelines that would likely be in jeopardy during an earthquake.

Self-help planning, preparation, and training should be key elements in any emergency-response plan for earthquake safety. Make sure that building managers and operations supervisors understand this fact and let them take the lead in the preparation of local emergency plans.

In the chapters that follow, each facet of a balanced earthquake safety program is discussed by engineers who have a great deal of experience with earthquakes. Each has considerable knowledge about subjects covered in other chapters so there is some overlap. However, it is intended that each chapter can be read independently of the others.

As one might expect, sometimes a healthy difference of opinion is expressed. These differences reflect the perspectives of different experts but tend to give managers valuable insight into the practical state-of-the-art. They also remind us that often there is more than one answer to a given problem. When a problem is particularly *sticky* and costs, risks, or liabilities are high, it is unquestionably worthwhile to get more than one opinion.

The question arises, how do managers resolve technical differences of opinion between two consultants on subject-matter about which managers feel inadequate.

The best answer lies in managers' usual role: managing the multidisciplinary functions of a technical complex such as a major research and development laboratory or a sophisticated production facility. The development of good communication and mutual trust with practical earthquake engineering consultants will provide managers with an extension of expertise in this specialized field much as it does in any other specialized field under their management. In the end, the responsibility must lie with managers, and it is important to realize ahead of time that technical differences of opinion are apt to arise about earthquake safety management. Responsible earthquake engineering consultants will be more interested in the primary goal of

practical earthquake safety rather than earthquake engineering for its own sake. This extension of expertise through selection of wise counsel is a challenge that most managers face in other facets of their responsibilities.

When questions regarding technical differences of opinion or criteria persist, it is important that they are resolved by some due process within a technical framework that will stand the test of future technical and legal review. Designers and managers should be reasonably protected by the due process involved, assuming that they each fulfill their professional responsibilities satisfactorily. This subject involves considerations in risk management that are discussed more thoroughly in Chapter 12.

Chapter

3

Earthquake Damage

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

In the aftermath of an intense earthquake, one can observe many obvious and convincing lessons that are almost impossible to describe later to the many people who will be involved in an earthquake safety program. Management, plant and safety engineers, administrators, and in particular, the operators and occupants of buildings and facilities must cooperate if such a program is to be successful. The earthquake experience is hard to imagine, and without some understanding of the devastation that can result from the combination of hazardous buildings and intense earthquakes, the motivation for cooperation is minimal. Also, people have short memories with respect to accounts of damage from earthquakes that happened somewhere else.

The best way to get attention focused on earthquake safety is to have all persons concerned view the results of a damaging earthquake first hand. Failing that, a presentation of earthquake damage by structural engineers experienced in earthquake investigation is the next best thing. The use of video and/or ample photographs to illustrate the effects of heavy shaking on buildings, contents, and other types of facilities that are familiar to the audience can be very effective in promoting an understanding of the requirements for earthquake safety and providing motivation for action and cooperation.

This Foreword and Chapter 3a depict typical failures and problems one must face in the aftermath of damaging earthquakes to illustrate the lessons available there. Subsequent chapters of the *Manual* focus on solutions to the problems illustrated to provide continuity for readers.

Recent earthquakes in California and Kobe, Japan provide graphic evidence of seismic devastation in urban areas. On January 17, 1994, a magnitude 6.7 earthquake struck Northridge, California in San Fernando Valley about 20 miles from downtown Los Angeles. It occurred on a blind-thrust fault, previously unknown in location, but of a type known to exist throughout the Los Angeles and San Fernando Valleys. Free-field ground accelerations reached 0.9g horizontal and 0.6g vertical 10 miles from the epicenter which was about one mile from Northridge (Refs. 1 and 2). Damage was widespread throughout the greater Los Angeles area impacting a large, moderately high density suburban environment. About 12,500 structures, dwellings, industrial and commercial buildings, as well as non-structural elements and highway structures, suffered severe damage or failure (Fig. 3-1).

In the immediate aftermath of the earthquake, structural engineers found few surprises. It was well known that unreinforced masonry and nonductile reinforced concrete frames posed severe seismic hazards (Fig. 3-2).

Another well known vulnerable building type was the multi-story wood-frame structure constructed over a platform of reinforced concrete or masonry walls with intermediate reinforced concrete columns (Fig. 3-3). Many large department stores, also damaged in the earthquake, were built in the 1970s using certain reinforced concrete structural systems now known to be hazardous. Although one concrete-frame department store in the Northridge Fashion Center had been retrofitted (inadequately), it suffered near total collapse (Fig. 3-4). Hundreds of people could have been killed had the earthquake occurred while the store was open (Ref. 3).

In the aftermath of the Northridge earthquake, the media and the general public were alarmed because of damage and failures in newer structures, particularly parking garages, commercial buildings and apartment complexes. These failures, however alarming, were not surprising to reputable earthquake engineers. Many of these structures, however new, were known by structural engineers to be vulnerable construction types, particularly when designed and constructed to meet the *minimum* prescriptive requirements of the building code (see Chapter 7). These structures are often the product of the *design-build* contractors who usually ignore *motherhood* provisions of the building code. There are great pressures by owners, developers, investors, and even public institutions to push aside good engineering practice in order to expedite schedules and ensure economic competition.

The parking garage that failed at California State University, Northridge is a graphic example of the consequences of *design-build* methodology (Fig. 3-5). The basic problem with this structure is an age old one, well known to structural engineers who have observed failures in buildings with similar defects in past earthquakes. Nonductile columns designed to resist vertical loads only, cannot be allowed to deform laterally beyond their capacity to handle their vertical loads; they must be protected from

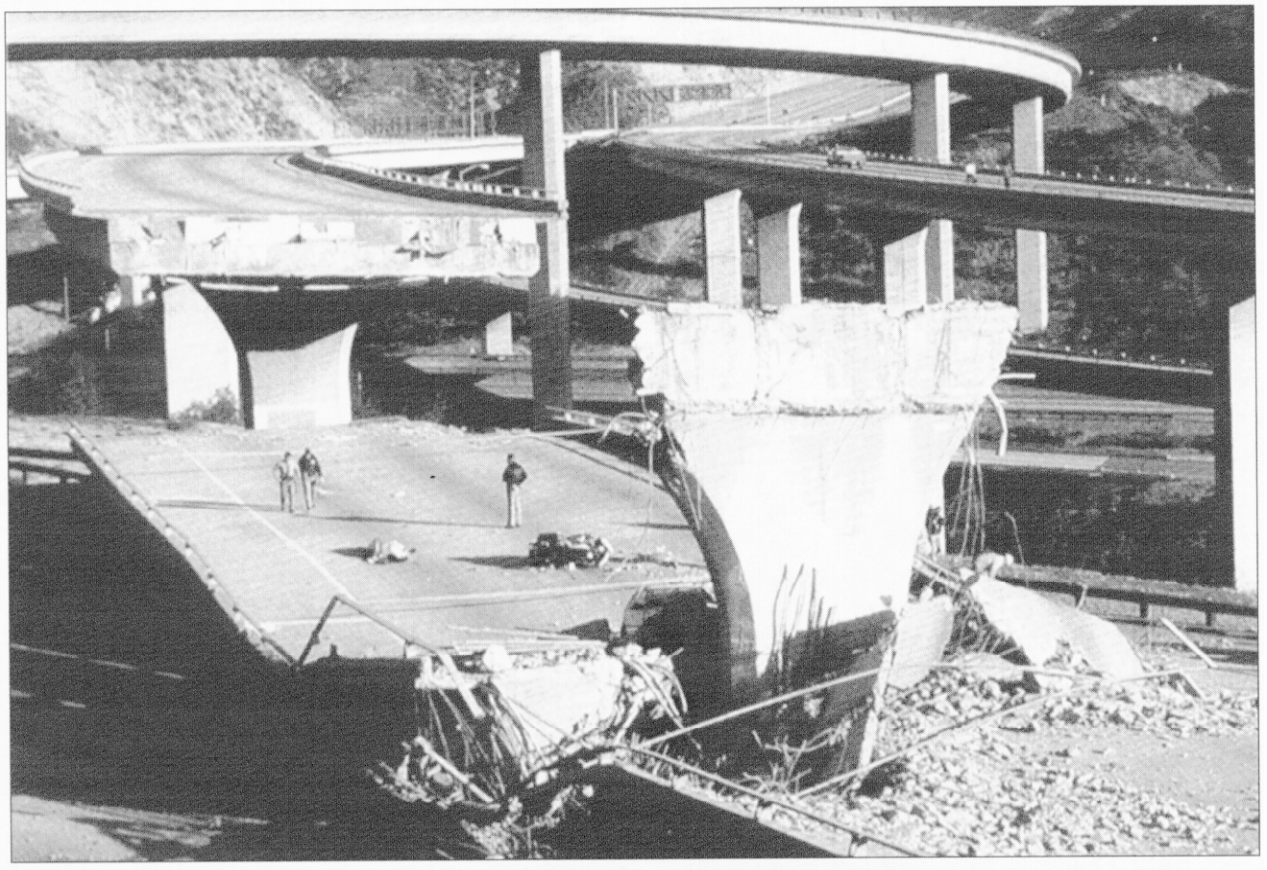


Fig. 3-1. Bridge collapse at Interstate 5 and State Route 14 interchange. Policeman was killed when his motorcycle ran off the severed freeway. Northridge, California earthquake, January 1994 (photo from Los Angeles Times, Ref. 1).

Fig. 3-2. Second story of this nonductile concrete frame building was completely crushed, and shear walls at both ends collapsed (top), close-up of crushed floor (bottom). *The January 17, 1994 Northridge, California Earthquake, An EQE Summary Report* (photo from EQE, Ref. 3).

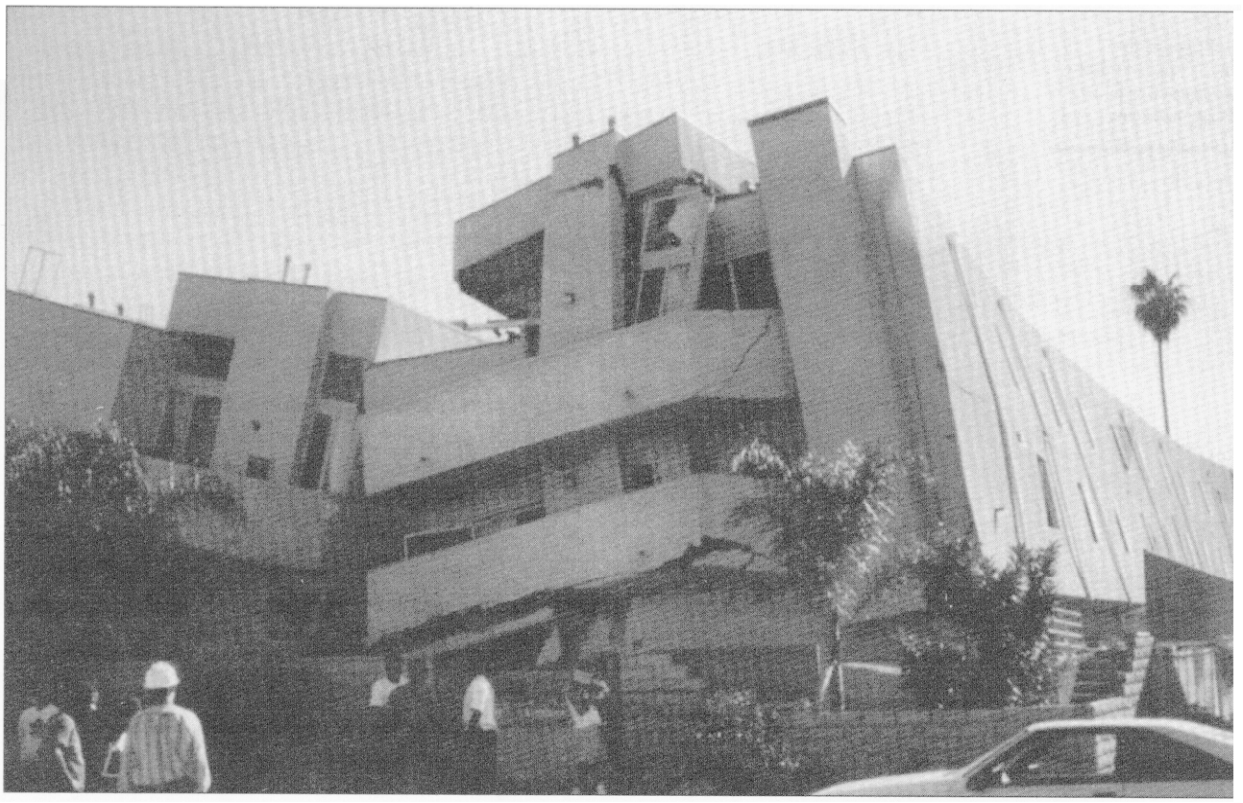


Fig. 3-3. Collapsed three-story apartment building in Van Nuys. The upper two-story wood-frame structure collapsed onto an inadequate concrete-block wall and cast-in-place, concrete column platform garage. Northridge, California earthquake, January 1994 (photo from EQE, Ref. 3).

excessive lateral deflection by limiting the deflection of those vertical members of the structure that are designed to resist lateral loads. This garage was a 5-level, 4-acre structure which was constructed by a *design-build* contractor who claimed the garage was code compliant in spite of being warned otherwise. The firm which reviewed the plans prior to construction pointed out its basic design deficiencies.

In the aftermath of the Northridge earthquake an independent engineering firm reported that there were no clear, unambiguous deviations from specific building code requirements; instead the design failed to address

Fig. 3-4. View of the interior of a 22-year-old concrete-frame department store in Northridge which partially collapsed. The remaining columns are 3 stories high. Small sections of the second and third floors remain in the background. Locations where the columns and pancaked floor slabs were previously connected can be seen on the column in the foreground. Northridge, California earthquake, January 1994 (photo from EQE, Ref. 3).

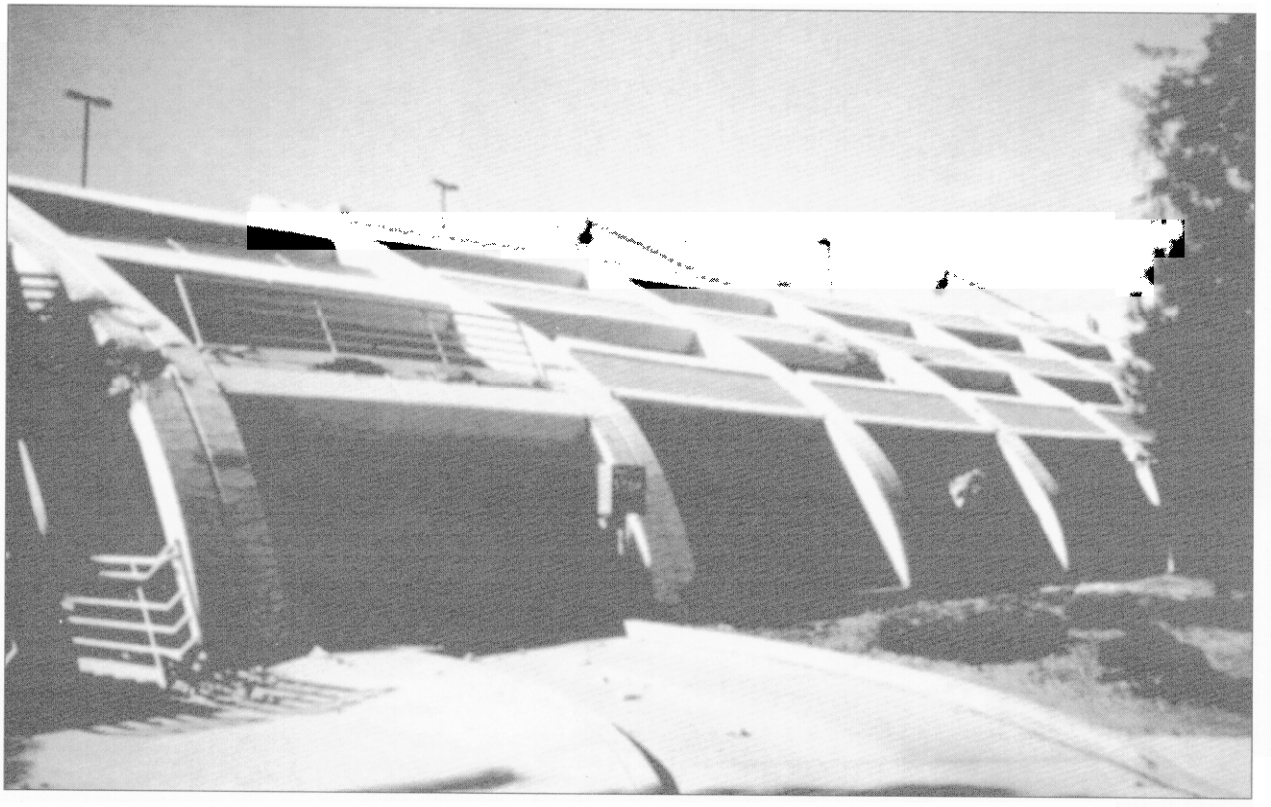


Fig. 3-5. A collapsed section of the 18-month-old parking structure at California State University, Northridge. This garage had precast moment-resisting ductile concrete frames on the exterior perimeter to take all the lateral earthquake loads. The interior precast columns were designed to carry only non-earthquake vertical loads. The nonductile interior columns could not resist the lateral deflections that the moment-resisting frames permitted. Northridge, California earthquake, January 1994 (photo from EQE, Ref. 3).

a series of general (intent) provisions in the code that led to noncompliance and failure. The report recommended not using design-build contractors for permanent buildings, adding that the advantages may not be worth the price of potentially low-quality designs, that even in a moderate earthquake may fail.

The collapse of this garage should provide motivation for appropriate use of the peer review process for projects of this nature. The subject of independent peer review is discussed in Chapters 1, 6 Foreword, 6a, 6d, and 12a. It is a vital element in seismic risk management.

Unfortunately, the Northridge earthquake exposed a very serious problem in buildings constructed with moment-resisting steel frames that surprised most earthquake engineers. A few weeks after the earthquake, it was reported that there was significant cracking in and around the welded (rigid) connections of such structures. This alarming news was slow to develop because these failures did not result in sufficiently large overall deformations to damage the architectural materials or concrete fire proofing that covered them. These hidden failures were brittle cracking in standard welded beam-to-column connections that have been used for many years in steel structures all over the world. About 7 months later a report, entitled *Performance of Steel Building Structures During the Northridge Earthquake,* was published by the Earthquake Engineering Research Center in August 1994 (Ref. 4). The same type of damage can be expected whenever such structures are subjected to strong earthquakes and, possibly, severe winds. As yet little has been done to investigate whether such damage actually occurred in welded structural steel joints in San Francisco during the magnitude 7.1 Loma Prieta, California earthquake.

Currently, this problem is under intensive investigation by SAC, a joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe). Under emergency authorizations obtained from the California Office of Emergency Services (COES) and the Federal Emergency Management Agency (FEMA), SAC has initiated a program to perform research aimed at resolution of issues related to steel frame damage and to publish *Design Advisories* and *Interim Guidelines* to assist engineers and building officials with practice until reliable new building code provisions can be developed.

Shortly after the unexpected damage to moment-resisting steel frame joints in the Northridge earthquake, the *International Conference of Building Officials* (ICBO), publishers of the *Uniform Building Code* (UBC), adopted an emergency code change to delete 1994 provisions for such joints and require that new joint designs demonstrate, by approved cyclic test results or calculations, the ability to develop the required strength criteria for the joint including the effects of steel overstrength and strain hardening (see Chapter 6a). Steel overstrength has become a serious problem for the *strong column-weak beam* concept, particularly where the column and beam are analyzed and designed using different strength steels. Steel producers generally ensure that a given steel meet its minimum strength requirement, but do not provide designers with a controlled range to maximum strength. For example, if a production run does not meet its minimum strength requirement, it could be marketed in the next lower strength steel classification even though it greatly exceeds that classification's mimimum strength requirement. The difficulty of designing a *strong column-weak beam* connection is increased under these circumstances, particularly when the consequences of inelastic behavior must be predictable.

Figure 3-6 shows the details of a typical welded steel moment-resistant connection. Steel fractures found in the connections of a 4-story commerical building (Fig. 3-7) damaged in the Northridge, California earthquake are shown in Figs. 3-8 and 3-9. Initially, these fractures were attributed to poor quality workmanship in the

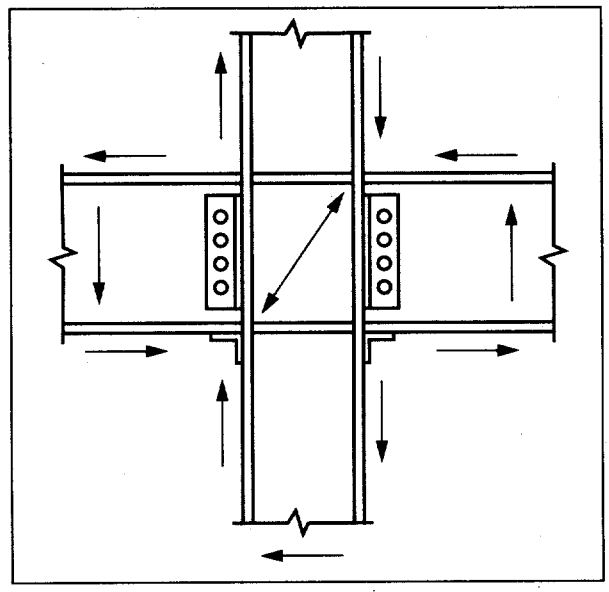


Fig. 3-6. Typical detail, steel moment resistant connection (from EQE, Inc.).

welding of connections, however, a review of historic test data, as well as a series of tests performed immediately following the earthquake, demonstrated that even connections with standard quality workmanship were vulnerable to this damage.

Materials in the weld and nearby beam-column webs and flanges cannot be so confined that the joint cannot deform in a ductile manner. In any case, the standard welded joint used for steel moment frames was shown to be brittle in the Northridge earthquake exposing a new and very hazardous flaw that exists in innumerable buildings throughout the United States and the world. Of special concern are high-rise structural steel buildings where the lateral resistance is concentrated in relatively few very large columns on the perimeter of such structures. Buildings where seismic and wind resistance is spread through all beam-column connections are thought to be less prone to joint failures.

When the ICBO issued the UBC emergency code change covering the steel moment frame joints, a project funded by the U.S. Department of Energy (DOE) to construct a Human Genome Laboratory at the Lawrence Berkeley National Laboratory was in the design stage, scheduled for bid in August 1995. Consequently, two full-size specimens of the steel joint (designed by Forell/Elsesser Engineers, Inc.) were tested (June

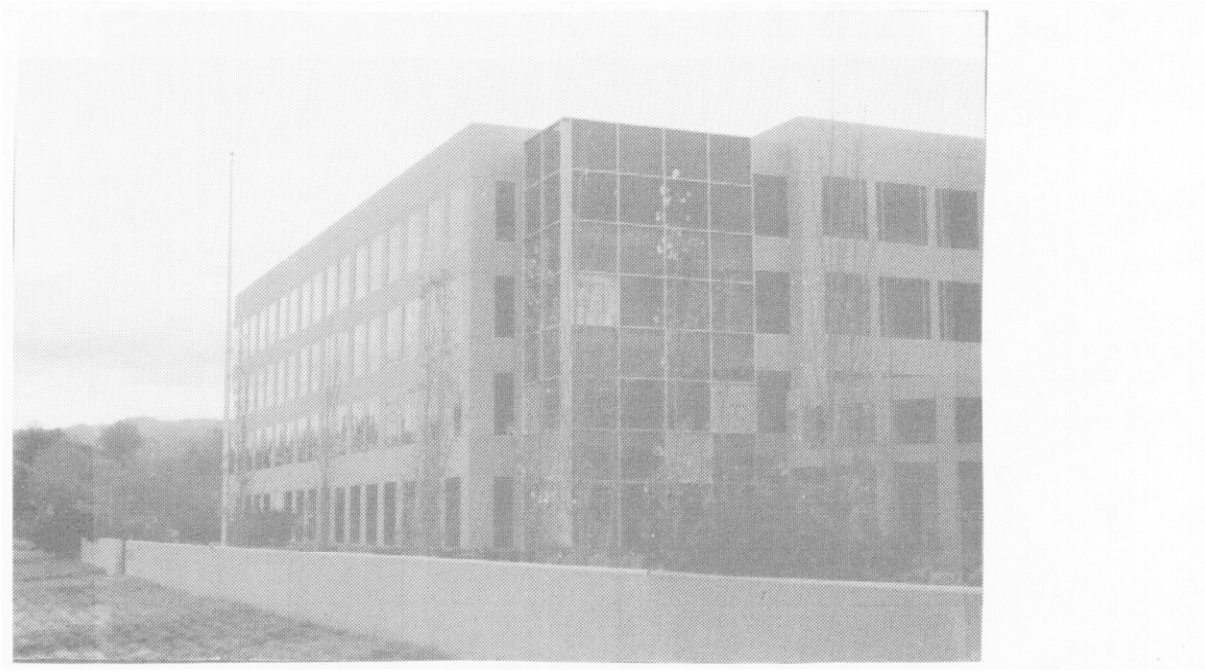


Fig. 3-7. The connections in the moment-resisting structural steel frame of this 4-story commercial building were seriously damaged in the 1994 Northridge, California earthquake (photo from EQE, Ref. 3).

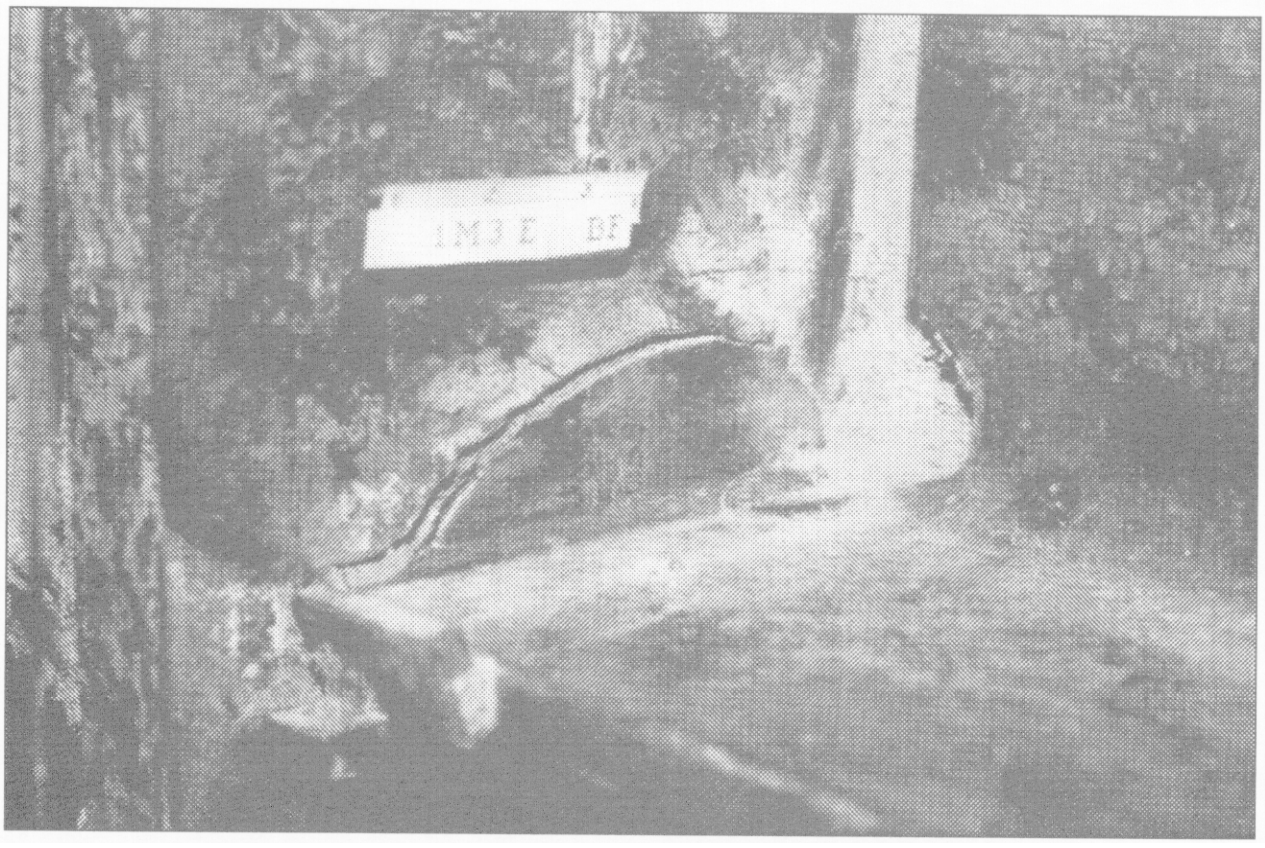


Fig. 3-8. Crack in a bottom flange of a beam-column connection within the 4-story commercial building. Northridge, California earthquake, 1994 (photo from EQE, Inc.).

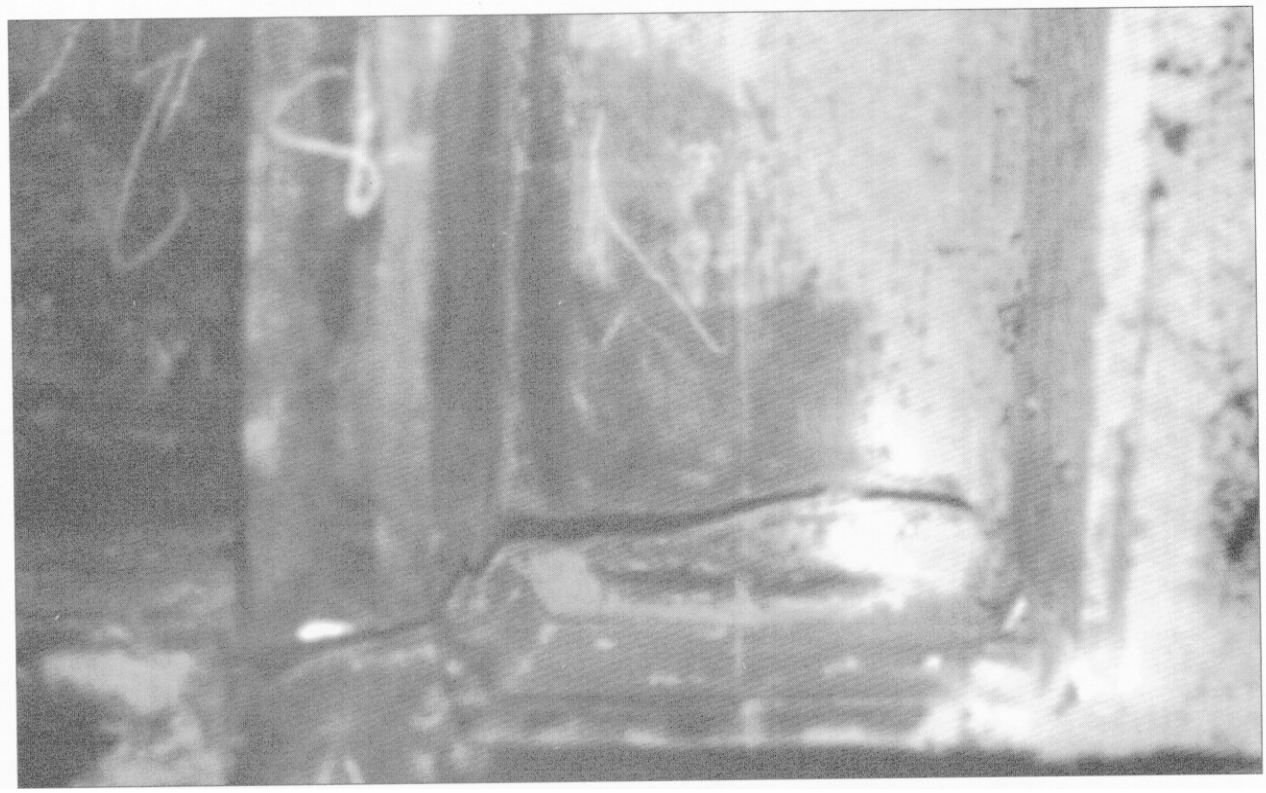


Fig 3-9. Crack in a column web-to-beam connection within the 4-story commercial building. Northridge, California earthquake, 1994 (photo from EQE, Inc.).

Fig. 3-10. Collapse of the Cypress Viaduct of Interstate 880 freeway in Oakland. The Loma Prieta, California earthquake, October 1989 (photo from Ref. 5).

26 and July 6, 1995) at the University of California, Berkeley (UCB) under the direction of Professor Emeritus, Egor P. Popov. These tests successfully proved the joint design in compliance with the UBC emergency code change acceptance criteria. This design was the first of numerous connections tested at UCB (during the previous months) to meet the new UBC criteria. Test data and design details are provided in a summary report by Forell/Elsesser in Appendix A of this Foreword.

The 1989 Loma Prieta, California earthquake was not the *big one* to be expected in the San Francisco Bay area because the epicenter was located in a sparsely populated mountainous area about 60 miles away. Nevertheless, the dramatic failures of the Cypress Viaduct (Fig. 3-10) on Interstate 880 in Oakland and the link span of the San Francisco-Oakland Bay Bridge (Fig. 3-11) provided a tragic *wake-up call* for the region. Over 1,300 buildings were destroyed, 20,000 damaged, and 18 bridges were closed to traffic following the earthquake. Loss of bridge access across the San Francisco Bay was a clear signal of the potential for much greater loss of transportation lifelines if a large earthquake strikes locally. Both the San Francisco Bay Bridge and the Golden Gate Bridge are susceptible to collapse in the *big one*; however, projects to retrofit both bridges are currently underway.

A 1990 report, entitled *Competing Against Time* (Ref. 5), by the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake provides a summary description of damage caused by the earthquake and recommendations to improve California's earthquake safety. In October 1994 a follow-up report, entitled The *Continuing Challenge, The Northridge Earthquake of January 17, 1994* (Ref. 6), was issued by the Seismic Advisory Board to the Director of the California Department of Transportation.

On Januray 17, 1995, one year after the Northridge, California earthquake, the magnitude 6.8 Hyogo-Ken Nanbu earthquake struck Kobe, Japan with disasterous results. Over 5,200 people were killed, over 26,000 injured and over 300,000 people were left homeless (Ref. 7). Approximately, 20,000 buildings were destroyed (100% damage) including buildings destroyed by the fire following the earthquake and about 35,000 buildings had severe damage (50% damage). An estimate of the total number of affected buildings in Kobe and

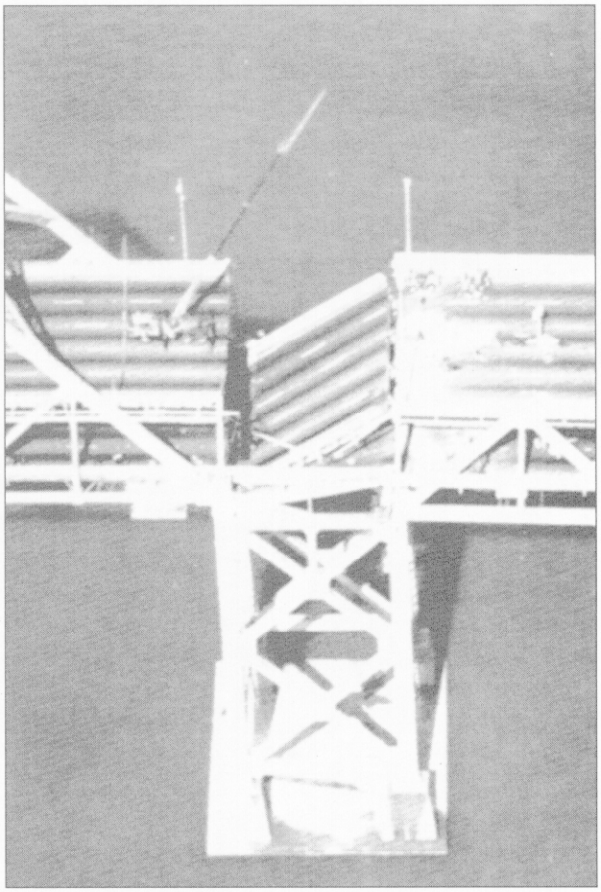


Fig. 3-11. Collapse of the link span of the San Francisco-Oakland Bay Bridge. The upper and lower closure spans fell when bolts attaching the truss span (right) were severed and the truss span moved to the right pulling the link spans off their supports. The Loma Prieta, California earthquake, October 1989 (photo from Ref. 5).

neighboring areas is roughly 800,000 including residential, commercial and industrial buildings. Financial losses are about 200 billion dollars, over ten times the losses in the Northridge earthquake. A significant proportion of the casualties is related to damage and collapse of traditional *Shinkabe* and *Okabe* dwellings which were constructed without the benefit of seismic and fire resistant design. The collapse of houses also triggered fires from broken gas lines (Fig. 3-12).

This earthquake was centered in a highly dense urban area (Fig. 3-13), more so by far than most American cities except areas like downtown Manhattan, New York where density is achieved in high-rise buildings. For example, the density in the traditional

Fig. 3-12. Damage to traditional housing in Nishinomiya. The Hyogo-Ken Nanbu, Japan earthquake, January 1995 (photo from EERI, Ref. 7).

Fig. 3-13. Nishinomiya in Kobe is an example of the dense aspect of Japanese cities. The 1995 Hyogo-Ken Nanbu, Japan earthquake (photo from EERI, Ref. 7).

Fig. 3-14. The Hanshin Expressway *rolled over* in places. The 1995 Hyogo-Ken Nanbu, Japan earthquake (photo from EERI, Ref. 7).

residential area of Oakland, California is about one-third that of Osaka, Japan (Ref. 6). Damage to transportation structures was extremely heavy (Fig. 3-14). At port facilities, where liquefaction, block sliding and soil compaction were caused by heavy shaking (lateral accelerations of 0.8g) major damage was widespread (Fig. 3-15).The tragic earthquake in Kobe provided an unusual opportunity to view the actual effects of a strong, near-field earthquake on modern engineered buildings in a high rise urban environment. The firm of Degenkolb Engineers, San Francisco, California, was fortunate to have highly qualified earthquake engineers in nearby Osaka when the temblor struck. The firm's reconnaissance team (Loring Wylie, David Bonneville, and John Halle) returned with many useful observations and insights which are summarized by David Cocke and reprinted with permission from David R. Bonneville, Principal, Degenkolb Engineers, San Francisco and Los Angeles.

Fig. 3-15. Liquefaction and block sliding at quay wall, Port of Kobe. The 1995 Hyogo-Ken Nanbu, Japan earthquake (photo from EERI, Ref. 7).

"What We Learned from the Kobe Earthquake"

"Buildings Within a Few Kilometers of the Fault

"Most of the significant damage was concentrated along a band, only a few kilometers in width, stretching along the fault line from Awaji Island northeast through downtown Kobe. It was remarkable how buildings collapsed predominantly in the north-south direction, i.e., at right angles to the fault. Directional effects are not normally considered in evaluating and designing buildings, but will undoubtely be taken into account in the future.

"How Building Shape Affected Performance

"As is always the case, buildings with regular shapes or configurations (vertically and in plan) fared much better than those with irregularities. The collapsed midrise concrete frame buildings in downtown Kobe most clearly demonstrated this. There were complete collapses in soft or weak stories, often at the lowest level where there were retail shops, and in stories where the structural system was changed. Although building codes recognize the effects of such irregularities, they probably do not sufficiently penalize them.

"Building Foundations in Soft Soil

"Some of the highest ground accelerations were recorded in the softer soil areas closest to the coast and on the off-shore man-made islands. There were no collapses of buildings with good lateral systems and supported on deep foundations. Even where there were ground displacements of several feet, structures on pile foundations did not suffer serious damage. This is valuable information since the building codes for foundation seismic capacity and ductility are currently being re-evaluated and data on foundation performance are scarce.

"Concrete Buildings of a Certain Age (20+ Years)

"Buildings of twenty years or more with concrete shear walls did much better than those with concrete frames. One of the reasons is that concrete frames of this age and older are generally non-ductile, that is, they lack the reinforcement necessary to prevent brittle failures. As long as the building shape was regular however, concrete wall systems did not collapse, even though they were not detailed to meet current code requirements. This confirms generally held opinions on concrete systems of this vintage.

"How Steel Moment Resisting Frame Buildings Fared

"The earthquake was a real test for steel buildings constructed like those in the United States over the past few decades. They were subjected to very strong ground motion and generally did not collapse although a few older ones, with details very different from U.S. practice, suffered partial collapse or were severely damaged. There was evidence of damage at the joints in some modern moment frames similar to that discovered after the Northridge earthquake.

"Modern Japanese Buildings

"The seismic provisions of the Japanese Building Code were significantly changed in 1971 and 1981. Among other things, the changes substantially improved the seismic capacity and ductility of reinforced concrete and steel buildings. Many of these changes have paralleled our own in the Uniform Building Code as there has been substantial collaboration between the two countries. As a result of these developments, damage to buildings constructed since the mid-'70s was much lower than in those constructed prior to that. In comparing modern building performance in Kobe to that in Northridge, although there were similarities with the steel moment frame problem as noted above, there were major differences in two other areas. The many collapses of concrete parking structures and tilt-up buildings in Northridge was not a factor in Kobe. This is due to the predominance of steel braced frame parking garages in Japan (rather than precast concrete) and the general absence of any system similar to our troubled tilt-up construction" (end of reproduced portion of report).

Reference 8, *The January 17, 1995 Kobe Earthquake, An EQE Summary Report,* provides a comprehensive description of the effects of the Hyogo-ken Nanbu earthquake including earth science aspects, buildings, industrial facilities, transportation, ports, other lifelines, fire following earthquake, economic impact, and societal impact as well as an explanation of important differences between the Kobe and Northridge earthquakes.

With the exception of Japan, U.S. earthquakes have had more impact on American engineers than foreign earthquakes. In Chapter 3a, however, James Stratta has used many examples of earthquake damage that occurred in foreign countries other than Japan. American engineers tend to discount foreign experience because they are unfamiliar with the foreign codes used for analysis and the quality control used in construction. For the most part, however, the lessons illustrated with foreign examples are rather common to U. S. experience. In a way, the foreign examples tend to reinforce an important point of this publication. That is, *that most problems in earthquake damage are simply the result of not implementing what has been known and observed about earthquake engineering for many years, and are not the result of lack of sophistication in analysis. We still need to understand and avoid the failures identified in old lessons.*

References

1. *Northridge Earthquake January 17, 1994, Preliminary Reconnaissance Report,* Earthquake Engineering Research Institute, Oakland, California, March 1994.

2. *Northridge Earthquake of January 17, 1994, Reconnaissance Report,* Vol. 1, Supplement C to Vol. 11, Earthquake Spectra, The Professional Journal of the Earthquake Engineering Research Institute, Oakland, California, April 1995.

3. *The January 17, 1994 Northridge, California Earthquake, An EQE Summary Report,* EQE International, San Francisco, California, March 1994.

4. *Performance of Steel Building Structures During the Northridge Earthquake,* UCB/EERC-94/09, V.V. Bertero, J.C. Anderson, H. Krawinkler, Earthquake Engineering Research Center, University of California, Berkeley, California, August 1994.

5. *Competing Against Time,* The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake, G.W. Housner, Chairman, State of California, Office of Planning and Research, North Highlands, California, May 1990.

6. *The Continuing Challenge, The Northridge Earthquake of January 17, 1994,* Report to the Director, California Department of Transportation by the Seismic Advisory Board, G.W. Housner, Chairman, Sacramento, California, October 1994.

7. *The Hyogen-Ken Nanbu Earthquake, January 17, 1995,* Preliminary Reconnaissance Report, Earthquake Engineering Research Institute, Oakland, California, 1995.

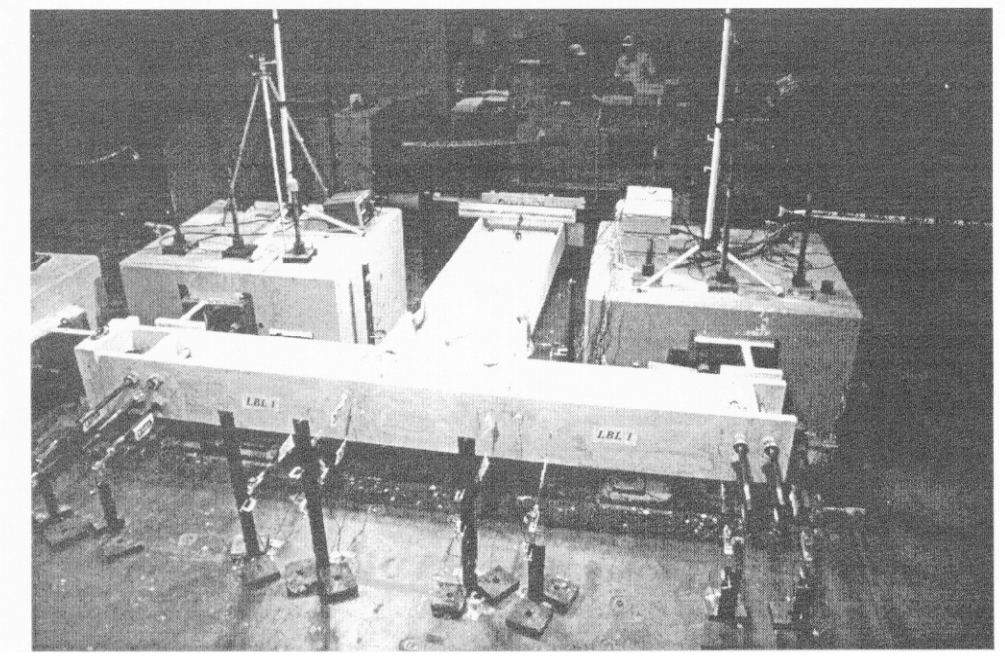
8. *The January 17, 1995 Kobe Earthquake,* An EQE Summary Report, EQE International, San Francisco, California, April 1995.

Appendix A - Chapter 3 Foreword

LAWRENCE BERKELEY NATIONAL LABORATORY

HUMAN GENOME LABORATORY
STEEL JOINT TEST

TECHNICAL BRIEF



FORELL/ELSESSER ENGINEERS, INC
160 PINE STREET #600
SAN FRANCISCO, CA. 94111

July 26, 1995

Lawrence Berkeley National Laboratory
Steel Joint Test - Technical Brief

July 26, 1995

Mark Jokerst, SE
Forell/Elsesser Engineers
160 Pine Street #600
San Francisco, CA 94111

In conjunction with normal structural engineering services provided for the design of a three building two- and three-story Human Genome Laboratory complex at the Lawrence Berkeley National Laboratory (LBNL) site, a study was undertaken to develop and test a beam-column connection detail for the Special Moment Resisting Frames (SMRF) selected as the structural lateral system for this facility. A technical summary of the connection design and testing is provided herein for public dissemination.

While there are no restrictions on the use of the concepts described herein, a warranty or guarantee is neither expressed nor implied in this report. Other engineers are cautioned that this detail is not suitable for all geometries and depends on limiting stresses in the column and panel zone to less than the elastic limit, depends on inelastic buckling of the beam flange, and depends on the use of high quality welding processes, materials, and diligent quality assurance personnel. It is highly recommended at this stage in the redevelopment of the SMRF joint connection, additional testing be performed to validate any design.

Connection Description: The connection detail is shown in Figure 1. The top and bottom cover plates are identical: Where the plate abuts the column flange, the plate width matches the column width dimension and tapers down to two inches greater than the beam flange width. The full width at the column interface is intended to present as uniform as possible distribution of stress in the cover plate and in the column joint region. This also allows easy removal of weld tabs and facilitates inspection of the weld end after grinding.

The cover plate is welded to the column flange with a complete joint penetration groove weld; the beam flange is *not* welded to the column. The bottom cover plate is shop welded to the column while the top cover plate is shop welded to the beam, using longitudinal fillet welds. This arrangement offers several positive features: The total cross sectional weld area is reduced which has cost and performance benefits. Inspection of the groove welds is facilitated because there is no plate-to-plate interface to cause erroneous discontinuity reflections. Beam fabrication time is lessened with the absence of flange groove preparations and only a single weld access hole at the top. Field erection time is lessened with only one down hand groove weld on the top flange (where the interruption of the beam web does not occur) and two down hand

fillet welds on the bottom flange.

The beam web is attached to the column using a trapezoidal web plate. The web plate is connected to the beam in the field with fillet welds and to the column in the shop with double sided vertical position groove welds. There is no weld between the beam web and column flange. The plate is tapered to facilitate welding of the top and bottom legs of the fillet weld.

Within the column, top and bottom continuity plates are positioned to align with the cover plates and welded with complete joint penetration groove welds. A quarter-circular backing bar was used at the column fillet to prevent welding into the fillet of the column. At the shop welded bottom cover plate, a plate stiffener is added to prevent inadvertent stressing of the groove weld during shipment or erection. This stiffener was removed prior to testing.

Connection Design Parameters: The detail is intended to force plastic action in the beam to occur away from the welded region. The length of cover plate is approximately one nominal beam depth from the column centerline, however, longer relative lengths may be required for beams of lesser depth in order to provide sufficient length of weld between cover plate and beam flange. Beams were proportioned to satisfy stress and drift requirements with the additional requirement that excessive beam strength must be minimized. Columns were proportioned to satisfy Uniform Building Code (UBC) panel zone strength requirements without the aid of column web doubler plates. Strong column-weak beam response was assured by plastic frame analysis of all possible mechanisms, accounting for the additional flexural demands imposed on the column by the eccentric plastic hinge location. Specific design parameters and specifications include:

Beams: ASTM A572 Grade 50, no cap. For design of the connection, the mean plus one deviation of the SAC 95-01 Table A (Appendix D) data (Mean yield strength of 57.6 ksi and Standard Deviation of 5.1 ksi), plus 10% strain hardening factor, less 5% web to flange strength conversion is used, yielding a total effective $F_y^* = 65.5$ ksi.

Columns: ASTM A572 Grade 50, fine grained fully killed, grain size 6 or finer, Charpy V-notch toughness determined at centerline intersection of web and flange of 20 foot-pounds at 40°F. In all calculations, possible material over-strength is neglected.

Plates: ASTM A572 Grade 50, rolling grain to be parallel to beam axis. Flexural stress at cover plate-column interface: 40 ksi.

Specified Welding Materials and Techniques: Project specifications include special provisions for all welds used in the SMRF connection. The term "SR Weld" for

Seismic Resistant Weld is used to indicate the application of these requirements on the drawings. These special requirements include:

Pre-fabrication and pre-erection conferences; submittal of detailed welding procedure specifications; submittal of welding inspection checklists; re-qualification of welders using high-grade materials; 100% ultrasonic inspection and the requirement that the inspection agency provide shop and field inspectors with mag particle or dye penetrant devices for their use.

For FCAW and SMAW welding processes: Charpy V-Notch toughness of 20 foot pounds at minus 20°F; preheat requirements roughly 50° higher than AWS D1.1; post-heat cool-down provisions; peening requirements; grinding of all thermal cut surfaces; removal of weld tabs; removal of weld backing strips.

Test Procedure and Acceptance Criteria: Two specimens were tested at the Davis Hall Laboratory at the University of California at Berkeley (UCB), under the direction of Professor Egor Popov and project manager Marcial Blondet. The testing arrangement and protocol are identical to the SAC test procedures.

Nonlinear dynamic analysis of the building frame was used to establish the test acceptance criteria. Building frames were designed in accordance with LBNL seismic criteria which specified an allowable stress design (ASD) static base shear of 20% for two buildings and 28% for the third building. The analysis indicated that plastic beam rotations of about 0.02 radian could be expected for a Hayward event. (For the buildings under consideration, the total beam rotations are in the range of 120% to 140% of the maximum interstory drift.) Allowing for the limited accuracy of structural calculations, and allowing for a higher than predicted event, test acceptance criteria were established to achieve at least three cycles that result in plastic beam rotations of 0.03 radian, computed at the plastic hinge location.

Specimen Features and Properties: Material strengths were verified by mill certificates and by independent lab testing by Signet Testing Laboratories. (All stress in ksi):

	Fy	Fu	Elong(%)	CVN[1]	Av. Grain Size	Tested by
W30x90 Beam:	60	71	26			Mill Certificate
Web	61.1	70.4	36.5			Signet Labs
Flange	50.3	66.1	41.0			Signet Labs
W14x283 Column:	51	70	29			Mill Certificate
Web	48.5	67.5	32.5			Signet Labs
Flange	45.5	67.5	32.5	21.4ft-lbs @40°F	#6-#7	Signet Labs
Cover Plates:	61.4	88.2	22			Mill Certificate
	61.5	87.5	28.0			Signet Labs
Web Plates:	60.8	82.2	23			Mill Certificate

(1) = Charpy V-Notch

Specimens were fabricated at The Herrick Corporation's Stockton, California facility. All shop welds were made using standard shop practices. Field erection was simulated in the shop with the column vertical and the beam temporarily shored. Groove welds were made with Lincoln NR-311 FCAW wire. Fillet welds were made with Lincoln NR-232 FCAW wire. Specimens were fitted with potentiometer, strain gauges and strain rosettes.

Specimen 2 was tested with a 4 inch long by 3/16 inch deep "crack" in the column flange, located on the flange centerline, starting at the cover plate weld and proceeding away from the connection. The crack was discovered by Davis Hall Laboratory technicians after grinding the area for instrumentation. The crack was identified as an acceptable (AWS D1.1) defect by Signet Testing Laboratories. The testing team elected to proceed with the test without repairing the crack.

Results of Testing: Figure 2 presents beam load versus tip displacement and beam load versus plastic beam rotation hysteresis curves for specimens 1 and 2. Both specimens met the acceptance criteria of three cycles of 0.03 radian plastic beam rotation. Generally, the tests proceeded as follows:

1 inch cycles: (Yield occurs at 1.1 inches) Very minor flaking of the paint.

2 inch cycles: ($\theta_p = .0005r$) Significant yield and plastic rotation, no buckling. Ram load peaks at 148 kips.

3 inch cycles: ($\theta_p = .0014r$) Flange buckles before reaching 3 inch displacement. Distorted flanges straighten upon reversed loading. Ram load repeats up to 148 kips in first cycle, subsequent ram loads show strength degradation.

4 inch cycles: ($\theta_p = .0024r$) Flanges and web buckle, distortion remains in constant form with each cycle, minor movement of flanges. Ram load diminishes to yield level (105 kips).

5 inch cycles: ($\theta_p = .0034r$) Same behavior as four inch, rotation peaks at $\theta_p = .0039r$. Ram load decreases uniformly with each cycle.

Specimen 1 was tested up to 3 cycles at the 5 inch displacement. Specimen 2 was tested up to 10 cycles at the 5 inch displacement whereupon a beam flange fractured in the buckled region due to high strain cyclic fatigue. Also, for specimen 2, there was evidence of minor yield near the pre-test column crack, but the crack did not appear to open or propagate during the tests.

Acknowledgments: This study, including value engineering, design, fabrication, and testing, was funded by the U.S. Department of Energy for the design and construction of the Human Genome Laboratory at the Lawrence Berkeley National Laboratory, as

Certainly one of the most important considerations in seismic design relates to the relative rigidities of the elements taking the lateral load and their capabilities to resist load. Lack of consideration for relative rigidities is exemplified in the type of construction utilizing infilled walls with concrete frames, which is used predominantly in many countries throughout the world. The infill material is usually hollow tile, block, or brick: very brittle, but not reinforced and usually not capable of resisting the load attracted by such rigid construction.

Fig. 3a-1 shows the Marionist School in the Chimbote, Peru earthquake of May 31, 1970 (Ref. 2). Because a relatively rigid transverse partition was located every third column, each attracted a large part of the load, but was unable to resist it. Note the severe damage to the column at the infill wall relative to the column between walls. It is quite probable that the intermediate column failed only after the column at the infilled frame failed.

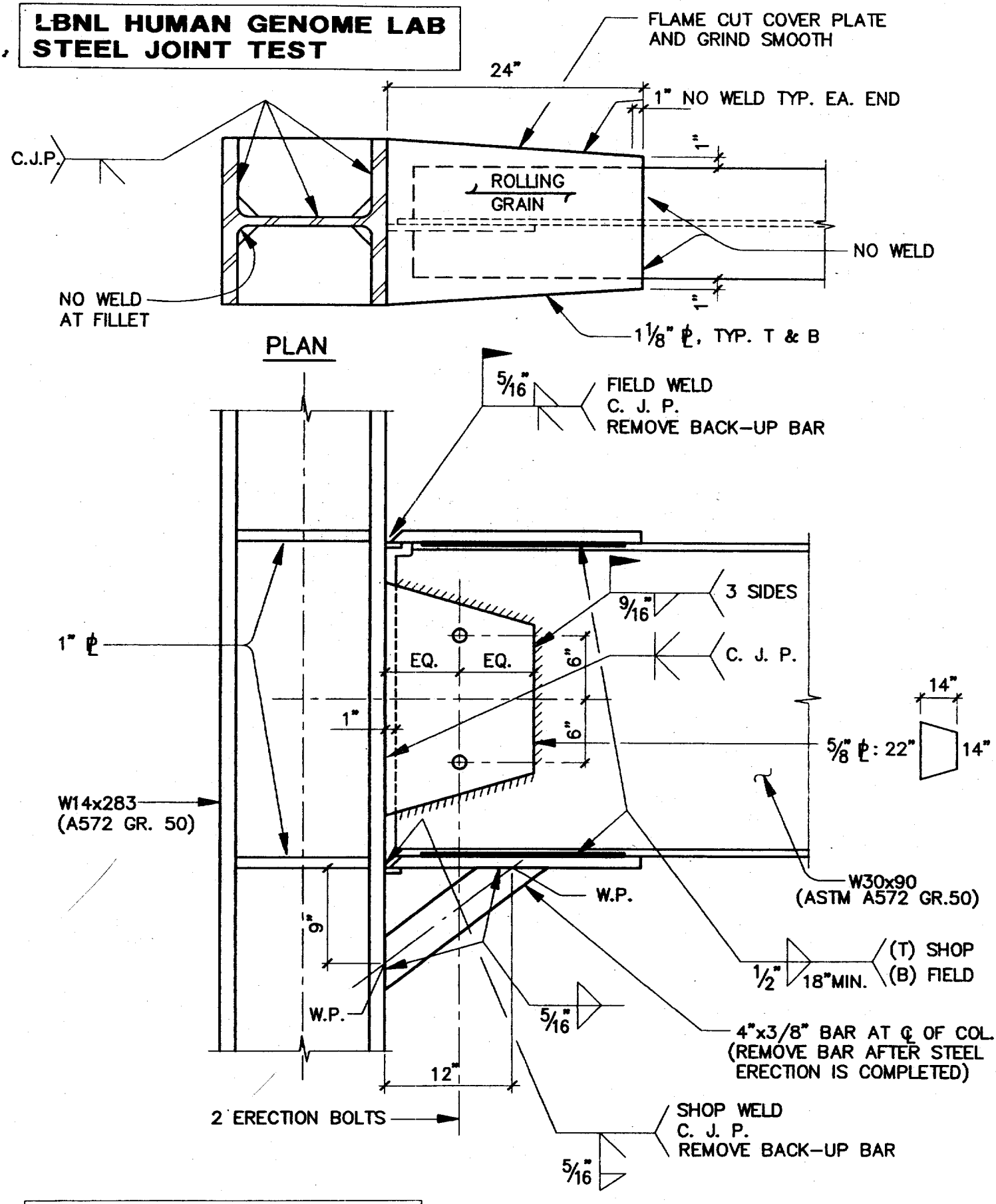
Fig. 3a-2 shows little discernible damage in the Elite Condominium after the Northern Italian earthquake of May 6, 1976 (Ref. 3), but Fig. 3a-3 shows otherwise in a closeup of the end wall column at the first story. Note that the end frame once had a tile-infilled wall, most of which has been destroyed. The rigid infill attracted the lateral load and caused the column failure. The interaction of the rigid infill with the less-rigid column was very damaging to the column. The entire structure totally collapsed in a later aftershock. Fortunately, all of the tenants had moved out.

In Fig. 3a-3, note the lack of ductile reinforcing in the column. Ductility is the ability to deform inelastically without abruptly failing. Early on, it was noted that ductility is necessary in concrete frames in order to develop toughness. In 1960 the Portland Cement Association published a book by Blume, Newmark and Corning (Ref. 4) demonstrating how to achieve ductility in concrete. Design for ductility and ductile reinforcing is a highly technical specialty. Designers who wish to become proficient in earthquake engineering must understand how buildings respond, deform, and distort, considering the effect of relative rigidities and other factors, and be able to cope with them in the design process.

The Philippine Bar Association Building (Fig. 3a-4), damaged in the Manila earthquake of 1968, dramatically illustrates the role of ductility. Fig. 3a-5 shows a spirally reinforced column, and Fig. 3a-6 shows a tied column.

Fig. 3a-1. Marionist School, Chimbote, Peru, May 31, 1970 earthquake.

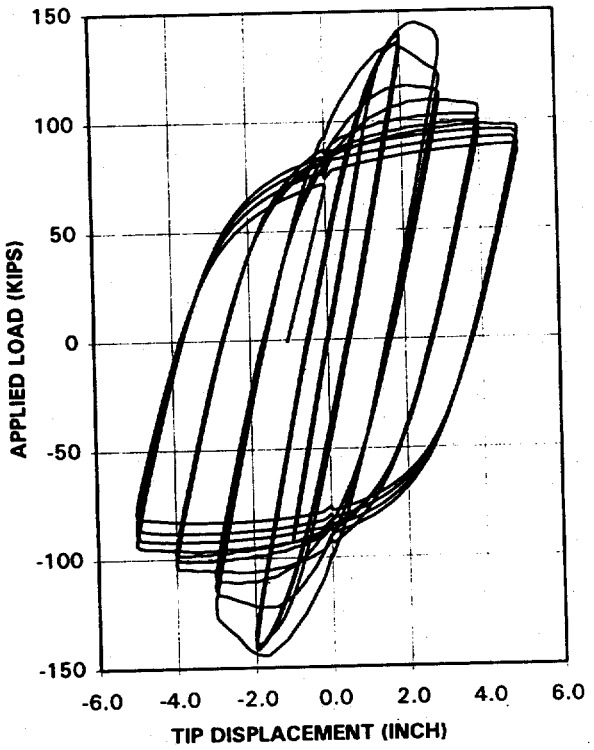
LBNL HUMAN GENOME LAB
STEEL JOINT TEST

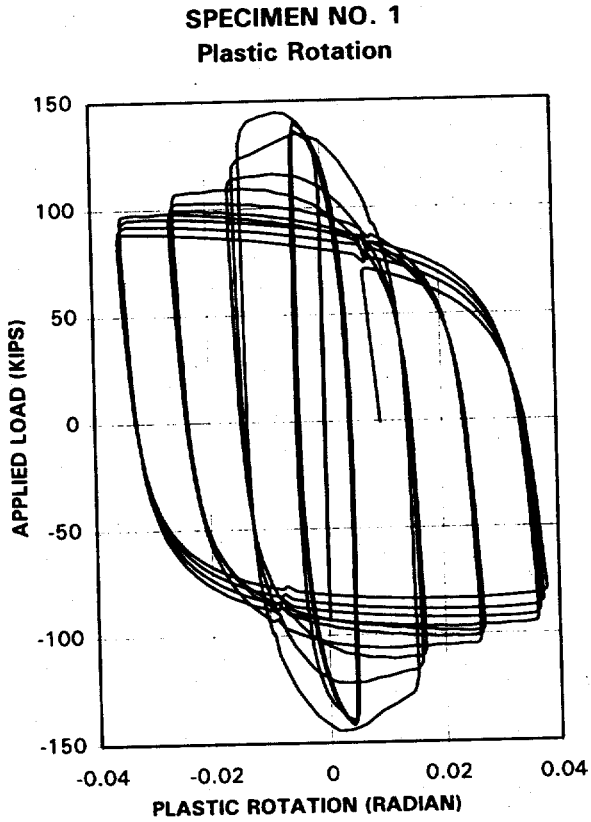


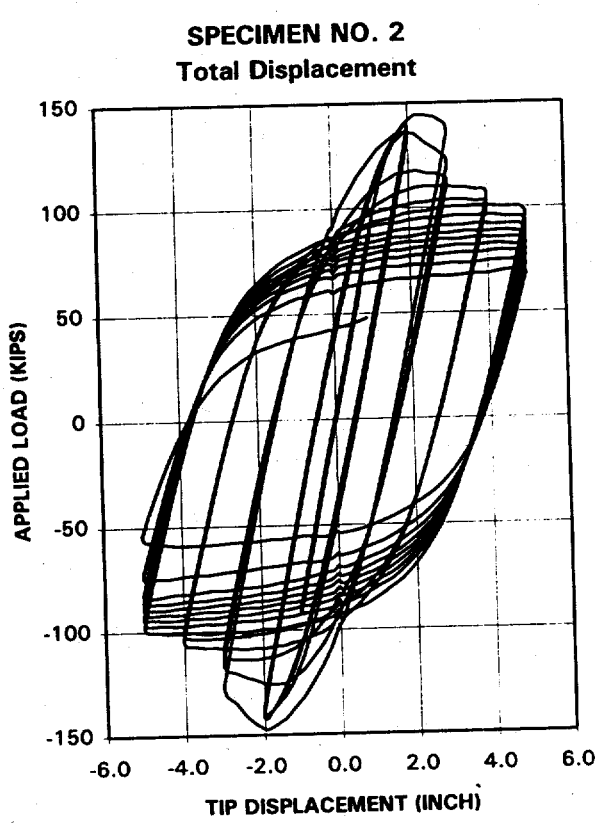
NOTE: ALL WELDS SHALL BE "SR" WELDS – SEE SPECIFICATIONS.

Figure 1
JOINT DETAIL

Forell/Elsesser Engineers, Inc.
March 24, 1995


SPECIMEN NO. 1
Total Displacement
APPLIED LOAD (KIPS)
150
100
50
0
-50
-100
-150
-6.0
-4.0
-2.0
0.0
2.0
4.0
6.0
TIP DISPLACEMENT (INCH)


SPECIMEN NO. 1
Plastic Rotation
APPLIED LOAD (KIPS)
150
100
50
0
-50
-100
-150
-0.04
-0.02
0
0.02
0.04
PLASTIC ROTATION (RADIAN)


SPECIMEN NO. 2
Total Displacement
APPLIED LOAD (KIPS)
150
100
50
0
-50
-100
-150
-6.0
-4.0
-2.0
0.0
2.0
4.0
6.0
TIP DISPLACEMENT (INCH)

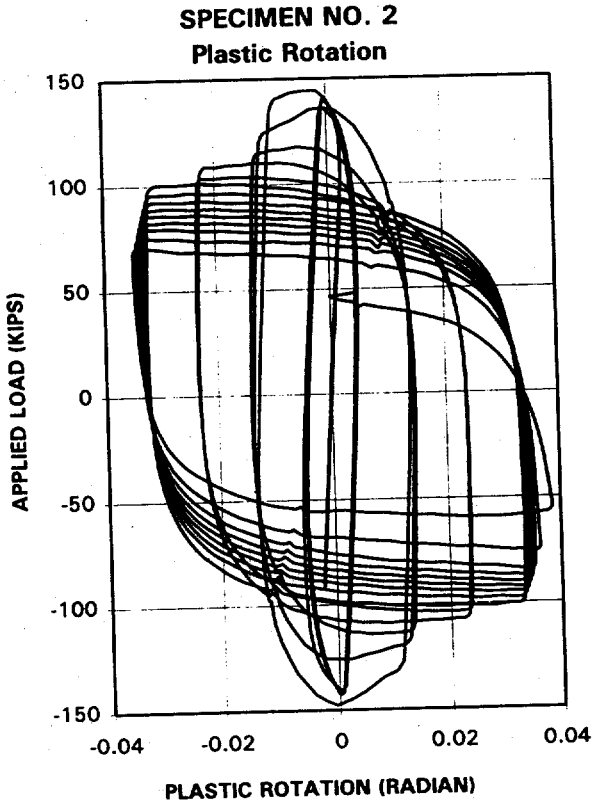

SPECIMEN NO. 2
Plastic Rotation
APPLIED LOAD (KIPS)
150
100
50
0
-50
-100
-150
-0.04
-0.02
0
0.02
0.04
PLASTIC ROTATION (RADIAN)

FIGURE 2

Chapter

3a

Types of Damage

James L. Stratta*

Introduction

Laboratory testing of materials has long been the engineer's best means of determining or predicting the physical properties of materials used for design purposes. Furthermore, full-scale assemblies of materials such as structural joints and wall sections are often tested to enable engineers to more accurately predict their behavior and rely upon certain assumptions necessary for design. Complete building structures, however, are very difficult and expensive to test, and it is virtually impossible to introduce realistic earthquake ground motions into such tests.

For the purpose of testing seismic performance there is no substitute for real earthquakes. They provide the ultimate in full-scale testing of large structures such as buildings, dams, bridges, roads, tanks, and utilities systems. Also, geologic phenomena such as faulting, liquefaction, heaving, subsidence, and grabens can be carefully observed and studied in the aftermath of an earthquake.

There is an old saying that *a picture is worth a thousand words*. A new saying is proposed: *A visit to the scene of a damaging earthquake is worth a thousand lectures*. It is a profound experience for structural engineers to observe earthquake damage first-hand. They can see what is good construction, what is bad, and study the actual modes of failures. The experience improves and develops their judgment for structural detailing and selecting building configurations, and generally upgrades the overall quality of their work. Unfortunately, many structural engineers do not have the opportunity to visit an area damaged by an earthquake, so they must study reports and pictures of earthquake damage to learn these lessons. The pictures and descriptions that follow are intended to communicate the earthquake investigator's point of view about earthquake damage.

The discussion is divided into several categories. Readers should keep in mind that seismic design is not a science, but an art. In many cases, varying reasons may be given, or varying assumptions made, as to the cause of earthquake damage. This is a healthy situation, because from many of these controversies have come the answers we were looking for. If readers should find what they consider to be contradictory statements herein, they should remember that we are dealing with an art.

Buildings

The interest of structural engineers lies predominantly in the building itself. Why has the building failure occurred? Why did other buildings not fail? What have been the most common or consistent types of failures? What force levels would cause damage? These and many more questions race through structural engineers' minds during an inspection trip.

* The late Mr Stratta is the author of the *Manual of Seismic Design* (Ref. 1).

Certainly one of the most important considerations in seismic design relates to the relative rigidities of the elements taking the lateral load and their capabilities to resist load. Lack of consideration for relative rigidities is exemplified in the type of construction utilizing infilled walls with concrete frames, which is used predominantly in many countries throughout the world. The infill material is usually hollow tile, block, or brick: very brittle, but not reinforced and usually not capable of resisting the load attracted by such rigid construction.

Fig. 3a-1 shows the Marionist School in the Chimbote, Peru earthquake of May 31, 1970 (Ref. 2). Because a relatively rigid transverse partition was located every third column, each attracted a large part of the load, but was unable to resist it. Note the severe damage to the column at the infill wall relative to the column between walls. It is quite probable that the intermediate column failed only after the column at the infilled frame failed.

Fig. 3a-2 shows little discernible damage in the Elite Condominium after the Northern Italian earthquake of May 6, 1976 (Ref. 3), but Fig. 3a-3 shows otherwise in a closeup of the end wall column at the first story. Note that the end frame once had a tile-infilled wall, most of which has been destroyed. The rigid infill attracted the lateral load and caused the column failure. The interaction of the rigid infill with the less-rigid column was very damaging to the column. The entire structure totally collapsed in a later aftershock. Fortunately, all of the tenants had moved out.

In Fig. 3a-3, note the lack of ductile reinforcing in the column. Ductility is the ability to deform inelastically without abruptly failing. Early on, it was noted that ductility is necessary in concrete frames in order to develop toughness. In 1960 the Portland Cement Association published a book by Blume, Newmark and Corning (Ref. 4) demonstrating how to achieve ductility in concrete. Design for ductility and ductile reinforcing is a highly technical specialty. Designers who wish to become proficient in earthquake engineering must understand how buildings respond, deform, and distort, considering the effect of relative rigidities and other factors, and be able to cope with them in the design process.

The Philippine Bar Association Building (Fig. 3a-4), damaged in the Manila earthquake of 1968, dramatically illustrates the role of ductility. Fig. 3a-5 shows a spirally reinforced column, and Fig. 3a-6 shows a tied column.

Fig. 3a-1. Marionist School, Chimbote, Peru, May 31, 1970 earthquake.

Fig. 3a-2. Elite Condominium, Northern Italy, May 6, 1976 earthquake.

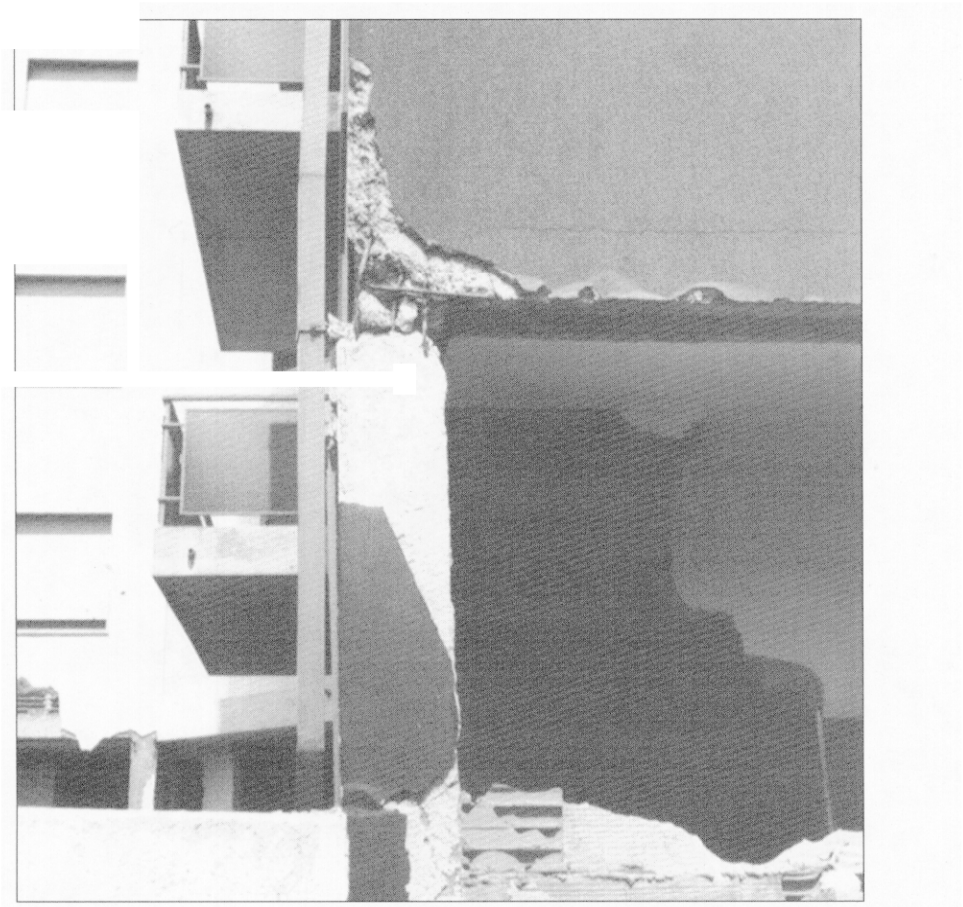


Fig. 3a-3. Lack of ductile reinforcing in first-story end-wall column, Elite Condominium.

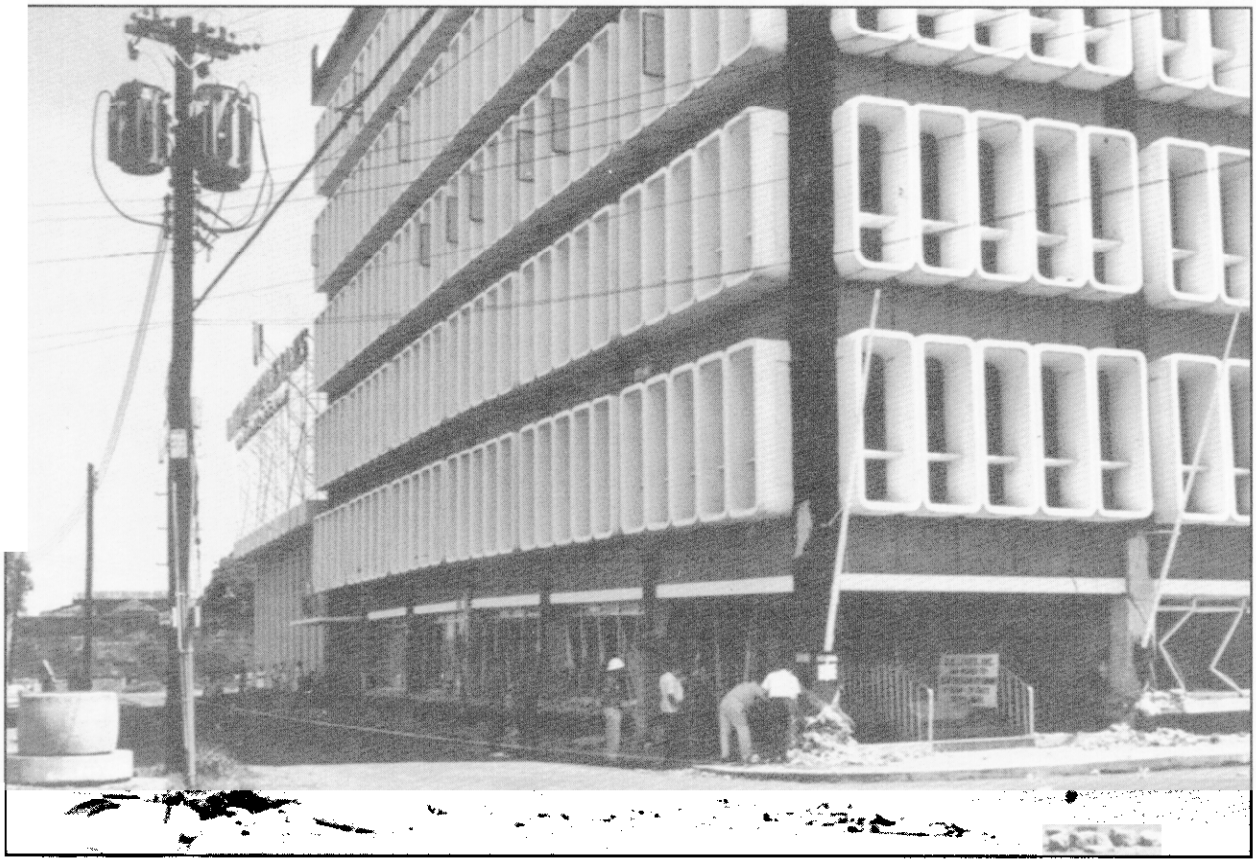


Fig. 3a-4. Philippine Bar Association building, Manila, 1968 earthquake.

Fig. 3a-5. Spirally reinforced column in Philippine Bar Association building.

Note the obvious capability of the spiral reinforcing to confine the concrete so the column can deform and continue to take load, ultimately failing in a ductile manner. On the other hand, in the tied column, the concrete has shattered and *flowed out* from between ties in abrupt failure.

Another of the many lessons learned from observation is the effect of torsion on buildings and how it may be introduced into a structure by the unsymmetrical location of load-resisting elements. Fig. 3a-7 shows the J.C. Penney building after the March 27, 1964, earthquake in Anchorage, Alaska (Ref. 5). Torsion was certainly an important factor contributing to damage.

It is not necessary to inspect large structures to understand what actually happens to buildings. The Mercado Modelo (Figs. 3a-8 and 3a-9) in Huarmey, Peru, shown after the May 31, 1970, earthquake (Ref. 2), is a classic example of the effects of varying rigidities combined with the effects of torsion and lack of ductile reinforcing. Reference 1 includes calculations for this structure.

Fig. 3a-6. Tied column in the Philippine Bar Association building.

Fig. 3a-10 shows a one-story classroom structure at the Agricultural University in Lima, Peru after the October 3, 1974, earthquake (Ref. 6). Here the structural combination of elements with different relative rigidities and a site situated in a localized area of high-intensity shaking (a microseismic area to be discussed later) were the chief causes of damage. Fig. 3a-11 pinpoints the failure at the columns just below the zone of closely spaced ties (ductile reinforcing) in this classroom building.

Information can be learned from undamaged elements as well as from those that failed. In the Mindanao, Philippines, earthquake of August 17, 1976 (Ref. 7), the New Society Hotel (Fig. 3a-12) rotated and collapsed. The Harvardian College Building (Fig. 3a-13) also collapsed. However, the Tison Building (Fig. 3a-14), designed by Filipino engineers using the California Code (exact year not known), suffered no structural damage. Flower pots on the roof (Fig. 3a-15) moved approximately 8 centimeters (3 inches),

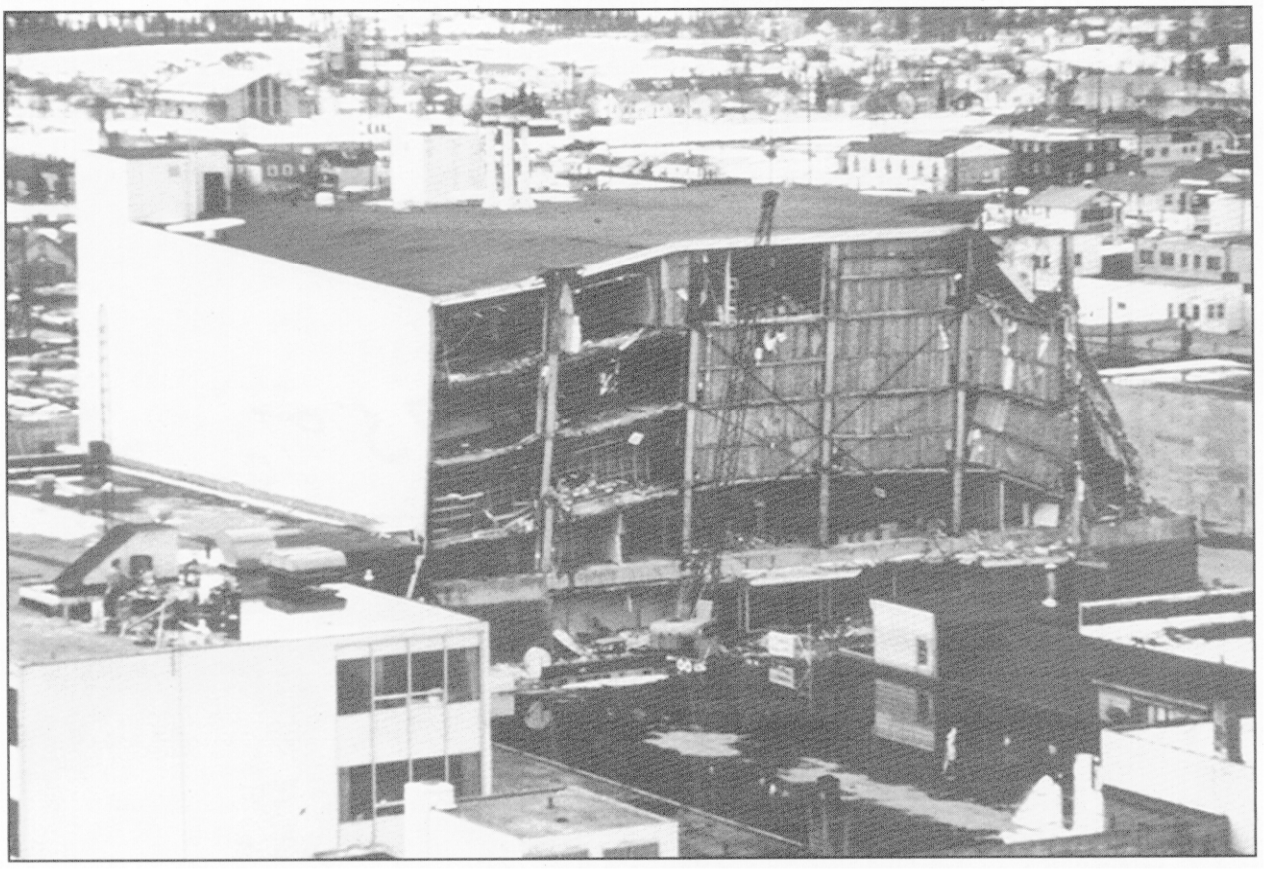


Fig. 3a-7. J.C. Penny building, Anchorage, Alaska, March 27, 1964 earthquake.

indicating that the building experienced a peak acceleration of about 40-50% or more at the roof.

This does not mean that a lateral force coefficient of 40% or more should have been used in the design because the building was designed for a much lower static lateral load, yet sustained no serious damage from the earthquake.

Earthquake engineers are not in agreement on the proper *force coefficients* to be used in design. Some feel the coefficients specified in the newest codes are higher than necessary, while others feel they are too low. Some feel dynamic analyses should be used for all structures, and others feel that knowledge about earthquakes has not progressed sufficiently to warrant this type of mathematical accuracy. Still others believe that we do not have all of the necessary and proper input for computer analyses.

Force coefficients and damage sustained during earthquakes are not always directly related to the magnitude or peak ground acceleration induced by an earthquake. The Earthquake Engineering Research Institute sent reconnaissance teams to both the 1970 (Ref. 2) and 1974 (Ref. 6) earthquakes in Peru. Each earthquake gave a surface wave magnitude (Ms) of about 7.5, yet the damage differed vastly. This difference was presented to Dr. Bruce A. Bolt, seismologist at the University of California at Berkeley, for explanation. He responded with a paper at the Earthquake Engineering Research Institute's annual meeting on February 8, 1975, which is published as Appendix A of Reference 6. This paper is strongly recommended to those interested.

In this paper, Dr. Bolt stated that, *the evidence is growing that there is no strong correlation, at least in the near field, between magnitude and peak acceleration.* Thus, such questions as: Will this building withstand a 7.0 magnitude earthquake? To what magnitude should we design this structure? are not very useful.

In the past two decades the development of many computer software programs has simplified the agony of some tedious engineering calculations. However, a strong word of caution is necessary to the users of software programs. It is not sufficient merely to learn how to use the

Figs. 3a-8 and 3a-9. Mercado Modelo, Huarney, Peru, May 31, 1970 earthquake.

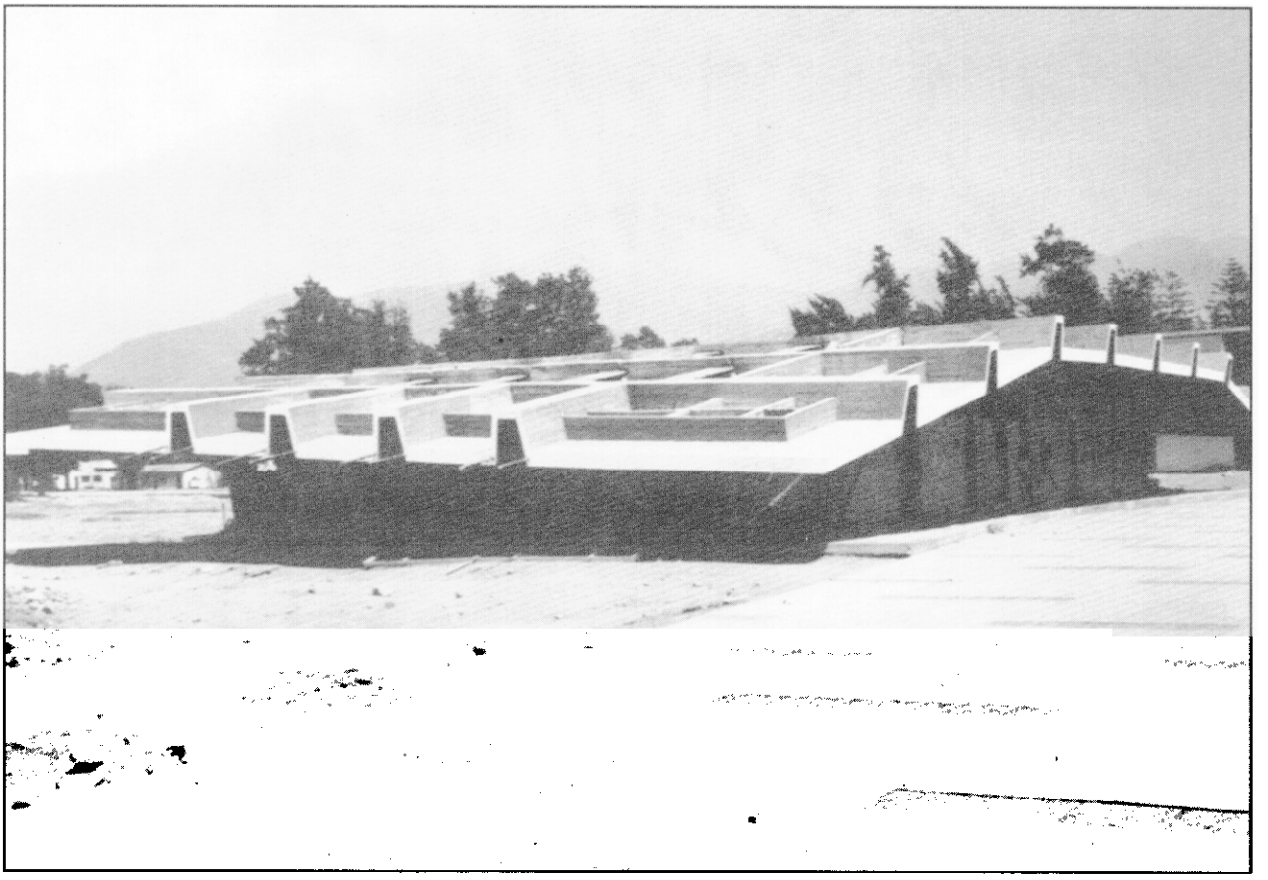


Fig. 3a-10. One-story classroom structure at the Agricultural University, Lima, Peru, October 3, 1974.

program; it is imperative to know how the program was developed and what assumptions have been made by those who wrote the program. Software users should go through the development of the program step by step to see whether or not they agree with and understand all of the assumptions made by the software writers.

For example, from the programs of truss design, one may find a program that assumes pin-ended conditions, another that assumes fixed-ended conditions, and another where users may assign some degree of fixity. Users must understand which assumption has been made. Also, for example, wood trusses may require a different program than that used for steel trusses.

The need to understand the assumptions incorporated in software for seismic analysis is very important because earthquakes find invalid assumptions very quickly, particularly when they cause large stress reversals and inelastic behavior. In the July 1985 issue of *Concrete International* (Ref. 8) an article showed that then existing programs could give results that differed by more than a factor of 2 (one could probably guess closer). *The lesson is that users of software programs should be thoroughly aware of assumptions and conditions that the writers are using.*

Plan Check and/or Peer Review

There are many facets of earthquake damage, and to determine methods for preventing such damage, experienced and well-qualified consultants should be engaged. Furthermore, some kind of review procedure should be adopted to check plans before implementation. While some cities and some governmental agencies have excellent review procedures, others have procedures that are almost meaningless. Usually, the task of making certain that plans are properly checked seems to fall to owners or facilities managers. Large or important structures, such as sporting arenas, large meeting halls accommodating several hundred or more persons, or important facilities housing nuclear material or other dangerous substances, should have an

independent review *(Peer Review)* made of at least the structural design of the building. This *peer review* should be made by a firm or engineer whose technical reputation is equal or superior to the firm designing the structure.

This type of peer review should not be merely a cursory review, rather it should be a very thorough review. To be truly effective, it must include a review of the selection of codes and specifications utilized, calculations, assumptions, plans, and specifications. It also should include consultations with the original designers should questions concerning any of the above issues arise. Chapters 6a and 12a have more detailed discussions of peer review.

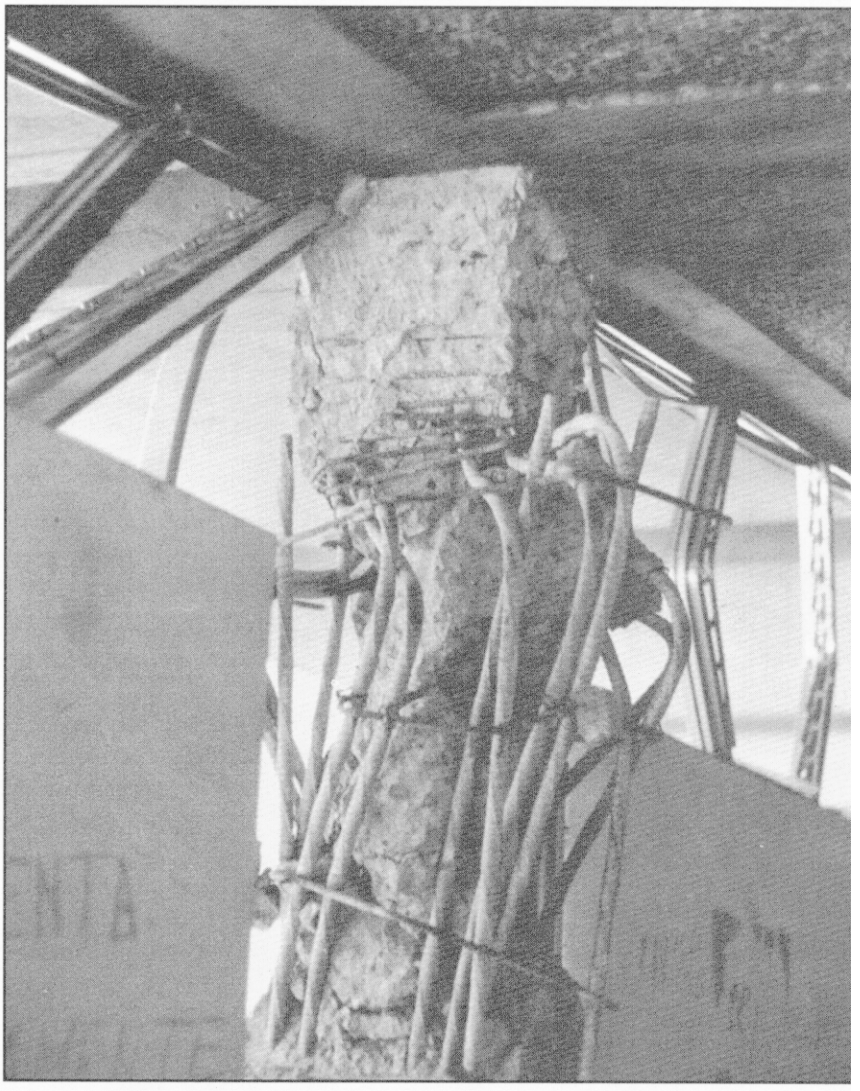


Fig. 3a-11. Column failure in Agricultural University class-room structure of Fig. 3a-10.

This may seem to be an extraordinary expense to owners or managers of facilities. The question is why would a firm be commissioned to design a structure if the design must be checked by another firm? This is indeed a good question; however, one has but to look back to major structures that have failed that were designed by competent firms to realize that engineers are not infallible. Peer review of the design for important facilities is not only a very realistic approach, it is cost-effective risk management. At times, firms from one part of the country are selected to design structures in another area. In these cases it is especially wise to review loading conditions on the structure caused by peculiarities of the locale (seismic, snow, foundation materials, or wind conditions resulting from hurricanes or tornadoes, etc.).

The review should not be limited to structural features only. In facilities where chemicals are stored and where toxic or volatile gases may be formed by mixing some chemicals, proper ventilation is a necessity in the aftermath of a damaging earthquake. These features should also be reviewed for proper design and necessary fail-safe features. Mechanical and electrical equipment, hung ceilings, boilers, transformers and emergency generators must be reviewed for proper bracing or restraint.

There remains no substitute for selecting competent design firms experienced in earthquake engineering, both for the original design and for peer review. Peer review is similar to the *second opinion* sometimes required in medical practice.

Fig. 3a-12. New Society Hotel, Mindanao, Philippines, August 17, 1976 earthquake.

Fig. 3a-13. Harvardian College building, Mindanao, 1976 Philipines earthquake.

Fig. 3a-14. Tison building, Mindanao, 1976 Phillipines earthquake.

Fig. 3a-15. Flowers pots on roof of Tison building.

Wood-Frame Construction

Most of the foregoing discussion of earthquake damage has been about concrete and masonry construction. Prior to the 1970s, wood-frame construction did not receive much attention because it was generally envisioned that *not much can happen to wood frame structures because they are so light and generally have many cross walls for bracing.*

However, beginning with the 1971 San Fernando, California earthquake (Ref. 9), deficiencies that lead to damage in wood-frame dwellings became more apparent. Damage caused by lack of adequate anchorage to footings, lack of bracing from the top of foundations to the first-floor structure, and large torsional distortions became very evident in the earthquakes of the 1970s, 1980s, and 1990s.

Many wood-frame structures damaged in these earthquakes had been constructed under *General Construction Provisions* in the codes and were neither engineered nor constructed specifically to resist lateral forces created by earthquakes. It is interesting to note that even today dwellings are being erected that have not been engineered or inspected to provide adequate lateral bracing required for proper seismic resistant construction.

Of all construction materials, wood requires the most thorough detailing and construction review by designers. This is because all connections must be physically made in the field with nails, screws, bolts, glue, or other similar connectors. In addition, carpenters, not being accustomed to such precise connection requirements, tend to overlook some details, and in other cases, not knowing the precise function the connection is intended to perform, do not install the connectors properly. This puts a lot of pressure on construction reviewers to check all of these necessary details prior to their being covered up by the remaining construction.

Remodeling of wood-frame buildings must be done with great care. Because it is so easy to cut through wood framing, the plans and/or structure must be very thoroughly scrutinized prior to work being done to be sure important bearing or shear walls are not disturbed, and if they are, that proper remedial construction is utilized to ascertain that the structure has been restored to at least its original lateral bracing capacity or upgraded to new codes.

Fig. 3a-16. House slid off of foundation, 1983 Coalinga, California earthquake.

Fig. 3a-16 shows a house that slid off its foundation in the 1983 Coalinga, California earthquake. This could have been caused by lack of sill bolts connecting the structure to the foundation or lack of proper bracing of the short walls (cripple walls) between the foundation and first floor, or both.

Fig. 3a-17 shows a house that badly rotated about one end as a result of lack of torsional resistance in the walls between the foundations and first floor. This occurred in the 1984 Morgan Hill, California earthquake. Torsion often is an important factor for hillside homes when the downhill side of the house has a cripple wall between the first floor and the foundation, and the uphill side has a shorter cripple wall or none. Torsion in this case is caused by the relative rigidity of the walls below the first floor.

Fig. 3a-18 shows new construction alongside older construction in the 1983 Coalinga earthquake. Note that the newer house has remained in place with no damage; the older house has fallen off its foundation in a pattern similar to the one in Fig. 3a-16.

Fig. 3a-19 shows the rotation of an apartment house in the 1971 San Fernando Earthquake. Because of parking requirements, apartments and condominiums frequently have partly open first floors conditions that impose large rotational forces in the structure.

One of the issues that owners should understand about wood-frame structures is the amount of time required for structural engineers to properly design them. It can take 4 to 5 times as long to complete the sketches, calculations, and details required to properly design the structures shown in Fig. 3a-17 as it does to design a 150-feet by 200-feet concrete tilt-up warehouse. Furthermore, construction review (inspection) for the dwelling could take several more visits than the warehouse to ensure compliance with details of the design. Consequently, most dwellings do not receive adequate engineering design or technical inspection, or they may be constructed under code requirements that do not mandate seismic calculations.

When selecting design professionals for wood-frame structures, it is important to select engineers who have had adequate experience in wood design and be prepared to compensate

Fig. 3a-17. House rotated about one end, 1984 Morgan Hill, California earthquake.

Fig. 3a-18. New house alongside older house. Older house slid off foundation, 1983 Coalinga, California earthquake.

Fig. 3a-19. Rotation of apartment house due to open end required for parking, 1971 San Fernando, California earthquake.

them for the extra time required to do the design and inspection properly.

Building Components and Systems

Not all damage to buildings is of a structural nature. Much damage usually occurs to architectural, mechanical, and electrical features. Fig. 3a-20 shows a corridor in the basement of the Olive View Hospital (Ref. 7) after the San Fernando earthquake of 1971. The extensive damage clearly indicated that ceilings and light fixtures must be properly anchored and braced to prevent collapse. Exterior facades also should be carefully supported. Fig. 3a-21 shows two exterior, precast concrete walls that completely collapsed on the J. C. Penney Building (Ref. 10).

Mechanical and electrical systems, including equipment and components, should be carefully reviewed for compliance with good seismic standards. The degree of care taken should be commensurate with the occupancy and usage of the structure. Fig. 3a-22 shows the four modes of equipment failure: sliding, overturning, inability of the equipment itself to withstand shaking, and pulling away from anchorage.

Fig. 3a-23 shows a boiler at the Olive View Hospital (Ref. 9) that moved in excess of 4 feet, sliding into the exterior wall and causing considerable damage. The air conditioning chillers also slid, breaking cast-iron valves and piping. The cost of *down time* and repairs was considerable, yet a few relatively inexpensive anchors, snubbers or restraints could have prevented this motion. Fig. 3a-24 shows the motor control center unit, which nearly overturned. It was restrained only by cable trays.

Sliding is the mode of failure illustrated in Fig. 3a-25. The emergency power generator tore away from its feeder cable, rendering it useless.

Fig. 3a-26 also illustrates failure caused by sliding. The main transformer at the substation tore away from its feeder. With both substation and emergency power generator out, no power was available to the Olive View Hospital (Ref. 9) in the aftermath of the 1971 earthquake.

Site Conditions

Although site conditions are more appropriately described by geologists, there

Fig. 3a-20. Corridor in Olive View Hospital, San Fernando, California, 1971 earthquake.

are some site problems that qualified foundation engineers consider when investigating a potential site. No site should ever be selected without prior investigation by qualified foundation engineers.

In the 1970 Peru , 1976 northern Italy , and 1976 Mindanao earthquakes, numerous slides were visible in the hilly and mountainous areas. The most dramatic was the Braulins slide in northern Italy, shown in Fig. 3a-27. Fig. 3a-28 shows boulders the size of two-story houses that came to rest at the toe of the slide. This slide completely buried several homes constructed at the base of the mountainside. It occurred during an aftershock two days after the initial earthquake and subsequent torrential rainfalls. The heavy, wet earth simply failed. Studies should have been made to assess such possibilities. Not only are slides dangerous in themselves, they can do double damage if, for example, they block an access road to a hospital. A well-designed hospital that has withstood an earthquake is of limited value if its access road is impassable.

Fig. 3a-29 shows the effect of a combination of soil liquefaction and rising water table. The soil has consolidated and subsided, squeezing water out of the pores of the soil up to the surface. The building has also subsided into the soil. This occurred in Chimbote, Peru, in 1970 (Ref. 2).

One of the strange phenomena evidenced in many earthquakes is the occurrence of small areas of much more intensive shaking than other areas in the same earthquake. These areas could be considered as microseismic areas, where ground waves seem to concentrate or focus to amplify the intensity of ground shaking. For example, Fig. 3a-30 shows the Solari Watch Works in northern Italy in 1976. Ground shaking was so intense that the precast side panels were thrown approximately one meter away from the base. All equipment on the floor moved between 0.5 meter (1.6 feet) and 1.0 meter (3.3 feet) in the same direction. No similar intensity was noted elsewhere in this earthquake. A similar situation occurred in the Lima, Peru, earthquake (Ref. 6) in which the

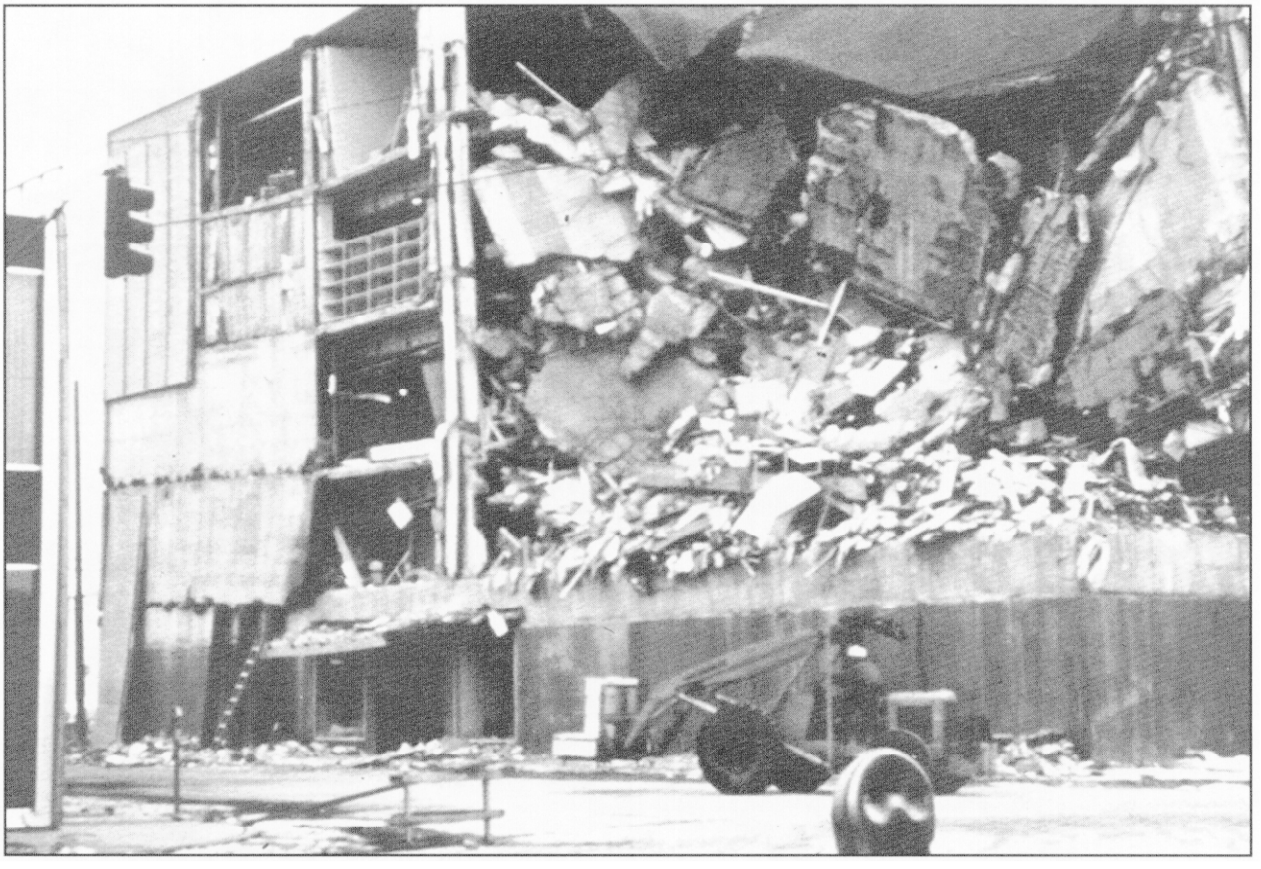


Fig. 3a-21. Two exterior walls, J. C. Penney building, 1964 Anchorage, Alaska earthquake.

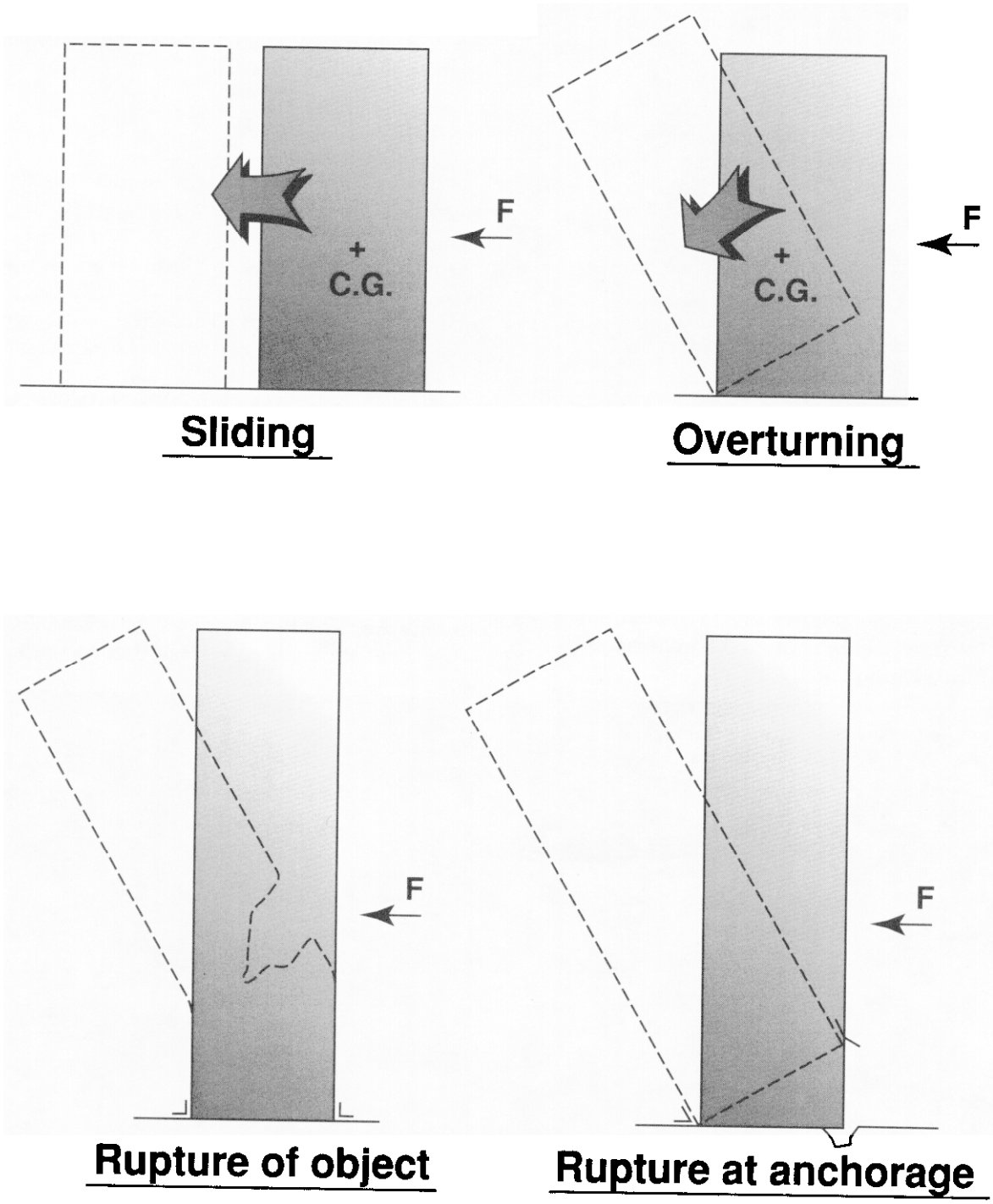


Fig. 3a-22. Modes of equipment failure.

Fig. 3a-23. Boiler at Olive View Hospital, slid 4 feet into the exterior wall, 1971 San Fernando, California earthquake.

Fig. 3a-24. Motor control units in Olive View Hospital, 1971 San Fernando earthquake.

University Classroom Building failed (Fig. 3a 10), yet within 1.5 kilometers (1 mile), adob dwellings suffered no damage. Most of thes microseismic areas seem to lie in lowlands, wit high water tables.

Contents

The effects of earthquakes on the contents c buildings can be devastating, depending o usage and contents. Fig. 3a-31 (San Fernand earthquake of 1971, Ref. 9) illustrates overturnin of equipment, and Fig. 3a-32 shows spillage c materials from shelving. These in themselves ar relatively minor kinds of earthquake damage.

In other situations, the damage ha ramifications that pose more serious problem For example, damage to computer room involves more than equipment damage. I general, computers are placed on raised floor that allow cables and, frequently, air-conditione air to pass under the various components. A shown in Fig. 3a-33, these computer floor usually are not laterally braced for seismic load This view shows the underside of a compute

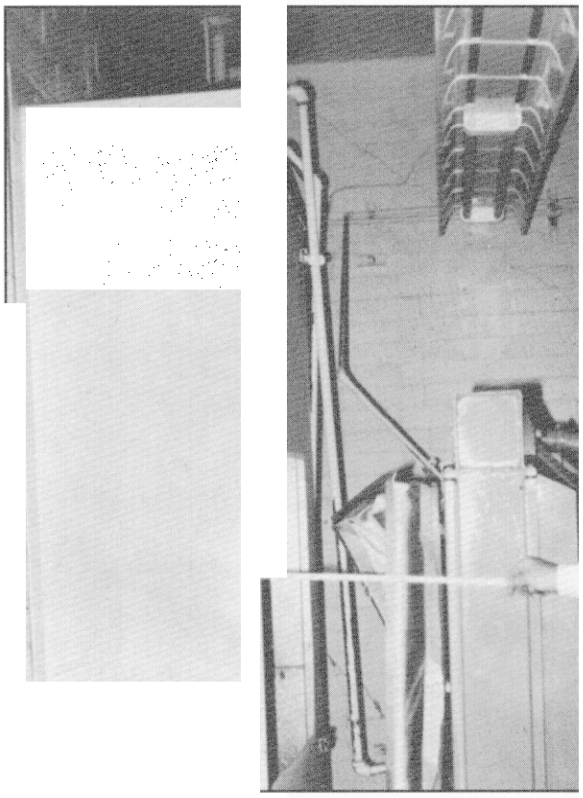


Fig. 3a-25. Emergency power generator at Olive View Hospital, torn away from feeder cable, 1971 San Fernando earthquake.

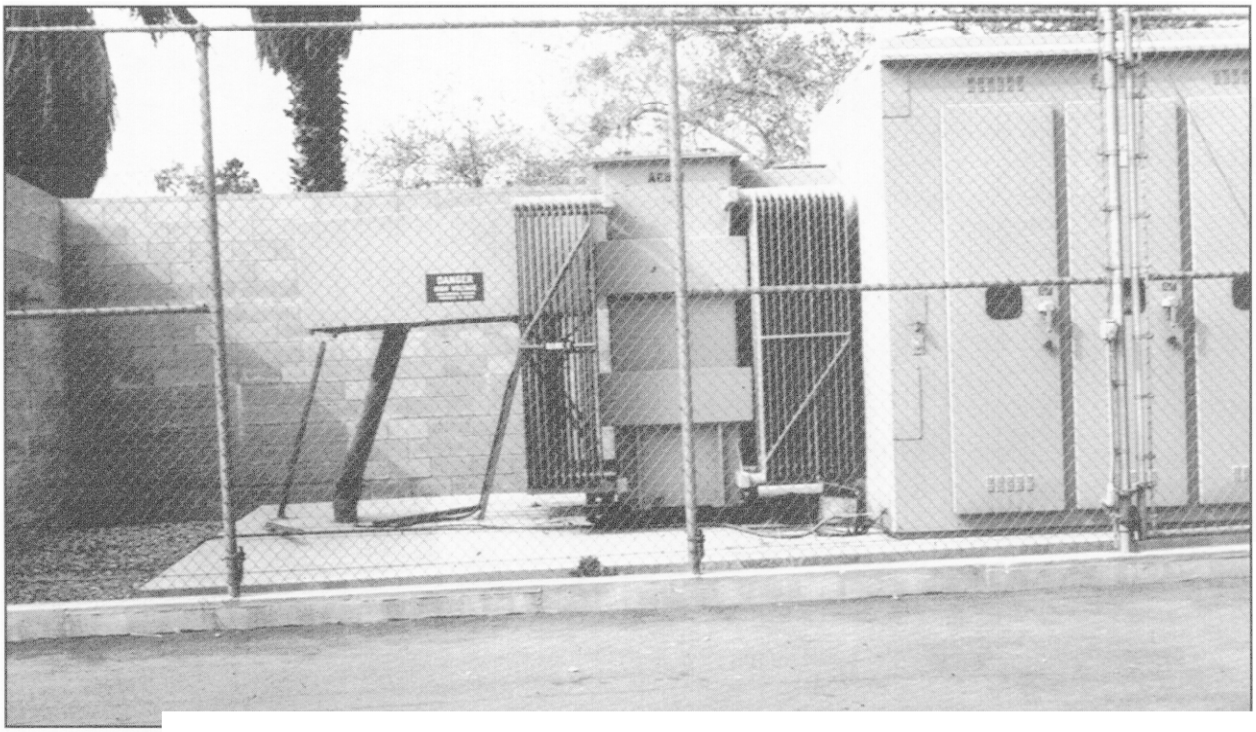


Fig. 3a-26. Oliver View Hospital (1971) substation main transformer torn away from feeder cable.

floor. If floors collapse, computers can overturn and fall to the subfloor, causing severe damage to the computer plus loss of stored data. In some computer centers millions of dollars worth of equipment and stored data could be lost, as well as the time necessary to restore the data and equipment. (For a detailed discussion of this subject, see Chapter 9c.)

In research and development centers and electronic facilities, large amounts of costly equipment are used on tables and benches. This equipment will probably fall to the floor during a mild shock. A severe shock could force some facilities to close for several months until damaged equipment can be replaced. Where integrated circuits and semi-conductor chips are stored on shelving, all could be lost.

Plating facilities also pose special problems because of the hazardous chemicals they store and employ. Usually, the cyanide tank is four tanks removed from the acid tank. During an earthquake, sloshing and spillage occur. If the cyanide and acid are combined, hydrogen cyanide, a deadly poisonous gas, is produced. A similar situation exists in chemical laboratories where chemical compounds are often stored in bottles in glass-door cabinets above the benches. Generally, both cyanide compounds and acids are

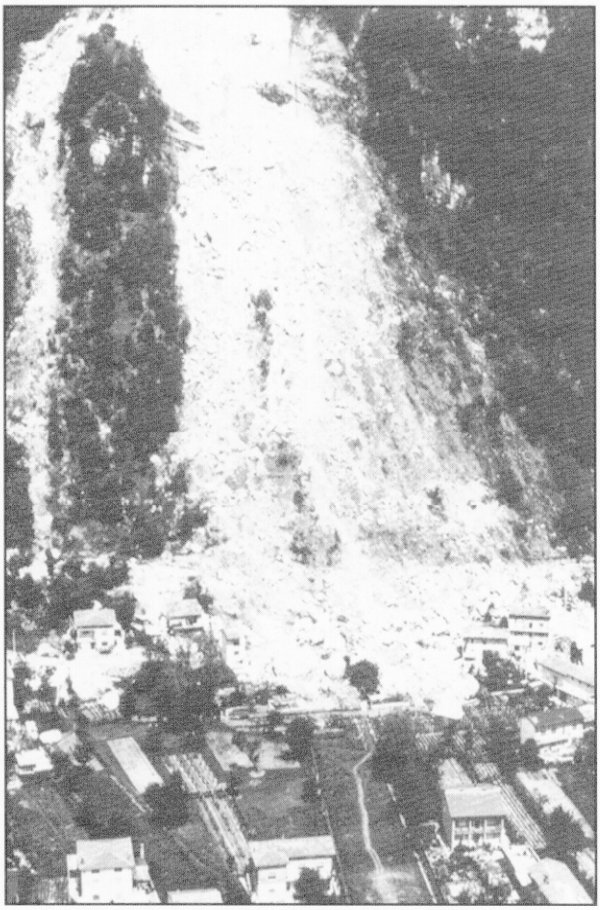


Fig. 3a-27. Aerial view of Braulins slide in Northern Italy, May 6, 1976 earthquake.

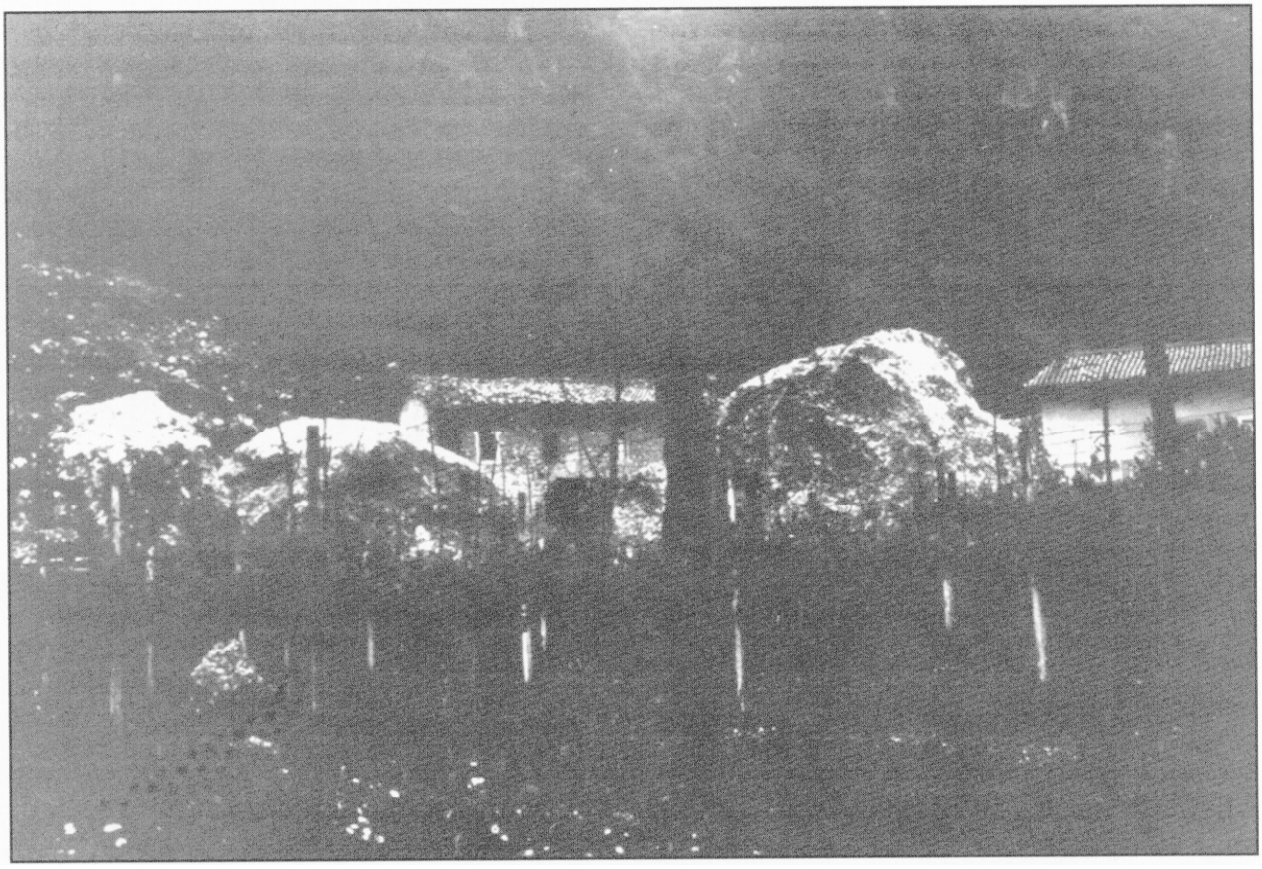


Fig. 3a-28. Two-story boulders came to rest at base of Braulins slide.

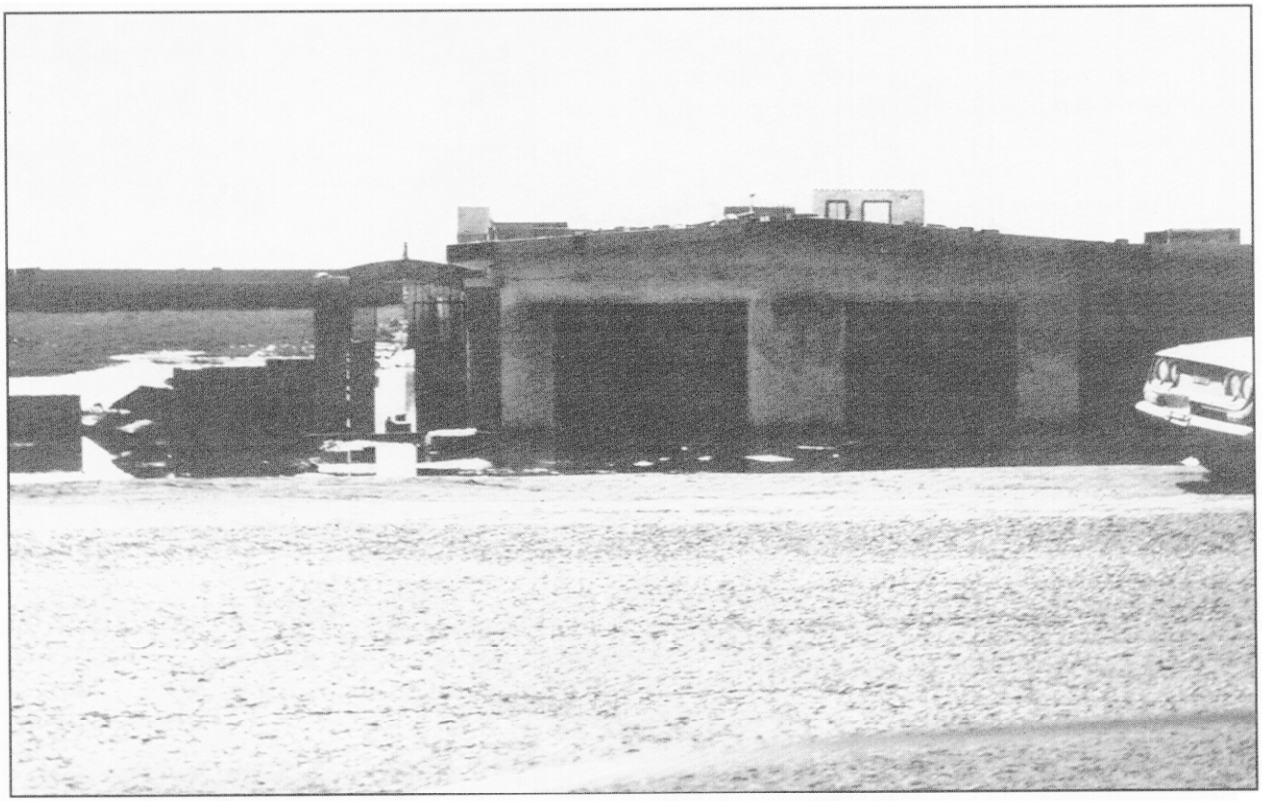


Fig. 3a-29. Effects of combination of soil liquefaction and raising water table, Chimbote, Peru, May 31, 1970 earthquake.

stored. If the earthquake shakes these bottles off the shelves, a situation similar to that in plating facilities could develop.

A related problem occurred in the Mindanao earthquake of August 17, 1976 (Ref. 7). When the reconnaissance team for the Earthquake Engineering Research Institute reached Cotobato City in Mindanao, they went to the Notre Dame University to study the damage. Fig. 3a-34 shows the Notre Dame Science Wing before the earthquake. Fig. 3a-35 shows the collapsed structure as seen by the reconnaissance team, and although the structure was blackened by fire, it was assumed the earthquake had caused the collapse. However, a local photographer produced Fig. 3a-36 showing that immediately after the earthquake, the structure, although severely damaged, had remained erect. A fire had broken out, presumably started by a mixture of chemicals that caused a flammable or explosive compound to be formed. This fire burned fiercely and caused the already weakened structure to collapse. *Obviously, the lesson to be learned is to store chemicals in such a manner that a mixture of any two will not create toxic, flammable, or explosive conditions.*

A similar situation arises when gases are stored in pressurized tanks in close proximity to a building. Figs. 3a-37 and 3a-38 show a small tank

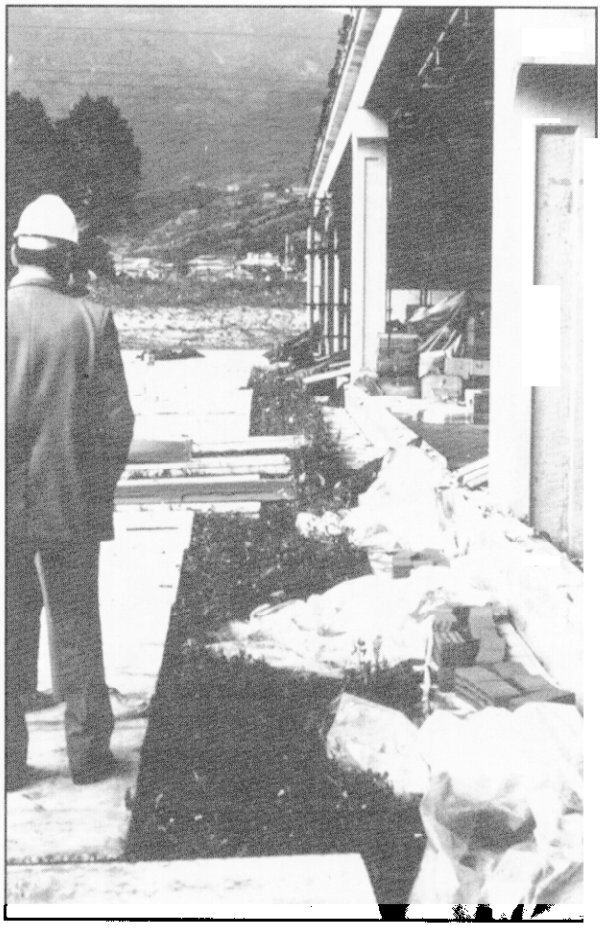


Fig. 3a-30. Solari Watch Works, Northern Italy, May 6, 1976.

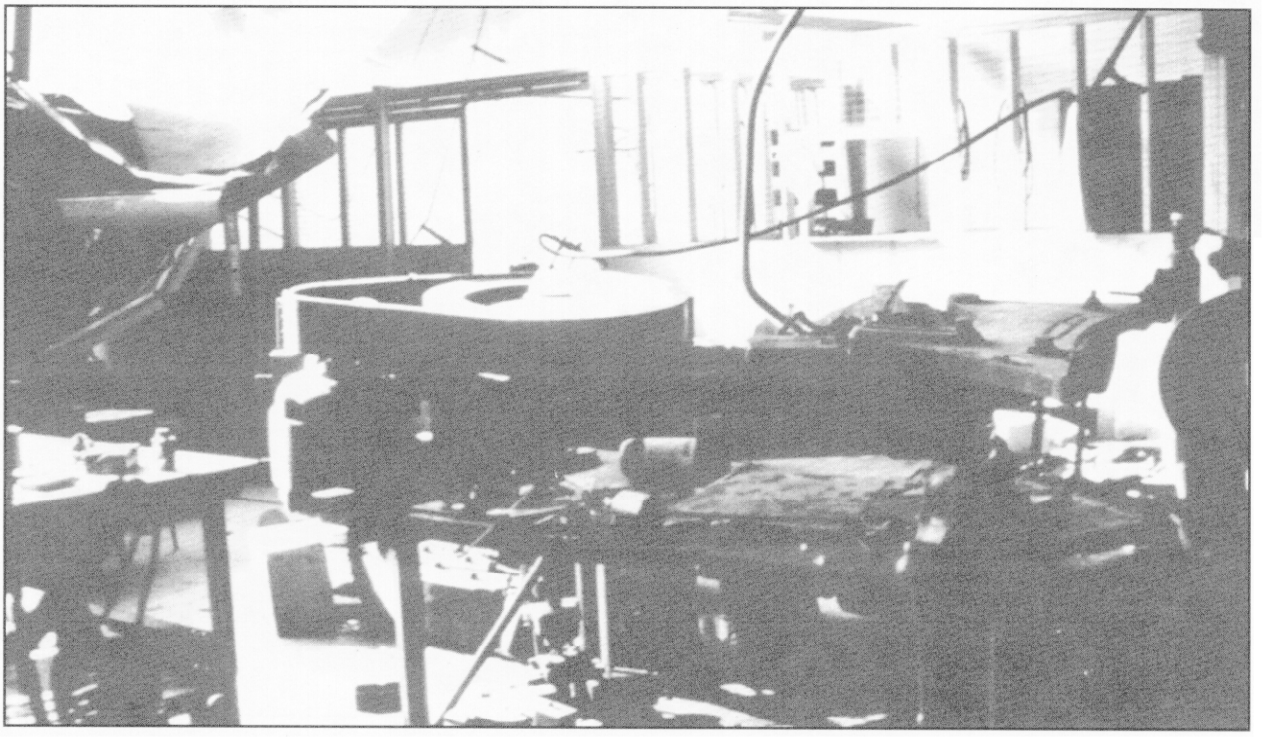


Fig. 3a-31. Overturned equipment, San Fernando earthquake, 1971.

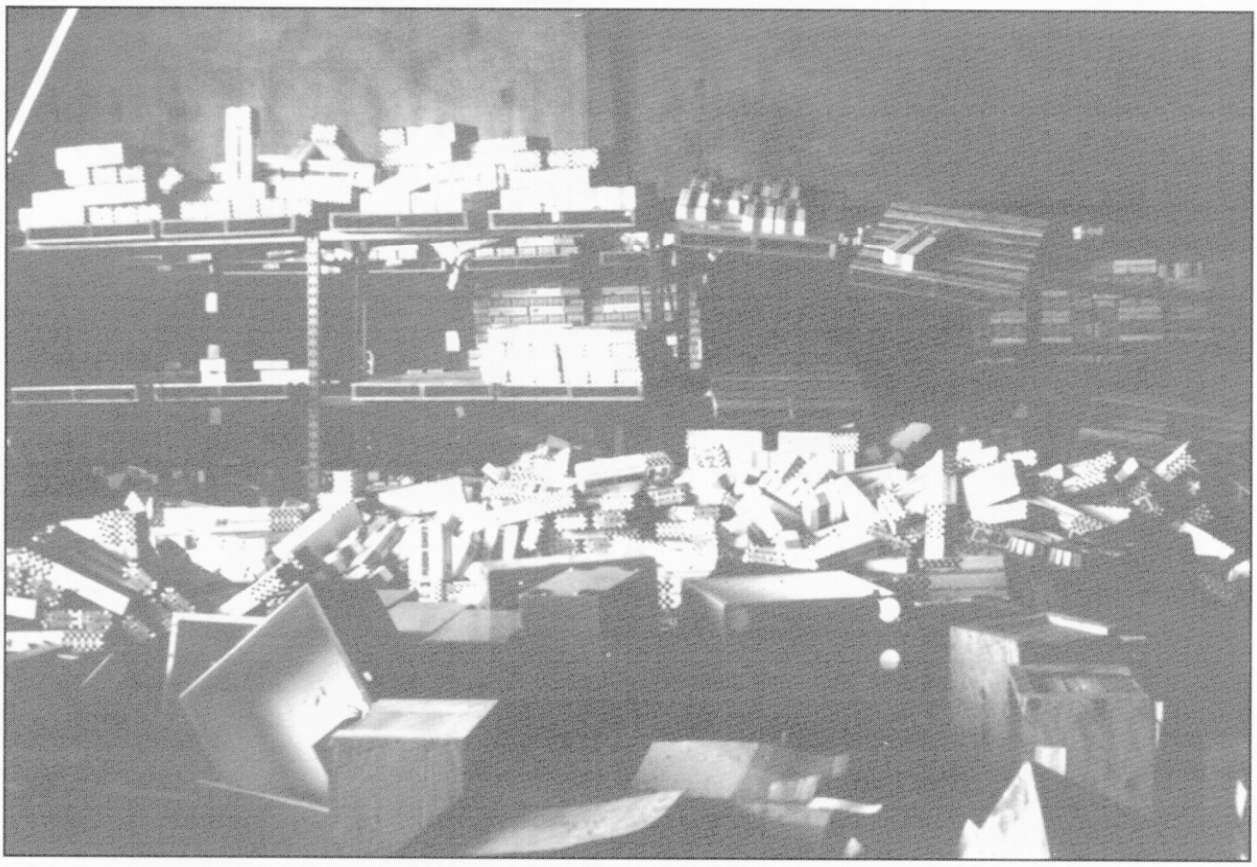


Fig. 3a-32. Spillage of materials from shelving, San Fernando earthquake, 1971.

Fig. 3a-33. Underside of a computer-room floor.

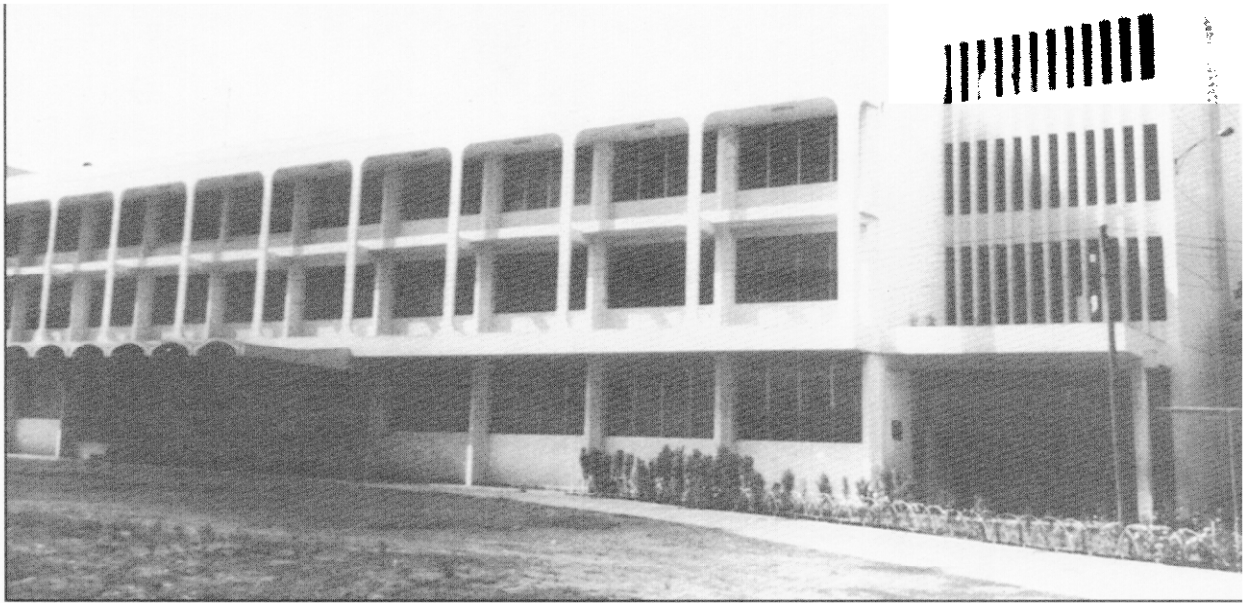


Fig. 3a-34. Notre Dame Science Wing, Cotobato City, Mindanao, prior to the 1976 earthquake.

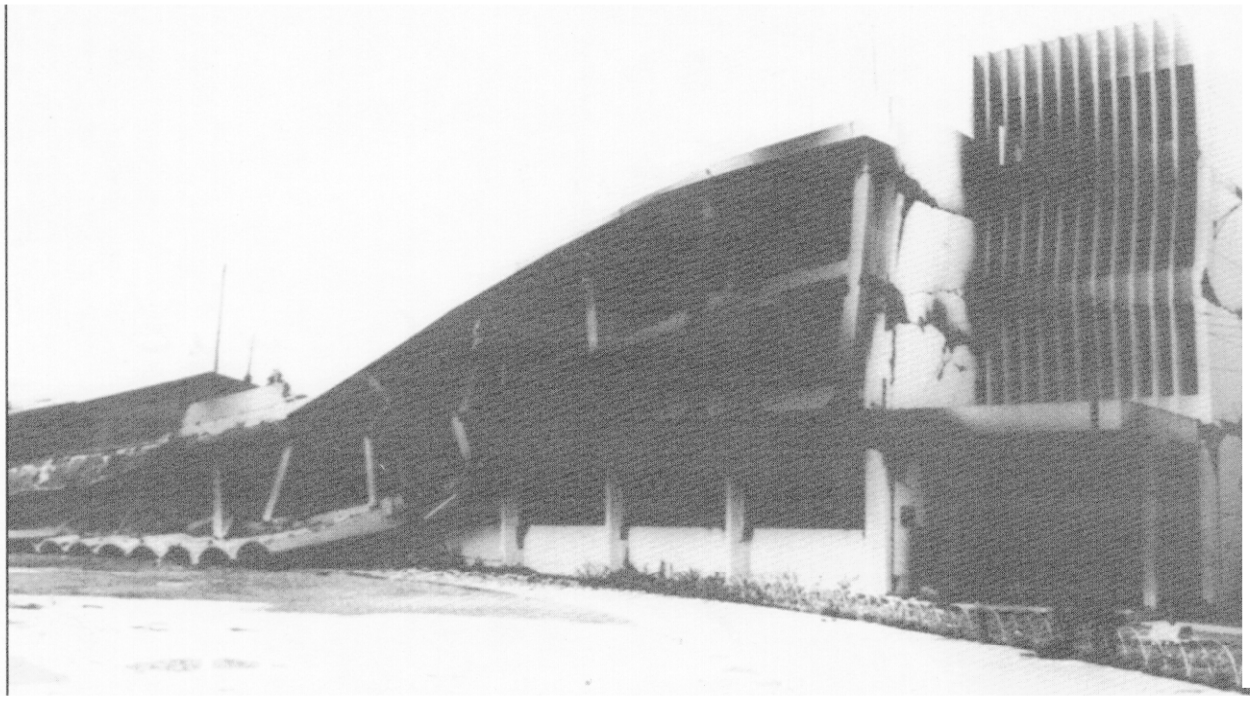


Fig. 3a-35. Notre Dame Science Wing blackened by fire. Photo taken a day or two after the 1976 earthquake.

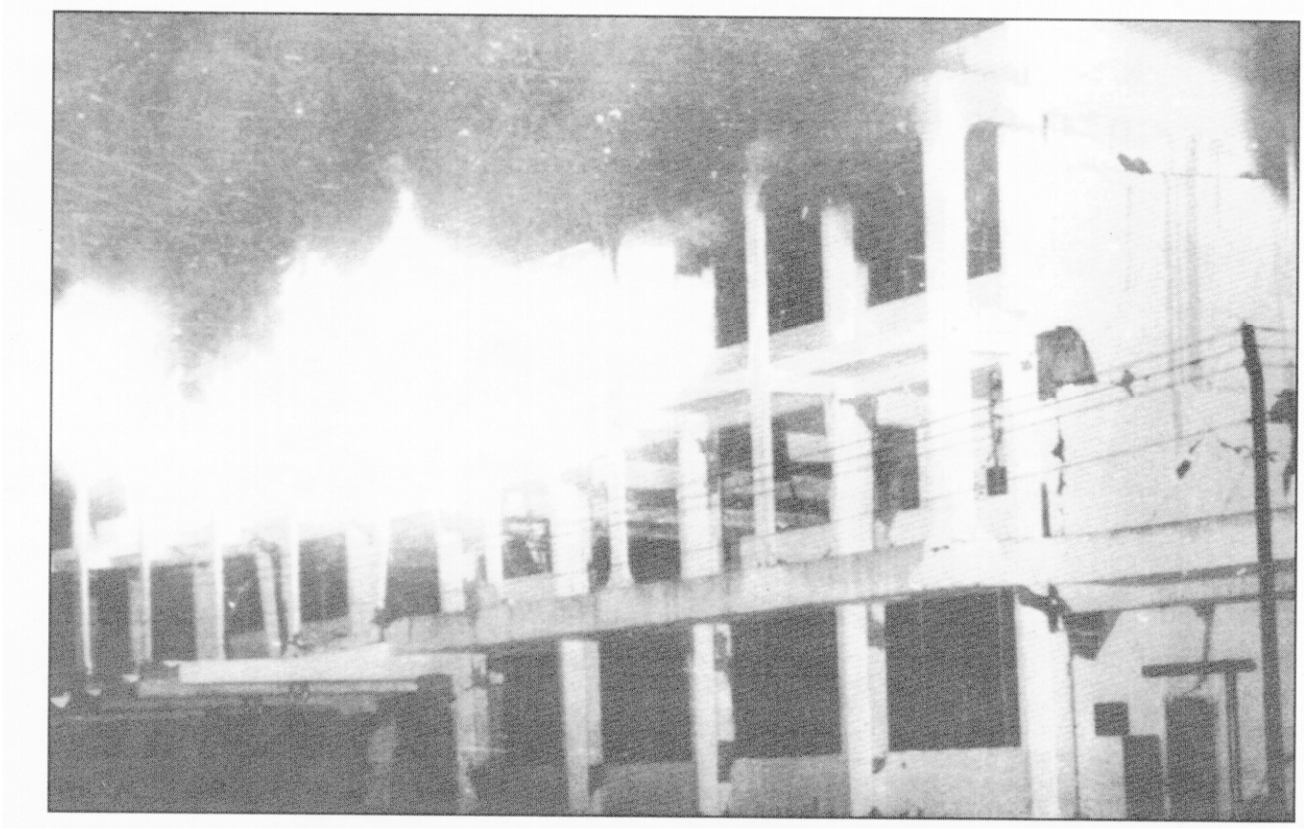


Fig. 3a-36. Notre Dame Science Wing during fire, photo taken immediately after the 1976 earthquake.

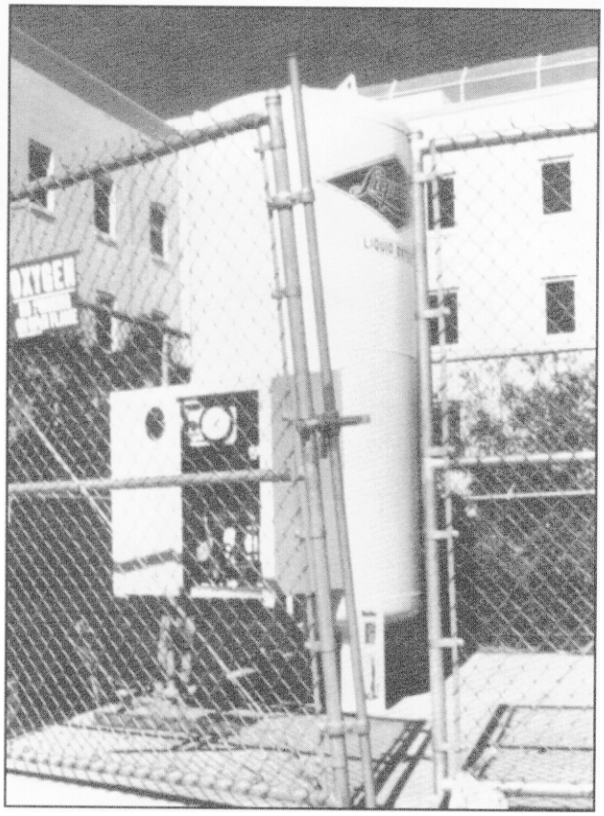


Fig. 3a-37. Tank outside Olive View Hospital slid during 1971 earthquake.

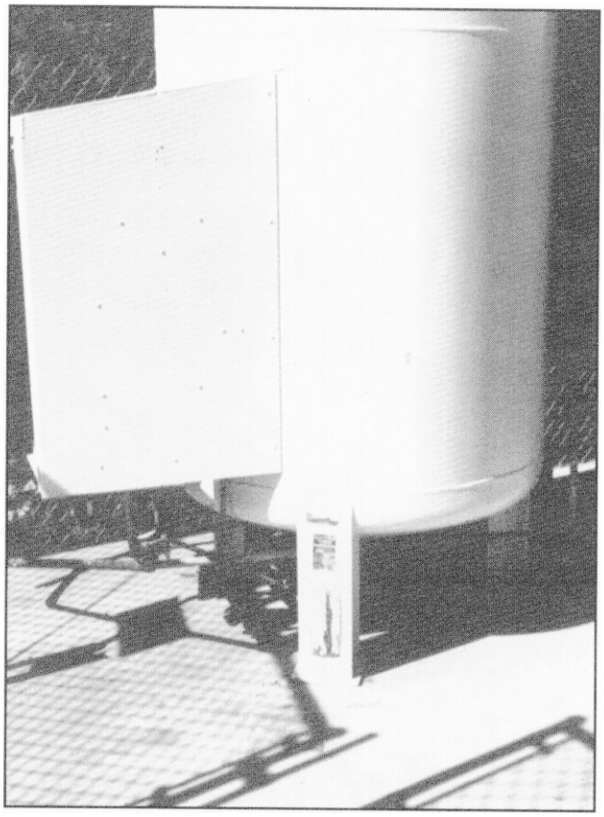


Fig. 3a-38. Tank outside Olive View Hospital was restrained from overturning during the 1971 earthquake.

tank outside the Olive View Hospital after the 1971 earthquake. Note how the tank slid, but was restrained by the fence from overturning. Such tanks should be properly anchored to reduce the possibility of damage.

Wherever toxic, flammable, or explosive gases are piped into a building, serious consideration should be given to the use of earthquake shutoff valves located outside the building. These valves are mechanical devices designed to shut off all supply to the building upon sensing earthquake motions above a set level. In this manner, only gases in the lines within the building confines will escape into the building in the event of a break in the lines, and the effects of toxic gases and possibilities for fires will be greatly reduced, if not eliminated. Earthquake valves also may be used for liquids when desirable.

In one case, an earthquake valve was used to actuate a solenoid valve to open a water main under a plating facility and flush the area to prevent the concentration of dangerous mixtures created by spillage.

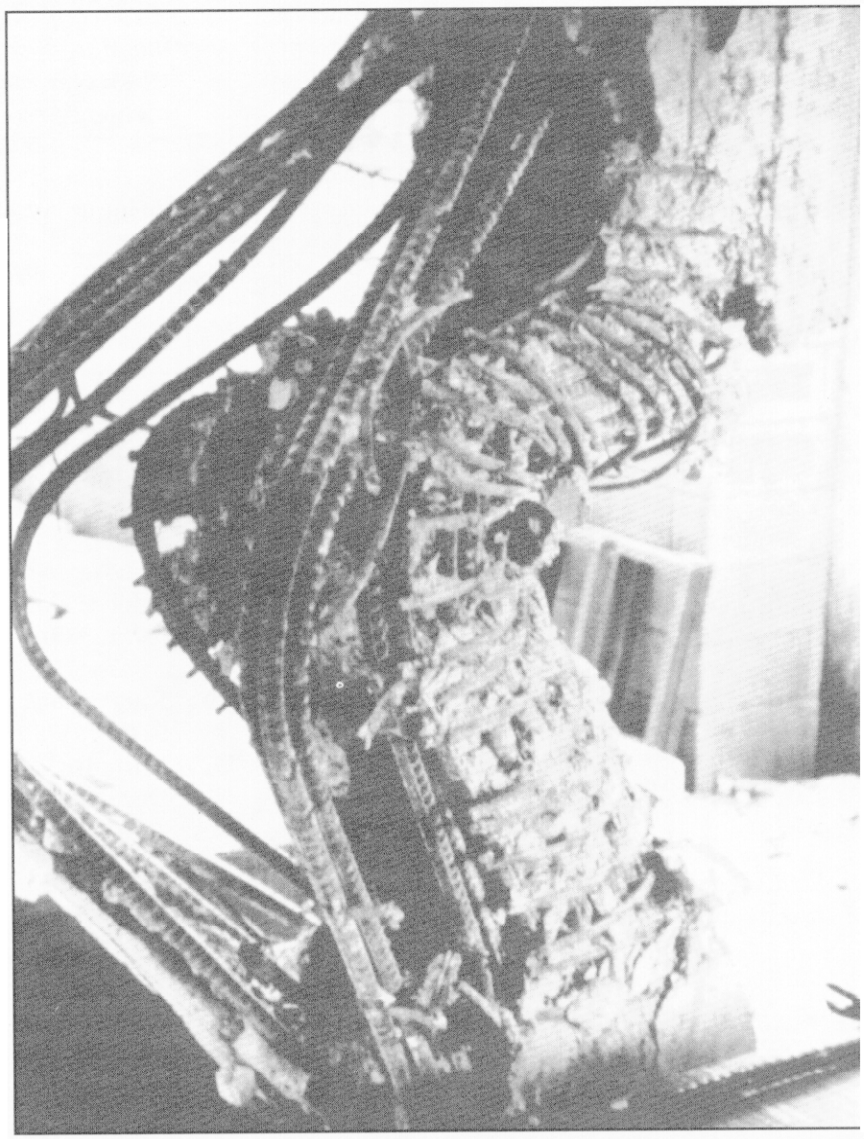


Fig. 3a-39. Exploded column in Phillipine Bar Association building. An overseer to check all details of construction is strongly recommended.

Supervision

The necessity for good field oversight by design engineers cannot be overemphasized. It is not of much value to go through painstaking calculations and detailed drawings and specifications, and then allow the construction work to be carried out improperly. After every damaging earthquake, lack of proper inspection is very noticeable. It is not possible to categorize this problem because it seems to occur in all areas. One specific example, however, illustrates the consequences. In the Philippine Bar Association Building failure, one of the columns literally exploded during the earthquake. Whether this was a cause or result of the damage that occurred can only be surmised. However, what actually occurred during construction is interesting. The plumbing contractor realized that he had forgotten to place a downspout in a column as required. The column reinforcing, however, was already in place, including the spirals from top to bottom of the column. The contractor opened up the column reinforcing by cutting the spiral with a torch, inserted a downspout, neatly closed the column reinforcing and tack-welded the spirals occasionally to maintain the steel in place. Consequently, during the earthquake a most artistically sculptured failure was developed (see Fig. 3a-39).

Some owners and governmental agencies often remove this overseeing requirement from engineering agreements to save money. This action can certainly be deemed *penny wise and pound foolish. It is strongly recommended that overseeing of the project be made an integral part of the contract with design engineers.*

Conclusion

It should be apparent that a complete dissertation on earthquake damage would require several volumes. It is hoped, however, that the few examples and discussions given here will develop in readers a sense of urgency for proper seismic design and construction.

The following references are recommended reading for a more detailed understanding of earthquake damage.

References

1. *Manual of Seismic Design*, J.L. Stratta, Prentice-Hall, Inc. (1987).

2. *Peru Earthquake of May 31, 1970*, Earthquake Engineering Research Institute, Berkeley, California (1970).

3. *Friuli, Italy Earthquake of 1976*, Earthquake Engineering Research Institute, Berkeley, California (August 1979).

4. *Design of Multi-Storied Reinforced Concrete Buildings for Earthquake Motion*, J.A. Blume, N.M. Newmark, and L.H. Corning, Portland Cement Association, Chicago (1961).

5. *Earthquakes*, American Iron and Steel Institute, Washington, D.C. (1975).

6. *The Lima, Peru Earthquake of October 3, 1974*, Earthquake Engineering Research Institute, Berkeley, California (1975).

7. *Mindanao, Philippines Earthquake of August 17, 1976*, Earthquake Engineering Research Institute, Berkeley, California (1977).

8. *Modeling of Floor Diaphragms in Concrete Shearwall Buildings*, R.R. Boppana and F. Naeim, Concrete International, vol. 7, No. 7 (July 1985).

9. *San Fernando, California, Earthquake of February 9, 1971*, Volume I, Parts A and B, Leonard M. Murphy, Scientific Coordinator, U.S. Department of Commerce (1973).

10. *The Prince William Sound, Alaska Earthquake of 1964 and Aftershocks*, Volume I, Part A, Fergus J. Wood, Editor-in-Chief, U.S. Department of Commerce (1967).

11. *Learning From Earthquakes–Planning and Field Guides*, Earthquake Engineering Research Institute, Berkeley, California (1977).

Chapter

3b

Assessment of Damage

Ronald P. Gallagher

Overview of Damage Assessment

Postearthquake damage assessment is a difficult, demanding task. Preparations for making damage assessments often receive little or no attention. Once a damaging earthquake strikes, however, those in charge are faced with overwhelming demands to inspect buildings, barricade hazardous areas, and take measures to prevent further risk to life and property.

This chapter provides guidance about how to organize a damage-assessment program and indicates what the essential components of such a program are. Much of what is presented is drawn from experience, including lessons learned from the very damaging 1989 Loma Prieta earthquake in northern California (Fig. 3b-1).

Before discussing DOE facilities, it is useful to consider what happens to a community after a damaging earthquake. There are typically four phases of damage assessment. The first phase usually starts with what can be called a *windshield survey*. This is done by groups such as fire fighters, police, or building department staff driving around looking for damaged areas. Early reports may come in by radio to a central emergency headquarters. Following this, local building departments begin organizing damage-assessment teams to start the task of formally inspecting buildings. This second phase is usually not started until several hours or a day has passed. The first inspections are often rapid and cursory in nature, because there is insufficient manpower to do more. Detailed inspections (the third phase) usually begin after sufficient numbers of structural engineers and building inspectors are mobilized; this is often a day or more after the event. Communities usually invoke mutual-aid pacts to obtain engineers and building inspectors from other cities. Often volunteer engineers must be flown into the area. All this takes time. After detailed evaluations are performed, buildings posted *questionable* and badly damaged buildings are normally turned over to the owner's engineer for further evaluation and repairs. The engineering evaluation phase is the fourth and final phase of damage assessment.

Most DOE sites include a number of buildings; some have as many buildings as a small city. They are typically staffed with an on-site facilities or plant engineering group that has responsibility for buildings and utilities. This group is analogous to the building department of a city and ordinarily has responsibility to make postearthquake building safety evaluations. Many DOE buildings are government owned, and local building departments have no jurisdiction over them and will not make inspections after an earthquake. Consequently, most DOE facilities can expect little or no help from the outside, partcularly during the initial period after the event. For this and other reasons,

managers of DOE facilities should prepare for postearthquake damage assessment.

Damage-Assessment Program for DOE Facilities

The following is a description of a damage-assessment program that can be used for most DOE facilities. Elements of this program are well-suited for some facilities, but not appropriate for others, and some adaptations or changes may be required.

This program consists of two levels of inspection: an *Initial Damage Evaluation* (IDE); and a *Detailed Evaluation*. These correspond to the second and third phases of damage assessment described earlier.

Immediately after an earthquake, there is an urgent need to inspect buildings. As previously mentioned, sufficient numbers of structural engineers are typically not available on-site, and it may take several days before they become available, particularly after a large event. The U.S. Postal Service was aware of this problem and devised an IDE procedure (Ref. 1). This simple procedure makes use of individuals normally on site to make the initial damage assessments, whether they are engineers or not. If visible damage is found, a building is posted *Closed Pending Evaluation*. If no damage or unsafe conditions are found, the building is posted *Open for Business*. The posting should be a readily visible placard located adjacent each entrance to the building (Fig. 3b-2).

Following the IDE, a second and more comprehensive assessment is done: the *Detailed Evaluation*. While the IDE is a conservative procedure in which the discovery of visible damage triggers closure of the building, the *Detailed Evaluation* is less conservative and much more thorough. It involves use of structural engineers and other specialists. Its purpose is to assess as fully as possible the condition of the building using visual appraisal techniques. Cosmetic damage (i.e., cracked plaster walls) and minor structural damage are not grounds for closing a building. The *Detailed Evaluation* seeks to determine if there has been a significant change in the ability of structures to carry their own weight or to resist subsequent earthquakes or high winds (i.e., lateral forces).

Fig. 3b-1. Disaster workers in San Francisco after the 1989 Loma Prieta earthquake. Damage-assessment programs should be organized before an earthquake strikes.

Three damage assessment techniques for DOE facilities are summarized in Table 3b-1. In addition to the IDE and *Detailed Evaluation* mentioned above, a third technique, *Engineering Evaluation*, also is used. Only the IDE and *Detailed Evaluation* are discussed further in this chapter.

Earthquake Response Team

IDE is accomplished by people who are normally on site during the work day and who have been organized into an *Earthquake Response Team* (ERT). When a damaging or possibly damaging earthquake strikes, the ERT members report to a designated place, usually without waiting for instructions. Often the designated place is an *Emergency Operations Center* (EOC). If an earthquake occurs while they are at home, they are instructed in advance to report for duty and not wait for instructions because telephone service may be disrupted.

The ERT assembles at the EOC (or other designated place), where they gather equipment and await assignment to buildings. Buildings with special concerns, such as those housing hazardous material, should be inspected by special teams including one or more hazardous materials specialist.

Initial Damage Evaluation

The IDE is designed to be a conservative procedure. The designated inspectors, who may not be structural engineers, are asked to look for visible damage (of a nontrivial nature). If they find such damage, they are to post the building *Closed Pending Evaluation.* To perform an IDE, a series of seven criteria is used. These are listed in Table 3b-2 and illustrated in Fig. 3b-3. If a condition described in Table 3b-2 is found, the building is posted closed using the *Closed Pending Evaluation* placard. If none of the seven conditions is observed and there is no other unsafe condition present, the *Open for Use* placard is used. Inspectors may also be asked to barricade unsafe areas. Initially, this may be done with yellow caution tape, but tape is normally replaced with more substantial wood or metal barricades.

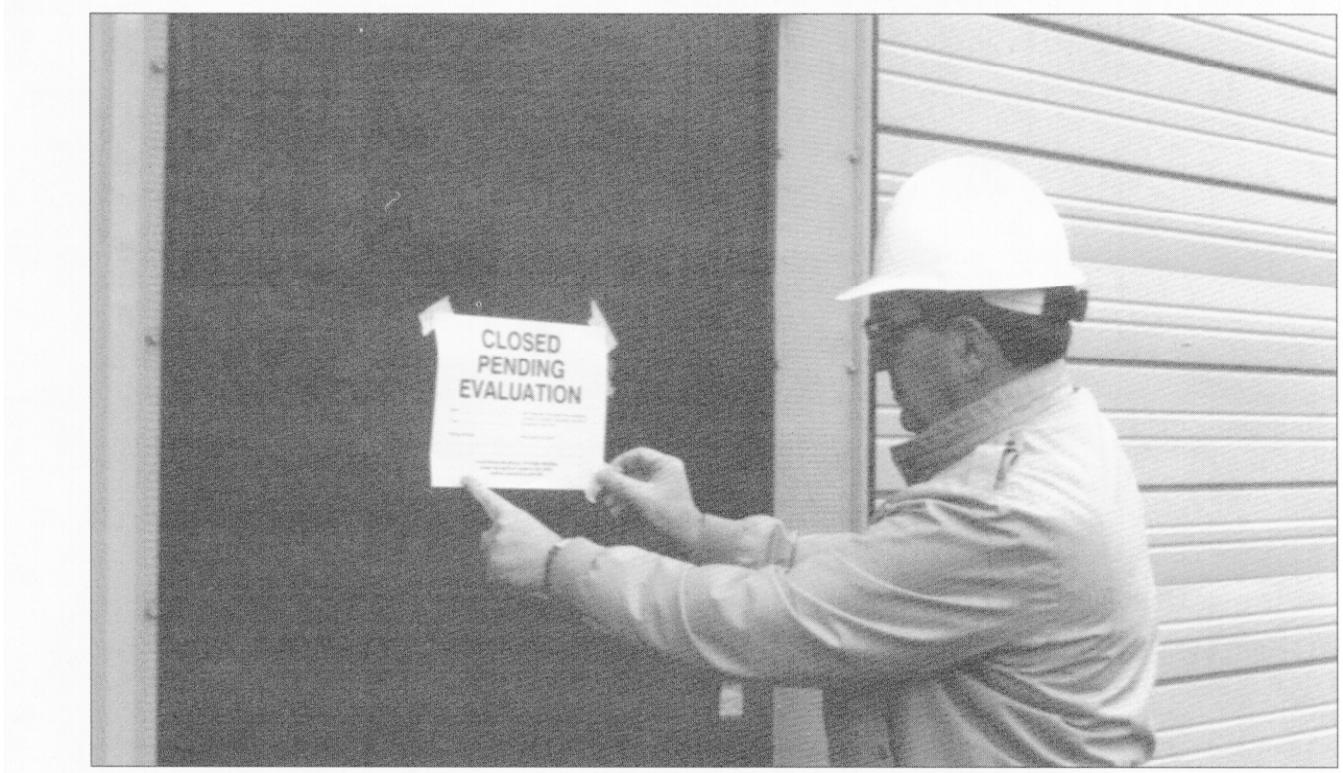


Fig. 3b-2. Posting a building closed. Whenever a building is to be closed after an Initial Damage Evaluation (IDE), the *Closed Pending Evaluation* placard is placed at every entrance.

Table 3b-1. Damage-assessment techniques for DOE facilities.

Technology	Required personnel[1]	Goal	Example time per building
Initial Damage Evaluation	Facility Manager (or designate)	Initial assessment of a building. Used to determine if any visible damage has occurred.	1/2 - 2 hrs.
Detailed Evaluation	Structural engineers	Thorough visual evaluation of a building. Used to identify restrictions on building use or the need for an Engineering Evaluation.	1 - 4 hrs. or more
Engineering Evaluation	Structural engineering consultant	Detailed engineering investigation of a damaged building for purposes of determining how to stabilize and repair it.	1 - 7 days

[1]Geotechnical or other specialists may be required for the assessment of nonstructural hazards (e.g., slope failure, toxic spill).

Table 3b-2. Initial Damage Evaluation (IDE) criteria.

	Condition	Posting
1.	The building or any part of the building has collapsed or moved off its foundation.	CLOSED
2.	The building or any story is leaning.	CLOSED
3.	There is damage on the outside of the building to walls, overhangs, or other building parts that makes them a falling hazard.	CLOSED*
4.	There is ground cracking or movement adjacent to the building.	CLOSED
5.	There are obvious indications of damage to structural elements such as walls, columns, trusses, or bracing, or other signs of severe distress are present.	CLOSED
6.	There is damage on the inside of the building to things overhead, such as light fixtures, piping, ducts, cranes, and hoists, that makes them a falling hazard.	CLOSED*
7.	Other identifiable hazards are present, such as damaged sprinkler systems, broken gas lines, damaged or downed power lines, or chemical spills.	CLOSED

*If damage is very localized, barricading the affected areas rather than closing the entire building may be the appropriate action.

Table 3b-3. Inspection procedure for Initial Damage Evaluation (IDE).

1. If an earthquake has occurred that may have caused damage to the facility, established evacuation procedures should be followed prior to conducting the IDE.

2. Examine the entire outside of the building, including the ground or pavement in the vicinity of the building.

3. Examine the interior of the structure. **Never enter an obviously unsafe structure.**

4. Report any hazardous material incidents immediately.

5. Evaluate the structure using the IDE criteria (Table 3b-2).

6. Post the structure according to the result of the inspection. Use one of the two placards (Closed Pending Evaluation or Open For Use). Post every entrance to a building that is to be closed.

7. Explain the significance of Closed postings to buildings occupants and advise them to leave immediately. Secure the building if this has not been done.

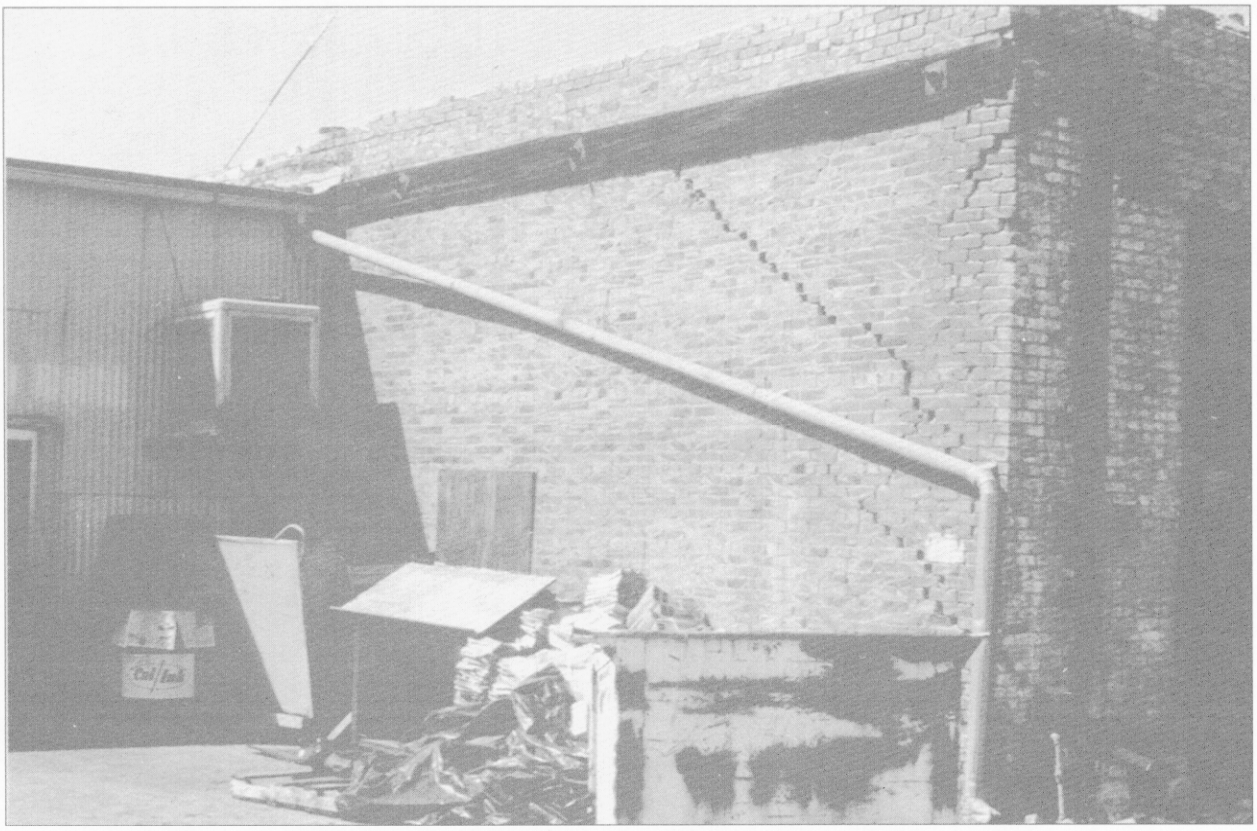
8. Communicate the assessment results to the appropriate authorities (e.g., Incident Commander) as rapidly as possible.

Table 3b-3 indicates the step-by-step procedure for conducting an IDE. Fig. 3b-4 shows two generic IDE placards, which may be modified to suit local conditions.

Fig. 3b-3 (Criterions 1 through 7) Examples of the use of the Initial Damage Evaluation Criteria. Criterion 1: Collapse, partial collapse, or moved off foundation. Post: Closed.

Fig. 3b-3. (continued)

Criterion 2: Building or story is leaning. Post: Closed.



Criterion 3: Obvious parapet or other outside falling hazard.
Post: Closed (or barricade area if appropriate). (Photo: Robert Reitherman)

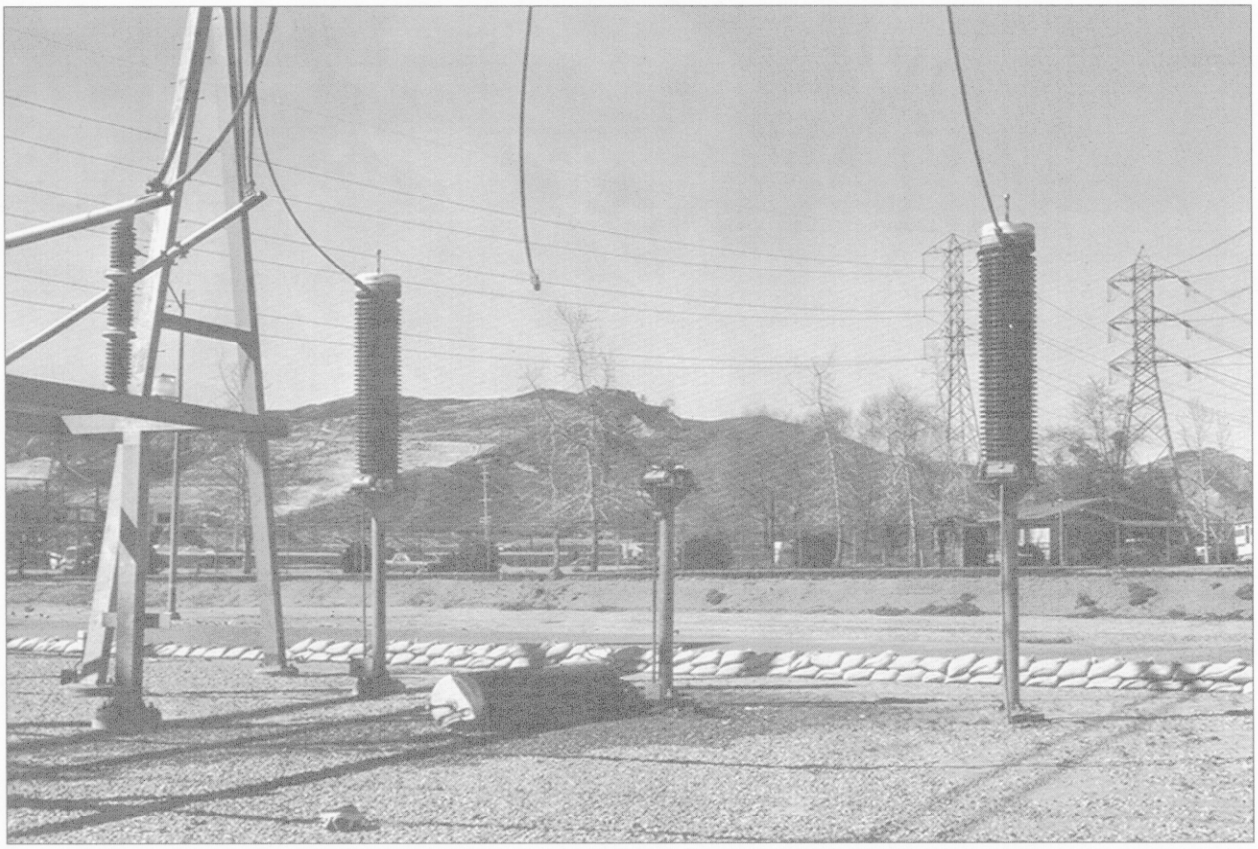
Fig. 3b-3. (continued)

Criterion 4: Ground cracking or movement. Post: Closed. (Photo: Les Youd)

Criterion 5: Obvious indications of damage to walls, bracing, or other structural elements. Post: Closed. (Photo: John Meehan)

Fig. 3b-3. (continued)

Criterion 6: Obvious damage to things inside the building that makes them falling hazards. Post: Closed (or barricade affected area if appropriate).



Criterion 7: Other hazards present (e.g., electrical hazard). Post: Closed (or barricade area).

Fig. 3b-3. (continued)

Detailed Evaluation

Detailed Evaluations are thorough visual examinations by qualified engineers. For buildings, these are accomplished by structural engineers (i.e., those qualified to design the building in the first place). Where geotechnical or other hazards are believed present, inspectors may be geologists, geotechnical engineers, or other specialists (e.g., fire-protection engineers, hazardous materials specialists, or nuclear engineers).

In a *Detailed Evaluation*, maximum use is made of visually available information. Because structural members are often obstructed by architectural finishes such as walls and ceilings, it is often necessary to remove them to permit better observation. In government-owned buildings, a certain amount of destructive exploration may be acceptable that may not be permitted in leased buildings without owners' permission. This may include removing plaster or drywall finishes to better evaluate key elements of the structural system.

The posting system used is that described in ATC-20, *Procedures for Postearthquake Safety Evaluation of Buildings* (Ref. 2). Three postings are used: *Inspected, Restricted Use,* and *Unsafe,* as shown on Fig. 3b-5. Table 3b-4 provides a description of each category.

In-depth guidance in performing a *Detailed Evaluation* is given in the ATC-20 and ATC-20-1 documents (Refs. 2 and 3, respectively). Reference 4 provides a brief summary of these documents. ATC-20 emphasizes evaluation of building structural elements. Procedures for dealing with hazardous materials released by an earthquake are only briefly covered.

Damage-Assessment Teams

It is recommended that damage-assessment teams consist of at least two people; experience has shown this to be the best approach because the decision-making process in the field is improved by having two persons instead of one make decisions, and if one person becomes trapped or injured, the other can go for help.

DOE facilities often have special chemical, radiological, or security concerns that can greatly complicate damage assessment. The IDE team for a laboratory building may require a different make-up than that for an office building. For example, at a high-security nuclear material facility, it may be necessary to have a three-person IDE team with representatives from health physics, security, and the ERT. Alternatively, members of groups like health physics and security can be assigned as members of the ERT. It may also be necessary to wear protective clothing and air breathing apparatus when making an IDE. Because of these complications, serious thought must be given to damage assessment for such special facilities. To wait until the day of the earthquake may be too late.

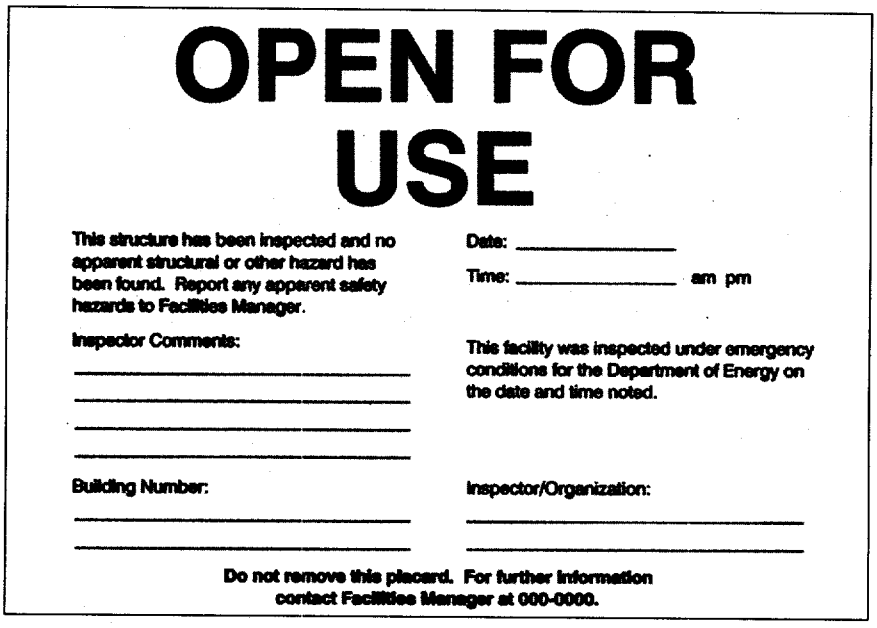
Establishing Inspection Priorities

It is desirable to establish a list of buildings in order of inspection priority. Factors that can place a building in a first-priority inspection category include known seismic hazards, hazardous contents, large number of occupants, and importance to facility mission. The place from which the immediate after-the-event activities are to be directed, usually the EOC, needs to be one of the first, if not the very first, building inspected. Similarly, facilities that will be used for recovery, such as fire stations, medical clinics, and communications centers, should be high on the priority list.

Having made up a list of inspection priorities beforehand does not mean the priorities cannot or should not be changed. Damage reports from windshield surveys, or from the occupants of buildings, may necessitate changes. In the absence of such information, a well thought-out plan will allow for the most judicious use of expected limited manpower.

Building-Specific Inspection Plans

A powerful tool in helping IDE and *Detailed Evaluation* inspectors do their job is the building-specific inspection plan. This is typically a set of brief written instructions, supplemented with simplified floor plan drawings, that provide a concise step-by-step set of instructions showing how a particular



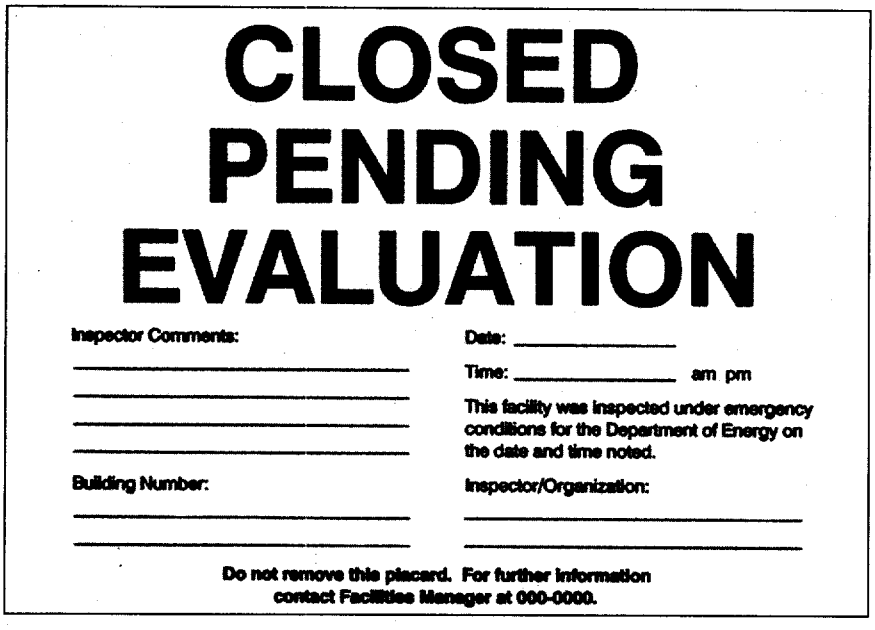


Fig. 3b-4. Initial Damage Evaluation (IDE) placards.

INSPECTED

NO RESTRICTION ON USE OR OCCUPANCY

This structure has been inspected and no apparent structural or other hazard has been found. Report any unsafe conditions to Facilities Manager; reinspection may be required.

Inspector Comments:

_____
_____
_____
_____

Date: _____

Time: _____ am pm

(Caution: Aftershocks since inspection may increase damage and risk.)

This facility was inspected under emergency conditions for the Department of Energy on the date and time noted.

Building Number:

_____
_____

Inspector/Organization:

_____
_____

Do not remove this placard. For further information contact Facilities Manager at 000-0000.

RESTRICTED USE

Caution: This structure has been inspected and found to be damaged as described below:

_____
_____
_____

Entry, occupancy and use are restricted as indicated below:

☐ Brief entry for emergency purposes only

☐ Other (specify restriction): _____

_____
_____

Date: _____

Time: _____ am pm

(Caution: Aftershocks since inspection may increase damage and risk.)

This facility was inspected under emergency conditions for the Department of Energy on the date and time noted.

Building Number:

_____
_____

Inspector/Organization:

_____
_____

Do not remove this placard. For further information contact Facilities Manager at 000-0000.

UNSAFE

DO NOT ENTER OR OCCUPY

This structure has been inspected, found to be seriously damaged and is unsafe to occupy, as described below:

_____
_____
_____
_____

Do not enter, except as specifically authorized. Entry may result in death or injury.

Date: _____

Time: _____ am pm

This facility was inspected under emergency conditions for the Department of Energy on the date and time noted.

Building Number:

_____
_____

Inspector/Organization:

_____
_____

Do not remove this placard. For further information contact Facilities Manager at 000-0000.

Fig. 3b-5. Detailed Evaluation placards.

Table 3b-4. Building safety evaluation classifications.

Posting Classification	Placard Color	Description
I. Initial Damage Evaluation (IDE)		
OPEN FOR USE	White	No significant damage found. No restriction on use or occupancy.
CLOSED PENDING EVALUATION	White	Visible damage found. Entry is not permitted, except by authorized personnel.
II. Detailed Evaluation		
INSPECTED	Green	No apparent hazard found, although repairs may be required. Original seismic resistance not significantly decreased. No restriction on use or occupancy.
RESTRICTED USE	Yellow	Hazardous condition exists (or is believed to exist) that requires restrictions on the occupancy or use of the structure. Entry and use have been restricted as indicated on the placard.
UNSAFE	Red	Extreme hazard present. Imminent danger of further damage or collapse from an aftershock. Unsafe for occupancy or entry, except by authorities.

building is to be inspected. Areas of special concern, such as hazardous-materials laboratories, can be identified in the inspection plan to alert inspectors to look in particular areas for certain kinds of damage or distress.

Developing building-specific inspection plans and training staff in their use, constitutes the highest level of postearthquake damage-assessment preparation. These measures are particularly well suited for critical and important facilities.

Aftershocks

Most earthquakes are followed by aftershocks, which typically are smaller than the main event. Because aftershocks can further damage already damaged buildings and can damage buildings that survived the main event undamaged, reinspection may be required. This requires people in charge of damage assessment to make decisions as to whether the aftershock was of sufficient strength to cause additional damage. There is no simple formula for this, but if an aftershock is known to have caused new damage, this should be considered grounds to order a reinspection of all buildings.

Pre-Event Activities

A number of steps can be taken to prepare for postearthquake damage assessment.

These include:

(1) Establish an EOC for postearthquake use. Select a structure with excellent seismic resistance.

(2) Develop a damage-assessment program that will endure and not fall into neglect with time. Specify who will be in charge. Assign teams to specific buildings.

(3) Establish a building inspection priority list. Identify structures with special needs (e.g., high-security buildings containing hazardous materials).

(4) Identify ERT members from among the on-site staff. Health physics, hazardous materials, and security personnel may need to be assigned to the ERT.

(5) Identify who will do the *Detailed Evaluation* (e.g., on-site structural engineers, consultants from nearby communities). If available on-site resources are insufficient, consider making pre-arrangements with local consultants.

(6) Stock the EOC with necessary post-earthquake supplies including placards, checklists, hardhats, protective clothing, two-way radios, and caution tape.

(7) Develop building-specific inspection plans if a high level of preparedness is desired. Keep a copy of these in the EOC.

(8) Maintain copies of building drawings, particularly structural drawings, in a readily accessible place (consider using the EOC for this). These may be needed for *Detailed Evaluations.*

(9) Hold periodic training exercises. *This is an absolute must.*

References

1. *United States Postal Service, Procedures for Post-disaster Safety Evaluation of Postal Service Facilities,* ATC-26-2, Applied Technology Council, Redwood City, California, 1992.

2. *Procedures for Post-earthquake Safety Evaluation of Buildings,* ATC-20, Applied Technology Council, Redwood City, California, 1989.

3. *Field Manual: Post-earthquake Safety Evaluation of Buildings,* ATC-20-1, Applied Technology Council, Redwood City, California, 1989.

4. *Post-earthquake Safety Evaluation of Buildings, Seismic Engineering: Research and Practice,* R. P. Gallagher, Proceedings, ASCE Structures Congress 1989, San Francisco, California, American Society of Civil Engineers, New York, New York, 1989.

INCH-POUND

DOE-STD-1020-94
April 1994
Change Notice #1
January 1996

DOE STANDARD

NATURAL PHENOMENA HAZARDS DESIGN AND EVALUATION CRITERIA FOR DEPARTMENT OF ENERGY FACILITIES

AREA FACR

U.S. Department of Energy
Washington, D.C. 20585

DISTRIBUTION STATEMENT A. Approved for public release; distribution is unlimited.

Chapter

4

Seismic Design and Evaluation Requirements for DOE Facilities

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

In December 1992, DOE Order 5480.28* (Ref. 1) covering *Natural Phenomena Hazards Mitigation* (NPH) was approved by the U.S. Department of Energy (DOE) Offices of Environment, Safety, and Health and Nuclear Energy. It was issued by the Secretary of Energy in January 1993. The Order states that it is the policy of the U.S. Department of Energy to design, construct, and operate DOE facilities so that workers, the general public, and the environment are protected from the impacts of natural phenomena hazards. It established new and more comprehensive protection against earthquakes, extreme winds, tornadoes, and flood hazards for all DOE facilities. It is consistent with the *National Earthquake Hazards Reduction Program* (NEHRP) and the President's Executive Orders on Seismic Safety. There are two Executive Orders; 12699 covering new building construction (Ref. 2), and 12941 covering existing buildings (Ref. 3).

The landmark policy embodied in DOE Order 5480.28 represented the culmination of 20 years of leadership by the department in the continuous development of improved seismic safety This policy has for its implementation a series of DOE Standards that were developed under the Office of Environment, Safety and Health with major support from the Lawrence Livermore National Laboratory, other DOE contractors and specialized consultants from the private sector. Order 5480.28 included extensive input and widely based concurrence by more than 50 DOE and contractor organizations.

Chapter 4a provides a road map for architects, engineers, and managers who must utilize DOE Order 5480.28 and its implementing standards to ensure seismic safety within the DOE complex. A summary of the DOE NPH Mitigation Policy and Standards is provided in Chapter 4b. Although the new DOE Orders and

* This document was completed in October 1995, at that time DOE was in the process of consolidating orders. DOE Order 5480.28, *Natural Phenomena Hazards Mitigation*, was being combined with three other orders to form a new DOE Order 420.1, Facility Safety. In addition, the draft *Corporate Facilities Management Order* has cancelled DOE Order 6430.1A.
A Natural Phenomena Rule, 10 CFR 830.215 and accompanying Safety Guide was also being drafted. These new directives retain the information contained in DOE Order 5480.28. DOE Standards 1020 through 1024 will continue to be referenced by the new directives.

Standards are comprehensive in that they cover earthquakes, extreme winds, tornadoes, flood hazards, and other natural hazards, this *Seismic Safety Manual* focuses on seismic safety only.

The overall approach for DOE NPH Mitigation is to provide protection against the impact of natural hazards graded in proportion to the consequences of failure of its facilities (including the potential release of hazardous material). *Facilities* include buildings, other structures, programmatic and process equipment, subsystems, infrastructure, and their individual components, all of which are referred to as *Structures, Systems, and Components* (SSCs) in NPH Orders and Standards. This approach is consistent with DOE's *Safety Analysis Reports* (SAR), which evaluate and classify each DOE building or facility by the level of potential hazard it presents based upon accident analysis.

The consequences of failure of a building (or SSC) affected by an earthquake depend upon the type of failure (e.g., catastrophic collapse versus bent out of shape), the occupancy within the building (e.g., people, process equipment, hazardous materials, etc.), the potential for loss of life or loss of use, and the impact on the public or the environment. Depending upon the results of a risk analysis, an individual facility (or SSC) is placed into one of five *Performance Categories* (PCs) that range from PC-0 to PC-4, each of which has defined performance goals. An SSC can be placed in category PC-0 only if the consequences of failure are none or negligible with respect to safety. Performance goals for categories PC-1 to PC-4 range from those required for ordinary occupancies to those required for facilities that pose a potential hazard to the public and the environment because radioactive or highly toxic materials could be released in the event of loss of confinement.

The performance goals for PC-1 and PC-2 are generally the same as those achieved if the provisions of *Uniform Building Code* (UBC) (Ref. 4) are followed for *Standard Occupancy Structures* and *Essential Facilities*, respectively. *Essential Facilities* generally include hospitals, fire and police stations, emergency operations centers (EOC), and supporting systems such as water tanks, emergency generators, communications centers, etc., that must continue to function in the aftermath of an earthquake. Also a facility (or SSC) may be designed to meet UBC requirements for *Essential Facilities* because it is of greater importance due to mission-dependent considerations or because it may pose a greater danger to occupants or other on-site personnel than *Standard Occupancy Structures*.

It should be noted that the intent of this *Seismic Safety Manual* is to focus primarily on conventional facilities that may adhere to the requirements of the UBC for *Standard Occupancy Structures* (PC-1) or *Essential Facilities* (PC-2) rather than those in categories PC-3 and -4. It is, however, necessary to explain all five categories in Chapter 4 so that readers have some understanding of the full nature and range of DOE's seismic safety policy and regulations. Only a small percentage of DOE facilities fall into PC-3 or PC-4. The seismic provisions for PC-3 are consistent with those used to design (or reevaluate) commercial plutonium facilities with conservatism greater than required by the UBC for *Essential Facilities* and less than required for civilian nuclear power plants. The seismic provisions for PC-4 approach those used to design (or reevaluate) civilian nuclear power plants.

The concept for graded performance provisions is to:

1. Estimate the level of intensity of shaking that will probably occur at a given site.

2. Categorize each of the site facilities (SSC's) (existing or proposed) into one of the five categories, PC-0 through PC-4, based upon its usage (or potential risk in the event of failure).

3. Evaluate existing facilities and design new facilities (or strengthen existing facilities) to meet the seismic and quality control standards required for the assigned Performance Category (PC).

The seismic and quality control standards required to meet the performance goals of each category are set to ensure that the likelihood of failure (as measured by the onset of structural distress or the onset of loss of function) is appropriate for the consequences of failure.

The earthquake loading for each site is defined at a level corresponding to its specified annual probability of occurrence, which is related

to the average return period of the design earthquake. The longer the average return period, the greater the design earthquake loading, and the less likely that loading will be exceeded in any one year. In essence, the input earthquake forces are defined on the basis of a specified annual probability that the design load will be exceeded.

Because experiences with *individual* earthquakes often surprise experts (and relate rather poorly to statistical averages), selection of earthquake loads is not a precise process even when large amounts of time and money are expended to investigate a given site. Probability analysis cannot be relied upon to pinpoint future earthquake loadings. Probability does, however, provide a reasonable basis for defining a level of loading compatible with risk calculations, given that both are based on probability assumptions.

From the structural engineer's point of view (and concern for liability), the idea that design loadings may be exceeded is not very inviting. The engineer-of-record for a given project would prefer to design a facility that will be safe under the most severe earthquake.

Fortunately, DOE seismic Standards incorporate conservative design and evaluation procedures, safety factors, and quality control measures that can be relied upon to ensure a specific level of seismic resistance based on known technology commonly practiced in the structural engineering profession. This *deterministic* level of seismic resistance in turn can be related to the target performance goals of each performance category and defined at a specified *seismic hazard exceedance probability* so that performance loads and risk probabilities can be related. In this way, the design (or evaluated) capacity can be compared in the same probabilistic terms as the risk analysis. This provides a practical methodology for setting performance standards appropriate to the hazard or to the consequences of failure due to the hazard.

It is one thing to accurately design for a given level of seismic resistance; it is another, and more difficult task, to accurately predict the specific level of loading at which failure will occur. Fortunately, it is practical to design a structure so that the mode of failure (if it ever happens) is ductile and not catastrophic. This concept is incorporated in the seismic provisions of the DOE Standards as well as those in the UBC. Consequently, deterministic techniques, not probabilistic, are used to further enhance seismic safety for facility occupants, the public, and the environment.

For facility managers, as well as structural engineers, the concept of seismic loadings and risk measurement in terms of *hazard exceedance probability* may be difficult to understand. However, in practice, the use of hazard assessment in terms of probability is neither rigorous nor very complex. It is simply a means of setting graded seismic input loads for normal structural analysis and design that are reasonably appropriate for the risks presented by the hazards involved.

A familiar application of this concept is embodied in the UBC to determine the minimum seismic forces for designing *Standard Occupancy Structures* and *Essential Facilities* that correspond to graded DOE Performance Categories PC-1 and -2, respectively. Seismic input forces are determined by the type of structural system, the facility's importance (standard occupancy or essential facility), and the probable intensity of ground shaking that may occur at the location of the site. For a given site, a "Z" factor is selected from a seismic zone map of the United States and applied to the seismic input based on the probability that an earthquake of a given intensity will occur there within a given period of years. This seismic zone map has been probability-based since the 1988 edition of the UBC.

For *Essential Facilities* that are necessary for emergency operations during and after a natural disaster such as an earthquake, the UBC applies an importance factor of 1.25 to the input base shear required for *Standard Occupancy Structures*. This factor, together with other provisions of the code that also add conservatism for *Essential Facilities*, significantly decreases the probability that the designed structure will suffer damage resulting in loss of function.

The UBC, like the DOE seismic Standards, provides seismic input loadings that are probabilistic-based, and require rigorous analysis and conservative design techniques to ensure that seismic resistance encompasses the potential deviations of the less precise probabilistic input.

The specifications of seismic hazard required by DOE are appropriate as long as the seismic design (or evaluation) of the SSCs for these earthquake levels is conservatively performed. The level of conservatism (or level of loading) should increase as one goes from PC-1 to PC-4. The criteria contained in DOE seismic Standards follow the philosophy of a gradual reduction in the probability that the design earthquake will be exceeded coupled with a gradual increase in the conservatism of analysis and design requirements going from PC-1 toward PC-4.

DOE Order 5480.28 includes a schedule for implementation of its supporting Standards, DOE-1020 through DOE-1024. These requirements are effective immediately for all new sites or new facilities. For existing facilities, it is required that an implementation plan for evaluation and upgrading be completed and submitted to DOE within one year of the effective date of the last Standard to be issued (Refs. 5 through 9). This plan must include a prioritized schedule for the sequence of evaluation and upgrade of existing facilities. Exemptions require DOE headquarters approval. Deviations may be granted by headquarters where it can be demonstrated that the deviation provides seismic protection equivalent to the DOE Standards.

A recent review by the Federal Emergency Management Agency (FEMA) found, *"At the DOE, an exemplary program has been established that contains all the elements of an agency seismic safety program. The DOE program can serve as the model for other agencies in leadership and program implementation activities of Executive Order 12699."*

References

1. *Natural Phenomena Hazards Mitigation*, DOE Order 5480.28, U.S. Department of Energy, Washington, D.C., January 15, 1993.
2. *Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction*, Executive Order 12699, Washington, D.C., January 5, 1990.
3. *Seismic Safety of Existing Federally Owned or Leased Buildings*, Executive Order 12941, Washington, D.C., December 1, 1994.
4. *Uniform Building Code*, International Conference of Building Officials, Whittier, California, 1994.
5. *Natural Phenomena Hazards Design and Evaluation for Department of Energy Facilities*, DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.
6. *Performance Categorization Criteria for Structures, Systems, and Components at DOE Facilities Subjected to Natural Phenomena Hazards*, DOE-STD-1021-93, U.S. Department of Energy, Washington, D.C., July 1993.
7. *Natural Phenomena Hazards Site Characterization Criteria*, DOE-STD-1022-94, U.S. Department of Energy, Washington, D.C., March 1994.
8. *Natural Phenomena Hazards Assessment Criteria*, DOE-STD-1023-95, U.S. Department of Energy, Washington, D.C., September 1995.
9. *Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites*, DOE-STD-1024-92, U.S. Department of Energy, Washington, D.C., December 1992.

Chapter

4a

Guidelines for Using DOE Orders and Standards for Earthquake Safety

Frank E. McClure

Introduction

Facilities managers for government-owned contractor-operated (GOCO) facilities, their staffs, and consulting architects and engineers involved with design and evaluation of U.S. Department of Energy (DOE) facilities must be familiar with applicable DOE Natural Phenomena Hazards (NPH) Orders and Standards. The number of interdependent provisions included in these documents may be rather intimidating to those unfamiliar with the DOE system. This chapter provides an overview or *road map* through the many Orders and Standards affecting seismic safety. Added guidance also is presented in terms of resource documents that provide technical information to help comply with these Orders and Standards.

Figure 4a-1 shows the relationship of federal requirements and DOE Orders for seismic safety. Examples of policy documents are Presidential Executive Orders that can be issued with or without congressional mandate. Presidential Executive Order 12699, *Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction* (January 5, 1990), and Executive Order 12941, *Seismic Safety of Existing Federally Owned or Leased Buildings* (December 1, 1994) establish the policy for the seismic design of new and existing buildings, respectively.

Rules formalizing policies are generally published in the *Code of Federal Regulations* (CFR).

DOE Orders are issued to assist in the implementation of policies. DOE Order 5480.28, *Natural Phenomena Hazards Mitigation*, sets forth DOE policy for the development of standards and guidance for designing or evaluating *Structures, Systems, and Components* (SSCs) subject to natural phenomena hazards.

DOE Standards provide technical background and criteria to carry out DOE Orders. Also, national, federal, and industry codes and consensus standards are referenced.

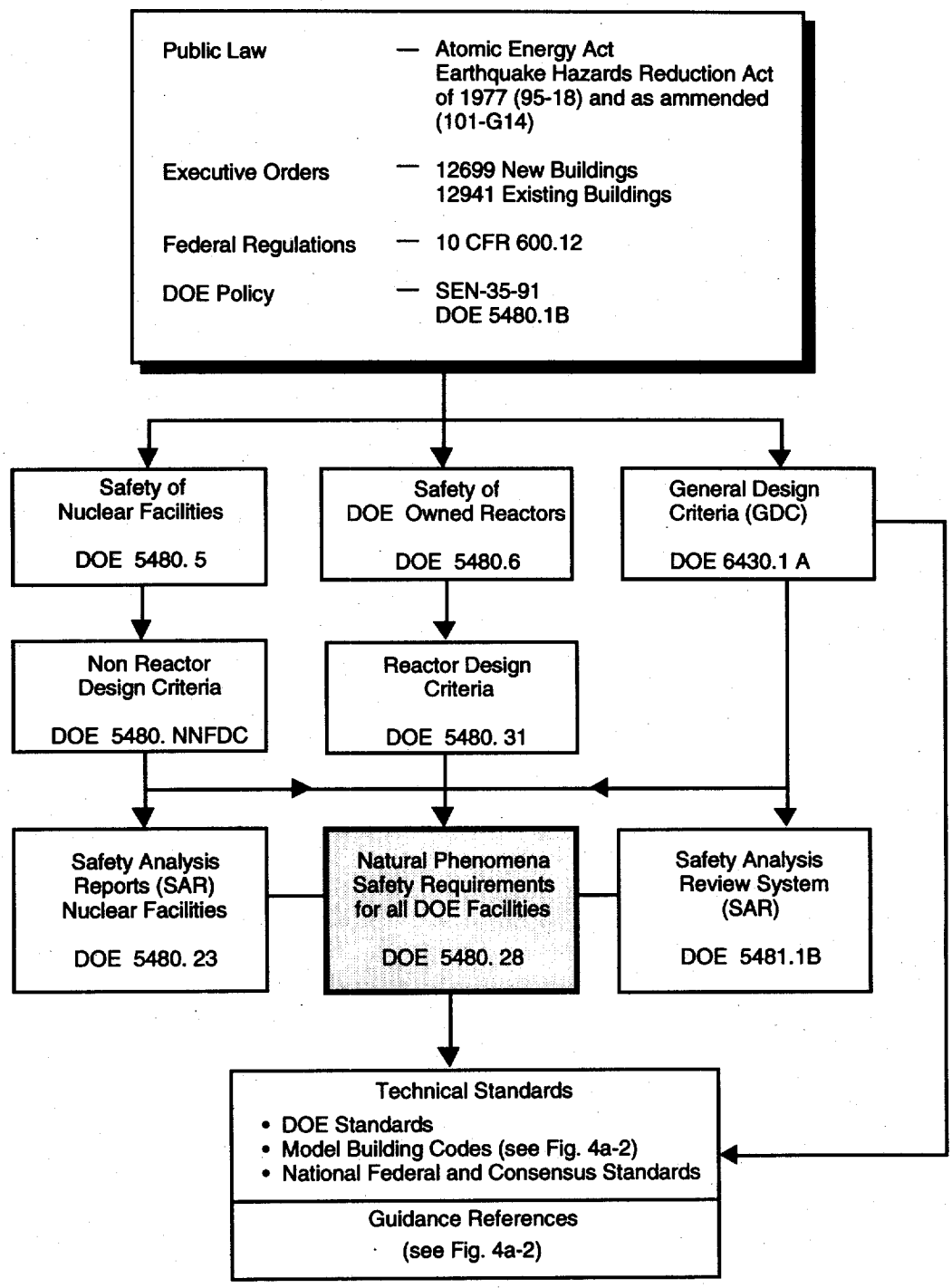


Fig. 4a-1. Relationships of federal requirements and DOE Orders for seismic safety.

Guidance consists of reference documents that address special topic areas to supplement information in the *Standards*.

Figure 4a-2 shows the relationship of standards and guidance used by DOE for seismic safety. Policies and Rules, as well as DOE Orders and Standards, are regulatory in nature. Guidance documents and industry standards, for example, promulgate good practice.

Applicable Executive Orders and DOE Orders and Standards are listed below in logical order, beginning with policy documents, followed by specific Orders, and ending with more detailed Standards.

- Presidential Executive Order 12699, *Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction* , January 1990.

- Presidential Executive Order 12941, *Seismic Safety of Existing Federally Owned or Leased Buildings,* December 1994.

- Interagency Committee on Seismic Safety in Construction (ICSSC), ICSSC RP 2.1-A, *Guidelines and Procedures for Implementation of the Executive Order on Seismic Safety of New Building Construction.*

- DOE Order 5480.1B, *Environment, Safety and Health (ES&H) Program for DOE Operations,* contains the requirements and responsibilities for ES&H programs.

- DOE Order 5480.23, *Nuclear Safety Analysis Reports,* establishes uniform requirements for the preparation and review of safety analysis reports for nuclear facilities.

- DOE Order 5480.28, *Natural Phenomena Hazards Mitigation,* establishes policy and requirements for NPH mitigation to comply with DOE Order 5480.1B. This Order establishes (1) policy requirements for mitigation of NPH at DOE facilities, (2) consistent requirements for NPHs (earthquake, wind and flood) for all DOE facilities, and (3) NPH requirements appropriate for facility characteristics and objectives within a graded approach.

- DOE Order 5481.1B, *Safety Analysis and Review System,* establishes uniform requirements for preparation and review of safety analyses. This order initially applied to all DOE facilities, but is now only applicable to non-nuclear facilities. Safety analyses for nuclear facilities are covered by DOE Order 5480.23.

- DOE Order 5700.6C, *Quality Assurance,* establishes quality assurance requirements and procedures for DOE activities.

- DOE Order 6430.1A, *General Design Criteria,* contains provisions for design and construction of DOE facilities. It does not reference DOE Order 5480.28 because 5480.28 is more recent.

- DOE-STD-1020, *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities,* specifies design and evaluation criteria for the mitigation of NPH effects and conforms to NPH mitigation requirements of DOE 5480.28. DOE-STD-1020 supersedes UCRL-15910, *Design and Evaluation Guidelines for DOE Facilities Subjected to Natural Phenomena Hazards.*

- DOE-STD-1021, *Natural Phenomena Hazards Performance Categorization Criteria for Structures, Systems, and Components,* provides criteria for selecting performance categories (PCs) of structures, systems, and components (SSCs) in accordance with DOE Order 5480.28.

- DOE-STD-1022, *Natural Phenomena Hazards Site-Characterization Criteria,* provides criteria for developing site characterization data to provide site-specific information needed for implementing DOE Order 5480.28.

- DOE-STD-1023, *Natural Phenomena Hazards Assessment Criteria,* provides criteria for the NPH assessments to be conducted to ensure that adequate design

STANDARDS

DOE Standards

DOE-STD-1020
DOE-STD-1021
DOE-STD-1022
DOE-STD-1023
DOE-STD-1024

National, Federal, and Industry Codes and Consensus Standards

Model Building Codes; UBC, NBC, SBC
NEHRP Provisions, FEMA 178 & 222
ASCE 4 and 7
ACI, AISC, ASME
NFPA, ASTM, ANSI, AWS
SEAOC Blue Book
NIST, ICSSC RP-4

GUIDANCE / REFERENCES

DOE Manuals for Seismic Safety

Seismic Safety Manual
Suspended Ceilings
Walkdown Guide
Seismic Hazard Studies
Structural Details
Equipment Tiedowns
Basis for Seismic Provisions

National, Federal, and Industrial Resource Documents

SQUG/EPRI GIP
NUREG 0800
DOD Manuals
ATC Manuals, 14, 22
FEMA Manuals
NIST, ICSSC RP-3

Fig. 4a-2. Standards and guidance used by DOE.

basis load levels are established to comply with DOE Order 5480.28.

- DOE-STD-1024, *Guidelines For Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites*, provides interim guidance on how studies developed by the Lawrence Livermore National Laboratory and the Electric Power Research Institute (EPRI) should be used to assess seismic issues for DOE sites east of 104W.

- DOE-STD-3009, *Preparation Guide for U.S. Department of Energy Nonreactor Nuclear Facility Safety Analysis Reports.*

The following discussion expands on the provisions of *Presidential Executive Orders* 12699 and 12941; DOE Order 5480.28, *Natural Phenomena Hazards Mitigation*; DOE Order 6430.1A, *General Design Criteria*; and DOE-STD-1020, *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities.*

Presidential Executive Orders

The Earthquake Hazards Reduction Act of 1977 (Public Law 95-124, as amended) was enacted by Congress to reduce risks to life and property from future earthquakes in the United States through the establishment and maintenance of an effective earthquake hazards reduction program. The *National Earthquake Hazards Reduction Program* (NEHRP) was created in response to this Act. Executive Order 12699, *Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction*, was prepared by the Interagency Committee on Seismic Safety in Construction (ICSSC) to implement certain provisions of the Earthquake Hazards Reduction Act. It was signed by President Bush on January 5, 1990.

The objectives of the Executive Order 12699 given in its Sections 1 and 2, are as follows:

> Section 1, New Federal Buildings: "The purposes of these requirements are to reduce risks to the lives of occupants of buildings owned by the Federal Government and to persons who would be affected by the failures of Federal buildings in earthquakes, to improve the capacity of essential Federal buildings to function during or after an earthquake, and to reduce earthquake losses of public buildings, all in a cost effective manner."
>
> Section 2, Federally Leased, Assisted or Regulated Buildings: "The purposes of these requirements are to reduce risks to the lives of persons who would be affected by earthquake failures of federally assisted or regulated buildings, and to protect public investments, all in a cost-effective manner."

The Earthquake Hazards Reduction Act of 1977 was amended by Public Law 101-614, to require the President to adopt standards for assessing and enhancing the seismic safety of *existing* buildings constructed for or leased by the Federal Government which were designed and constructed without adequate seismic design and construction standards.

Executive Order 12941, *Seismic Safety of Existing Federally Owned or Leased Buildings*, signed by President Clinton on December 1, 1994, adopted minimum standards developed by the ICSSC to mitigate unacceptable seismic risks in those buildings.

Executive Order 12941 requires each federal agency to submit cost estimates to the *Federal Emergency Management Agency* (FEMA) for mitigating unacceptable seismic risks in its buildings within 4 years (by December 1, 1998). FEMA, in consultation with ICSSC, is then to report to Congress about how to achieve seismic safety in these buildings in an economically feasible manner. Thereafter, FEMA is to report to Congress on the implementation of Executive Order 12941 on a biennial basis. ICSSC is to update the minimum standards within two years of the publication of the first edition of FEMA's *Guidelines for Seismic Rehabilitation of Buildings and Commentary* and at least every five years thereafter.

DOE Order 5480.28 Natural Phenomena Hazards Mitigation

DOE Order 5480.28 establishes policy for design, construction, and operation of DOE facilities so that workers, the general public,

and the environment are protected from the impacts of natural phenomena hazards on DOE facilities. The goals of the Order are to achieve the following goals in a cost-effective manner:

- Provide for safe workplaces
- Protect against property loss or damage
- Provide for continued operation of essential facilities
- Provide for continued public health
- Protect the environment against exposure to hazardous materials.

Order 5480.28 establishes DOE policy and requirements for mitigation of natural phenomena hazards (NPH), including seismic, wind, and flood, for all DOE sites and facilities, using a graded approach.

Its requirements cover:

- Assessment of NPHs for new and existing buildings
- Natural phenomena effects to be considered
- Design of new facilities
- Evaluation and upgrade of existing facilities
- Design of additions and modifications
- Target performance goals
- Graded approach and performance categories
- Interaction and common cause effects
- Instrumentation
- Evaluation of NPH events and lessons learned.

DOE Order 6430.1A General Design Criteria

DOE Order 6430.1A implements the policies of DOE Order 5480.1B, *Environment, Safety, and Health,* and DOE Order 5481.1B, *Safety Analysis and Review System.* DOE Order 5480.28, *Natural Phenomena Hazards Mitigation,* is not referenced in 6430.1A because it is more recent.

DOE Standards, Guidance, and practices are developed and promulgated by DOE 6430.1A to provide a level of design for occupant life-safety, reduction in loss of government property, continued functioning of essential operations, and confinement of hazardous material.

For *Standard Occupancy Structures* and *Essential Facilities,* DOE Order 6430.1A references the seismic requirements of the *Uniform Building Code* (UBC). Recent evaluations under ICSSC procedures have found that the standards used by DOE (DOE-STD-1020) for design and evaluation of buildings are *substantially equivalent* to the *NEHRP Provisions* developed by ICSSC for use under Executive Order 12699 and are substantially more conservative for levels of design beyond those judged acceptable for life safety.

DOE Order 5480.28 does not supersede DOE Order 6430.1A requirements related to seismic and other natural phenomena hazard mitigation, but governs where there are inconsistencies between these two Orders.

DOE-STD-1020 Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities

DOE-STD-1020 is the primary reference for structural engineers performing design and evaluation of Structures, Systems, and Components (SSCs) for conformance with DOE Order 5480.28.

DOE-STD-1020 was developed from and supersedes the 1990 version of UCRL 15910, *Design and Evaluation Guidelines for DOE Facilities Subjected to Natural Phenomena*

Hazards, which was originally specified for use by DOE 6430.1A, *General Design Criteria*.

The earthquake design and evaluation criteria provided in DOE-STD-1020 set forth procedures for achieving the performance goals specified in DOE Order 5480.28. These criteria are intended to result in uniform design and evaluation for protection against NPH at all DOE sites. The goal is to ensure that DOE facilities can withstand the effects of natural phenomena hazards, such as earthquakes, extreme winds, tornadoes, and floods. These criteria control the level of conservatism introduced in the design/evaluation process so that all NPH (earthquake, wind, and flood) are treated on a reasonably consistent and uniform basis.

Fig. 4a-3 illustrates the flow of steps through DOE Orders and Standards to determine NPH design criteria for SSCs.

DOE-STD-1020 criteria also employ a graded approach by ensuring that the level of conservatism and rigor in design/evaluation is appropriate for facility characteristics, such as importance, cost, and hazards to people on and off site and to the environment.

For each NPH covered, these criteria generally consist of the following:

- Performance categories and target performance goals
- Specified probability levels for which NPH loadings on structures, equipment, systems, and components must be developed
- Design and evaluation procedures to evaluate response to NPH loads and criteria to assess whether or not the computed response is permissible.

Seismic performance goals are related to the annual probability of exceedance of acceptable behavior limits for structures and equipment as a result of earthquakes. Different criteria are provided for each of four performance categories, each of which has a specified performance goal. Performance categories and performance goals range from *Standard Occupancy Structures* (the lowest seismic occupancy category in the UBC) to highly hazardous occupancy uses (approaching nuclear power plant provisions).

Earthquake design and evaluation criteria in DOE-STD-1020 are aimed at meeting the performance goals described in Table 4a-1.

For SSCs in Performance Category 1 (PC-1), the primary goal is preventing major structural damage or collapse that would endanger personnel. The cost to repair or replace a SSC or the ability of a SSC to continue to function after an earthquake is not a controlling factor.

SSCs in PC-2 are of greater importance because of mission-dependent considerations. In addition, these SSCs may pose a greater danger to on-site personnel than those in Performance Category 1 because of the type of operations or materials involved. The performance of SSCs in PC-2 should allow only relatively minor structural damage that can be easily and readily repaired following the event, and only minimal interruption to operations. The performance goal is slightly more conservative than that for UBC *Essential Facilities* occupancies (e.g., hospitals, fire and police stations, centers for emergency operations, etc.).

SSCs in Performance Categories 3 and 4 pose potential hazards to public safety and the environment because highly toxic materials may be present. Design goals for these categories are intended to limit SSC damage so that hazardous materials can be controlled and confined, occupants and the public are protected, and the SSC will continue to function during and after the natural hazard occurs. For these performance categories, damage must be confined within specified barriers (e.g., buildings, glove boxes, storage canisters, vaults, and ventilation systems). Filtering and monitoring control equipment must continue to operate effectively in the event of severe earthquakes, winds, or floods.

Design and evaluation criteria are aimed at target probabilistic performance goals that require probabilistic NPH assessments. Design loads are specified from such assessments by developing NPH annual probabilities of

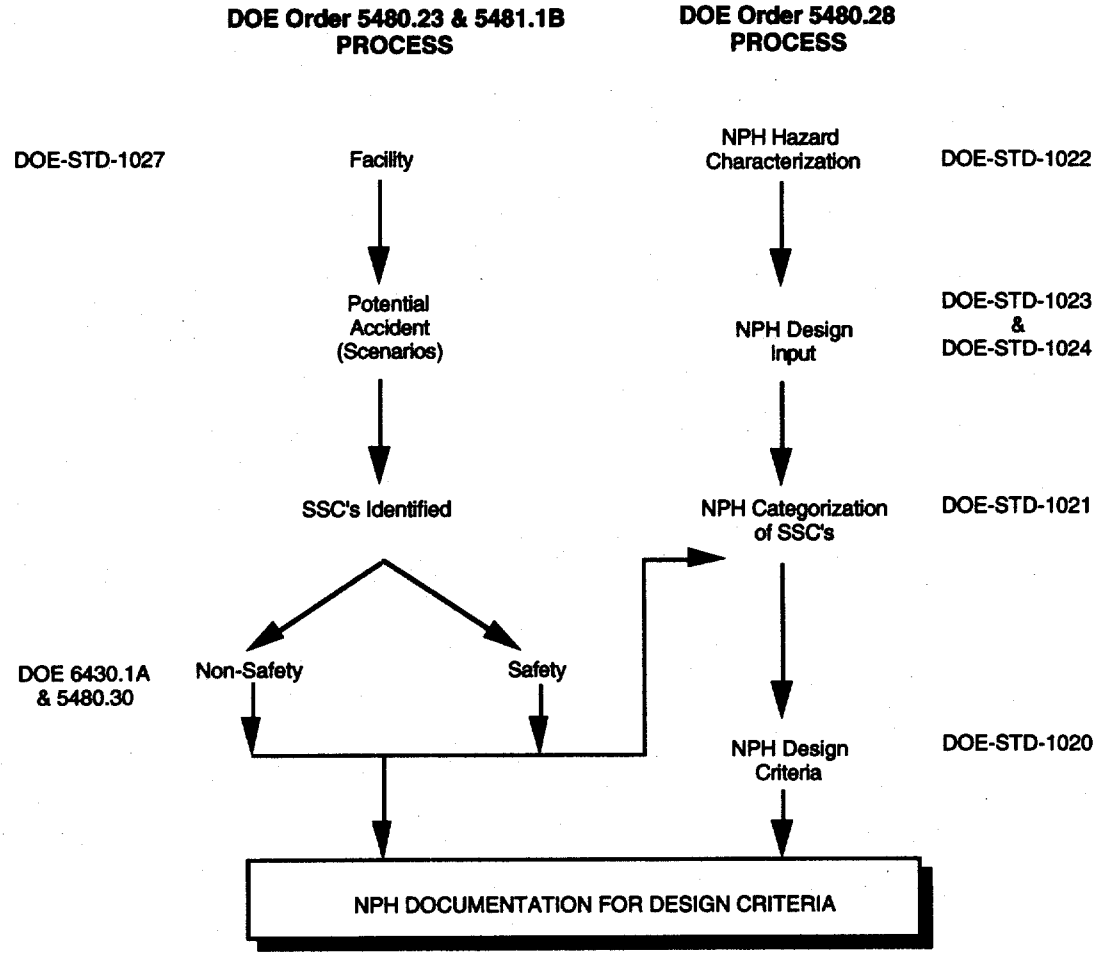


Fig. 4a-3. Flow chart for determining NPH design criteria for SSCs.

exceedance. Table 4a-2 presents target seismic performance goals, seismic hazard exceedance probabilities, and the risk reduction ratios for the four performance categories. Related earthquake return periods are also shown (see DOE-STD-1020 for details).

Performance goals may then be achieved by using the conservatively selected probabilistic loads, combined with deterministic design and evaluation procedures that provide a consistent and appropriate level of conservatism. To ensure that performance of a SSC will meet intended goals, the deterministic details of its design must encompass the potential exceedance of the selected probabilistic input.

Design/evaluation procedures closely conform to standard practices so they are easily understood by most engineers.

DOE Guidance Documents, Courses, Workshops, and Conferences

DOE has prepared a number of guidance documents to support its NPH regulations as shown in Fig. 4a-2. The following manuals are very helpful for design and evaluation of DOE facilities for seismic safety.

Table 4a-1. Seismic performance goals for structures, systems, and components (SSCs)[1] for various performance categories.

Performance Category	Performance goal description	Seismic target performance goal, annual probability of exceeding acceptable behavior limits
0	No precautions for safety, mission, or cost required	No requirements
1	Maintain occupant safety	$\simeq 10^{-3}$ of the onset of SSC damage to the extent that occupants are endangered
2	Occupant safety, continued operation with minimum interruption	$\simeq 5x10^{-4}$ of SSC damage to the extent that the component cannot perform its function
3	Occupant safety, continued operation, hazard confinement	$\simeq 10^{-4}$of SSC damage to the extent that the component cannot perform its function
4	Occupant safety, continued operation, confidence of hazard confinement	$\simeq 10^{-5}$ of SSC damage to the extent that the component cannot perform its function

(1) SSC refers to structure, distribution system, or component (equipment).

Table 4a-2. Seismic performance goals and specified seismic hazard probabilities.

Performance category PC	Target seismic performance goal		Seismic hazard exceedance		Risk reduction ratio, R_R
	Probability P_F	Return period, yrs	Probability P_H	Return period, yrs	
1	$1x10^{-3}$	1,000	$2x10^{-3}$	500	2
2	$5x10^{-4}$	2,000	$1x10^{-3}$	1,000	2
3	$1x10^{-4}$	10,000	$5x10^{-4}$ ($1x10^{-3}$)[1]	2,000 (1,000)[1]	5 (10)[1]
4	$1x10^{-5}$	100,000	$1x10^{-4}$ ($2x10^{-4}$)[1]	10,000 (5,000)[1]	10 (20)[1]

(1) For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC, which are near tectonic plate boundaries.

- UCRL-CR-106554, *Structural Concepts and Details for Seismic Design*, Lawrence Livermore National Laboratory, Livermore, California, September 1991.

- UCRL-15815, *Practical Equipment Seismic Upgrade and Strengthening Guidelines*, Lawrence Livermore National Laboratory, Livermore, California, September 1986.

- UCRL-15714, *Suspended Ceiling Systems Survey and Seismic Bracing Recommendations*, Lawrence Livermore National Laboratory, Livermore, California, August 1985.

DOE Headquarters, Office of Risk Assessment and Technology, working with Lawrence Livermore National Laboratory, has sponsored many courses, workshops, and conferences that have resulted in valuable notes and conference proceedings. Some are listed below.

- Notes on *Short Course on Seismic Base Isolation* held in Berkeley, California, in 1992.

- Notes on *Walkdown Procedures To Mitigate Natural Phenomena Hazards*, workshops held at various locations in the United States.

- Notes from *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, workshops held at various locations in the United States.

- *Conference Proceedings* from DOE Natural Phenomena Hazards Mitigation Conferences held at Las Vegas, Nevada, 1985; Knoxville, Tennessee, 1989; St. Louis, Missouri, 1991; Atlanta, Georgia, 1993; and Denver, Colorado, 1995.

Chapter

4b

DOE Natural Phenomena Hazards Mitigation Requirements and Standards

James R. Hill
Robert C. Murray

Natural Phenomena Hazards Mitigation Order DOE 5480.28

As part of a coordinated effort to ensure proper performance of DOE facilities subjected to natural phenomena hazards (NPH), *Natural Phenomena Hazards Mitigation* Order DOE 5480.28 (Ref. 1) was developed to establish:

- Policy requirements for mitigation of NPHs at DOE facilities
- Requirements consistent with building codes and national standards, including the *National Earthquake Hazards Reduction Program* (NEHRP) *Provisions*, for all DOE facilities
- NPH protection requirements appropriate for facility characteristics and objectives within a graded approach.

A graded approach is one in which facilities are placed into categories so that the required level of performance is commensurate with:

- The relative importance of the facility to safety and the environment
- The expected magnitude of natural hazards the facility may be subject to
- The importance of the programmatic mission of the facility
- The particular characteristic of the facility
- The cost and replaceability of the facility.

Order DOE 5480.28 embodies the principles incorporated in DOE Order 6430.1A, *General Design Criteria* (Ref. 2), and UCRL-15910, *NPH Design and Evaluation Guidelines* (Ref. 3), which had been in use at DOE facilities for a number of years. Order DOE 6430.1A contains criteria for the design and construction of DOE facilities. DOE 5480.28 does not supercede DOE 6430.1A, but it does decree that its implementing standard DOE-STD-1020-94, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, supercedes UCRL-15910. In addition, through its implementing standards, it establishes as requirements good practices that have been used at many of the DOE sites. These requirements cover:

- Assessment of NPHs for new and existing sites
- Natural phenomena effects to be considered
- Design of new facilities
- Evaluation and upgrade of existing facilities
- Design of additions and modifications
- Target performance goals
- Graded approach and performance categories
- Interactions (colateral damage) and common cause effects
- Instrumentation.

The goals of Order DOE 5480.28 are to:

- Provide for safe workplaces
- Protect against property loss or damage
- Provide for continued operation of essential facilities
- Protect public health and the environment against exposure to hazardous materials (off-site consequences).

The key DOE NPH standards required to implement the order are listed here and summarized in the text that follows:

- DOE-STD-1020, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (Ref. 4).
- DOE-STD-1021, *Natural Phenomena Hazards Performance Categorization Criteria for Structures, Systems, and Components* (Ref. 5).
- DOE-STD-1022, *Natural Phenomena Hazards Site Characterization Criteria* (Ref. 6).
- DOE-STD-1023, *Natural Phenomena Hazards Assessment Criteria* (Ref..7).
- DOE-STD-1024, *Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites* (Ref. 8).

For new facilities, DOE Order 5480.28 became effective in January 1993. For existing facilities, implementation is planned over several years because of resource constraints and programmatic mission considerations. To start the process of evaluating and upgrading existing facilities, the order requires contractors/operators to establish an implementation plan that contains a prioritized schedule for evaluation of current and future NPH mitigation actions. This plan is to be submitted to the DOE Headquarters Program Office within one year after all implementing standards have been issued.

The issuance and implementation of DOE Order 5480.28 provide the framework for meeting Presidential Executive Order 12699 (Ref. 9) for seismic hazard mitigation of new federal facilities. Executive Order 12941 (Ref. 10) requires inventory, screening, and evaluation of existing facilities.

Fig. 4a-1 in Chapter 4a shows documents used in the DOE NPH mitigation program.

Natural Phenomena Hazards Design And Evaluation Criteria DOE-STD-1020

Design and evaluation criteria to implement DOE Order 5480.28 requirements are provided in DOE-STD-1020. The conceptual and technical basis for this standard was developed in UCRL-15910 over several years of use, review, and revision. The basis for the seismic provisions in DOE-STD-1020 are presented in Ref. 11.

The NPH Mitigation Order requires that each DOE facility, including *Structures, Systems, and Components* (SSCs), be assigned to *Performance Categories* (PC) numbered PC-0 through PC-4, each with a qualitative performance goal for behavior (i.e., maintain structural integrity, maintain ability to function, maintain confinement of hazardous materials) and a quantitative target probabilistic performance goal (expressed as an annual probability of exceedance of acceptable behavior limits [i.e., behavior limits beyond which damage/failure is unacceptable]). DOE-STD-1020 provides four sets of NPH design and evaluation criteria (explicit criteria are not needed for PC-0). These criteria range from those provided by

model building codes (normal occupancy) for PC-1 to those approaching nuclear power plant criteria for PC-4. Table 4b-1 illustrates how DOE-STD-1020 criteria for the performance categories defined in DOE 5480.28 compare with NPH criteria from other sources.

Design and evaluation criteria in DOE-STD-1020 use deterministic procedures that establish SSC loadings from probabilistic NPH curves; specify acceptable methods for evaluating SSC response to these loadings; provide acceptance criteria to judge whether the computed SSC response is acceptable; and provide detailing requirements so that behavior is as expected, as illustrated in Fig. 4b-1. These criteria are intended to apply equally to the design of new facilities and to the evaluation of existing facilities. In addition, criteria cover buildings, equipment, piping, and other structures and combine probabilistic and deterministic methods to achieve performance goals.

The annual probability of exceedance of SSC damage as a result of a NPH event (i.e., performance goal) is a combined function of the annual probability of exceedance of the event, factors of safety introduced by the design/evaluation procedures, and other sources of conservatism. The ratio of the hazard annual probability of exceedance and the performance goal annual probability of exceedance is called the *risk-reduction ratio*, R_R, in DOE-STD-1020. This ratio establishes the level of conservatism to be employed in the design or evaluation process. For example, if the performance goal and hazard annual probabilities are the same ($R_R = 1$), the design or evaluation approach would introduce no conservatism. However, if conservative design or evaluation approaches are employed, then the hazard annual probability of exceedance can be larger (i.e., more frequent) than the performance goal annual probability ($R_R > 1$). Seismic criteria are provided in DOE-STD-1020 that achieve risk-reduction ratios, R_R, of 2, 5, 10, and 20.

The report entitled *Basis for Seismic Provisions of DOE-STD-1020* (Ref. 11) demonstrates that performance goals are achieved when:

- The design/evaluation basis earthquake is defined from probabilistic seismic hazard curves, and
- Conservatism in acceptance criteria results in less than 10% probability of unacceptable performance at a *Scaled Design Basis Earthquake* (SDBE).

The SDBE probabilistic definition is used to account for the potential wide variation in site-specific seismic hazard curve slopes, while using a single deterministic acceptance criterion for the range of all potential hazard curve slopes. DOE-STD-1020 encourages obtaining site-specific probabilistic NPH assessments, and requires them for facilities assigned to PC-3 and higher. The required factor of safety, F_R, to achieve risk-reduction ratios, R_R, of 2, 5, 10, and 20 is a function of the hazard curve slope, proximity to tectonic plate boundaries, and uncertainties in seismic capacities of SSCs. For the potential range of hazard curve slopes and capacity variabilities, values of the factors have been evaluated for each desired R_R.

For PCs 1 and 2, the risk-reduction ratio, R_R, of 2 is achieved by F_R of unity which, in turn, is judged to be achieved by following a seismic-acceptance criterion in accordance with the *Uniform Building Code* (UBC). For PC-3 and PC-4, risk-reduction ratios, R_R, are 5 and 10, respectively, except for DOE sites near tectonic plate boundaries where they are 10 and 20, respectively. The required factors for these values of R_R are achieved in the seismic-acceptance criteria by specifying conservative material strengths, structural capacities, inelastic energy-absorption factors, and a seismic scale factor.

DOE-STD-1020 seismic provisions differ from those in UCRL-15910 in that a single value of *Inelastic Energy Absorption Factor*, F_u, is specified for both PC-3 and PC-4, and a scale factor, SF, is used to increase the inelastic demand forces by 1.25 for PC-4 over that required for PC-3. This change was needed because UCRL-15910 specified F_u of 1.0 for brittle modes for both moderate and high-hazard categories. As a result, different performance goals for each category were not achieved for these brittle-failure modes. For PC-4, this change results in slightly more liberal criteria for ductile-failure modes and slightly more conservative criteria for brittle-failure modes.

DOE-STD-1020 also includes damping ratios provided at several response levels to be used in structural evaluations for the purpose of

Table 4b-1. Comparison of performance categories from various sources.

Source	SSC categorization				
DOE-STD-1020	0	1	2	3	4
Uniform Building Code		Standard occupancy structures	Essential facilities		
DOD Tri-Services Manual for Seismic Design for Essential Buildings				High risk	
Nuclear Regulatory Commission				Evaluation of fuel facilities	Evaluation of existing reactors

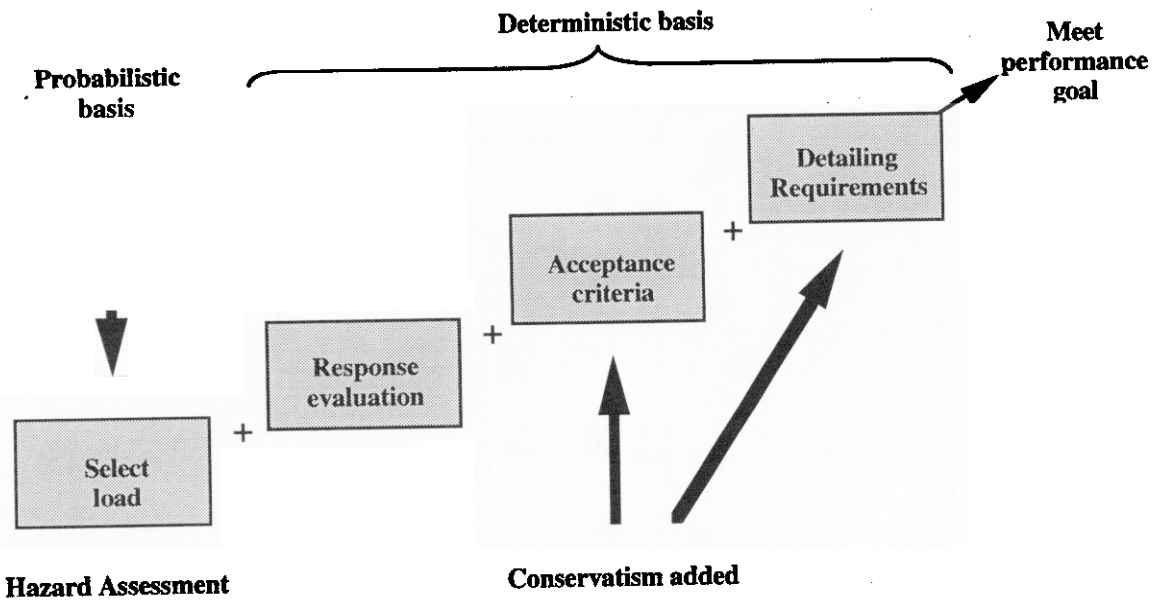


Fig. 4b-1. DOE-STD-1020 combines probabilistic and deterministic methods to achieve performance goals.

determining in-structure response spectra. As well, seismic interaction (colateral damage) is explicitly addressed.

Natural Phenomena Hazards Performance Categorization for Structures, Systems and Components DOE-STD-1021

Safety engineers and facility managers responsible for the design or evaluation of SSCs need to select levels of NPH and corresponding amounts of conservatism in NPH design and evaluation criteria that are appropriate for SSC designs. Design criteria are selected on the basis of the failure consequences of the SSC. For example, if the failure of a large dam may destroy a city, it is likely to be designed for a long recurrence-period precipitation (larger storm). In contrast, because far fewer potential fatalities would result from the collapse of a warehouse roof, that roof may be designed for a much smaller recurrence-period precipitation (smaller storm). Still, if both designs are optimally performed, members of the public will be exposed to relatively the same level of risk from the dam as from the warehouse roof.

To attain similar uniformity in risk from the potential failure of various SSCs that are present in DOE facilities, DOE Order 5480.28 requires that all SSCs be placed in one of the five PCs commensurate with the consequences of failure. Performance goals for these categories are specified in terms of *target* annual failure probabilities. DOE-STD-1021 provides guidelines for placing SSCs into these performance categories. The basic categorization process (excluding system interaction effects) is outlined in the flow chart in Fig. 4b-2.

DOE-STD-1021 uses the results of SSC safety classification, facility hazard classification, and accident analyses that are performed in conformance with other DOE Orders.

Performance Category 4 (PC-4) SSCs are defined as safety class SSCs in facilities with sufficient quantities of radioactive/highly toxic materials and (explosive) energy that, if released, can potentially result in prompt off-site fatalities.

Performance Category 3 (PC-3) SSCs are defined as safety class SSCs in facilities with sufficient quantities of radioactive/highly toxic materials and energy that necessitate on-site emergency planning.

Performance Category 2 (PC-2) SSCs include safety class SSCs in essential facilities or those with sufficient quantities of radioactive materials to be reportable to the Environmental Protection Agency (EPA). Additionally, PC-2 SSCs must perform their emergency functions to preserve the health and safety of those working with hazardous materials and co-located personnel.

Building structures or components with potential human occupancy and SSCs whose failure can potentially cause a life-threatening situation for occupants are placed in PC-1. SSCs that have no effect on safety, mission, or cost need not be designed to withstand NPH loads and may be placed in PC-0.

Problems of system interactions (colateral damage) caused by seismic shaking are important, but often overlooked. Therefore, DOE-STD-1021 places special emphasis on the consideration of system interaction effects in the process of NPH categorization. Simple-to-use, but conservative, rules are provided to preclude adverse effects of lower category SSCs (sources) that may impact on or damage higher category SSCs (targets). These rules satisfy the basic requirement that the performance goal of the target SSCs must not be compromised. Several examples are provided in DOE-STD-1021 to illustrate the use of the interaction rules.

Natural Phenomena Hazards Site Characterization Criteria DOE-STD-1022

The purpose of this standard is to provide criteria for developing site characterization and identifying site-specific information needed for design and evaluation of DOE facilities subject to natural phenomena hazards. The standard covers geologic, seismologic, geotechnical, hydrologic, and meteorologic aspects of site characteristics. The seismic-related hazards include site earthquake ground shaking; tectonic site deformation; ground failure induced by ground shaking including liquefaction, differential compaction, and landsliding; and earthquake-induced flooding.

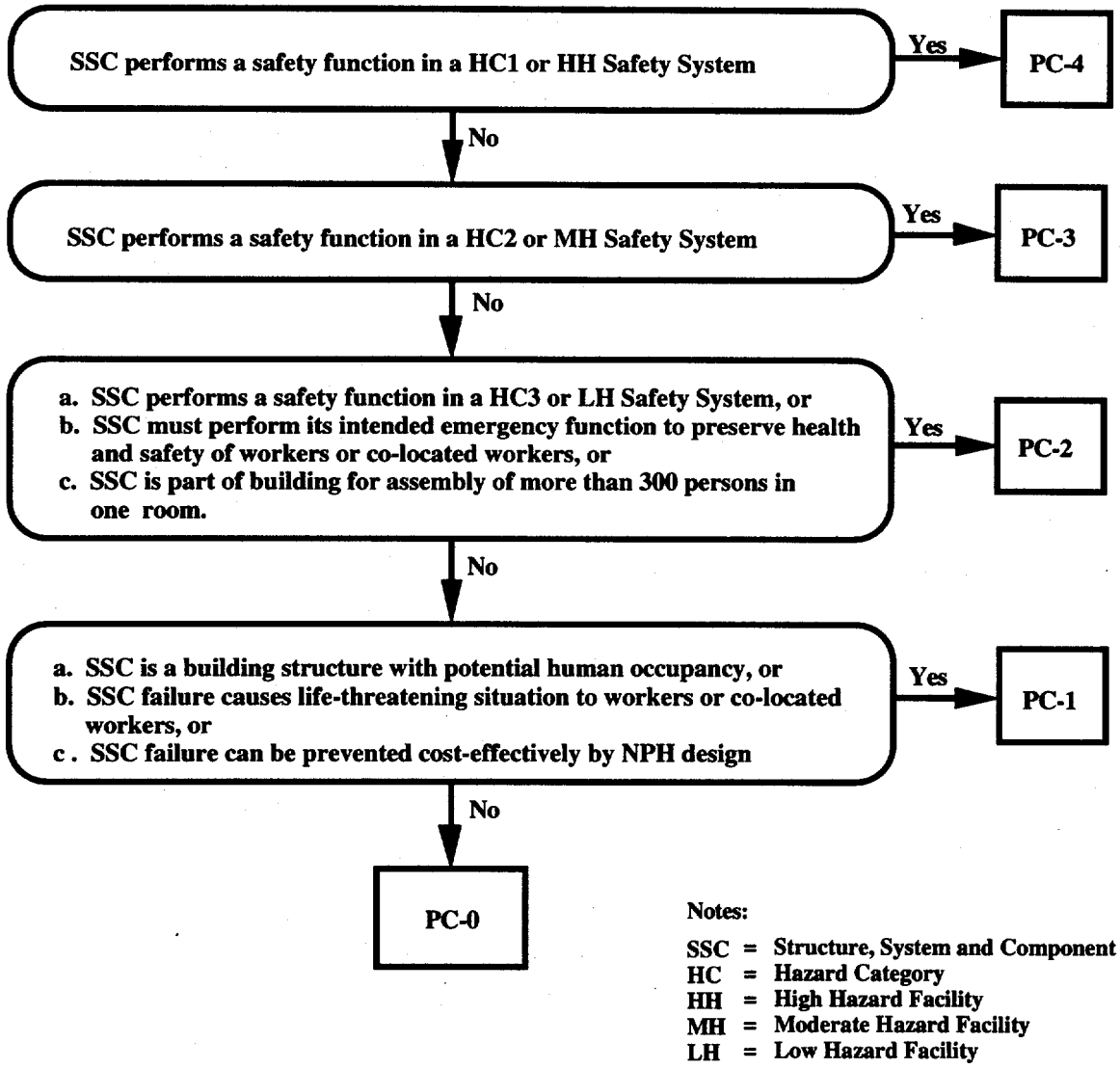


Fig. 4b-2. Basic criteria for preliminary NPH performance categorization of structures, systems, and components.

Considerations of geologic and seismologic aspects include identifying and characterizing seismic sources and resulting site ground motions, evaluating the potential for tectonic and other site deformation, and earthquake-induced flooding, and evaluating volcanic hazards. Major considerations of geotechnical aspects involve defining site soil properties needed for hazard evaluation and seismic engineering analyses, evaluating site soil-amplification effects on ground motions, conducting seismic soil-structure interaction analyses, and evaluating the potential for site ground failure induced by ground shaking.

Hydrologic and meteorologic aspects of site characterization criteria also are important. Site studies of hydrologic aspects include determination of ground-water conditions, flood runoff, drainage, and other hydrologic characteristics that could influence the design or operation of DOE facilities. Ground-water conditions include ground-water levels, flow patterns, permeability, porosity, and hydraulic gradients at the site, as well as the chemical analysis of the ground water. Site studies on meteorologic aspects should be performed to provide sufficient information for the design and evaluation basis of wind and tornado hazards.

Seasonal weather conditions of the local site and region, including temperature, precipitation, relative humidity, and prevalent wind direction, need to be determined. The occurrences and intensity of heavy rain, snow, ice storms, and thunderstorms, as well as strong wind, tornadoes, and hurricanes, need to be determined.

The scope and degree of detail of investigations to address these natural hazards depend on several factors, including the hazard classification of the facilities; the subsurface conditions at the site; the seismologic, hydrologic, and meteorologic environments of the site region; and the extent of prior knowledge, investigations, and data regarding the site and site region. An appropriate scope of investigation should be developed for a particular facility after considering all these factors. For example, although more detailed investigations are generally appropriate for facilities having higher hazard classifications, investigations of lesser scope and detail may be appropriate when existing knowledge of the site and region is relatively high. Similarly, although less-detailed investigation would generally be commensurate with lower-hazard facility classification, more comprehensive investigations may be needed if investigations to define the hazards have not been conducted previously.

Natural Phenomena Hazards Assessment Criteria Standard DOE-STD-1023

DOE Order 5480.28 requires a probabilistic assessment of the likelihood of future NPH occurrence. The level of probabilistic NPH assessment to be conducted must be appropriate for the performance category being considered, consistent with the graded approach. For sites containing facilities with SSCs in PC-3 and PC-4, a site-specific probabilistic NPH assessment must be conducted. For sites containing facilities with SSCs in PC-1 and PC-2 only and having no site-specific probabilistic NPH assessment, it is sufficient to utilize NPH maps from model building codes or national consensus standards.

DOE Standard DOE-STD-1023 provides criteria for NPH assessments to ensure that adequate design-basis loadings are established for design and/or evaluation of DOE facilities. It provides general assessment criteria for all NPHs, as well as specific criteria for the assessment of seismic hazard, wind, tornado, and flood. The emphasis is on site-specific probabilistic NPH assessment, i.e., development of NPH curves. Criteria for development of design response spectra to define the seismic input motion are also provided.

There are five steps in probabilistic seismic hazard assessments, as shown in Fig. 4b-3. These include:

a. Development of zonation maps around the site indicating faults and seismic source areas

b. Development of recurrence relations indicating how many earthquakes of specific magnitude are possible in each source area. Zonation and recurrence are developed by seismicity experts

c. Attenuation relationships indicating peak ground acceleration for earthquakes of specified magnitude as a function of distance from the source. These are developed by ground motion experts

d. The above information is combined probabilistically to develop seismic hazard curves showing *Peak Ground Acceleration* (PGA) as a function of *Probability of Exceedence* (PE). Uncertainity in the parameters is included

e. In addition to the hazard curve, response spectra showing the frequency content of the expected ground motions also are developed.

Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites DOE-STD-1024

This standard provides guidance in the use of seismic hazard curves developed by the Lawrence Livermore National Laboratory (LLNL) and the Electric Power Research Institute (EPRI) for estimating seismic hazards at DOE sites in the eastern United States (East of longitude 104W).

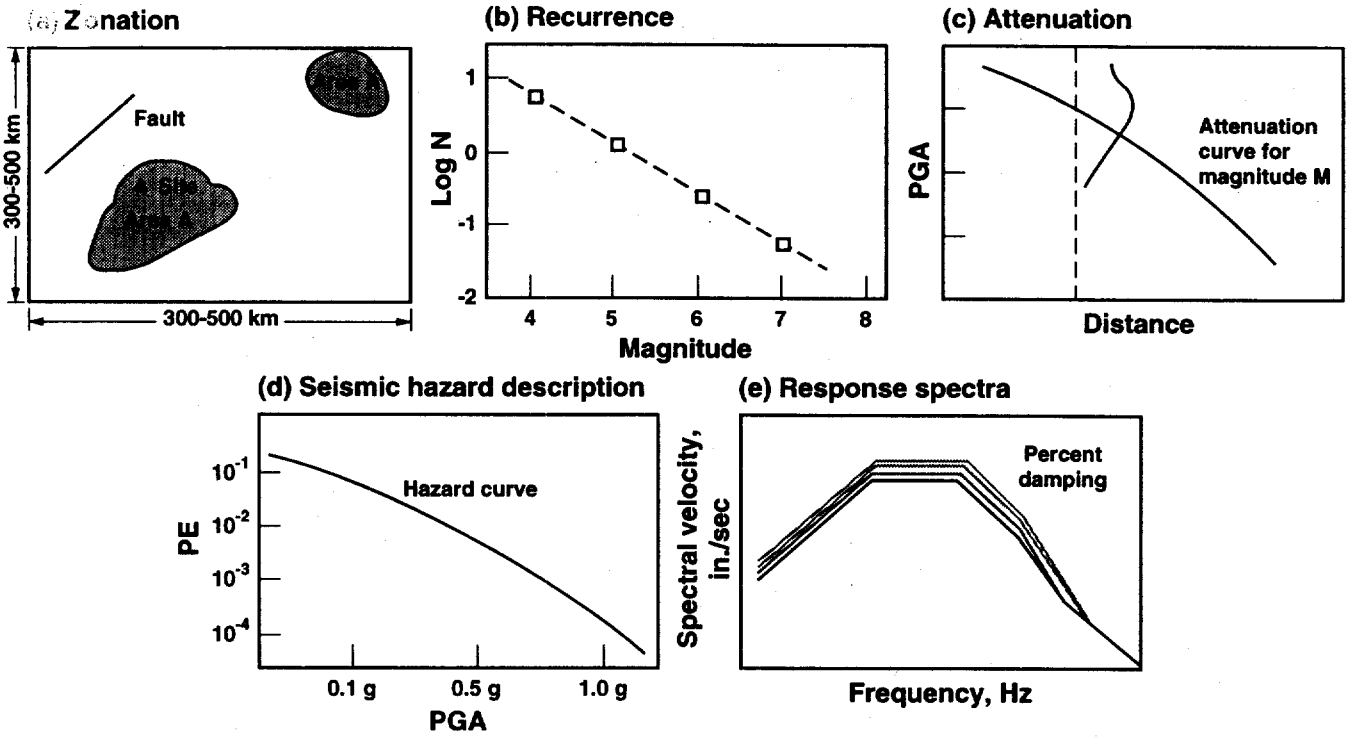


Fig. 4b-3. Steps in probabalistic seismic hazard assessment.

References

1. *Natural Phenomena Hazards Mitigation*, DOE Order 5480.28, U.S. Department of Energy, Washington, D.C., January 15, 1993.

2. *General Design Criteria*, DOE Order 6430.1A, U.S. Department of Energy, Washington, D.C., April 6, 1989.

3. *Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards*, UCRL-15910, R. P. Kennedy, S. A. Short, J. R. McDonald, M. W. McCann, R. C. Murray, J. R. Hill, Lawrence Livermore National Laboratory, Livermore, California, June 1990.

4. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.

5. *Performance Categorization Criteria for Structures, Systems, and Components at DOE Facilities Subjected to Natural Phenomena Hazards*, DOE-STD-1021-93, U.S. Department of Energy, Washington, D.C., July 1993.

6. *Natural Phenomena Hazards Site Characterization Criteria*, DOE-STD-1022-94, U.S. Department of Energy, Washington, D.C., March 1994.

7. *Natural Phenomena Hazards Assessment Criteria*, DOE-STD-1023-95, U.S. Department of Energy, Washington, D.C., September 1995.

8. *Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites*, DOE-STD-1024-92, U.S. Department of Energy, Washington, D.C., December 1992.

9. *Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction*, Executive Order 12699, Washington, D.C., January 5, 1990.

10. *Seismic Safety of Existing Federally Owned or Leased Buildings*, Executive Order 12941, Washington, D.C., December 1, 1994.

11. *Basis for Seismic Provisions of DOE-STD-1020*, UCRL-CR-111478, R. P. Kennedy, S. A. Short, Lawrence Livermore National Laboratory, Livermore, California, April 1994.

Chapter

5

Site-Use Planning for Earthquake Safety

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

The extent to which a site should be investigated varies with the types and degrees of natural hazards present and the possible consequences of damage. It also varies directly with the complexity of the geology and seismology of the site and/or the difficulty of diagnosis. One important point to keep in mind is that it is easy to dissipate funds in site investigation work before the problems, priorities, and direction of the broader earthquake safety program are fully understood. Detailed work should always be carried out after the other facets of earthquake safety have been considered and the objectives of further work are more clearly defined.

An important practical goal of site-use planning for earthquake safety is to identify potential natural hazards such as unstable slopes and existing landslides; areas subject to dynamic subsidence, liquefaction, or strength loss under ground shaking; and of course, fault movement. The object of such an investigation is to avoid seismic hazards, if possible, or to mitigate them if it is not practical to avoid them.

One needs to think in terms of shaking the whole site, then visualizing what might happen in a geotechnical sense. For professionals with experience in soil dynamics, this is not a difficult task, and it need not (and cannot) be precise. Uncertainties always remain even after exhaustive investigations. *The important thing is to recognize that a particular hazard does indeed exist so that it can be dealt with in the planning stage.* This is true whether it is a new site to be developed or an existing site involving the retrofit or expansion of existing facilities.

Given that a specific hazard exists, further investigation can be undertaken to more thoroughly understand the threat if it cannot be avoided. Uncertainties that remain can be dealt with under the envelope of the more definitive process of structural design rather than in the less certain process of predicting seismic input.

Development of site-specific criteria for seismic design is one of the more sensitive processes that must be carefully managed to avoid technical and political pitfalls. Usually, the need to develop site-specific criteria relates to dynamic analysis rather than static lateral force analysis. Unfortunately, there is persistent misunderstanding and confusion about the meaning and use of ground-surface acceleration as a measure of the earthquake resistance of buildings. This applies not only to public perception, but also to most facility managers and engineers who do not have the technical insight and experience of earthquake engineers. This confusion is amplified through continued, but

usually unintentional, misrepresentation of the issue by public communication media. The seismic input values of ground acceleration used in dynamic analysis are generally much greater than the values used for static analysis. However, when each type of analysis is properly conducted, the resulting designs should provide equal protection against earthquakes. People who do not understand this tend to be critical of the lower ground accelerations used for *equivalent* static design analyses. One sometimes hears or reads that the values used for static design are in error because they are lower than real peak ground accelerations. The following example illustrates this issue:

A low-rise building having a ductile lateral-force-resisting system analyzed for a 0.2g static lateral force, and having been well-designed and constructed to code, should resist actual ground accelerations of 0.8g without collapse.

An explanation for this apparent paradox is contained in *Earthquake Design Criteria*, G.W. Housner, and P.C. Jennings, Division of Engineering and Applied Science, California Institute of Technology; a monograph published by the Earthquake Engineering Research Institute, September 1982. Fig. 5-1 taken from this reference provides insight into the capacity of typical structures (designed for earthquakes) to resist strong earthquake motions. The authors state, that the most significant result in the figure, is that structures of this type can, or on the average, be expected to resist base shears that are two to three times larger than the code (static) design values without major structural damage. They also point out that the margin of safety against collapse of these structures was not tested by the San Fernando earthquake, but the data and the condition of the buildings suggest that responses equivalent to five or more times the design (static) base shear could have been resisted without collapse, though severe damage would have resulted.

In the above example (in italics), the static analysis/design procedure utilizes an input static base shear force of 0.2 times the weight of the building to be fully resisted by the members of the lateral-force-resisting system using code allowable working stresses.

An analysis using structural dynamics in conjunction with 0.8g ground acceleration assumes that all forms of structural work (not just stress) act to resist earthquake forces. This total resistance is calculated in terms of kinetic energy inducing vibration of the building mass, strain energy causing elastic and inelastic deflections of materials, and damping energy from friction between moving parts and internal molecular friction within the construction materials. In effect, earthquake-resistant design using dynamic analysis balances the ultimate resistance of the building against the *real* earthquake input forces *estimated* for the site.

Many assumptions or decisions made by engineers responsible for the analysis and design may be more or less conservative with respect to the *real* earthquake and the actual resistance of the building. The competence of engineers responsible for the analysis and design is much more important than the specification of input criteria.

Although site-specific earthquake ground motions are not very predictable in an engineering sense (see Foreword to Chapter 6), there seems to be a compulsion to be extremely rigorous when estimating the maximum earthquake or the maximum ground acceleration that a site might experience. Possibly this rigor is a *spin-off* from the nuclear power industry, where the rigorous determination of a site-specific earthquake for each reactor site is a regulatory requirement. For whatever reason—political, academic, or psychological—too much time and money often go into estimating and predicting the size of the earthquake and the specific ground motion to be expected, even though it may be unrewarding in a practical sense. There are examples of sites where millions of dollars have been expended over many years to presumably refine potential ground motion approximations while relatively simple (but hazardous) structural problems await attention (or the inevitable damaging earthquake).

If one looks at actual experience with damaging earthquakes, it is indeed rare to find that the predicted size of the earthquake was the major deficiency revealed by the damage. The reality is that most problems are found to be the result of structural deficiencies, such as a missing or brittle link in the lateral-force-resisting system, or simply the lack of a formal, predictable lateral-force-resisting system. *The point is that excessive time and money should not be spent refining the estimated size of a future earthquake and its ground motion.* These refinements provide very dubious cost benefits and worse, are apt to be misleading. For facility managers this is an important pitfall

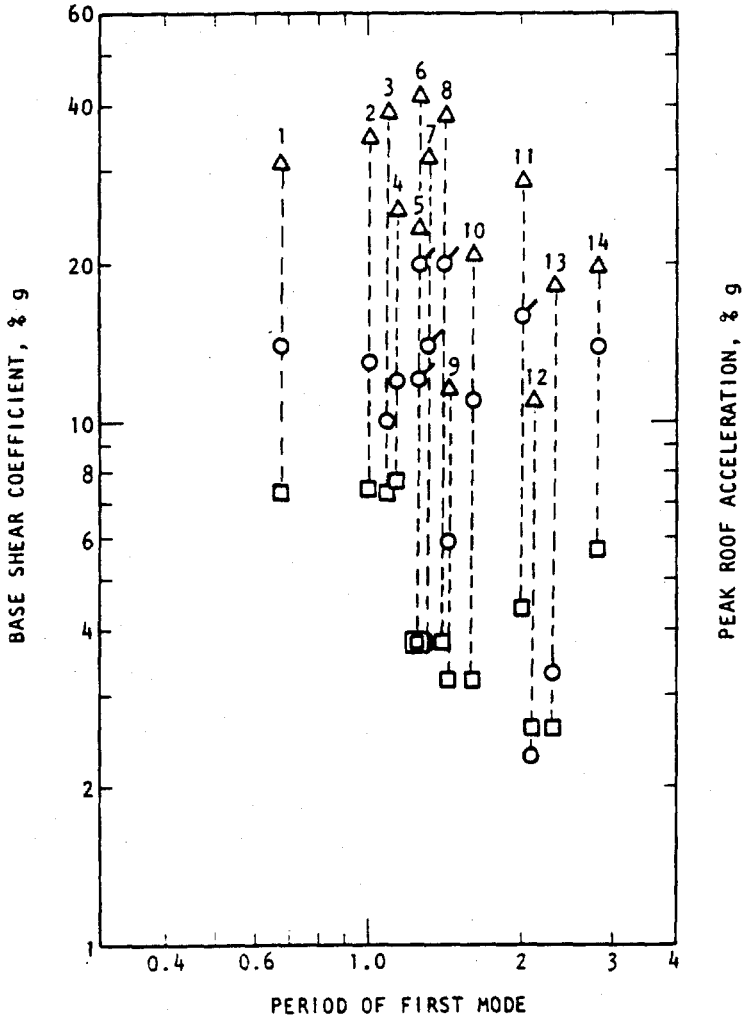


Fig. 5-1. Plot of responses of 14 multistory concrete frame buildings during the San Fernando earthquake (Feb. 9, 1971). The triangles represent the peak acceleration recorded on the roof; the circles represent the peak base shear of the first mode as calculated from the recorded response of the building; the squares represent the design base shear as specified by the building code in Los Angeles. The ticketed circles represent buildings that sustained some structural damage. The damage was not severe and none of the buildings were close to collapse. These were all relatively modern structures designed under the Los Angeles building code.

to be avoided. The money can be more wisely spent on fixing buildings before the earthquake strikes.

This is not to say that thorough geotechnical investigations are not important, nor that predictions of ground motion are not needed. It is to say that the process of developing seismic input data, as well as geotechnical design data, should be practical, economical, and timely and should not unduly delay the objective of safety for building occupants or others threatened by potential seismic hazards. The extreme example to be avoided is the situation in which unreinforced brick buildings (with occupants or hazardous materials) await rehabilitation (or seismic destruction) while pursuing lengthy and sophisticated ground-motion studies.

When site-specific seismic criteria are required, the work should be carried out by experienced geotechnical specialists working in close coordination with structural engineers who will use the results. Generally, seismologists and geologists have limited understanding and little control over how the site-specific criteria will be utilized for structural design. It is impossible for them to take this into account if they must set criteria in a structural vacuum. Worse, when this happens, structural engineers may be saddled with unrealistic criteria that will, in turn, make the results of the analysis unrealistic or the design solution impractical.

Public debate over seismic criteria is commonplace with respect to facilities that house hazardous materials. Because the estimate of earthquake size is at best still subject to professional conjecture, it is an ideal battleground for political debate. It is a poor place to make a firm stand for less stringent criteria. The cost to provide extra strength and ductility in a new structure to resist a major earthquake versus that required for minimal earthquake resistance is usually small: about 2 or 3% of the cost of most buildings. It is not worth arguing about, if this cost is balanced against the high costs that are usually required to justify data to support site-specific criteria for an event smaller than a major earthquake. As a practical matter, the added cost resulting from several months' delay is apt to be more than the cost of providing extra strength for larger earthquake.

Politically, the costs of prolonged public debate are significant and damaging. It does not make economic or technical sense to undertake extensive studies that have the object or possibility of establishing less stringent site criteria in an area where potentially damaging earthquakes have been part of recent geological time and can be anticipated in the future.

Chapter

5a

Site Evaluation

Maurice S. Power

Introduction

In this chapter, a brief discussion of earthquakes and earthquake hazards is presented as background information. This discussion is followed by guidance for evaluating earthquake ground shaking and the other earthquake hazards that could affect site facilities.

Background Information on Earthquakes and Earthquake Hazards

Earthquake Generation

Earthquakes are caused by a sudden slip along a zone of weakness in the earth's crust, termed a fault. For example, some of the major faults in California are shown in Fig. 5a-1.

Fig. 5a-1. Major active faults in California (after Wesnousky, 1986).

Earthquakes are also caused by volcanic action, but these are relatively rare in comparison with earthquakes caused by fault slip.

There are several types of earthquake faults as illustrated in Fig. 5a-2. They are distinguished by the primary sense of relative displacement between the two sides of the fault. Strike-slip faults are characterized by horizontal movement; reverse or thrust faults involve relative upward movement of the crustal block above the fault plane; normal faults involve relative downward movement of the block above the fault plane; and oblique faults are characterized by both strike-slip and reverse or normal types of movement.

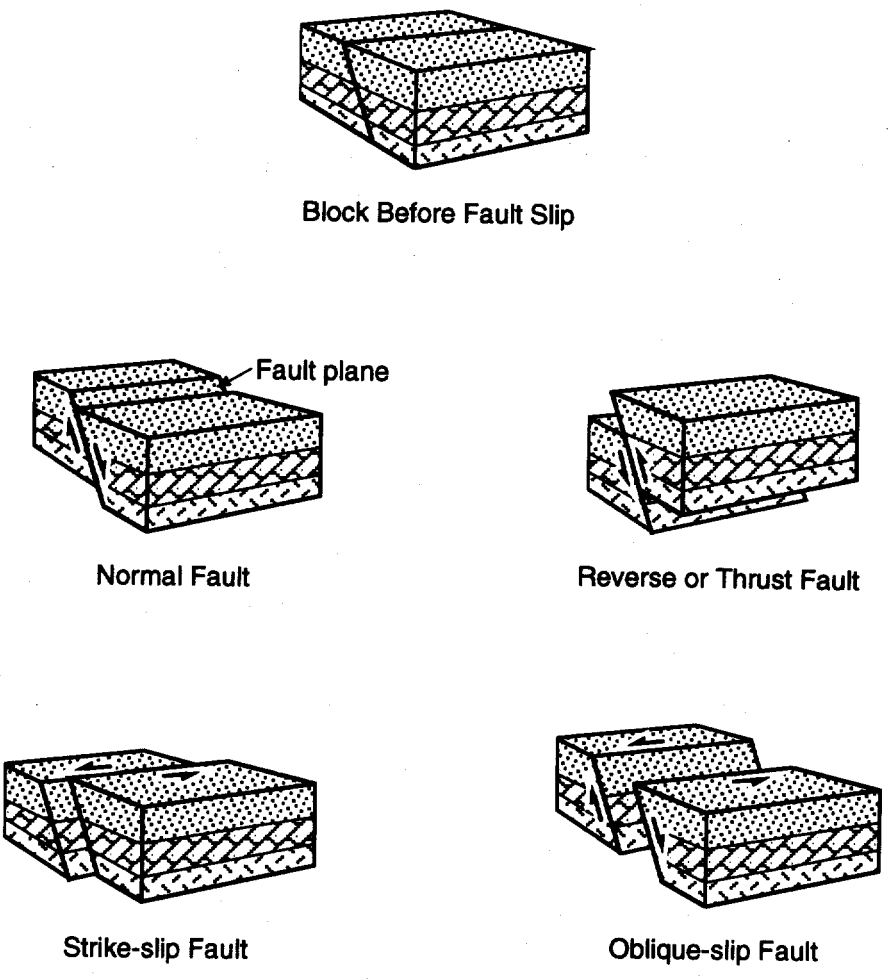
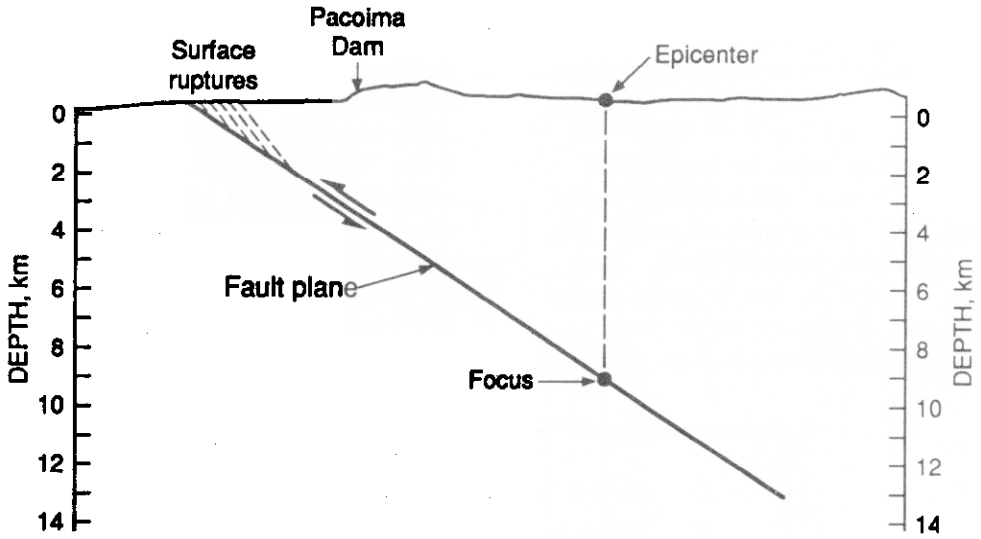
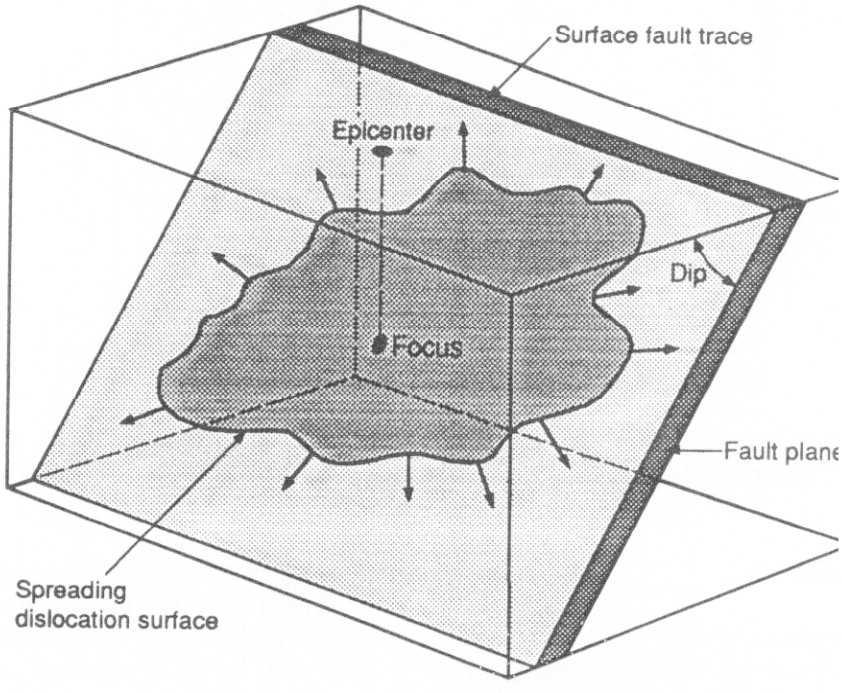


Fig. 5a-2. Types of faults.

The subsurface point on the fault at which fault slip (or *rupture*) initiates is termed the *focus*, and the vertical projection of this point to the ground surface is termed the *epicenter* (Fig. 5a-3a). The rupture spreads away from the focus along the *fault plane* (Fig. 5a-3b).



(a) Earthquake focus and epicenter of 1971 San Fernando, California, earthquake.



(b) Earthquake rupture propagation (after Bolt, 1978).

Figs. 5a-3. Earthquake initiation and rupture propagation.

Magnitude is a quantitative measure of the size of an earthquake. It is determined by measuring the largest ground motion recorded on a seismograph during the arrival of a seismic wave type and applying a standard correction for distance to the epicenter. The magnitude scale is logarithmic; an increase of one unit on the magnitude scale represents ten times the measured wave amplitude. Three common types of magnitude are Richter (or local) (M_L), body wave (m_b), and surface wave (M_S) magnitude. Another type of magnitude is moment magnitude (M_W), which is determined from the seismic moment of an earthquake. The seismic moment is proportional to the energy released by an earthquake and is equal to the product of the area of rupture on the fault, the average slip displacement on the fault, and the shear modulus of the rocks in the region of fault rupture. M_W is being increasingly used as a measure of earthquake size, partly because of its relationship to fundamental faulting parameters and earthquake energy.

A typical correlation of earthquake moment magnitude with rupture area is shown in Fig. 5a-4. As shown, a one-unit increase in magnitude is associated with an approximately eightfold increase in rupture area. The slip displacement on a fault and the amount of energy released by an earthquake also increase rapidly with increase in earthquake magnitude. Approximately thirty times as much energy is released for each one-unit increase in magnitude.

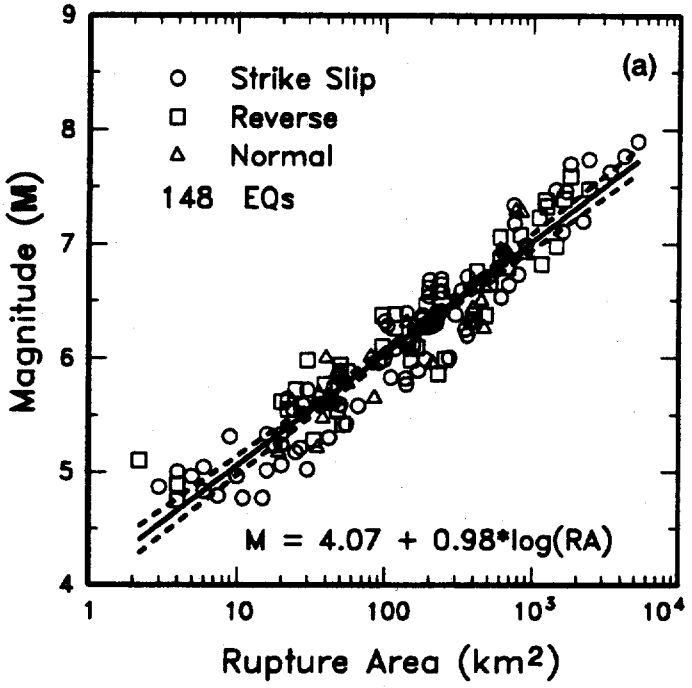


Fig. 5a-4. Relationship between earthquake magnitude and rupture area (after Wells and Coppersmith, 1994).

Ground Shaking Hazard

The fault rupture causes energy to be released in the form of seismic waves that propagate away from the fault through the geologic media. Ground shaking occurs when these seismic waves arrive at a site. As schematically illustrated in Fig. 5a-5, ground shaking is affected by the magnitude and other characteristics of the earthquake source, the travel path between the source and the site, and the local subsurface and topographic conditions at the site.

Earthquake ground shaking at a specific site is described in terms of time histories of

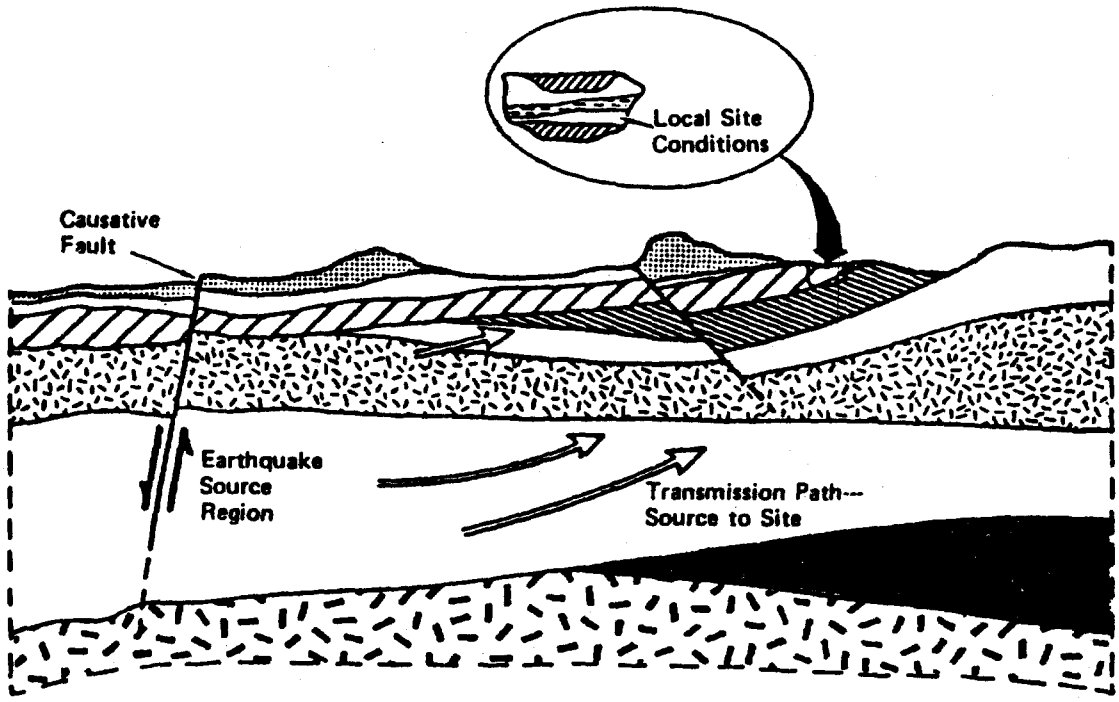
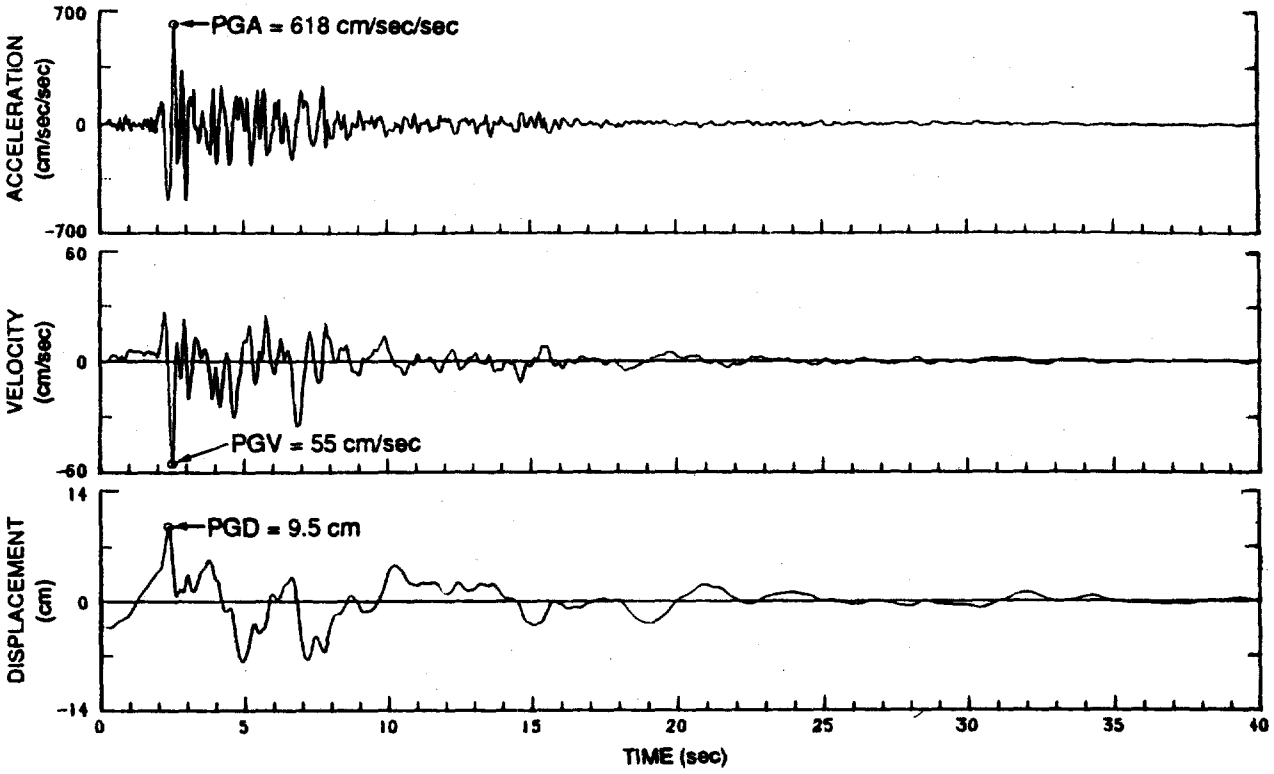


Fig. 5a-5. Seismic wave transmission (modified from Hudson, 1972).

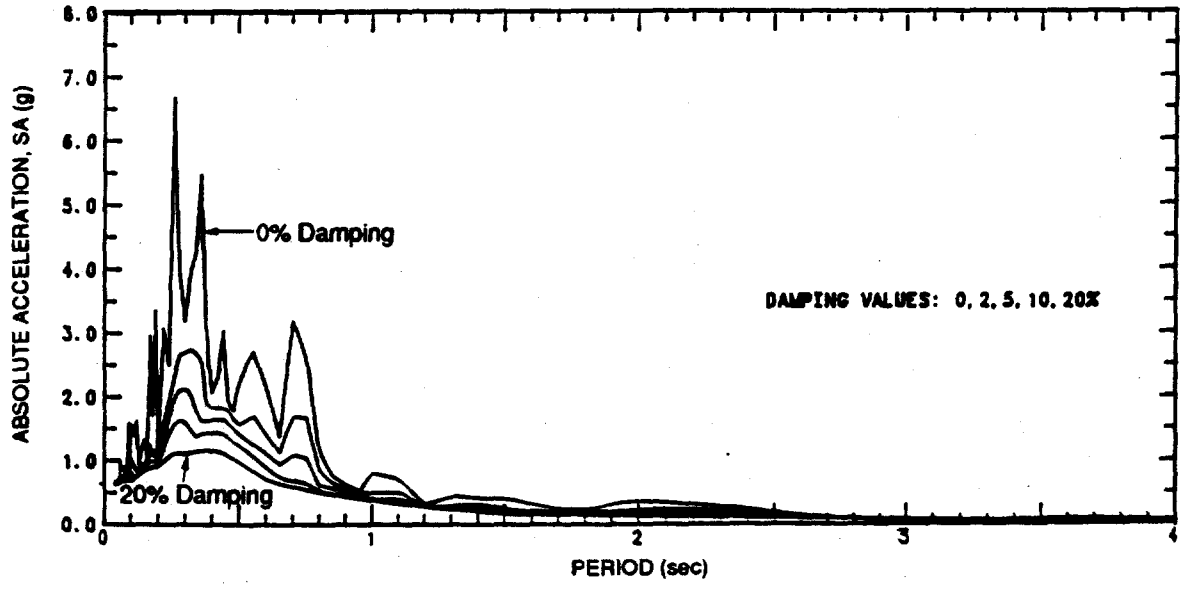
acceleration, velocity, and displacement. Strong-motion accelerographs are deployed at numerous sites in the United States by the U.S. Geological Survey (USGS), California Division of Mines and Geology (CDMG), and other organizations, and these accelerographs have recorded numerous acceleration time histories (accelerograms) during earthquakes. An example of a ground motion acceleration time history and the corresponding velocity and displacement time histories obtained from integration of accelerograms is shown in Fig. 5a-6a. Various measures of the intensity of the ground shaking can be obtained from accelerograms, either directly or by further analysis of the recording, such as:

- Peak ground acceleration (PGA)
- Peak ground velocity (PGV)
- Peak ground displacement (PGD) (Fig. 5a-6a)
- Response spectrum, which is a measure of the maximum amplitude of shaking for different structural periods of vibration and amounts of structural damping (Fig. 5a-6b) (the response spectrum is discussed in Chapter 6b)
- Duration of strong shaking.

It is important to note that, with increasing earthquake magnitude, not only does the amplitude of the ground shaking increase, but so does the duration of shaking. For example, the duration of strong shaking for a magnitude 7 earthquake is typically on the order of 12 to 25 seconds, compared to 4 to 10 seconds for a magnitude 6 earthquake. Furthermore, the size of the region subjected to strong ground shaking increases rapidly with earthquake magnitude as a result of the much larger dimensions of fault rupture and greater amounts of energy released for larger magnitude earthquakes.



(a) Acceleration, velocity, and displacement time histories.



(b) Acceleration response spectra.

Fig. 5a-6: Corralitos ground motion recording, component 0° October 17, 1989, Loma Prieta, California, earthquake (after California Division of Mines and Geology, 1989).

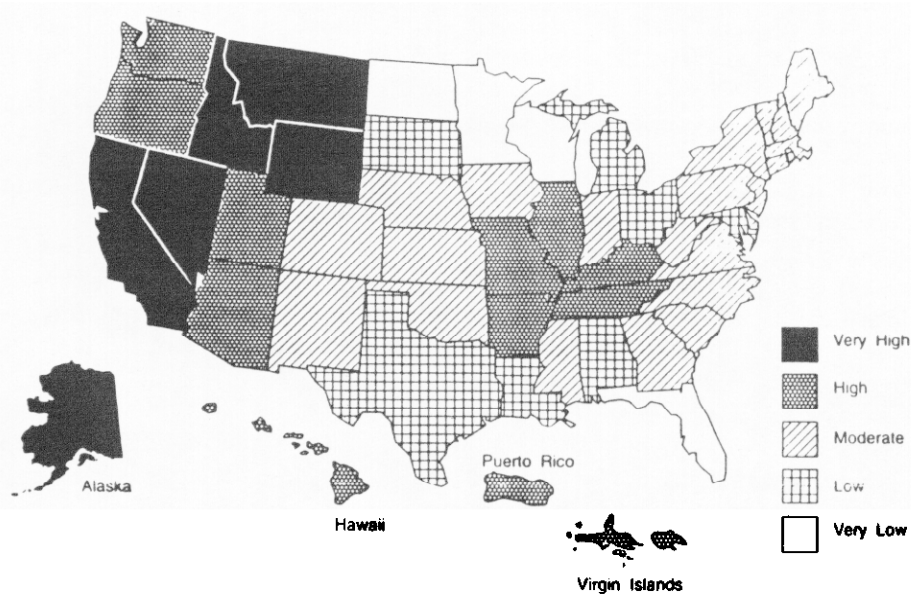


Fig. 5a-7. Seismic hazard of United States (based on NEHRP, 1994).

Earthquake ground shaking (and the resulting inertial forces induced in structures by ground shaking) is the major cause of damage to structures from earthquakes. Because earthquakes occur more frequently in some regions of the United States than others (most frequently in California and Alaska), the likelihood of strong ground shaking is higher in some regions. A moderate to high earthquake risk exists in most of the United States (Fig. 5a-7).

Modified Mercalli Intensity (MMI) is a qualitative measure of the effects of an earthquake on structures of various types, the ground, and people. There are other similar intensity scales, but the MMI scale is the most widely used scale in the United States. The MMI scale is reproduced in Table 5a-1. The assignment of intensity values is subjective and is influenced by the quality of construction, the ground surface condition, and the perception of the observer. MMI has been correlated to peak ground acceleration, and MMI in the epicentral area (epicentral intensity) has been correlated to earthquake magnitude. Because of the qualitative and subjective nature of the intensity scale as well as the inherent variability in ground shaking, these correlations involve considerable uncertainty.

In the eastern part of the United States, the lower likelihood of earthquakes (compared to the western United States) may be offset by a tendency for eastern United States earthquakes to affect a larger area than earthquakes of similar size in the western United States. This is illustrated in Fig. 5a-8 by a comparison of areas of equal Modified Mercalli Intensity for two eastern United States earthquakes and two western United States earthquakes. The area affected by the 1811 New Madrid, Missouri,

Table 5a-1.

MODIFIED MERCALLI INTENSITY SCALE

I. Not felt except by a very few under exceptionally favorable circumstances.

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

III. Felt indoors; hanging objects swing; vibration similar to passing of light trucks; duration may be estimated; may not be recognized as an earthquake.

IV. Hanging objects swing; vibration similar to the passing of heavy trucks or sensation of a jolt similar to a heavy ball striking the walls; standing motor cars rock; windows, dishes, and doors rattle; glasses clink and crockery clashes; in the upper range of IV, wooden walls and frames creak.

V. Felt outdoors; direction may be estimated; sleepers wakened, liquids disturbed, some spilled; small unstable objects displaced or upset; doors swing, close, or open; shutters and pictures move; pendulum clocks stop, start, or change rate.

VI. Felt by all; many frightened and run outdoors; walking unsteady; windows, dishes, and glassware broken; knick-knacks, books, etc., fall from shelves and pictures from walls; furniture moved or overturned; weak plaster and masonry D* cracked; small bells ring (church or school); trees and bushes shaken (visibly, or heard to rustle).

VII. Difficult to stand; noticed by drivers of motor cars; hanging objects quiver; furniture broken; damage to masonry D, including cracks; weak chimneys broken at roof line; fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments); some cracks in masonry C*; waves on ponds; water turbid with mud; small slides and caving in along sand or gravel banks; large bells ring; concrete irrigation ditches damaged.

VIII. Steering of motor cars affected; damage to masonry C or partial collapse; some damage to masonry B*; none to masonry A*; fall of stucco and some masonry walls; twisting and fall of chimneys, factory stacks, monuments, towers, and elevated tanks; frame houses moved on foundations if not bolted down; loose panel walls thrown out; decayed piling broken off; branches broken from trees; changes in flow or temperature of springs and wells; cracks in wet ground and on steep slopes.

IX. General panic; masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged; general damage to foundations; frame structures if not bolted shifted off foundations; frames cracked; serious damage to reservoirs; underground pipes broken; conspicuous cracks in ground; in alluviated areas sand and mud ejected, earthquake fountains and sand craters appear.

Table 5a-1.
(continued)

MODIFIED MERCALLI INTENSITY SCALE

X. Most masonry and frame structures destroyed with their foundations; some well-built wooden structures and bridges destroyed; serious damage to dams, dikes, and embankments; large landslides; water thrown on banks of canals, rivers, lakes, etc.; sand and mud shifted horizontally on beaches and flat land; rails bent slightly.

XI. Rails bent greatly; underground pipelines completely out of service.

XII. Damage nearly total; large rock masses displaced; lines of sight and level distorted; objects thrown into the air.

* Masonry A, B, C, and D as used in MM scale above.

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

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

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

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

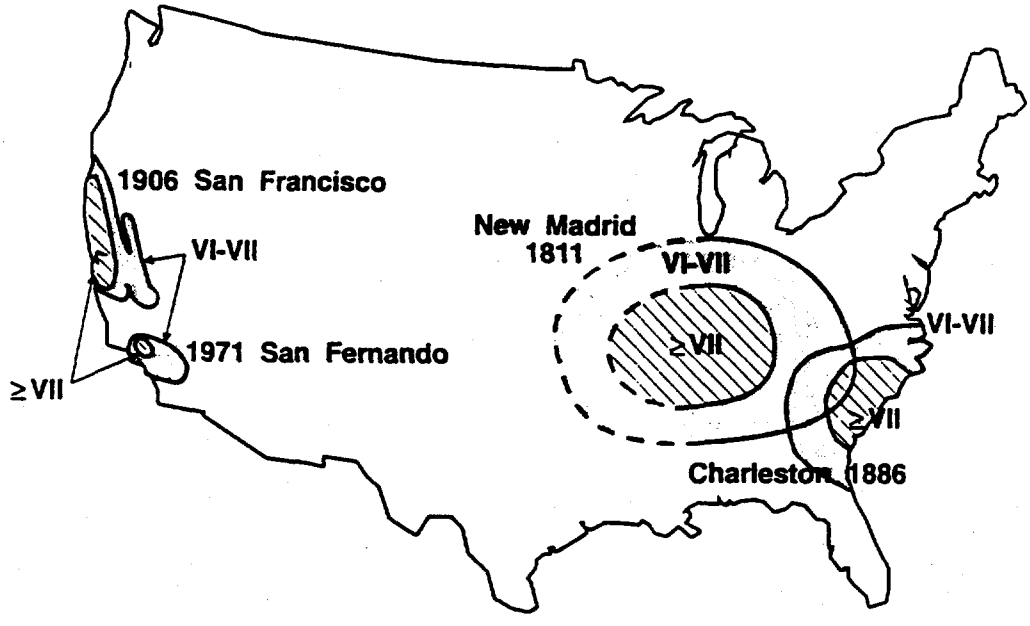


Fig. 5a-8. Areas of equivalent earthquake damage for eastern and western U.S. earthquakes (after Nuttli, 1979).

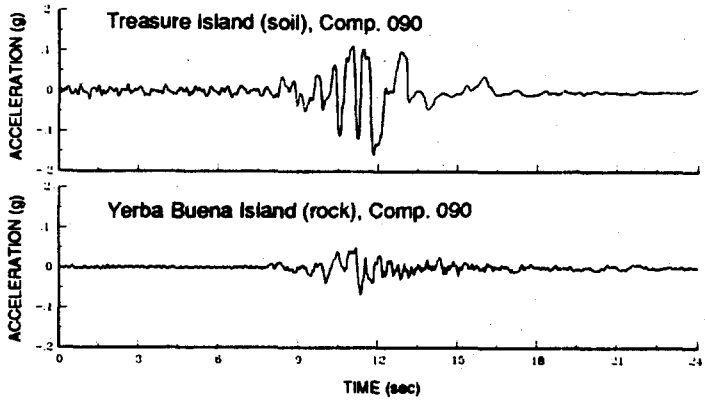
earthquake greatly exceeded the area affected by the 1906 San Francisco, California, earthquake, both earthquakes being of approximately magnitude 8. It is also of interest to note that two earthquakes of a size comparable to the 1811 earthquake occurred in the New Madrid area in 1812. Also shown in Fig. 5a-8 are areas of equal Modified Mercali Intensity for the 1886 Charleston, South Carolina, and the 1971 San Fernando, California, earthquakes. The Charleston earthquake was of magnitude approximately 7 to 7 1/2 and the San Fernando earthquake of magnitude approximately 6 1/2.

The local soil conditions at a site can have an important effect on the strength and character of earthquake ground shaking. An example of this effect is illustrated in Fig. 5a-9 by the ground motion recordings (in San Francisco Bay) obtained at a rock site (Yerba Buena Island) and an adjacent deep, soft soil site (Treasure Island) during the 1989 Loma Prieta, California earthquake. As shown, the effect of the soft sediments was to greatly amplify the ground shaking.

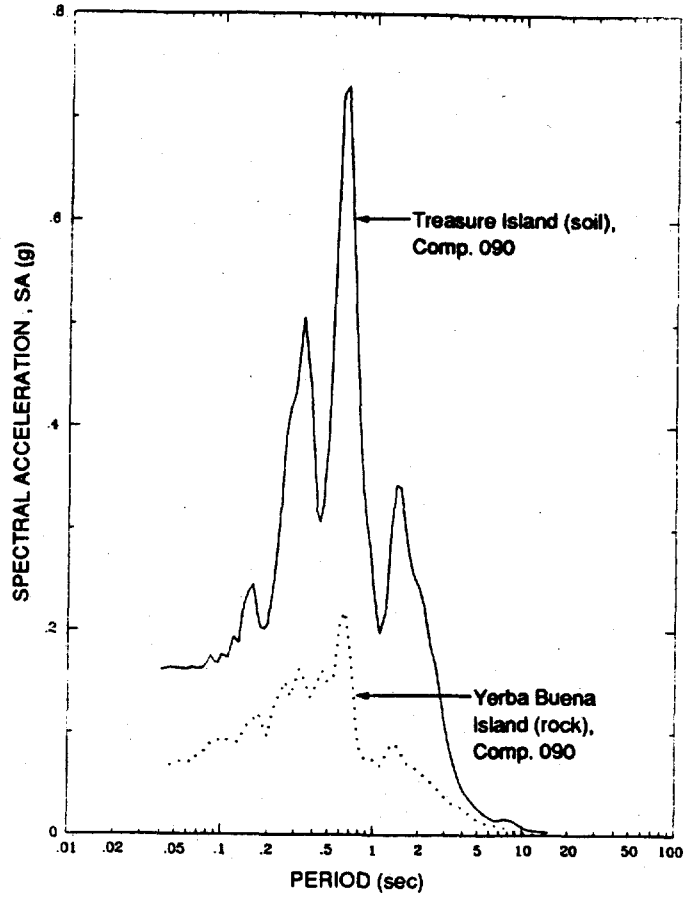
Other Earthquake Hazards

Earthquake hazards in addition to ground shaking that can potentially affect a facility include *surface fault rupture, soil liquefaction, soil differential compaction, landsliding, and flooding*. These are briefly described below.

Surface fault rupture is the displacement occurring along the ground surface trace of the fault (see Figs. 5a-2 and 5a-3). Generally, fault rupture extends to the ground surface only during moderate-to-large-magnitude earthquakes (magnitudes approximately equal to or greater than 6). The resulting displacements may range from a fraction of an inch to several feet or more, depending on the earthquake magnitude and other factors. Surface fault rupture can be catastrophic to structures situated directly astride the rupture zone.



a) Acceleration time histories.



b) Acceleration response spectra.

Fig. 5a-9. Site amplification of ground motion at Treasure Island, compared with Yurba Buena Island, October 17, 1989, Loma Prieta, California earthquake.

Fig. 5a-10 illustrates surface fault rupture that occurred in the 1992 Landers, California, earthquake. During this magnitude 7.3 earthquake, the displacement was mainly of the strike-slip type (see Fig. 5a-2) and the maximum observed horizontal displacement along the fault was 18 feet. Fig. 5a-11 illustrates damage to a structure astride the surface fault rupture of the 1971 San Fernando, California, earthquake, which was of the reverse- or thrust-faulting type (see Figs. 5a-2 and 5a-3a). More than 6 feet of combined vertical and horizontal displacement occurred along the surface trace of the fault during the earthquake. The 1994 Northridge earthquake was of the same type of faulting as the 1971 San Fernando earthquake. However, it occurred on a buried or *blind-thrust* fault at depth and the primary fault displacement did not extend to the ground surface.

In general, the potential for surface fault rupture is much higher in the western United States (WUS) than in the eastern United States (EUS) (east of the Rocky Mountains). In the WUS, active faults that extend to the ground surface are fairly common. In the EUS, active faults typically do not extend to the ground surface. Although surface faulting has been documented in the EUS, (e.g., the Meers fault in southwestern Oklahoma), historical or geologically recent surface faulting earthquakes are rare in the EUS.

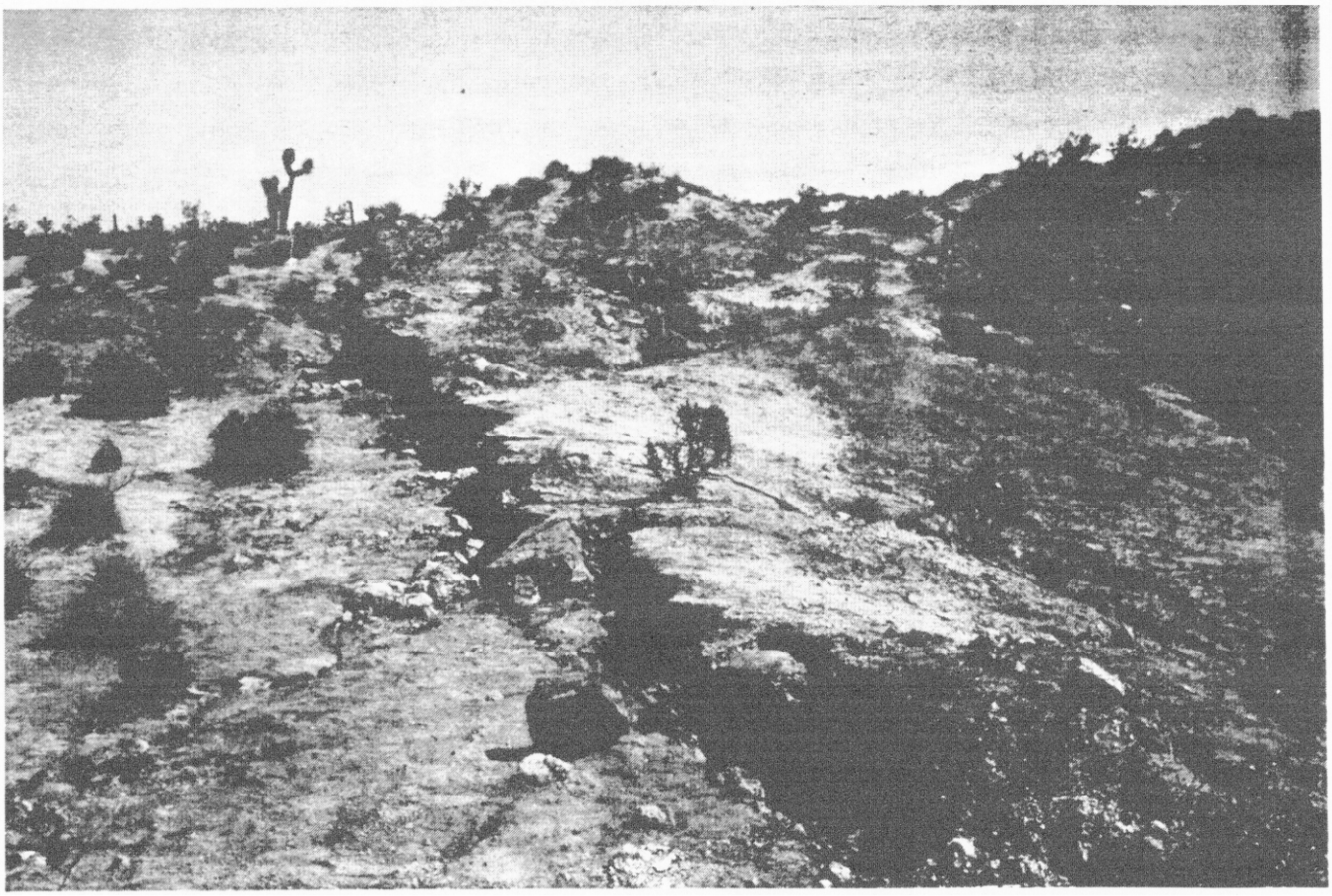


Fig. 5a-10. Surface faulting accompanying Landers, California, earthquake of June 28, 1992.

Soil Liquefaction is a phenomenon in which soil below the groundwater table loses a substantial amount of strength as a result of strong earthquake ground shaking. Recently deposited (i.e., geologically *young*) and relatively loose natural soils and uncompacted or poorly compacted fills are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible. Loose silts and gravels also have potential for liquefaction. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clay soils are generally not susceptible, except for highly sensitive clays found in some geographic locales.

Examples of the effects of liquefaction are illustrated in Figs. 5a-12 and 5a-13. In Fig. 5a-12, liquefaction caused loss of bearing capacity of the foundation soils beneath apartment buildings during the 1964 Niigata, Japan, earthquake, causing the buildings to settle into the ground and tilt. In Fig. 5a-13, liquefaction resulted in lateral spreading beneath a facility during the 1989 Loma Prieta, California, earthquake; the movements pulled the structure apart. Liquefaction-induced landsliding and lateral spreading can occur on extremely flat slopes, less than 1% slope in some cases. The potential for lateral movements is increased if there is a *free face*, such as a river channel or the sloping shoreline of a lake or bay, toward which movement can occur. Other potential effects of liquefaction include flotation of lightweight structures (such as tanks) embedded in liquefied soil and increased lateral pressures on walls retaining liquefied soil.

Fig. 5a-11. House damaged by ground displacement caused by surface faulting accompanying San Fernando, California, earthquake of February 9, 1971.

Soil Differential Compaction (or densification of soils) may accompany strong earthquake ground shaking. Localized differential settlement or subsidence can cause serious problems for buildings, lifelines and other structures.

Landsliding can occur as a result of the loss of soil strength accompanying liquefaction, as mentioned above. However, landsliding can also occur in soils and rocks on hillside slopes and in poorly enginneered embankment fills in the absence of liquefaction, caused by the inertia forces induced by the ground shaking. The consequences of landsliding include differential lateral and vertical movements of a structure located within the landslide zone, or landslide debris impacting a structure located below a landslide. An example of a structure within a zone of earthquake-induced landsliding is shown in Fig. 5a-14. Fig. 5a-15 illustrates the hazard of landslide material (rockfall debris in this case) impinging on a structure below a slope. Even a single large boulder dislodged from an uphill slope can cause considerable damage to a structure below.

Earthquake-induced flooding at a site can be caused by a variety of phenomena including seiches, tsunamis, and dam, levee, and water

Fig. 5a-12. **Bearing capacity failure caused by liquefaction, Niigata, Japan, earthquake of June 16, 1964.**

Fig. 5a-13. **Lateral spreading failure caused by liquefaction, University of California Marine Laboratory Building at Moss Landing, Loma Prieta, California, earthquake of October 17, 1989.**

Fig. 5a-14. House and street damaged by several inches of landslide displacement caused by San Fernando, California, earthquake of February 9, 1971.

Fig. 5a-15. Damage to store front caused by rock fall during San Fernando, California, earthquake of February 9, 1971.

storage tank failures. Seiches are waves induced in a body of water such as a bay, lake, or reservoir by interaction of the water body with the arriving seismic waves. Seiches can be caused by earthquake occurrences either in the region of a site or as far away as thousands of miles. Seiche waves may reach several feet in height and can be damaging to facilities located at or very near the shoreline.

Tsunamis are ocean waves generated by vertical seafloor displacements associated with large earthquakes. Tsunami waves at a site may be produced by local or distant events, and wave heights may reach tens of feet at some coastal locations. Within the United States, the tsunami hazard is greatest along the coasts of northwest California, Oregon, and Washington, southern parts of Alaska, and Hawaii. Another type of flooding hazard is that caused by earthquake-induced failure of a dam, levee, or water storage tank.

Guidance for Site Evaluations

The objectives of geotechnical site evaluations are:

- To specify the earthquake ground shaking characteristics as needed to provide the seismic input loading for a facility's design

- To evaluate the potential for other earthquake hazards that could pose a risk to a facility.

These evaluations are most typically carried out at the same time as geotechnical studies conducted to develop foundation design criteria for a new facility, or as part of a seismic evaluation of an existing facility. Guidance for conducting these evaluations is presented below.

Estimating Site Ground Shaking

For seismic design purposes, site ground shaking is usually characterized by *peak ground acceleration* (PGA) and *response spectra.* In many cases, it is also useful to characterize peak ground velocity and displacement and the duration of strong ground shaking and to select or develop acceleration time histories of ground motion.

DOE publications (DOE-STD-1020, *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities;* DOE Order 6430.1A, *General Design Criteria;* DOE Order 5480.28, *Natural Phenomena Hazards Mitigation for DOE Facilities*) provide criteria and guidelines for developing seismic loading parameters for analysis and design of facilities. These criteria are a function of the performance goal and performance category for the facility. Performance categories (PC) are determined in accordance with DOE-STD-1021, *Natural Phenomena Hazards Performance Categorization Criteria for Structures, Systems, and Components.* Performance Categories 1 through 4 have progressively smaller performance goals. Minor facilities not having the need for specific performance goals are in Performance Category 0.

For facilities in PC-1 or -2, site-specific estimates of PGA and response spectra are not required; generic data given by *Uniform Building Code* (UBC) provisions may be used. Earthquake loadings based upon a site-specific geotechnical study are preferred to the general seismic zonation maps of the UBC and may be used alternatively. However, if these loads turn out to be less than those determined by UBC provisions, the difference must be justified, resolved, and approved by DOE.

For facilities in PC-3 and -4, it is required that site-specific estimates of peak ground acceleration (PGA) and response spectra be utilized. Site-specific criteria are provided in DOE-STD-1022, *Natural Phenomena Hazards Site Characterization Criteria.* DOE-STD-1023, *Natural Phenomena Hazards Assessment Criteria,* provides detailed requirements for seismic hazard assessment. For many sites, site-specific ground motion estimates have been developed through previous DOE studies, specifically as contained in UCRL-53582 (Coats and Murray, 1984); these ground-motion estimates are also summarized in Appendix C of DOE-STD-1020. DOE-STD-1024, *Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites,* indicates that, although the UCRL-53582 hazard estimates are outdated relative to the current state-of-the-art, they can continue to be used in the interim until new studies are completed. Alternatively, and also as an interim approach, DOE-STD-1024 permits use of recent Lawrence Livermore National Laboratory (LLNL) and Electric Power Research Institute (EPRI) seismic hazard results at eastern sites for which results are available.

DOE Order 5480.28 requires that the need for updating site-specific ground motion estimates for a site be reviewed at least every ten years. Such a review is appropriate because of changes in the state of knowledge and/or practice since the estimates were developed. For example, there may be new knowledge of seismic sources in a region from more recent geologic and seismologic studies, new relationships developed to characterize ground motion attenuation or site amplification, new approaches developed to conduct ground motion analysis, etc.

The following paragraphs provide guidance for developing site-specific ground motions. Three basic aspects are discussed:

- Approach to estimating site ground motions
- Assessing the influence of site subsurface conditions on ground motions
- Developing acceleration time histories.

Estimating Site Ground Motions

When site-specific ground motion estimates are developed, the basic estimates should be probabilistically based; that is, the levels of site ground motion should be estimated as a function of their probability of exceedance. DOE-STD-1020 provides criteria regarding the appropriate probability of exceedance levels for ground motions for the four different facility performance categories. The higher the

performance category, the lower the probability of exceedance that is specified and the higher are the corresponding ground motions.

Fig. 5a-16 diagrammatically illustrates the process of probabilistic ground motion analysis, commonly termed *probabilistic seismic hazard analysis* or PSHA. There are two basic inputs for a PSHA: seismic source characterization (Fig. 5a-16a); and ground motion attenuation characterization (Fig. 5a-16b).

Seismic sources that have the potential to generate significant ground shaking at a site are characterized in terms of their location, geometry, rate of seismic activity (i.e., frequency of occurrence of earthquakes of different magnitudes), and maximum earthquake magnitude (Fig. 5a-16a). When possible, specific active faults should be identified and characterized based on geologic, geophysical, and seismologic studies. Generally, specific faults can be identified in the western United States and such faults are modeled in a PSHA as lines or planes (Fig. 5a-16a). However, where the sources of earthquakes cannot be associated with a specific fault (because the fault does not have surface expression or other evidence for its exact location), then it becomes necessary to model earthquakes as occurring at random locations within source areas (or volumes) (Fig. 5a-16a) that are considered to have a potential for earthquakes different from that of adjacent areas. The locations and dimensions of seismic source areas are established based on studies of the geologic structures, tectonic processes, and historical seismicity of a region. For PSHAs conducted in the eastern United States, it is usually necessary to model earthquake sources as area sources.

Ground motion attenuation is characterized by attenuation curves, as illustrated in Fig. 5a-16b. Attenuation curves express the relationships between the amplitude of ground motions and earthquake magnitude and distance from the earthquake source. Ground-motion attenuation can vary significantly in different regions of the United States, and it is important to use relationships applicable to the specific region. For example, attenuation relationships have been developed for the western United States, eastern United States, and subduction zones. Subduction zones are regions where segments of the earth's crust known as tectonic plates are underthrusting beneath adjacent plates. In the United States, subduction zones occur along coastal northwest California, Oregon, Washington, and Alaska, where the oceanic Pacific plate is thrusting beneath the continental American plate.

In characterizing seismic sources and ground-motion attenuation for PSHA, it is important to incorporate the various uncertainties involved, (e.g., uncertainty as to maximum earthquake magnitudes, earthquake activity rate, most appropriate attenuation relationship, etc.). The characterization of earthquake sources and ground motion attenuation are then combined in a probabilistic analysis to assess ground-motion levels for different probabilities of exceedance (Fig. 5a-16c).

Typical outputs from a PSHA are illustrated in Figs. 5a-16c and 5a-17. The curves relating ground motion level to probability of exceedance are termed *hazard curves*. From hazard curves, ground motions (PGA and response spectra, Fig. 5a-16d) can be assessed for the probability of exceedance that is specified for the particular facility performance category.

DOE-STD-1023 defines a procedure to be followed in developing design basis earthquake response spectra using the results of a PSHA. The PSHA results are first analyzed to determine the earthquake magnitudes and source-to-site distances that are the dominant contributors to the probabilistic seismic hazard. Response spectral shapes are then evaluated for magnitude and distance pairs that are found to be the dominant contributors in difference period-of-vibration ranges. These response spectral shapes may be selected based on statistical analysis of ground motion records, attenuation relationships for response spectral values, or numerical ground motion modeling. The spectral shapes are then scaled, using the PSHA results, to the level of ground motion for

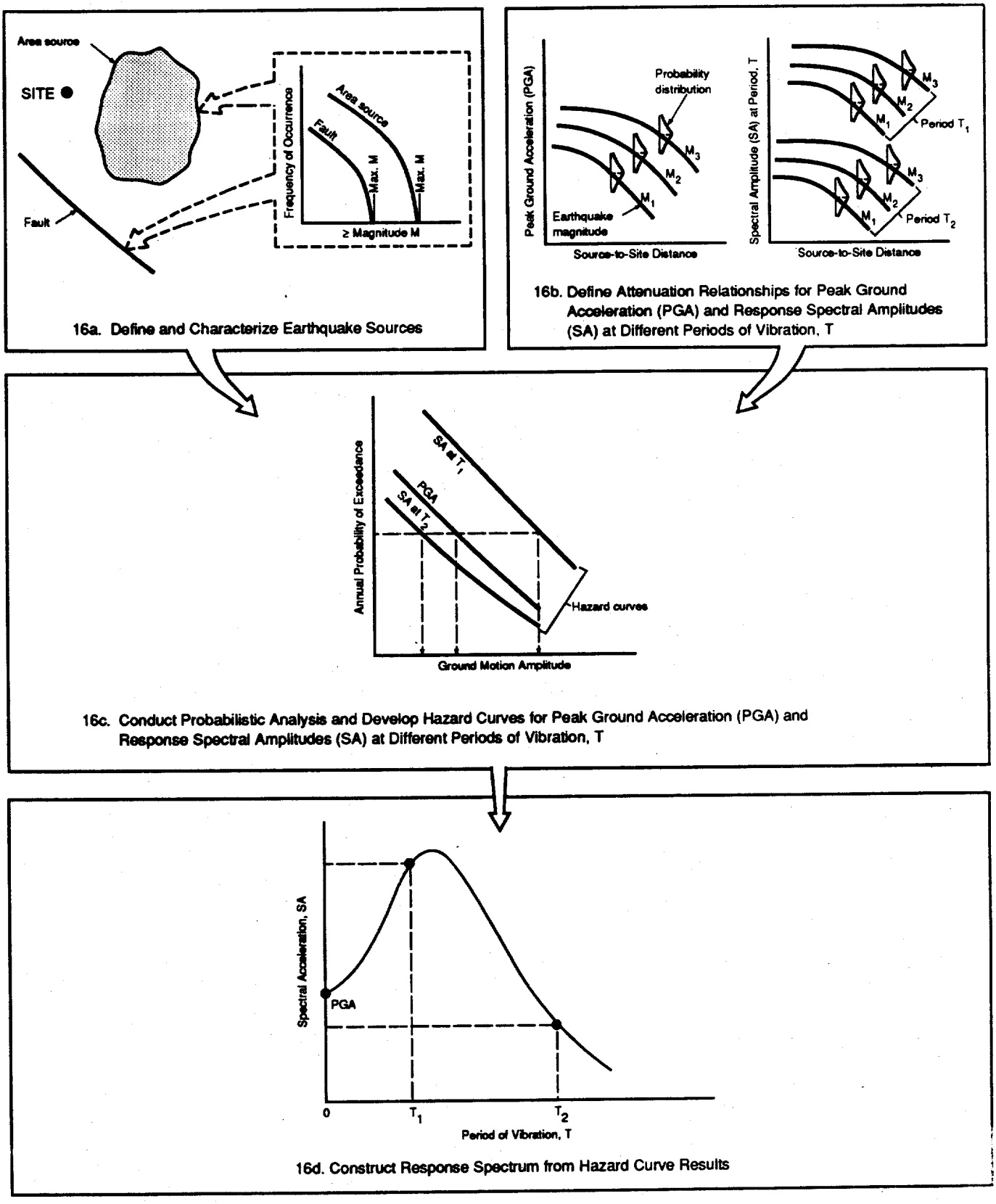


Fig. 5a-16. **Process of probabilistic ground motion analysis and development of hazard curves and response spectra.**

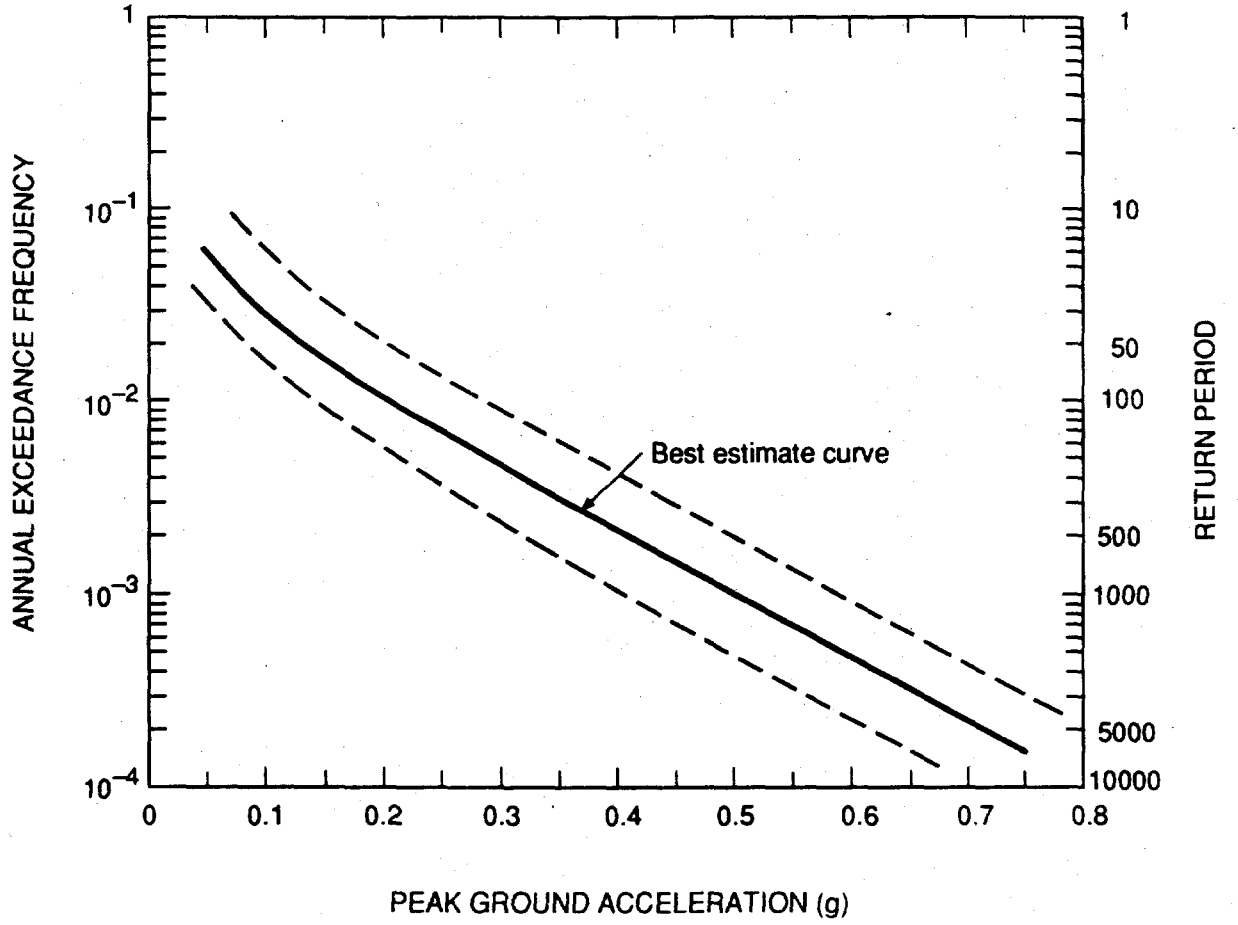


Fig. 5a-17. **Example of seismic hazard curve showing relationship between peak ground acceleration at a site and annual frequency of exceedance.**

the probability of exceedance specified for the performance category of the structure.

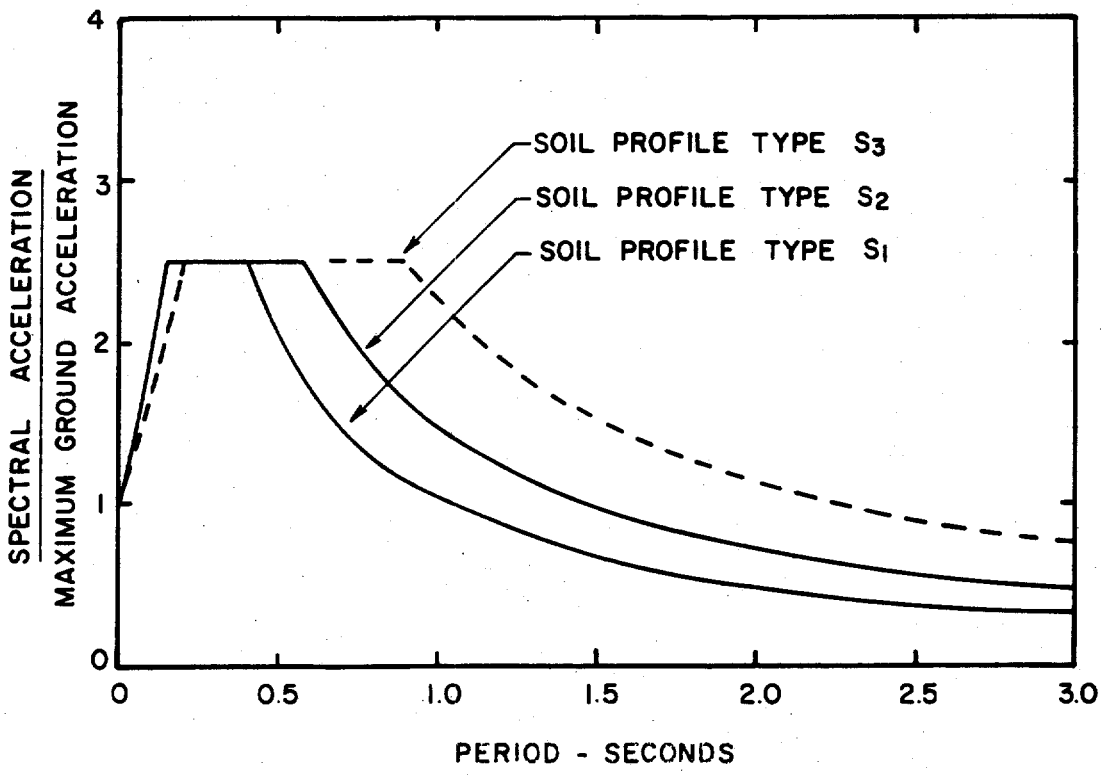
Assessing the Influence of Subsurface Conditions on Site Ground Motions

As discussed earlier, the subsurface soil conditions at a site can significantly influence the ground motion characteristics, particularly the response spectrum of ground motions. Thus, the local site conditions should be taken into account when estimating the site ground motions.

It is often sufficient to characterize the effects of the subsurface conditions on ground motions using generalized relationships. These relationships include attenuation relationships for response spectral values that have been developed for certain broad categories of subsurface conditions (e.g., rock or firm soils) or response spectral shapes such as those illustrated in Fig. 5a-18. The curves and site classifications in Fig. 5a-18 are part of the 1994 *Uniform Building Code* (UBC) provisions. A curve is not presented in the UBC for soil profile Type S_4 because the intent of the UBC is that site-specific methods should be used to develop response spectra for this soil category.

The spectral shapes shown in Fig. 5a-18 were developed based mainly on ground motion data from the western United States. Applying these curves to eastern United States sites should be approached with caution. Also, the curves are applicable to moderate magnitude earthquakes (magnitude ≈6-1/2) and may underestimate long-period spectral response for large-magnitude earthquakes.

Recently a reexamination has been made of soil profile types and their corresponding effects on response spectra for application in building codes (NCEER/SEAOC/BSSC, 1992; Rinne, 1994)(Ref. 9). Based on this work, a new set of soil profile types and site coefficients has been



Soil Profile Type	Description
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) Stiff or dense soil conditions where the soil depth is less than 200 feet.
S_2	A soil profile with dense or stiff soil conditions where the soil depth exceeds 200 feet.
S_3	A soil profile 40 feet or more in depth and containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay.

Fig. 5a-18. **Normalized response spectra and soil profile types given in the 1994 Uniform Building Code.**

adopted into the 1994 edition of the *National Earthquake Hazard Reduction Program* (NEHRP) *Recommended Provisions for Buildings*, (NEHRP, 1994), but has not yet been adopted into the UBC. The new site profile categories are defined in Table 5a-2. Although they are basically differentiated by the shear wave velocity at the site (average shear wave velocity for the upper 100 feet of geologic materials below the ground surface), the soil profiles can also be differentiated using blow count data or shear strength data from soil borings using guidelines given in the *NEHRP Provisions*. The new site coefficients for these soil profiles are given in Table 5a-3. The site coefficients are ratios between the response spectra for a given soil profile to the response spectra for rock type B. These new site coefficients differ from the characterization in the UBC in two ways: (1) there are separate sets of site coefficients for the short-period and the long-period parts of the response spectrum whereas the UBC site coefficients affect only the long-period part (i.e., affect only the declining branches of the spectra in Fig. 5a-18); and (2) the site coefficients are dependent on the level of ground acceleration, increasing for lower acceleration levels, whereas the UBC spectral shapes in Fig. 5a-18 are independent of acceleration level. As a result of this second difference, the *NEHRP Provisions* result in greater ground motion amplification in the eastern United States than in the western United States because of lower mapped acceleration values in the eastern United States.

In some cases, it may be desirable to obtain more detailed information on the site soil profile and properties and carry out a site response analysis, rather than using generalized relationships. If the site is underlain by soft clay soils, it is particularly desirable to carry out site response analyses, especially for longer-period structures. In a site response analysis, the site motions at bedrock are defined and are then propagated through an analytical model of the site soil column to determine the modifying influence of the soils on the ground motions. The site response analysis procedure is illustrated schematically in Fig. 5a-19. The spectra obtained from site response analyses typically contain sharp peaks and valleys; these spectra should be smoothed for use in design.

Another situation where special consideration may need to be given to the subsurface conditions is at eastern United States sites that are underlain by hard bedrock. Such sites may experience ground motions that have high amplitudes of short-period (high frequency) motion relative to typical western United States ground motions. Additional amplification is typically in the period range of less than 0.2 seconds (frequencies greater than 5 Hz). However, if the facility and/or contained equipment are not sensitive to such motions, as determined by the structural engineer, then no special effort is needed to characterize these high-frequency motions.

Developing Acceleration Time Histories

For dynamic analysis of some facilities, acceleration time histories of ground motions may be required in addition to response spectra. Depending on the desire of the structural engineer, such time histories may be either multiple sets of recorded motions that, in aggregate, envelope the adopted design response spectrum or one or more sets of time histories that have been spectrally modified to approximately *match* the design spectrum. In either case, it is important that the time histories have a duration of strong shaking that is representative for the earthquake magnitude(s) and distance(s) under consideration. In the case of nearby earthquakes (distances from site to earthquake source less than about 15 km), it may be important to include a long-period pulse in the time history that is often present in near-source ground motions.

Evaluating Other Earthquake Hazards

Of equal importance to developing site design ground motions is ensuring that there is a low risk of a critical seismic-geologic hazard at the site, or if the risk is significant, taking action to mitigate it. These hazards, described previously, include surface fault rupture, soil liquefaction, soil differential compaction, land sliding, and flooding. DOE criteria (DOE Orders 6430.1A and 5480.28; DOE-STD-1020)

Table 5a-2.

DEFINITION OF SOIL PROFILE TYPES IN 1994 NEHRP PROVISIONS

Soil Profile Type	Description
A	Hard rock with measured shear wave velocity, V_S > 5,000 ft/sec
B	Rock with 2500 ft/sec < $V_S \leq$ 5,000
C	Very dense soil and soft rock with 1,200 ft/sec $\leq V_S \leq$ 2,500 ft/sec
D	Stiff soil with 600 ft/sec $\leq V_S \leq$ 1,200 ft/sec
E	A soil profile with V_S < 600 ft/sec or any profile with more than 10 ft of soft clay defined as soil with plasticity index (PI) > 20, water content, w $\geq$ 40 percent, and undrained shear strength, S_u < 500 psf
F	Soil requiring site-specific evaluations: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H > 10 ft of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays (H > 25 ft with PI > 75) 4. Very thick soft/medium stiff clays (H > 120 ft).

Table 5a-3

SITE COEFFICIENTS IN 1994 NEHRP PROVISIONS

Short-period Site Coefficient, F_a

	Shaking Intensity				
Soil Profile Type	**PGA = 0.1g**	**PGA = 0.2g**	**PGA = 0.3g**	**PGA = 0.4g**	**PGA = 0.5g**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	(--)[1]
F	(--)[1]	(--)[1]	(--)[1]	(--)[1]	(--)[1]

Long-period Site Coefficient, F_v

	Shaking Intensity				
Soil Profile Type	**PGA = 0.1g**	**PGA = 0.2g**	**PGA = 0.3g**	**PGA = 0.4g**	**PGA = 0.5g**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	(--)[1]
F	(--)[1]	(--)[1]	(--)[1]	(--)[1]	(--)[1]

[1] Site-specific geotechnical investigations and dynamic site response analyses should be performed.
Note: PGA refers to estimated peak ground accelerations on rock.

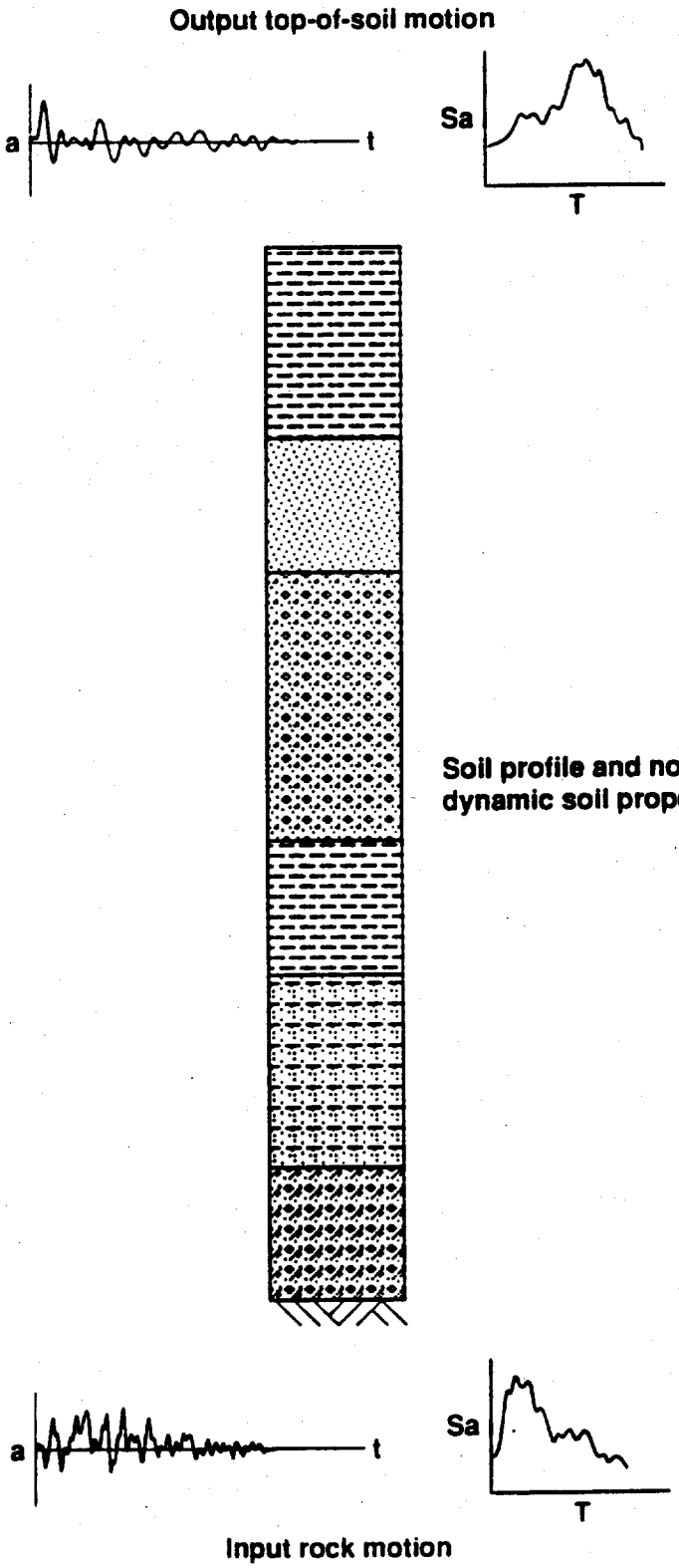


Fig. 5a-19. Schematic of site response analysis.

require that the potential for these hazards be evaluated.

To conduct these evaluations, information about the site conditions must be obtained. For a new facility, the subsurface explorations normally carried out as part of the foundation investigation will also usually provide the basic data needed to evaluate the hazards of soil liquefaction and differential compaction. For an existing facility, sufficient information about the subsurface soils may already be available from prior investigations and general geological information. By working with geotechnical specialists experienced in conducting earthquake hazard assessments, it is possible to determine the most appropriate approach to obtaining site information.

Assessment of seismic-geologic hazards should always include a check as to whether any of the hazards has occurred historically at the site or in the near vicinity. If there is evidence of historical occurrence of a hazard at the site, then a more detailed assessment may be needed.

For some regions and communities of the United States, earthquake-related hazards have been mapped and rated on a regional basis by federal or state agencies, such as the urban-area mapping conducted under the U.S. Geological Survey's *National Earthquake Hazard Reduction Program* (NEHRP). For example, regional liquefaction potential mapping has been conducted in parts of California, Oregon, Washington, Utah, Tennessee, and South Carolina under NEHRP. Geologic hazard maps should be consulted where available.

The following paragraphs provide guidance regarding evaluating specific hazards and measures for hazard mitigation.

Surface Fault Rupture

Generally, a significant hazard from surface fault rupture exists only where a structure is located directly across or within the zone of an active fault that has experienced movement during recent geologic time. Thus the thrust of the evaluation is to ascertain whether active faults or fault-related features (such as fault-related anticlinal folds) traverse the site. In most cases, an assessment by qualified geologists based on a review of existing geologic maps, reports, and aerial photographs, and a limited field reconnaissance is sufficient to demonstrate the absence or possible presence of active faults or fault-related features. If an initial assessment suggests that a potential hazard exists, detailed investigations to determine fault location, activity, and potential amount and sense of displacement may be necessary.

Hazards caused by surface fault rupture can be mitigated by avoiding active fault traces (i.e., establishing appropriate set-back distances from the fault), or demonstrating that the nature and amount of deformation expected on the fault can safely be accommodated in the design of the facility. In some cases, an analysis may indicate that the probability of occurrence of surface fault rupture is acceptably low. The approach used to mitigate a potential fault hazard will depend upon:

- The severity of the hazard
- The function of the facility
- The potential consequences of failure.

Where facilities are particularly sensitive to fault rupture and the consequences of failure are great, avoidance generally has been the strategy to mitigate surface faulting hazards. For an existing facility, change in usage or function may also be considered.

Liquefaction

The first step in assessing liquefaction potential is to examine available geotechnical data about the local geology (particularly the age of the local geologic units) and the subsurface soil and groundwater conditions. In some cases, examination of these data by geotechnical specialists may be sufficient to indicate that conditions conducive to liquefaction are not present. In other cases, use of simplified, empirically based geologic correlations and analytical procedures using blow count data from soil borings is sufficient to evaluate liquefaction potential. Occasionally,

when dealing with soil types for which empirical correlations are less applicable (such as silts and gravels), it may be necessary to conduct special field and/or laboratory investigations.

In general, liquefaction potential and its consequences decrease with increasing age of the deposit, increasing clay content of the soils, and increasing depth to groundwater. Natural soils older than very late *Pleistocene* in geologic age (i.e., older than about 20,000 years) typically have low susceptibility to liquefaction. However, older soils have been found to have liquefied in certain regions, specifically soils as old as a few hundred thousand years in parts of South Carolina. Liquefaction potential is usually low in clay soils (except highly sensitive clays) or if the depth to groundwater exceeds 30 feet.

Assessment of the potential consequences of liquefaction (type and amount of ground movement) is extremely important, because the consequences determine the severity of the hazard and may affect the mitigating measures.

Mitigation of a potential liquefaction hazard can include such options as:

- Designing the structure for the predicted ground movements (if small)
- Selection of appropriate foundation type and depth (including foundation modifications in the case of existing structures) so that the ground movements do not significantly adversely affect the structure (e.g., mat foundation to increase foundation rigidity; deep piles or piers to extend below the zone of liquefaction, etc.)
- Soil stabilization to eliminate the potential for liquefaction or control its effects (e.g., removal and replacement of liquefiable soils; in situ stabilization by grouting, densification, or dewatering; buttressing of lateral spreading zones, etc.).

In general, the consequences of lateral movements associated with liquefaction are more difficult to mitigate than those of vertical movements. This is because structures are typically more vulnerable to lateral displacements than vertical displacements, and the mitigating measures, to be effective, may, in some cases, need to be employed beyond the area of the specific facility.

Differential Compaction

Evaluating the hazard of differential compaction is similar to evaluating liquefaction potential. Available data on the local geology and subsurface conditions should first be reviewed. Simplified procedures can be used to estimate settlements, given knowledge of the soil profile and properties and the design level of ground shaking. For cases of predicted significant differential settlements at a facility, mitigation options are similar to those mentioned above to mitigate liquefaction hazards, that is:

- Designing for predicted ground movement
- Selection of appropriate foundation type and depth
- Soil stabilization.

Landsliding

This section addresses the evaluation of the potential for landsliding of hillside and embankment slopes. Factors affecting slope stability include the intensity and duration of ground shaking, slope angle, slope height, soil and rock type, joints and bedding, groundwater conditions, and past instability. If a site is underlain by competent soils or rock that are not susceptible to liquefaction, geologic conditions are favorable (e.g., rocks do not contain weak zones or bedding planes with an unfavorable orientation), slopes are not overly steep, and the site and adjacent areas have not been involved in previous historic or prehistoric landsliding (whether statically or seismically induced), then the risk of future landsliding is usually low.

Evaluation of the stability of hillside slopes involves review of available information on the geology, soils, and slope

performance; reconnaissance of the site; and examination of aerial photographs in some cases. Simplified deformation analyses for design levels of ground shaking may be useful in some cases. If potential problems should be indicated by these initial assessments, then more detailed studies may be warranted.

Sites requiring special attention for slope stability considerations are those located near shorelines of lakes or bays and underlain by soft soils. Simplified deformation analysis methods are useful in these cases.

If a significant landslide risk to a facility exists, it is generally difficult to design the structure or its foundation to withstand the landslide movements. Mitigating measures typically involve some form of slope stabilization such as soil removal and/or replacement, buttressing, or in situ stabilization. If the slope cannot be stabilized, the alternatives of facility relocation, or change in function or usage for an existing facility, can be considered.

Flooding

The assessment of a potential flooding hazard caused by tsunami, seiche, or failure of a water-retaining structure involves evaluating either or both (1) whether there is any exposure of a facility to the hazard (e.g., whether a facility is located adjacent to a body of water, located downstream of a dam, etc.) and (2) whether there is a significant likelihood of the hazard occurring to a degree sufficient to cause flooding of the facility site (e.g., whether there is significant risk of a tsunami wave high enough to cause flooding, significant risk of a dam failure, etc.). Generally, the assessment of the potential for earthquake-induced flooding involves making a judgment based on a technical understanding of the phenomenon and the physical setting of the facility, experience and data (for example, there are considerable worldwide data on tsunami occurrence). If a significant flooding hazard exists, potential mitigation approaches include siting the facility above the anticipated flood level, preventing flooding by constructing dikes, or accepting and preparing for the flooding effects.

References

1. *Earthquakes*, B.A. Bolt, W.G. Freeman and Company, New York, New York, 1978.

2. *Plots of the Processed Data for the Interim Sets of 14 Records from the Santa Cruz Mountains (Loma Prieta) earthquake of October 17, 1989: CDMG*, California Division of Mines and Geology (CDMG), California Strong Motion Instrumentation Program, Sacramento, California, December 13, 1989.

3. *Natural Phenomena Hazards Modeling Project: Seismic Hazard Models for Department of Energy Sites*, D.W. Coats, R.C. Murray, UCRL-53582 Rev. 1, Lawrence Livermore National Laboratory, Livermore, California, 1984.

4. *Strong Motion Seismology*, D.E. Hudson, Proceedings, vol. 1, pp. 29-60, International Conference on Microzonation for Safer Construction: Research and Application, Seattle, Washington, 1972.

5 *The Site Response Workshop*, Proceedings, National Center for Earthquake Engineering Research, Structural Engineers Association of California, and Building Seismic Safety Council, Los Angeles, California, 1992 (in press).

6. *National Earthquake Hazards Reduction Program (NEHRP), Recommended Provisions for the Seismic Regulations for New Buildings*, 1994 Edition, Federal Emergency Management Agency publication FEMA 222A, May 1995.

7. *Seismicity of the Central United States*, O.W. Nuttli, A.W. Hatheway, C.R. McClure, (eds), *Geology in the Siting of Nuclear Power Plants*, Reviews in Engineering Geology, vol. 4, pp. 67-93, Geological Society of America, Boulder, Colorado, 1979.

8. *Development of New Site Coefficients for Building Codes*, E.E. Rinne, Proceedings,

vol. III, pp. 69-78, Fifth U.S. National Conference on Earthquake Engineering, Chicago, Illinois, 1994.

9. *New Empirical Relationships Among Magnitude, Rupture Length, Future Width, Rupture Area, and Surface Displacement*, D.L. Wells, K.J. Coppersmith, Bulletin of the Seismological Society of America, vol. 84, no. 4, pp. 974-1002 (1994).

10. *An Evaluation of Seismic Hazard in California*, S.G. Wesnousky, Proceedings, vol. 1, pp. 185, Third U.S. National Conference on Earthquake Engineering, August 24-28, 1986, Charleston, South Carolina, 1986.

11. *Uniform Building Code*, International Conference of Building Officials, Whittier, California, 1994.

12. *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities*, DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.

13. *General Design Criteria*, DOE Order 6430.1A, U.S. Department of Energy, Washington, D.C., April 1989.

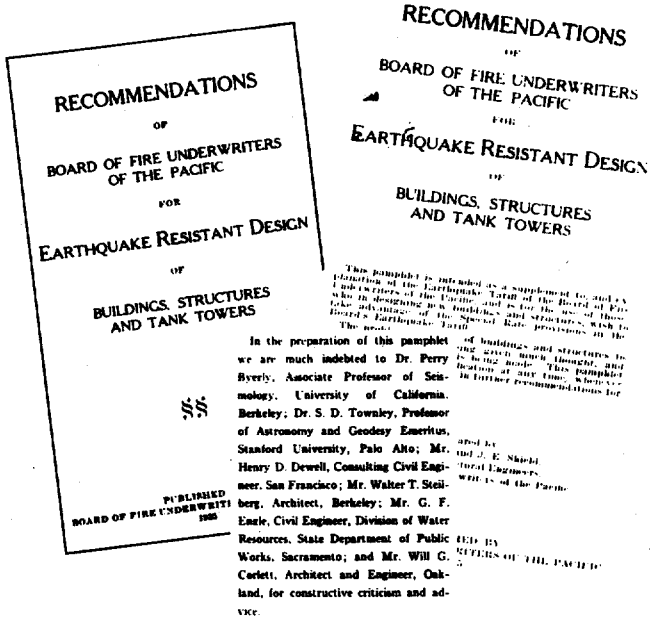
14. *Natural Phenomena Hazards Mitigation*, DOE Order 5480.28, U.S. Department of Energy, Washington, D.C., January 1993.

15. *Natural Phenomena Hazards Performance Categorization Criteria for Structures, Systems, and Components*, DOE-STD-1021-92, U.S. Department of Energy, Washington, D.C., December 1992.

16. *Natural Phenomena Hazards Site Characterization Criteria*, DOE-STD-1022-94, U.S. Department of Energy, Washington, D.C., March 1994.

17. *Natural Phenomena Hazards Assessment Criteria*, DOE-STD-1023-95, U.S. Department of Energy, Washington, D.C., September 1995.

18. *Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites*, DOE-STD-1024-92, U.S. Department of Energy, Washington, D.C., December 1992.


RECOMMENDATIONS
OF
BOARD OF FIRE UNDERWRITERS
OF THE PACIFIC
FOR
EARTHQUAKE RESISTANT DESIGN
OF
BUILDINGS, STRUCTURES
AND TANK TOWERS

PUBLISHED
BOARD OF FIRE UNDERWRITERS
1935

In the preparation of this pamphlet we are much indebted to Dr. Perry Byerly, Associate Professor of Seismology, University of California, Berkeley; Dr. S. D. Townley, Professor of Astronomy and Geodesy Emeritus, Stanford University, Palo Alto; Mr. Henry D. Dewell, Consulting Civil Engineer, San Francisco; Mr. Walter T. Steilberg, Architect, Berkeley; Mr. G. F. Eagle, Civil Engineer, Division of Water Resources, State Department of Public Works, Sacramento; and Mr. Will G. Corlett, Architect and Engineer, Oakland, for constructive criticism and advice.

A booklet entitled "Recommendations of (the) Board of Fire Underwriters of the Pacific, for Earthquake Resistant Design of Buildings, Structures and Tank Towers" was prepared by H. M. Engle and J. E. Shield, Civil and Structural Engineers, in 1935. The Foreword from this publication, which is reproduced here, still provides a valid philosophy for today's earthquake engineers.

FOREWORD

We live on an unstable coast. It is a region of great topographic extremes, where mountain peaks reach elevations of 14,000 ft. and over, within 75 miles of a depression whose floor is 275 ft. below the level of the sea; where the huge range of the Sierra Nevada lies in fairly close proximity to the Pacific Ocean and the depths beyond the Continental Shelf; where the Sacramento and San Joaquin Rivers year by year move vast tonnages of silt through the great central valleys from the mountains to the sea. Pent up behind the rampart of the Sierra Nevada lies the central plateau, with its alternating mountain ranges and valleys extending far to the East. Much of this plateau is 4,000 ft. above sea level, with the mountain ranges reaching elevations of more than 10,000 ft.

Although this region appears solid and substantial, it is often shaken by tiny tremors, and at not infrequent intervals by stronger vibrations, which, when felt by people are called earthquakes. This whole area seems to be in a state of unbalance, with readjustments constantly taking place. What causes these cannot be gone into here. For the present purpose it is enough to know that they do take place, as evidenced by the records of the seismographs, and that earth vibrations are felt strongly in various places from time to time. The readjustments generally occur on definite planes of weakness in the earth's crust, called faults, which in reality are long though not open cracks along which the movement takes place. When this movement is great it is sometimes visible at the surface, showing horizontal and vertical displacements of many feet in some cases. A considerable number of these faults are now well located, and many more approximately so.

The San Andreas Fault is the best known and one of the longest active faults in the world.[1] It comes into the Pacific Coast at Point Arena in Mendocino County, runs to the south through Tomales and Bolinas Bays, crosses outside the Golden Gate, again reaches the Coast at Mussel Rock, traverses the Spring Valley lakes, follows the Coast Range to the south, passes to the East of Los Angeles and San Bernardino and within 8 miles of the latter, curves easterly along the shore of the Salton Sea and possibly extends to the Mexican Border. A great fault runs along the east base of the Sierra Nevada, past Mt. Whitney, and curves westerly toward the Coast. The Hayward Fault follows the base of the Berkeley Hills, passes to the east of San Jose, and appears to join the San Andreas system to the south. The Inglewood Fault, on which two strong movements have occurred in recent years, lies southerly from Los Angeles along the coast line, and passes through Long Beach. These faults are only a few of those known to exist.

A condition must be faced from which there is no escape. Earthquakes have occurred as far back as records have been kept; they will continue through our lifetime and far into the future. They are dangerous solely because we make them so by erecting buildings and houses which can be shattered or shaken down. In themselves earthquakes have little hazard for us. They are dangerous only when we build things so weakly that they can be damaged or wrecked. It is just as easy to build them so they will not be shaken down. We have had ample proof in the past that it can be done and is economically feasible. The decision rests with us. If we choose, an earthquake can be made an interesting but not dangerous occurrence. If we choose otherwise it will always be a thing of terror, something to be dreaded, something which may kill us, and cause misery and suffering. The choice is our own. Unfortunately, up to the present a course leading to terror and misery has largely been followed. The San Francisco disaster of 1906, the shock of 1892 at Dixon, Vacaville and Winters, the Santa Barbara shock of 1925, and now the Long Beach shock of 1933 are ample proofs of this. If we are to be classed as an intelligent people a new course must be chosen. The past cannot be undone in a moment, but the future can slowly be made brighter along the road toward safety. The choice is up to us, and better late than never we had best do something. It is no disgrace to admit past errors in which nearly all have shared. Much has been learned since 1906. It will be an everlasting disgrace if from this time forward we do not actively take steps to safeguard ourselves.

The courage and foresight of the Japanese, following the terrible Tokyo shock of 1923, in completely replanning and rebuilding their city with rigid adherence to earthquake resistive construction affords an object lesson worthy of thoughtful consideration.

The late shock at Long Beach is very definitely the handwriting on the wall, giving notice that a public attitude of indifference must give way to one of realization and constructive endeavor. Nature has been kind to us on this Coast in the times of occurrence of our notable shocks. Instead of trying to push our luck too far, the time has come when we must do something positive for our own safeguard. On our decision rests the lives and happiness of many people in the future. We believe that the following recommendations, when intelligently carried out, will result in structures which will adequately safeguard life in severe shocks, and in which damage will be minimized and held to rather moderate proportions.

[1] Faulting also occurred near Pt. Delgado, Humboldt Co. There is some doubt as to whether this faulting was part of the San Andreas Fault, or a separate parallel fault.

Chapter

6

Seismic Design of New Facilities

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

The process of setting seismic criteria for a given site is an inexact science. Earthquakes, being unpredictable in nature, continue to bring surprises to engineers and seismologists as they attempt to correlate earthquake magnitude with earthquake ground motion and damage.

The 1971 San Fernando, the 1979 Imperial Valley, and the 1994 Northridge earthquakes in California produced ground accelerations that were recorded at higher levels than most researchers believed reasonable for *moderate* earthquakes. The startling effects on freeway structures, dams, and lifelines (e.g., utilities and roads) in the San Fernando earthquake stimulated many changes in seismic codes including the introduction of a soil-structure interaction factor. In contrast the damage during the Imperial Valley earthquake was relatively light, when one considers the recorded peak ground accelerations of 0.8g and higher.

In 1980, three relatively small earthquakes near Livermore, California, took place within two minutes, producing a duration of heavy shaking generally associated only with major earthquakes. Although the effects were very localized, the surprisingly long duration of heavy shaking caused considerably more damage than one would expect from the instrumental magnitude of these events.

The epicenter of the 1971 Loma Prieta earthquake, magnitude 7.1, is located about 60 miles south of San Francisco. Although it was not the great earthquake expected for the San Francisco Bay region, ground motions at soft soil sites were much greater than anticipated, even by the most up-to-date geotechnical estimates and building codes.

The 1994 Northridge, California earthquake (magnitude 6.7) was generated on a blind-thrust fault 18 km beneath the San Fernando Valley; thus the principal rupture did not break the surface. It has been only since the 1987 Whittier Narrows, California earthquake that the occurrence of blind-thrust fault earthquakes through the Los Angeles and San Fernando Valleys has been widely accepted. Because blind-thrust faults usually do not rupture the surface, unless they have generated a historically recorded earthquake, they are generally unmapped. There may be numerous blind-thrust faults of unknown location in the Los Angeles basin capable of earthquakes in the range of 6.0 to 7.0 magnitude. Many other areas of complex geology throughout *earthquake country* potentially harbor these unknown seismic hazards.

This unpredictable nature is an important characteristic of earthquakes. For most structures, spending much time or money to

improve the estimate of maximum ground acceleration or duration of shaking is not cost-effective. On the other hand, a truly in-depth study could be considered to constitute such *complete and accurate* input criteria, that those involved may be lulled into a sense of false security. In reality, the ultimate test is applied to the structure during an earthquake, not to the prediction of ground motion.

In the analysis and design of special or essential facilities such as reactors or major hospitals, it is extremely important that detailed investigations and the best techniques be used to ensure that the worst possible ground shaking is taken into account. The risk of facility failure must be reduced to a minimum in the process of setting seismic criteria for design and construction as well as for dynamic analysis. For ordinary structures, however, it is impractical to spend limited resources in an attempt to predict accurately the degree of ground shaking. It is a better use of time and money to concentrate effort on achieving quality structural design of the building.

For most buildings, the lateral force coefficient to be used for the equivalent static force analysis can be selected from an appropriate seismic risk map and building code. *It is of utmost importance to focus on the design of the lateral-force-resisting system to ensure its continuity, ductility and completeness.* That is, the system should provide a continuous path to transfer all forces from their point of application to the final point of resistance. The system must not have missing links, inadequate joints or anchorages, or brittle elements. The actual magnitude of seismic forces may be unpredictable, but they can be counted upon to seek out weak spots or missing links in a lateral-force-resisting system. Experience has shown that inadequate structural performance in earthquakes can usually be attributed to neglect of this simple concept.

If the level of shaking is so unpredictable, how can we design for earthquakes? A practical approach is to envelope the problem by designing for some appropriate lower level of resistance that can be achieved elastically and economically, and at the same time make provisions for inelastic behavior without sudden or brittle failure at the upper level of forces that could occur. It is important to recognize that intense shaking may stress the materials in the lateral-force system beyond yield and cause stress reversals as well. Therefore, yielding of members must not result in sudden collapse. The structural system should be ductile. *It is important to realize that regardless of how sophisticated the analysis may be, earthquake resistance is ultimately achieved by proper selection, designing, detailing, and construction of the lateral-force-resisting system. It is essential that connections and materials of the lateral-force-resisting system perform in the way assumed in the analysis.*

Among the lessons learned from observation of the effects of prior earthquakes on various facilities is the importance of detailing and the need for engineers to remain responsible for ensuring that the constructed details actually carry out the design philosophy. Experience with California earthquakes has also established the importance of plan-check procedures and the so-called *third-party review*. The requirement that plans and calculations for schools must be submitted for formal review to the California Division of the State Architect has been a prime factor in the excellent seismic performance of modern public school buildings (elementary, high school, and community colleges) in California. (Universities are not required to submit their designs for peer review or to the state architect.) The lack of an adequate third party review was apparent in the poor performance of buildings and parking structures at California State University at Northridge.

In California, city or county jurisdictions require all those who wish to construct facilities to submit plans, specifications, and design calculations for review and approval before issuance of a building permit for construction. In most California jurisdictions Building Officials perform a *plan check* utilizing in-house staff, a third-party consultant or the professional staff of the *International Conference of Building Officials* (ICBO), publishers of the *Uniform Building Code.* The choice depends on the competence or availability of the in-house staff, its workload and/or economic factors. As a result, designers of most construction in California receive a third-party review of their designs. Exceptions are those entities over which state and local jurisdictions have no authority to require building permits. These exceptions include federal and state facilities (including state universities) not constructed on private property. More specifically, a DOE site located on federal or state property is not required to obtain a building permit from a local jurisdiction such as a city or county in order to construct a building. Thus, there is no built-in process or *Building Official* to

perform a *third-party plan check* for a DOE sites *unless* the manager for the site establishes one.

To ensure a fully effective code enforcement program, it is essential that a single individual (knowledgeable and licensed as an engineer or architect) be formally delegated responsibility for review and approval of all design and construction at each DOE site. In this way the individual so appointed is the *Building Official* for the site, is responsible for interpretations of the code, and can be liable if negligent in the fulfillment of his or her responsibilities. The official delegation of this responsibility not only provides the *Building Official* with sufficient authority to enforce the code, but prevents other on-site individuals from making code decisions without the knowledge of the site manager or director. It also ensures that architectural-engineering (A-E) firms who design projects for the DOE site know that code or regulatory interpretations must be approved by the *Building Official* for the site rather than made unilaterally.

Once an individual has been delegated authority as *Building Official*, this person should become a member of the model code organization designated for use at the site. In this way he or she will be knowledgeable about current activities of the model code body, have access to its professional staff for advice about code matters and interpretations, be conscious of potential code changes, and receive its publications. This subject is discussed in more detail in Chapter 13.

The Lawrence Berkeley Laboratory's long experience with *third-party* reviews has been enlightening and to some extent alarming. LBL instituted third-party reviews in 1971 after a seismic review of its existing buildings was stimulated by the destructive San Fernando earthquake. In spite of the fact that these buildings were designed by licensed structural engineers using the latest edition of the *Uniform Building Code*, many contained serious design flaws that could have been easily detected in a third-party plan check. Since 1971, it has been LBL policy to impose third-party reviews on all facility design projects that involve life-safety whether designed by consulting A-E firms or in-house A-E staff. Subsequently, through the years, these third-party reviews have uncovered significant structural deficiencies in a surprising number of new projects designed by well-known A-E firms. Such errors usually result from lack of appropriate review procedures within the designer's office that are omitted because of competitive pressures and schedules. Fortunately, the costs for third-party reviews are very nominal, generally on the order of $10,000 or 0.1% of construction cost for a building with a $10 million construction cost.

The third-party review is often referred to as an *independent review*, and questions sometimes arise whether reviewing engineers can be truly independent if hired by a facilities manager who is also the site *Building Official*. The answer is no, because the *Building Official* cannot relinquish his or her institutional responsibility as *Building Official* for the facility to a *third party*. The object of third-party review is to provide both the *Building Official* and the designers with technical (not legal) protection against errors, omissions, lack of experience, and other problems that plague structural design, including lack of quality control in the design process.

Often reviewers make use of simplistic analytical processes to detect gross errors and those arising at the interfaces between disciplines. Except for certain construction types such as precast concrete, building redundancy usually prevents collapse unless there are gross overloads or gross calculational errors. A gross error is more likely to occur when sophistication obscures an understanding of the actual physical process. Thus, a simplistic analysis or approximate check is a very valuable element of the third-party review.

Frequently the third-party review raises technical differences of opinion or questions of code interpretation and application. The resulting resolution of such issues is, of course, vital to the performance of the structure. Finally, the mere fact that design engineers' work will receive *peer* review has a very positive effect on the quality of their design.

Earthquake engineering has developed rapidly and effectively in *earthquake country* because the lessons have been taught by actual earthquakes. Most structural engineers in California are convinced that their designs will, in fact, be subjected to the test during their professional careers. From a manager's point of view, it is important that engineers selected for design of the lateral-force-resisting system have such awareness. If they are not experienced in earthquake engineering, it is strongly recommended that the plan-check or third-party review be performed by qualified structural

engineers with considerable seismic design and field experience.

It is further recommended that the procedures for design and construction of earthquake-resistant structures include the following:

1. Submission of complete structural calculations, with lateral force analysis and design, signed and stamped by the licensed engineer in responsible charge and including:

   a) statement of design criteria, lateral-force levels, and design stresses to be used (this data should be repeated on the first sheet of the structural drawings)

   b) clear written description of the lateral-force-resisting system and the functional philosophy of the design

2. A plan-check by a third-party professional, experienced in earthquake engineering

3. Field inspection of the construction, with special attention being given to the lateral-force-resisting system by engineers responsible for the design

4. The filing of a statement by the engineer in responsible charge at the project's completion, certifying that construction complies with construction documents.

The cost premium to provide good earthquake resistance in new buildings in the United States varies with the seismicity, the locale, and the building type being considered. Table 6-1 was prepared in 1970 by the Structural Engineers Association of California for the *Task Force on Earthquake Hazard Reduction* chaired by Karl Steinbrugge. Although too generalized for specific proposals, it is useful both as a guide and to put the cost premium for seismic-resistant design in proper perspective. The original intent for this table was to compare cost premiums for buildings of similar types located in different seismicity zones. Although the table does not reflect code changes made since 1970, those changes have little effect on cost premiums except for buildings requiring ductile concrete frames and those designed with precast prestressed concrete elements. The design and construction requirements for these specific structural types have become much more stringent since the chart was published.

Table 6-2 provides more recent cost data from a study by Stephen F. Weber in 1985 entitled *Cost Provisions for the Design and Construction of New Buildings* and reported in *Societal Implications: Selected Readings*, prepared by the Building Seismic Safety Council, FEMA Earthquake Hazards Reduction Series 14. Table 6-2 has been more recently published as Table C1-2 in the 1991 Edition of *NEHRP Provisions* because the modifications made to the *Provisions* since 1985 have little effect on cost impacts. Table 6-2 and the summary that follows the table are reprinted from the Commentary, Part 2, of the 1991 Edition of the *Provisions*.

The cost to retrofit existing buildings for improved seismic resistance varies, primarily with the quality of design, the construction materials used, and the type of lateral-force-resisting system incorporated. Normally, if buildings were initially designed with earthquakes in mind, it would be easier and less costly to strengthen them than if no thought were given to earthquakes.

During the last decade, a great deal of progress in earthquake engineering has taken place as a result of research and development and as a consequence of destructive testing by actual earthquakes, particularly in California. Methods of analysis have greatly improved and personal computer software has been expanded to make dynamic analysis almost as easy for structural engineers as conventional analysis. During this period the soil-structure interaction (SSI) analysis has come to the fore and seismic isolation and energy-dissipation systems have come out of research into practice. Also, the field of data processing has become such an integral and important link in the continuity of operations in public and private institutions that the need to design data-processing systems for earthquakes has become critical. Each of these subjects is discussed in more detail in Chapter 6.

New regulations to control the accidental dispersal of toxic materials have greatly expanded the scope of design responsibility (and manager liability) for the safe performance of research and production facilities utilizing hazardous materials. Before 1988, the responsibility for proper installation and operation of such facilities was left primarily to

research or production people and largely ignored by facilities managers and designers. Now the seismic safety of these systems is specifically covered by model building codes; consequently, those responsible for enforcing these codes must ensure that research and production facilities utilizing hazardous materials are properly designed, installed, and operated to resist earthquakes.

Table 6-1.

Estimated Increased Costs to Provide Earthquake Resistance in Buildings.

Prepared by the Structural Engineers Association of California (1970).

AREA (ZONE) / BLDG. TYPE:	Areas where earthquake regulations are now enforced	Areas which now enforce design for hurricane, cyclone, tornado or abnormally high winds	Other areas to meet Zone 3 requirements	Other areas located in Zones 0, 1 & 2 to bring up to Zone 2 requirements as a minimum
1 & 2 story wood frame	0	1/2%	2%	1%
1, 2, 3 story brick or conc. block	0	4%	8%	4%
4 story & up brick or conc. block	0	5%	10%	5%
Concrete	0	2%	5%	2%
Steel frame	0	1/2%	3%	1%

1. Zones are those shown in the 1970 edition of the *Uniform Building Code*.
2. Percent increase includes design, inspection, and construction cost.
3. Table assumes:
   - Basic design requirements of the *Uniform Building Code* (or equivalent) for wind forces are recognized and enforced at present in the area where the building is located.
   - Includes extra architect and engineer design and inspection costs.
   - Compared construction is of same quality, construction material and fire resistance.

Table 6-2.

Percentage changes in structural cost and total building cost for the trial designs by city and city group with and without seismic provisions in current local codes in 1983-84.

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

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

[b]Projected percentage change in total building construction cost from the local code to early version of the *Provisions*, derived from estimated structural cost changes by using the following McGraw-Hill's, *Dodge Construction Systems Cost* (1984) data on structural cost as a percent of total building cost: low-rise residential, 18.1%; high-rise residential, 30.0%; office, 28.1%; industrial, 33.7%; commercial, 29.5%.

In summary, Weber's study of the results of the BSSC trial design program provides some idea of the approximate cost impacts expected from implementation of the *NEHRP Recommended Provisions*. For the 29 trial designs conducted in the 5 cities (Chicago, Ft. Worth, Memphis, New York, and St. Louis) whose local building codes had no seismic design provisions, the average projected increase in total building construction costs was estimated to be 2.1 percent. For the 23 trial designs conducted in the 4 cities (Charleston, Los Angeles, Phoenix, and Seattle) whose local codes had seismic design provisions, the average projected increase in total building construction costs was estimated to be 0.9 percent. The average increase in costs for all 9 cities was estimated to be 1.6 percent.

Chapter

6a

Conventional Analysis and Design

Roland L. Sharpe

Introduction

This chapter covers the analysis and design of new buildings and examines pertinent building code provisions including major changes since the 1983 edition of the *Seismic Safety Guide*. The following issues are discussed:

- Earthquake behavior of buildings
- Determination of criteria for facilities
- Construction materials
- Design steps and design considerations - structural
- General design requirements
- Design considerations - nonstructural
- Design considerations - lifelines
- Independent engineer peer review.

The bases for seismic codes has expanded over the last decade. The *Structural Engineers Association of California* (SEAOC) *Bluebook* (Ref. 1) is the basis for two major seismic codes: the *Uniform Building Code* (UBC) (Ref. 2), and the *Tri-Services Manual - Seismic Design for Buildings* (Ref. 3). A supplementary Tri-Services document relating to essential buildings was published in 1986 (Ref. 4).

ATC-3 *Tentative Provisions for the Development of Seismic Regulations for Buildings* (Ref. 5) published in 1978 is the basis for the *NEHRP National Earthquake Hazard Reduction Program* (NEHRP) *Recommended Provisions for the Development of Seismic Regulations for New Buildings* (Ref. 6) published by the *Federal Emergency Management Agency* (FEMA) in 1985 with updates in 1988, 1991 and 1994. The *NEHRP Provisions* were recommended by the *Interagency Committee on Seismic Safety in Construction* (ICSSC) in ICSSC RP 2.1-A (Ref. 7) as a minimum safety standard for all federal agencies to follow in meeting the intent of Presidential Executive Order 12699 of January 5, 1990, *Seismic Safety of Federal, and Federally Assisted or Regulated New Building Construction*, (Ref. 8).

Subsequently the UBC 1991 Edition, the *National Building Code* (NBC) 1992 Supplement (Ref. 9) and the *Standard Building Code* (SBC) 1992 Amendment (Ref. 10) have been judged by the ICSSC to be equivalent to the *NEHRP Provisions* for seismic safety. The

NBC and SBC adopted the 1991 *NEHRP Provisions* with modifications as seismic safety appendices in the fall of 1991.

DOE-STD-1020 (Ref. 11), also determined equivalent to the NEHRP Provisions, provides *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy (DOE) Facilities.* Both DOE-STD-1020 and its predecessor document, UCRL 15910, *Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards,* (Ref. 12) use the UBC for Performance Category 1 (UBC *Standard Occupancy Structures*) and Performance Category 2 (UBC *Essential Facilities*). DOE Order 6430.1A *General Design Criteria* (Ref. 13) provides general criteria for the design of DOE facilities. DOE Order 5480.28, *Natural Phenomena Hazards Mitigation,* (Ref. 14) requires that the need for updating site seismic hazard assessments be reviewed at least every 10 years.

Earthquake Behavior Of Buildings

The horizontal and vertical accelerations of the ground during an earthquake induce inertial forces in a building. These inertial forces are proportional to the building's weight. For most earthquakes their effects are primarily horizontal (lateral), and must be resisted by the building's lateral-force-resisting system. In some earthquakes, vertical accelerations may exceed the force of gravity, i.e., 1994 Northridge, California earthquake. If a structure cannot resist the forces induced by the ground motions, it suffers severe damage to both structural and nonstructural elements, and portions may collapse.

The floor and roof framing, beams, columns, and walls of a structure form a lateral-force-resisting system that must resist induced horizontal earthquake forces. The floors and roofs, when properly constructed and adequately connected to the vertical elements, transmit induced horizontal earthquake forces to resisting vertical elements. The vertical elements (columns and walls) transfer the horizontal forces induced in upper stories down to the foundation. Walls, referred to as *shear walls* in an earthquake engineering context, resist lateral earthquake forces. Shear walls must have enough in-plane strength and stiffness to prevent excessive lateral displacement of the building as it resists the seismic forces. Columns and beams, when connected with rigid end connections, act as moment-resisting frames to resist earthquake forces. Moment-resisting frames rely on the flexural strength of the columns and beams to resist lateral forces.

Inertial lateral earthquake forces are induced by the weight of the elements of the building when the building is subjected to ground motions. Much of the self weight (or *dead load*) occurs in the floor(s) and roof structure with the walls contributing additional weight, which may be substantial when the walls are constructed of reinforced concrete or precast concrete panels.

Lateral forces generated by the weight of the floor, roof, and wall elements are transferred to the shear walls by *diaphragm* action of the floor and roof slabs. Floor and roof diaphragms act as horizontal beams spanning between vertical shear walls or moment frames. Vertical elements, when adequately attached to these horizontal beams or diaphragms, are laterally supported. When the direction of ground motion is perpendicular to the plane of a wall (out-of-plane), the wall may fall away from the building unless it is adequately attached to a diaphragm or the structural framing. When a wall or precast panel is attached to a diaphragm, the out-of-plane seismic forces acting on the wall or precast panel are transferred to the diaphragm by the wall-to-diaphragm connections. The diaphragm receives this load as well as loads from other attached elements and transfers the accumulated load horizontally to the shear walls or moment frames.

Diaphragms, like shear walls, must have enough strength to resist lateral seismic forces without excessive damage and have enough stiffness to prevent excessive out-of-plane lateral displacements of the floor and/or roof. However, having shear walls or moment frames and diaphragms with adequate strength and stiffness is not sufficient (in themselves) to prevent damage or collapse in an earthquake. All elements of the structural system, diaphragms and walls or moment frames, must be properly connected or tied together in order to resist earthquake forces. A

building with strong walls (or moment frames and floors), but without adequate connections between them, may be no safer than a building with weak walls (or moment frames) and floors.

When designing structures for seismic resistance, it is important to recognize that design forces derived from code formulas represent the best judgment of competent groups of professional engineers aided by considerable input from researchers in earthquake engineering. In the design of a new building, a minimum design lateral force level is established, usually by code provisions. Building codes recognize that during a severe earthquake, a building's lateral-force-resisting system can be subjected to lateral forces that far exceed the design forces. Modern earthquake engineering practice is to design buildings to resist forces that are substantially less than the forces that develop during a severe earthquake.

Observations of damage in past earthquakes have shown that properly designed and constructed buildings can resist strong earthquake ground motions generated by a major earthquake without collapse even though designed for substantially lesser forces. This design approach is also based on past and continuing physical testing and research on building structures and elements. The seismic deformations to which a building or structure may be subjected during an earthquake are really unknown, but in most cases will greatly exceed the design deformations calculated using code provisions. As a result, major elements of buildings or structures could be severely overstressed. However, for most structures it is not economically feasible to design for the maximum loadings that might be imposed, consequently, most buildings are designed to resist forces that are substantially less than those that develop during a severe earthquake.

To achieve the type of behavior described above, building structures must be properly designed and detailed to possess some degree of *ductility* (the ability to sustain loads and remain stable after the material begins to yield). Major deficiencies in buildings built before the development of modern earthquake engineering standards are usually the result of low lateral force design levels, low ductility, and/or lack of connections between elements. When subjected to a severe earthquake or prolonged shaking of a moderate-intensity earthquake, nonductile (or brittle) elements yield (or fracture) and progressively lose their seismic resistance capacity. Numerous examples of this type of damage in older unreinforced masonry buildings (UMB) have been observed and documented in past earthquakes. Similar failures occurred in older nonductile concrete structures in the January 17, 1995 Hyogo-Ken Nanbu earthquake near Kobe, Japan (author's on-site observations). Thus, yielding and fracturing of nonductile structural elements must be prevented.

The structure and its appendages must remain stable when undergoing horizontal deformations that considerably exceed yield deflections. It is therefore basic to good design that in conjunction with providing minimum design strength, the performance of the structural system at very large overloads and deformations be carefully considered. Adequate performance may be provided in most cases by careful detailing of joints and members to ensure that the structure remains as a unit even when subjected to these very large deformations and stresses. Also, it is essential that redundancy be provided in structural framing systems. Structural elements should be designed so less critical elements are overstressed first, thus absorbing and dissipating energy and providing protection for more critical members. A building or structure having this capability is said to have *ductility*. Finally, it is essential that a continuous load path (or paths) having adequate strength and stiffness be provided so that all of the seismic-induced and gravity loadings can be transferred to the foundations or final point of resistance without significant failure of any of its elements.

Determination of Criteria for Facilities

Several factors should be considered in developing seismic criteria for building sites. The most important are:

- General site characteristics, type and depth of underlying soil, proximity of major faults
- Site seismicity, recurrence interval

- Peak ground motion and duration; acceleration, velocity, displacements, frequency content
- Building or facility use
- Type of structural system; required or expected performance of structure
- Applicability of code formulas.

These factors are discussed in the following sections

General Site Characteristics

When selecting a new site, an investigation should be made to determine the existence of any active earthquake faults and the potential for landslides, liquefaction, or dynamic consolidation of foundation soils when subjected to vibratory ground motions.

An active or potentially active fault at or near a site can cause damage to structures by ground rupture and/or by intense ground motions. The January 17, 1994 Northridge and the January 17, 1995 Hyogo-Ken Nanbu earthquakes demonstrated the intense ground motions and extensive damage that can be caused by nearby fault movements.

A nearby fault also can cause ground motions containing fairly high-frequency pulses and a long period, large-amplitude pulse or *fling*. The 1994 UBC recognizes this large pulse amplitude potential in Appendix Chapter 16, Division III governing design of seismic-isolation systems for buildings. The UBC allows static analysis design procedures for base-isolated structures more than 15 km from an active fault, but requires dynamic analysis for closer facilities.

Landslides, liquefaction, or soil consolidation can damage buildings by causing vertical and/or horizontal displacements of the foundations, differential settlements, or overturning. Most of the property damage in the 1964 Alaska Earthquake was caused by these phenomena. Extensive ground failures at port facilities and adjacent areas, due primarily to liquefaction in the Hyogo-Ken Nanbu earthquake, caused failures in quay walls, retaining walls, dock side equipment and adjacent buildings (author's on-site observations).

For an existing site, all available foundation investigations and geological reports should be reviewed and large differences in elevation studied. Soils-foundation reports should be assessed with the assistance of qualified geotechnical professionals to determine possible liquefaction potential of underlying soils and potential for soil compaction/consolidation when subjected to vibratory motions.

The types of soil underlying the site (soft or firm, granular or clayey, or rock-like) and their depths should be evaluated. If the predominant frequency (or natural period) of the site is or could be, fairly close to that of the building or structure, a resonant condition could occur that would significantly amplify the structure's response. Materials underlying the site have a major effect on the predominant period of the site ground motion. During the 1985 Mexico City earthquake, a region underlain with soft clayey soils was subjected to low-amplitude, long-duration ground motions. Many buildings in the four- to ten-story range were seriously damaged and/or collapsed. Buildings of these heights have fundamental periods of vibration within the range of the predominant ground motions that occurred in the area. There were a few similar situations around the San Francisco Bay region during the 1989 Loma Prieta, California earthquake on a much smaller and less intense scale. Some high-rise buildings (ten to twenty stories) in Los Angeles and surrounding areas suffered nonstructural damage during the June 28, 1992, Landers, California earthquake (magnitude 7.4). These buildings were more than one-hundred miles from the earthquake epicenter. The Landers earthquake was noteworthy for its large magnitude and long duration of strong ground motions (35 to 40 seconds). In the Hyogo-Ken Nanbu earthquake ground motions were greatly amplified by so-called *soft soils* and extensive damage to buildings and infrastructure resulted.

Detailed field investigations may be necessary to determine the existence and extent of any of the above hazards. The extent to which such investigations are employed depends on the size and importance of a

proposed facility, the probability of safety-related or large economic risks, and the possibility or suspicion of potential site hazards.

Site Seismicity

It is appropriate to select earthquake design coefficients for normal-usage facilities based upon the seismic zoning shown in the local building code. However, if the importance of a proposed facility and/or the potential risks involved so warrant, a special study of site seismicity should be performed. It should include a review of the historic seismicity within the surrounding area to at least a 50-mile radius. Probable frequencies of earthquake occurrence and probable ground motions at the site should be determined.

An important factor in determining site seismicity is the degree of acceptable risk or the performance required of the facility. Should the facility be able to function after occurrence of the maximum vibratory ground motions predicted to occur within 50 years, 100 years, 500 years, or 1,000 years? Should it be designed to sustain minimum damage and maintain functionability if any of these events occur? This decision should be made by management, not left to seismic consultants or engineers. DOE-STD-1020 (Ref. 11) and DOE 5480.28 (Ref. 14) address these questions. DOE 5480.28 requires the decision to be made by systems/safety engineers and facility managers. Earthquake return periods are specified for required performance categories.

Peak Ground Motion and Duration

A facility that must be able to function during and after a major earthquake, or still be functional after a major event, should have the *peak ground motions* (PGM) for design (including probable duration of strong ground motion) determined as a part of the site study. For most sites, the maximum possible PGM will be several times greater than the static building code coefficient. Therefore, the PGM amplitude used for design will depend on the degree of risk considered acceptable for the facility, i.e., usually the PGM for a 50-year mean return period will be much less than for a 500-year mean return period. There are exceptions, such as near the Hayward, California fault, where strain buildup has created the potential for a large earthquake in the near term.

Peak acceleration, velocity, displacement values and duration of strong ground motions appropriate for the selected risk should be determined. After the PGM is determined, response spectra should be developed considering soil and foundation conditions at the site and distances to potential earthquake sources. Acceleration time history plots should be prepared for facilities where continued functioning is required or a seismic isolation or passive energy dissipation system is being planned for some of the buildings or structures at the site.

Building or Facility Use

The planned usage or importance of a facility and its potential safety-related or economic risks should be considered when establishing an acceptable degree of seismic-damage. (See discussion in Chapter 4 and DOE-STD-1020). A laboratory or processing facility handling hazardous materials (such as highly toxic or radioactive materials) should be planned with more restrictive seismic design criteria than for a normal office-administration building or a warehouse. If special treatment is not warranted, appropriate building code design provisions can be followed. The UBC (Ref. 2) provides an importance factor for increasing the seismic design coefficient for structures of special importance or occupied by large numbers of people; for the highest categories, the seismic coefficient is multiplied by 1.25. The *NEHRP Provisions* (Ref. 6) do not increase the seismic force for structures of increased or special importance, but require more restrictive detailing for structural joints and members.

DOE-STD-1020 requires that each of DOE's facilities, including *Structures, Systems, and Components* (SSCs) be assigned to *Performance Categories* (PC) numbered PC-0 through PC-4 each with a quantitative goal for behavior (i.e., maintain structural integrity, maintain ability to function, maintain confinement of hazardous materials). Important features of the criteria include graded design, analysis, and quality assurance requirements increasing with seismic hazard and the quality of safety

necessary for facility use. DOE-STD-1020 allows the use of the UBC for PC-1 and PC-2 corresponding to UBC occupancy categories for *Standard Occupancy Structures* and *Essential Facilities*, respectively. Facilities assigned to PC-3 and PC-4 have much more stringent performance goals than required by the UBC. Refer to Chapter 4 for a more detailed discussion of *Seismic Design and Evaluation Requirements for DOE Facilities.*

Type of Structural System

The structural system used for a building, (moment-resisting space frame, eccentric braced frame, shear wall or braced frame, or bearing wall system) largely governs the type of response a building will exhibit during an earthquake. The structural configurations, both in plan and vertically, also play a major role in a building's response.

A moment-resisting frame (MRF) is usually the most flexible structure. It absorbs/dissipates energy by its members deforming in bending or flexure. Because a flexible structure tends to *give* when subjected to vibratory motions, less seismic energy is imparted to it. If the structural frame is ductile, i.e., has redundancy and capability to remain stable when stressed beyond yield levels, then a lower total seismic design coefficient can be used for design. However, nonstructural components and systems must be designed to accommodate greater structural frame deformations.

Accommodation for such movements should include connection and support details so that the frame can move relative to other elements without damaging exterior walls, windows, ceilings, or interior partitions. Similarly, provisions to accommodate building deformations should be provided in the design of mechanical and electrical systems in a moment-resisting frame structure. These provisions add to construction costs; however, a moment-resisting space frame may be the best solution for meeting building function and use requirements, if building deformations are properly accommodated.

The Loma Prieta, California earthquake again demonstrated the need to provide for differential movements between the structural framing and architectural and other nonstructural systems. For example, the executive offices of a major computer company, a 375,000-sq-ft, four-story building, was evacuated and not available for use for nearly ten months after the earthquake in order to make repairs and add provisions for movement between the structural framing and non-structural systems in the building. The building was designed and constructed as a ductile moment-resisting space frame for 1988 UBC seismic force levels. Unfortunately, little provision was made for differential movements and varying stiffnesses of building elements. There was no damage to the structural framing per se, but the owner was not pleased that it was necessary to evacuate this almost new building. The disruption to operations was significant.

The Hyogo-Ken Nanbu earthquake vividly demonstrated the need to consider nonstructural elements, mechanical/electrical systems and furnishings, when designing buildings. Many buildings had little or no structural damage, but contents were destroyed. One large American company was forced to move 2600 employees about 20 miles to a building in Osaka because the contents of its highrise building were trashed. The structural framing of this building performed in accordance with its design (on-site communication to author).

Eccentric braced frame (EBF) technology for providing energy absorption/dissipation in buildings has been developed in the past few years. The eccentric brace system provides some of the benefits of both the shear wall or braced-frame system and the moment-resisting frame. Special *link beams* provide ductility, while the bracing members provide stiffness. The system is called EBF because at least one end of each brace must intersect a beam eccentric to the beam-brace intersection. The section of the beam between the opposing braces or between a brace and the beam column intersection is called a *link beam.* The link beam is a ductile element where the structural system yields in lieu of the braces buckling when the dynamic loads on the frames exceed their elastic strength. Research has shown that this type of frame can exhibit ductile performance similar to ductile moment-resisting frames if the EBF is properly designed.

The EBF system can be used for many types of facilities. However, diagonal brace elements often intrude into areas that would be unimpeded space with an MRF system. It becomes a matter of trade-off between facility function and project cost.

A shear-wall structure is quite rigid and deflects less than a comparable moment-resisting frame when subjected to seismic ground motions. Therefore, more energy is imparted to the structure and a higher seismic force coefficient is required for design. Most of the seismic-induced energy is dissipated by shearing distortion, and less ductility is available. Because shear-wall structures deform considerably less than comparable MRF structures, there can be savings by using simple rather than more complex movement-allowable connections to the building structure for exterior cladding, windows, and interior partitions. Shear-wall buildings normally have a structural frame for supporting gravity loads.

A bearing-wall building has many of the characteristics of a shear-wall system; however, some of the bearing walls must be designed as shear walls to withstand or resist lateral earthquake forces. In a bearing-wall structure, gravity loads are mostly carried by the walls. A high seismic force coefficient, similar to that used for shear-wall buildings, is required for the design of bearing-wall structures because of their rigidity.

The final choice of a moment-resisting frame versus an eccentric-braced frame, shear wall or bearing wall, is most often controlled by functional and other design considerations.

Applicability of Code Formulas

Seismic design force formulas given in building codes apply to typical buildings; they are not intended for offshore towers, processing equipment, dams, or special facilities such as those involved with high-risk processes., i.e., nuclear power or processing plants. In all such cases, special consideration should be given to use of site-specific criteria, the dynamic characteristics of the structures, and the degree of risk involved. DOE-STD-1020 requires site specific seismic input for Performance Categories 3 and 4.

Construction Materials

Four basic materials are used for seismic-resistant structural systems: structural steel, reinforced concrete, reinforced masonry, and wood or timber. Buildings constructed of any of these materials (or combinations) can be designed to withstand major seismic motions. Each has physical characteristics that are conducive to certain types of buildings and functions. Pros and cons are discussed from a seismic design viewpoint in the following text.

Structural steel is a homogeneous material with excellent stress-strain properties. Steel has excellent ductility and can deform many times its yield point stress before failure. Structural steel moment-resisting and non-moment-resisting frames generally deform more when subjected to earthquake motions than other types of structural framing systems. As a result, unless adequate precautions are taken in the design to allow for these deformations, considerable damage can occur to nonstructural elements such as partitions, exterior walls/cladding, windows, ceilings, and mechanical and electrical systems (Refs. 15 and 16). Structural steel framing systems can be stiffened by using steel cross bracing or other types of bracing, or shear walls to reduce deformation induced by seismic motions. Nonstructural components are less likely to be damaged in such systems.

As previously noted , braced-steel framing has much less ductility than a moment-resisting steel framing. Bracing members act primarily in tension; when failure occurs, it is somewhat abrupt. Unlike a moment-resisting frame, a braced frame has little capability for dissipating energy by continued deformation. Steel eccentric braced frames, in contrast, allow ductile deformation because of the link beam design.

The damage to steel girder/column connections in the 1994 Northridge earthquake and damage to steel frame buildings during the 1995 Hyogo-Ken Nanbu earthquake have raised serious concerns about the design, fabrication and earthquake performance of welded structural steel connections. Significant cracking in or adjacent to welds for some moment-frame connections in buildings ranging from two stories to twenty stories in height was

uncovered in the aftermath of the 1994 Northridge earthquake. (See Chapter 3 Foreword for more detail.) These concerns have led to a multi-million dollar, several year research effort by the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe) (SAC joint venture) to assess the causes of the often brittle cracking/failures, to conduct experimental studies and develop design guidelines.

Meanwhile the International Conference of Building Officials (ICBO) adopted an emergency code change to delete 1994 provisions for steel moment frame joints and insert the following new requirements:

2211.7.1.1 Required Strength

The girder-to-column connections shall be adequate to develop the lesser of the following:

1. The strength of the girder in flexure.
2. The moment corresponding to development of the panel zone shear strength as determined by Formula (11-1).

2211.7.1.3-2 Connection Strength

Connection configurations utilizing welds and high strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotations and to develop the strength criteria in Section 2211.7.1.1 considering the effects of steel overstrength and strain hardening.

The full text and reasoning for this change and a detailed discussion of the problem including possible interim solutions are presented in *Interim Recommendation Number 2*, entitled *Interpretation of Moment Frame Connections 1994 UBC Emergency Code Change*, published in January 1995 (see Appendix A of this chapter).

Based on observations of damage in the Hyogo-Ken Nanbu earthquake it appears that steel buildings in Japan may have similar problems.

Conventional reinforced concrete construction, either as a moment-resisting frame or shear-wall structure, does not have as much ductility as moment-resisting steel frames. However, reinforced concrete moment-resisting frames can be designed and constructed to exhibit excellent ductility. Because these frames generally have larger member sizes than corresponding structural steel-frame members, they often are more rigid and deflect less when subjected to earthquake ground motions. As a result, the smaller deformations should cause less damage to nonstructural components. When subjected to major earthquake motions, concrete-moment frames tend to crack and, in some cases, the concrete cover over the reinforcing steel cracks and spalls. However, properly designed, detailed, and constructed reinforced concrete moment frames perform well during earthquakes.

The 1994 Northridge and the 1995 Hyogo-Ken Nanbu earthquakes again demonstrated that reinforced concrete joints, columns and connections must be carefully detailed. Adequate confining steel (transverse reinforcing) must be provided at beam-column joints and in columns. Consideration must be given to possible tensile stretching of longitudinal bars during overturning modes and subsequent buckling of the bars when the cycle reverses. It is the author's opinion from 1995 observations in Japan, that spiral confinement should be used either singly or overlapped in first story columns or piers that are the primary lateral-force-resisting elements.

Reinforced concrete buildings constructed with shear walls and building elements well tied together generally exhibit good performance during earthquakes. Shear wall reinforcing must be well anchored into boundary members and structural framing above and below. Because these structures are quite rigid, nonstructural components properly anchored to the structural framing suffer little damage.

Reinforced-masonry structures (either concrete unit masonry or clay brick) can be designed to function satisfactorily during a major earthquake. In UBC Seismic Zones 2A, 2B, 3, and 4, masonry must have steel reinforcing. When properly designed and constructed, reinforced masonry structures perform well during seismic motions.

Wood or timber structures generally exhibit excellent performance when subjected to major earthquakes. However, fire and other

building code safety requirements usually limit their size and height. Wood construction has substantial inherent energy-dissipating capability. When shear panels and wood diaphragms are properly utilized, wood structures have very good seismic resistance. The floors and/or roof are designed to distribute horizontal seismic forces to vertical load-resisting panels or shear walls. They act as *diaphragms*, which are analogous to a plate girder laid in a horizontal plane, where the floor or roof acts as the girder web, floor or roof beams function as web stiffeners, and the peripheral beams act as flanges. Anchorage of vertical shear elements to the foundation with anchor bolts and tie down anchors for overturning is a must.

The shear walls (or panels) are used to resist and transmit the lateral seismic forces to the foundation in the plane of the wall. A shear wall acts similar to a cantilevered plate girder standing on end; the wall acts as the web, and vertical boundary members function as the flanges. Shear walls may be constructed of wood, reinforced concrete, reinforced masonry, or structural steel.

Design Steps And Design Considerations - Structural

Some general design factors to be considered are building shape and geometry, framing type, basic construction materials, arrangement and type of nonstructural components, and adequacy of connections of various building parts and components. To construct an efficient, economical, seismic-resistant building, it is essential that architects and engineers collaborate as a team during the conceptual design stage. Often architects develop the conceptual plan, arrangement, and aesthetic design prior to discussing it with structural, mechanical, and electrical engineers who must design required systems. Involvement of these engineers during the conceptual design stage can often avoid costly and sometimes inadequate solutions for effective seismic resistance (Refs. 15 and 16).

Certain factors should be considered during the conceptual design phase:

1. The basic geometry of a building has a major effect on its inherent seismic resistance. It is desirable for seismic resisting systems to be symmetrical or have symmetry about each building axis. Symmetry should be considered in the arrangement of lateral-force-resisting elements including the location of shear walls and wall openings, and size and spacing of columns. Large eccentricities of building mass should be avoided (Ref. 17). If potential seismic force effects are considered in the initial layout, significant cost savings can be made without detracting materially from a building's function or appearance.

2. Re-entrant corners, such as those occurring in L-, T-, or U-shaped buildings , are locations of great stress during an earthquake and should be avoided or reinforced appropriately.

3. The potential effect of nonstructural components on the response of a building should be considered in the initial layout. Elements such as filler walls, interior partitions, and exterior cladding can add to lateral stiffness depending on the type of connection or attachment to the structural framing. Initially, the added stiffness caused by nonstructural components can induce higher seismic forces in the building than may have been contemplated in the design. For example, filler walls not symmetrically located may interact with the framing system, inducing torsional (rotational) moments in the structure, resulting in excessive stresses in columns or other shear-resisting elements.

4. The relative stiffnesses of different stories in a building can materially affect seismic response. The first story is often taller than others, with many interior walls omitted to present a more pleasing open appearance. This produces a flexible or *soft* first story, which could suffer excessive deformations and subsequent adverse effects.

   The Olive View Hospital provided a vivid example of the *soft-story* problem during the 1971 San Fernando, California earthquake. The upper stories were stiffened with concrete shear walls, but the first story depended on concrete columns to resist tributary lateral forces by flexure.

Large first-story deflections caused extensive damage and almost total loss of the vertical load-carrying capacity of the columns. Although the hospital was only about four months old, it was razed and replaced with a new facility. The second Olive View Hospital performed very well during the 1994 Northridge, California earthquake, resisting ground accelerations slightly greater than those which caused the original hospital to fail.

Observations of building damage after the 1995 Hyogo-Ken Nanbu earthquake disclosed innumerable buildings with excessive displacements in *soft* first floor stories. Many of these buildings collapsed.

5. After the conceptual plan layout is determined, construction materials are selected. Factors other than seismic resistance are considered, including fire-resistance characteristics, aesthetics, and construction cost. Architects and engineers should investigate various framing systems and construction materials (including life-cycle costs) to determine which will provide the required functions most economically.

6. A careful comparison of a moment-resisting space frame (MRSF) system versus a shear wall or EBF is often a first step. (Refer to the earlier discussion about MRSF joints in this chapter.) For buildings with few or no brittle finishes, such as a warehouse or shop building, any of these systems work equally well for seismic resistance. If there are many relatively brittle elements such as interior partitions, stairwells, and/or exterior glazing, the cost of stiffening the MRSF and/or providing connections to accommodate relative movements between the structural framing and these elements should be considered. In many cases, shear walls or bracing can be used to limit lateral deformations without sacrificing function.

7. Concurrently with the above effort, seismic design criteria for the site should be established. If a building code is used, appropriate seismic zone and design factors are selected from code provisions. *See Appendix B of this Chapter for applicable UBC tables and figures.* If a facility requires more detailed criteria because of its size, importance and/or safety-related or economic risks, detailed studies should be made by qualified professional consultants.

8. The following steps should be followed in the project design process:

   a. Select the site (if not already done) and evaluate potential seismic hazards at the site including liquefaction, land-sliding, and active fault proximity

   b. Determine the facility's function, importance, and potential safety-related or economic risks

   c. Determine the site seismicity (from building code or special studies)

   d. Determine the degree of acceptable risk (amount of property damage acceptable, necessity for maintaining function, potential hazard if function is lost, and potential hazard to occupants/public)

   e. Establish the geometry and layout of the facility (in plan and vertically; see prior discussion)

   f. Determine type of framing system and construction materials

   g. Establish specific seismic design criteria to be followed (based on building usage, importance, size, and type of framing system). These factors may be selected from the building code, DOE-STD-1020 or determined by independent studies

   h. Select the appropriate reduction factor (R or R_W) from the building code or DOE-STD-1020

   i. Make preliminary design of the structural framing system to determine tentative sizes of beams, columns, walls, foundations, etc.

   j. Calculate a preliminary value for the fundamental building period, T (either from code formula or computer program)

k. Calculate the seismic coefficient, C, and base shear, V as follows:

	UBC and DOE-STD-1020 (Refs. 2 and 11)	NEHRP (Ref. 6)
Coefficient	$C = 1.25\ S/T^{2/3}$	$C_S = 1.2\ C_V/RT^{2/3}$
Base Shear	$V = ZICW/R_W$	$V = C_S W$

where:

C = ≤ 2.75

S = soil profile factor for site; 1.0, 1.2, 1.5, or 2.0

A_v = seismic coefficient from NEHRP maps; varies from 0.4 to 0.05

T = fundamental period of the building in the direction under consideration

Z = seismic zone factor from map; 0.4 for zone 4, 0.3, 0.2, 0.15 and 0.075, respectively, for Zones 3, 2B, 2A, and 1 (UBC)

C_V = $F_V A_V$ where F_V is a site coefficient varying with shaking intensity (see discussion in Chapter 5a)

I = importance factor; varies from 1.0 to 1.25 (UBC)

R = response modification coefficient, varies from 8 to 1.25*

R_W = response coefficient, varies from 4 to 12**

W = total gravity load of building.

* NEHRP based on limit stresses
** UBC based on allowable stresses

l. Distribute base shear over height of building

UBC	NEHRP
$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i}$	$F_t = \frac{V w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$

where:

F_x = the lateral force induced at level x

F_t = the concentrated force (in addition to F_n) applied at the top of a building.

= 0.07TV and need not exceed 0.25V nor be considered for buildings with T less than 0.7 seconds

w_i, w_x = that portion of W located at or assigned to level i or x

h_i, h_x = height in feet above base to level i or x

k = exponent related to building period; 1 for T ≤ 0.5 seconds, and 2 for T ≥ 2.5 seconds, interpolated for intermediate values of T.

m. Recalculate the vertical and horizontal member sizes using the gravity loads and seismic shear forces calculated at each floor level. Distribute the seismic shears and torsional shear forces to the seismic resisting system in proportion to the relative stiffnesses of the vertical components and the floor or roof diaphragms. In addition to the calculated torsional moment, an accidental torsional moment equal to the story shear times a distance equal to not less than 5% of the maximum dimension of the building at that level (perpendicular to the direction under consideration) must also be distributed to the lateral-force-resisting system.

n. Design the building to resist overturning effects caused by wind or earthquake forces. The overturning moment at each floor level should be calculated using the seismic shear

forces and moment distributed to the vertical resisting elements (columns or walls). UBC and DOE-STD-1020 follow this procedure.

The *NEHRP Provisions* allow a reduction in overturning forces for buildings higher than ten stories. The full overturning moment is applied for the top ten stories and is reduced to 80% for the twentieth story from the top. Linear interpolation may be used for intervening stories.

o. Review the design to see if there is adequate redundancy in the structural framing system. There are many uncertainties in the amplitude and frequency characteristics of the earthquake ground motions, in the detailed behavior of materials and systems as they resist seismic loadings, and in the methods of analysis. Therefore, it is good earthquake engineering practice to provide as much redundancy as possible in the building's seismic resistance system. In a structural system without redundant components, every component must remain operative to preserve the integrity of the building structure. In a system with considerable redundancy, one or more redundant elements may fail and the structural system will retain its integrity and continue to carry vertical loads and resist lateral forces, although with some reduced effectiveness.

In a frame system, redundancy can be obtained by making all of the joints of the vertical load-carrying frame moment resisting as part of the seismic resisting system. A moment-resisting space frame has considerable load-carrying ability even if deformed beyond yield point stress. Redundancy can also be provided by using more than one type of seismic resisting system in a building; for example, utilizing a dual system of moment-resisting frames and shear walls or bracing. A back-up system can prevent catastrophic effects if the primary resisting system undergoes excessive deformations.

p. Consider the vertical component of earthquake ground motion in the design of cantilevered horizontal members and horizontal prestressed components (Ref. 6). In view of the high vertical accelerations recorded in the Northridge and Hyogo-Ken Nanbu earthquakes, possible adverse effects of vertical accelerations on critical vertical load carrying components should be considered for facilities in close proximity to active faults.

q. Check the design to determine if there are significant discontinuities in strength between adjacent stories, that could cause adverse response in the building. Normally the size, length, or strength requirement of a resisting member is then determined; if more than the required strength is provided, so much the better. Extra strength in a story, if significantly different than the strengths in adjacent stories, can produce responses that vary greatly from those calculated. UBC and NEHRP provisions prohibit weak stories for the higher performance structures by limiting the ratio of strength provided in each story to that required for adjacent stories. Over one-hundred-ten multi-story buildings (varying from six to twelve stories in height) suffered collapse of an intermediate story during the Hyogo-Ken Nanbu earthquake (author's on-site observation). An approach for considering this problem is presented in Appendix C of this chapter.

r. Calculate the *drift* in each story. Story *drift* is the displacement of one floor relative to the floor immediately below. The lateral displacement or deflection of a building relative to its base is not story drift, and should not be used for drift control or stability considerations because it could give a false impression of the effects of critical stories. The total building deflection is, however, important when considering seismic separation between adjacent buildings. Limitations provided in the UBC and DOE-STD-1020 on story drift (not to exceed

$0.04/R_W$ nor 0.005 times the story height for buildings having a fundamental period less than 0.7 seconds and not to exceed $0.03/R_W$ nor 0.004 times story height for buildings having a fundamental period of 0.7 seconds or greater) should be followed. (Table 6a-1 compares the *NEHRP Provisions* for story drift limitations with those of the UBC).

Story drift must be controlled to ensure building stability under maximum earthquake conditions. Large horizontal deflections can cause secondary stress effects resulting from eccentricity of the gravity load-inducing moments and stresses in the vertical supporting members. Moments and stresses caused by gravity loads are augmented by the stresses induced by bending moments equal to the lateral drift times the total weight of the structure above the level under consideration. Commentaries of Reference 1 (Section 1.E.8) and Reference 6 (FEMA 223 Section 2.3.7.2) summarize recent work in this area. In high-seismicity areas, seismic drift considerations control the lateral force design for moment-resisting frame buildings up to medium height. In low-seismicity areas and for very tall buildings in high seismic risk zones, wind loadings can control.

Table 6a-1. Allowable Story Drift, Δ_a[(a)] (in. or mm).

Building	NEHRP			UBC [(d)]	
	Seismic Hazard Exposure Group			T<0.7 sec.	T≥0.7 sec
	I	II	III		
Buildings, other than masonry shear wall or masonry wall frame buildings, four stories or less in height with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025\ h_{sx}$[(b)]	$0.020\ h_{sx}$	$0.015\ h_{sx}$	No limit	N.A.
Masonry cantilever shear wall buildings[(c)]	$0.010\ h_{sx}$	$0.010\ h_{sx}$	$0.010\ h_{sx}$	$\frac{0.04\,h_{sx}}{R_w}$ or $0.005\ h_{sx}$	$\frac{0.03\,h_{sx}}{R_w}$ or $0.004\ h_{sx}$
Other masonry shear wall buildings	$0.007\ h_{sx}$	$0.007\ h_{sx}$	$0.007\ h_{sx}$		
Masonry wall frame buildings	$0.013\ h_{sx}$	$0.013\ h_{sx}$	$0.010\ h_{sx}$		
All other buildings	$0.020\ h_{sx}$	$0.015\ h_{sx}$	$0.010\ h_{sx}$		

(a) h_{sx} is the story height below Level x.

(b) There shall be no drift limit for single-story buildings with interior walls, partitions, and exterior wall systems that have been designed to accommodate the story drifts (NEHRP).

(c) Buildings in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

(d) DOE-STD-1020 follows UBC for Performance Categories 1 and 2.

s. Review member sizes and seismic-resisting elements on each floor for conformance with the initial design assumptions including combinations of loadings required by UBC, DOE-STD-1020, or NEHRP requirements. If they do not conform, the procedure described above should be repeated using new sizes. If significant differences in mass and stiffness of adjacent stories exist, a dynamic analysis should be made (see Chapter 6b, *Dynamic Analysis* and Ref. 18). The general design approach for each basic material follows.

Structural Steel Frames

The requirements of two specifications of the American Institute of Steel Construction (AISC), *Load and Resistance Factor Design Specification for Structural Steel Buildings* (LFRD) (Ref. 19) and *Allowable Stress Design and Plastic Design Specification for Structural Steel Buildings* (ASD) (Ref. 20) provide the major criteria to be followed for the design of steel buildings.

For structural steel buildings in Seismic Zones 3 and 4, moment-resisting frames should be designed in accordance with the AISC-LFRD or -ASD specifications. Modifications and detailing requirements listed in the UBC should be followed, including connections of beams to columns, developing full plastic capacity of the beam, special consideration for possible local buckling in members when stressed beyond yield, and panel zone requirements for beam-column joints. (See earlier discussion concerning joints in steel moment-resisting frame in this chapter.)

Braced frames or eccentric-braced frames (EBF) should be designed to LFRD or ASD specifications for Seismic Zones 1, 2A, and 2B and with required UBC modifications for Seismic Zones 3 and 4 or modifications as specified in the *NEHRP Provisions* as appropriate. The story-drift limitations should not have much effect on braced-frame or EBF designs.

Reinforced Concrete

Reinforced concrete buildings designed to resist significant seismic forces usually have moment-resisting frames, shear walls, or combined moment-resisting frame and shear-wall construction. Because actual seismic forces may greatly exceed design forces, buildings may be deformed beyond the elastic limit to dissipate input energy. Therefore, ductile design detailing is necessary for structures to sustain gravity and other loads without catastrophic failure. Even though a structure's energy-dissipation capacity (work done in deforming the structure) is related to its inelastic deformation capabilities, over-reliance on such capabilities is inappropriate. Excessive structural deformations can cause significant damage to both structural and nonstructural elements. Either the deformations must be limited, or structural and nonstructural components must be designed to tolerate large drift deformations without significant damage (Ref. 21).

Concrete without steel reinforcing is brittle and, when overstressed, fails suddenly and sometimes catastrophically. Therefore, concrete structures should be reinforced and designed for minimum possibility of concrete compressive failure, concrete shearing failure, or loss of reinforcing anchorage. Compression failure can be controlled by requiring confinement or special transverse reinforcing around longitudinal reinforcing bars. Confinement of concrete increases strain capacity as well as the compressive, shear, and bond strengths of concrete. Maximum confinement should be provided near beam and column connections. Shear failures can be controlled by sufficient shear reinforcement and stirrup-ties or hoops. (See earlier discussion in this chapter about confining transverse reinforcing and overturning.) Anchorage failures can be controlled by following the special anchorage requirements given in the building code.

Building code provisions limit member size proportions for moment-resisting frames to ensure adequate seismic performance. For example, the ratio of minimum to maximum column dimensions must not be less than 0.4, and the ratio of width to depth for flexural members must not be less than 0.3. The minimum dimension for frame columns is 12 inches and for the width of flexural members is 10 inches. The use of light-weight concrete is

limited by the maximum permissible 28-day compressive stress of 4,000 psi.

Shear walls must have specially reinforced boundary members at their edges and around openings when the maximum total stress exceeds a given amount. Horizontal reinforcing in shear walls must be well anchored into boundary members.

Both the UBC and *NEHRP Provisions* include ductile concrete design and detailing requirements intended to inhibit failures from compression, shear, or loss of anchorage. References 21 and 22 include detailed discussions about seismic resistance design requirements for reinforced concrete structures. Reference 23, *Structural Concepts and Details for Seismic Design*, includes figures showing structural concepts and details for seismic design, limitations on dimensions for special moment-resisting concrete frames (SMRF), reinforcing bar splices, longitudinal reinforcement, girder web reinforcement, and column transverse reinforcement, girder column joint analysis, and reinforcement details for SMRF concrete frames. Reference 23 also contains recommendations concerning the design of shear walls and floor and roof diaphragms.

Reinforced Masonry Construction

Buildings constructed of masonry should be reinforced for seismic resistance. This applies to concrete block masonry as well as to grouted brick masonry. Numerous publications deal with these two types of masonry and References 2, 3, 6, 23 and 24 contain information regarding design details for reinforced masonry construction. Masonry walls should be firmly anchored to floors and roofs. There should be extra reinforcing at the tops and bottoms of openings, at changes of direction in building geometry, and at points where excessive stress might occur when the building is subjected to earthquake motions.

The UBC does not require special seismic design and detailing for masonry structures in Seismic Zones 0 and 1. DOE-STD-1020 follows UBC for Performance Categories 1 and 2. Special design and detailing are required for such structures in Seismic Zone 2. Additional provisions apply to structures in Seismic Zones 3 and 4. Requirements with similar intent are included in the *NEHRP Provisions*.

Wood or Timber Buildings

Fire-safety code requirements usually limit the area and height of wood or timber buildings, however, as noted previously, they generally perform well when subjected to major earthquake ground motions. Wood construction practices have not been codified in a standardized form in the United States. Reference 25 provides allowable stresses for members and connections. Chapter 23 of the 1994 UBC and Chapter 9 of the *NEHRP Provisions* describe excellent details and are good documents to follow. Wood has inherent energy-absorbing capacity and, especially for small structures, has performed well in earthquakes when all components are adequately tied together, the structure is anchored to the foundation, and the foundation rests on reasonably firm soil or rock.

General Design Requirements

Building codes do not cover all the design requirements necessary to make a structure fully earthquake-resistant; therefore, considerable engineering judgment based on experience is needed to ensure adequate resistance. The following discussion of special seismic requirements should be helpful to those less experienced in earthquake engineering.

1. Building components should be tied together to act as a unit. This not only aids in earthquake resistance, but also provides added resistance to high winds, floods, explosions, progressive failures, and foundation settlement. As a general requirement, a section passed through any part of a structure should be tied to the rest of the structure to resist a force at least equal to 5% gravity and, in higher seismic zones for at least 10% of the weight of the portion of the building being connected. In addition, beams should be tied together, to their supports or columns, and columns should be tied to the footings for a minimum of 5% of the dead and live load reaction.

2. Concrete and masonry walls should be anchored to all floors and roofs for lateral support. A common event noted during major

earthquakes is the pulling away of heavy masonry or concrete walls from floors or roofs. As a minimum, such walls should be anchored for a force equal to at least 200 pounds per lineal foot or the appropriate building code requirement, whichever is larger.

3. Shear walls or other bracing elements in buildings often are not uniformly spaced around the floor or roof diaphragms. Collector members must be provided to collect the shear forces and transmit them to these shear resisting elements. Collectors are composed of reinforced concrete beams for concrete slabs, steel members for steel diaphragms, and continuous wooden members for timber structures.

4. Diaphragms, as previously noted, act as horizontal deep beams or trusses. They distribute lateral loads to vertical resisting elements and are subject to shears, bending moments, direct stresses, and deformations. In some cases, deformations must be controlled because they could overstress the walls to which they are connected. Diaphragm deflections must not exceed the ability of those walls normal to the direction being analyzed to deflect without failure. The stress at wall anchorages tends to tear off diaphragm edges and, therefore, ties must be extended into the diaphragm to develop adequate anchorage. For openings in shear walls and diaphragms, chord stresses must be provided for, and the chord members anchored (to develop chord stresses) by embedment.

   A diaphragm should be tied together so it will act as a unit. Floor and roof diaphragms should be designed to resist forces to which they may be subjected. The UBC provides formulas for calculating these forces.

5. Bearing walls, like concrete and masonry walls, should be anchored to floor and roof diaphragms. It is important that the wall elements and interconnections have sufficient ductility or rotational capacity or strength to remain as a unit when subjected to inertial forces and accompanying deformations. Consideration should be given to the detrimental effects of shrinkage or differential settlement which can erode this capability.

6. Walkways into buildings or interconnections between buildings are often constructed with a roof slab and a single row of columns. These are referred to as inverted pendulum-type structures because a large portion of their mass is concentrated near the top. Where such structures incorporate heavy concrete slabs, lateral seismic motion may cause a rotation of the roof slab that can result in vertical accelerations acting in opposite directions on the slab overhang. Hence, a bending moment is induced at the top of the column. One way to cope with this is to apply one-half of the calculated foundation bending moment at the top and vary the moments along the column from 1.5 times the base moment at the base to 0.5 times the base moment at the top. This recommendation is based on background work performed during the development of Ref. 5.

Design Considerations — Non-structural

Most of the seismic design requirements in building codes are meant for structural framing systems. Experience in earthquakes over the past few decades has shown that the cost of damage to architectural, mechanical and electrical systems and components sometimes exceeds the cost of structural damage. Furthermore, costly nonstructural damage can occur without significant damage to the structural framing system. Enclosure systems (such as infill walls, curtain walls, spandrel beam covers, and precast panels), finish systems (such as partitions, ceilings, and veneers), and service systems (such as heating, lighting, air conditioning, communications, and transportation) affect and possibly alter the response of a building and its components during an earthquake. Any of these components may initially act structurally until overstressed, whether designed as part of the structural framing system or not. Although they are traditionally referred to as nonstructural components, they can behave structurally and improve or impair the building's basic ability to endure an earthquake.

The degree to which any structural or permanent nonstructural component may interact with any other or all of the building's component parts should be considered in determining whether a given component can be incorporated into the lateral force-resisting system. If so, this could reduce the initial cost of the structural system and enhance building performance during an earthquake. Architects should collaborate with engineers during the conceptual design stage so that such components can be incorporated into the traditional structural system to improve the building's response to earthquakes and help all components to better endure induced forces and deformations.

Unfortunately several categories of building components are typically not incorporated into the structural systems of most buildings. They are either overlooked or left out for economical or functional reasons. These categories include:

- Components that are not considered permanent
- Permanent or major components for which structural incorporation would be too expensive
- Components having mass, stiffness, or configurations that would probably have a detrimental effect on the building response or would cause unacceptable problems in the building's functional layout or aesthetic concepts.

Nevertheless, some components in the above categories, present in a given building, interact with others and affect the building's earthquake response. To provide a high degree of seismic safety in a building, architects should work closely with structural engineers when designing and detailing such systems.

Regardless whether a component is part of the structural system, consideration should be given in its design to improving earthquake resistance. For example, a partition that is connected to the floor and ceiling must be able to accommodate differential motion between the ceiling slab or floor above and the floor on which it is supported, as well as be compatible with motions that may be induced in the ceiling or in mechanical or electrical equipment systems. References 15 and 16 contain discussions about the interaction of building components during earthquakes and architectural seismic design of such components. Reference 26 presents an early discussion of design of seismic anchorage and restraints for mechanical and electrical components.

As a minimum, architectural components and mechanical and electrical systems and components should be designed to resist seismic forces to which they may be subjected; this is especially true in UBC Seismic Zones 3 and 4 and NEHRP map areas where A_v equals or exceeds 0.15. UCRL 15815 (Ref. 27) provides practical guidelines and details for seismic upgrade of equipment.

The UBC requires parts and portions of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure to be designed to resist lateral forces in accordance with the formula:

$$F_p = ZI_pC_pW_p$$

where:

F_p = the seismic force applied to the component at its center of gravity

Z = the seismic zone coefficient

I_p = the importance factor varying from 1.5 for life safety systems and toxic material containers to 1.0 for other components

C_p = the seismic coefficient (or horizontal force factor) for the component.

Equipment weighing less than 400 pounds and furniture are excepted.

See Appendix B of this chapter for applicable UBC tables and figures.

The above UBC formula is for use with elements and components and for rigid and rigidly supported equipment. Rigid or rigidly

supported means equipment for systems with a fundamental period equal to or less than 0.06 seconds. The lateral forces calculated for equipment or systems with a longer period shall consider the dynamic properties of both the equipment and structure supporting it, but the C_p value must not be less than the value listed for rigid elements. In lieu of a dynamic analysis, the C_p value for a nonrigid or flexibly supported equipment located above grade can be taken as two times the value listed for a rigid system but need not exceed 2.0. Piping, ducting, and conduit systems constructed of ductile materials and connections may use the C_p values for rigid systems.

For elements, components, and equipment laterally self-supported at or below ground level, the lateral force may be two thirds of the value for rigid systems but not less than V = 0.5 ZIW.

DOE-STD-1020 follows UBC requirements except for equipment located at grade; the coefficient C_p for non-rigid or flexibly supported items is twice the value for rigid or rigidly supported equipment.

The *NEHRP Provisions* (Ref. 6) also include detailed requirements for the design of architectural, mechanical, and electrical systems. The basic formulas used are of similar types. However, factors are included for required performance, amplification of force with increased height in the building (earthquake response varies with height), and type of attachment system for mechanical and electrical systems.

In the design of architectural, mechanical, and electrical components and systems, design consideration should be given to the differential displacement in each story (or story drift) during earthquakes. Because story drifts are relatively small in a shear wall or braced frame building, they probably do not need to be considered except in Seismic Zones 3 and 4. However, for most frame structures, provisions should be made to accommodate story drift. As noted previously, consideration should also be given to possible interaction between architectural, mechanical, and electrical systems when the building deforms.

Partitions, ceilings, and filler walls should be designed to resist seismic forces normal to their plane. The UBC specifies minimum factors for which these elements should be designed and minimum design requirements for architectural, mechanical, and electrical systems. Reference 16 discusses design of attachments for mechanical and electrical systems. Chapter 8 of Reference 4 and corresponding commentary discuss seismic design of architectural, mechanical, and electrical components.

Consideration should be given to providing seismic restraints or anchorage for furnishings in buildings. Furnishings which can move during an earthquake can present life safety hazards. Trashed contents inhibit operations for some time after a damaging earthquake.

Design Considerations — Lifelines

Facility lifelines include means of ingress and egress such as stairways and elevator systems; critical mechanical and electrical systems; and systems external to a facility, such as utilities, communications, road access, bridges, and fire protection. Seismic design of lifelines should include means for building occupants to evacuate safely during and after earthquake emergencies. Utilities such as natural gas could be ruptured and fires ignited. Liquid fuels or other flammables may leak from broken lines, or electrical short-circuit currents in excess of the normal protective device's capabilities may occur. Consideration should be given to devices that will automatically stop fuel flows or safely interrupt electrical currents in the event that severe earthquake motions occur; this recommendation is especially applicable to Seismic Zones 3 and 4. Gas and high-temperature energy supplies to buildings can be interrupted by installing seismic shut-off valves at service connections. Interruption in electrical service can be accomplished by shunt-tripping the main circuit breakers when activated by a sensor that can detect excessive ground motion. This means of electrical service shutdown should be carefully evaluated for any adverse effects on the electrical system.

The overall site electrical distribution network should be reviewed to ensure that the fault current potential that existed when the

site was first developed has not increased sufficiently to exceed the capability of supply and distribution equipment to adequately handle such loads. This problem is of concern because phase-to-phase or phase-to-ground faults can develop during a seismic event, and equipment may not be adequately designed to prevent destruction of service entrance equipment, service protection equipment, or distribution equipment, and could represent a significant source of fire. Consideration should be given to this problem to avoid a potential safety hazard. Further discussion is provided in commentary Section 8.3.6 of Ref. 5.

Generally, utility piping systems should avoid unstable ground and not traverse soil conditions with widely varying compaction or consolidation potential. Care should be taken when piping systems pass from natural ground onto an unstable fill area. In Seismic Zones 2B through 4, consideration should be given to providing two independent sources of water supply for major facilities. If a public utility company is providing the service, there is a good chance that little or no consideration has been given to the system's seismic resistance. A dependable water supply is essential to combat fires that might occur during or after major earthquakes (see Chapter 11a). The adverse impact of loss of water supply was vividly demonstrated during the 1989 Loma Prieta, California and 1995 Hyogo-Ken Nanbu, Japan earthquakes. For critical processes requiring continued functioning, an alternative gas supply such as liquid petroleum gas or an emergency power supply (diesel-powered generator) system should be provided.

It is essential that fire-fighting equipment have ready access to any facility that might be damaged by an earthquake. If access is by bridge or at the edge of steep slopes that might slide during an earthquake, consideration should be given to rerouting access, or improving the seismic resistance of road structures.

Chapter 10 in Reference 3 provides seismic details for various site utility systems.

Independent Engineer Peer Review

The proliferation of activities and associated equipment now employed in almost any building, compared with a few decades ago, has greatly complicated architectural and mechanical design for seismic safety. Many facilities are now composed of complex systems, or the construction procedures used may be innovative or untried. Designers are experiencing increasing pressure to produce work on accelerated schedules for low fees. This makes it very difficult for them to devote as much attention to design as they might like. Because of low fee criteria, the most qualified designers may not be selected. Therefore, it is more important than ever to have structural and seismic designs reviewed by independent consultants. Practical plan check procedures have been developed to help ensure good design and construction, and also to facilitate the reviews themselves, as well as inspections and approvals.

Independent seismic reviews should be made at least at two stages for major facilities such as those having potential risk to life safety because of the processes contained therein, or facilities with a potentially large economic loss. The first review should be made at the end of the conceptual design (pre-Title I services), and the second, separate review made when the final design is submitted for review before bids are taken. DOE order 6430.1A (Ref. 13) specifies that an independent review of the seismic design must be made for facilities and buildings where a seismic event can have a potential risk to operator lives, to public safety, or of large economic loss. DOE-STD-1020 (Ref. 11) requires independent peer review for facilities (structures, systems, and components) which are designed to performance categories PC-2 or higher.

Scope of Review - The scope of the *peer review* (PR) and how it is interpreted by the PR members and the *structural engineer of record* (SER) can become a major bone of contention. The scope must be carefully delineated prior to start of the review and care must be taken by the PR to ensure during the review that there is no encroachment on the design responsibility of the SER. Where the building code or DOE manual does not define the extent of the peer review, it may be only a limited review or detailed review assistance throughout the project including construction. It is essential that there be a clear understanding of whether the PR is responsible for only (a) a general review of details or (b) detailed checking of

calculations, drawings, details, and contract bid documents and specifications. Structural/seismic design reviews should include design philosophy, configuration, framing system, construction materials, seismic criteria, and any other factors pertinent to the seismic capability of the proposed facility. Particularly important in the review is to check for a continuous load path, or paths, and for the adequacy of their strength and stiffness to transfer seismic forces from point of application to the foundations. Also it is important to check the foundation to verify that it has been designed to accommodate forces developed or movements induced in the building by the design ground motions. Peer review by independent consultants need not provide a detailed check of the spacing of reinforcing bars, but rather an overview to help identify oversights, errors, conceptual deficiencies, and other elements likely to cause problems during and after construction. The purpose is to better ensure:

- Quality of design and design approach
- Quality of project documentation
- Structural performance
- Project objectives.

PR Qualifications - The success of PR is directly related to the experience, qualifications, and ability of individual reviewers and the PR as a team. The SER should be consulted during the selection because it is essential that PR members be respected by the SER. SEAONC, 1991 (Ref. 28) suggests the following qualifications:

- Be independent from design team and have no involvement in the project before or after the review
- Ability to cooperate with others
- Ability to conduct review in an unbiased and constructive manner
- Equal or higher level of technical expertise than SER
- California structural engineer as lead
- Familiarity with governing regulations for project.

ACEC/ASCE 1990 (Ref. 29) further suggests that the team be composed of registered professionals with at least 15 years of relevant experience. In addition, each structural reviewer should have substantial experience in the analysis *and* design of major buildings. An understanding of how a building will perform during an earthquake is essential. Often there is a tendency to believe the results of complex analyses, but they must be related to reality. The results of dynamic analyses depend in large part on the assumptions made while preparing the mathematical model(s). A major benefit of retaining a PR team is the added input in engineering judgment and intuition. Engineering intuition is an off-shoot from seasoned judgment. Design calculations or computer printout may look beautiful, but intuition may indicate something is wrong. Questioning by experienced PRs will usually reveal the problem.

Analysts often view a building from a numerical perspective; structural designers view it as a physical structure composed of many integral elements. Experienced structural engineers view the results of analysis and calculations as a guide to judgment.

Examples of peer review procedures include those employed by the U. S. Corps of Engineers advisory boards to review dam designs (either performed in-house or by engineering firms), and by the Bureau of Reclamation, which has developed similar procedures. The U.S. Nuclear Regulatory Commission has a staff and an advisory committee on reactor safety that perform certain independent peer review functions for nuclear power plants. Many utilities involved with major facilities have independent design reviews made. These may seem to be unnecessary expenses, but peer reviews can catch costly design mistakes in judgment, calculations, or philosophy (Refs. 30 and 31). For a major facility, an independent peer review could more than pay for itself by uncovering design deficiencies before they are cast in concrete or constructed in steel.

Submittal of Data - Another important requirement is that architect-engineers submit a complete set of structural calculations, as-

built drawings and specifications, an explanation of the lateral-force-resisting system, and a listing of the design loads and criteria. It is extremely helpful, after a building has been constructed for some time, to be able to utilize as-built drawings to determine how a proposed change in the building structure would affect its lateral force resistance. Design loads and criteria should be listed either on the drawing cover sheet or on a sheet near the front of each set because calculations are sometimes separated or misfiled separately from the plans. Similarly, an explanation of the lateral-force-resisting system should be listed on an early sheet of the drawings.

When required, calculations for seismic resistance of mechanical and electrical systems should also be submitted, and a listing of design loads and criteria given on the mechanical and electrical drawings. *Lastly, architect-engineers should certify that calculations and drawings have been checked by their staff.* These requirements should be part of fee negotiation because the use of cursory checks or spot checks of calculations and drawings may result from competitive fee negotiation.

DOE-STD-1020 (Ref. 11) specifies that independent design or evaluation reviews must include design philosophy, structural system, construction materials, design and evaluation criteria used, and other factors pertinent to the seismic capacity of the facility. Refer to Chapter 12a for a more detailed discussion of *Quality Assurance by Peer Review.*

References

1. *Recommended Lateral Force Requirements and Commentary,* Seismology Committee, Structural Engineers Association of California, San Francisco, California, 1990.

2. *Uniform Building Code,* 1994 Edition, International Conference of Building Officials, Whittier, California.

3. *Seismic Design for Buildings,* Army TM5-809-10, Navy NAVFAC P-355, Air Force AFM 88-3, Chapter 13, Departments of the Army, Navy and Air Force, Washington, D.C., October 1992.

4. *Seismic Design Guidelines for Essential Buildings, a Supplement to Seismic Design for Buildings,* Army TM5-809-10.1, Navy NAVFAC P-355.1, Air Force AFM 88-3, Chapter 13.1, Departments of the Army, Navy, and Air Force, Washington, D.C., February 1986.

5. *Tentative Provisions for Development of Seismic Design Regulations for Buildings,* ATC-3-06, Applied Technology Council, Palo Alto, California, June 1978.

6. *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings,* 1994 Edition, FEMA 222 and *Commentary* FEMA 223, Federal Emergency Management Agency and Building Seismic Safety Council, Washington, D.C., March 1995.

7. *Guidelines and Procedures for Implementation of the Executive Order on Seismic Safety of New Building Construction,* ICSSC RP 2.1-A, NISTIR 4852, National Institute of Standards and Technology, Gaithersburg, Maryland, June 1992.

8. *Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction,* Executive Order 12699, Washington, D.C., January 5, 1990.

9. *National Building Code,* 1992 Supplement, Building Officials Conference of America, Chicago, Illinois, 1992.

10. *Standard Building Code,* 1992 Amendment, Southern Building Code Conference International, Birmingham, Alabama, 1991.

11. *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities,* DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.

12. *Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards,* UCRL-15910, Lawrence Livermore National Laboratory, Livermore, California, June 1990.

13. *General Design Criteria,* DOE Order 6430.1A, U.S. Department of Energy, Washington, D.C., April 1989.

14. *Natural Phenomena Hazards Mitigation,* DOE Order 5480.28, U.S. Department of Energy, Washington, D.C., January 15, 1993.

15. *The Interaction of Building Components During Earthquakes,* MBT Associates and Engineering Decision Analysis Company, Inc., National Science Foundation Grant Research Report, San Francisco, California, January 1978.

16. *Architectural Design of Building Components for Earthquakes,* MBT Associates, National Science Foundation Research Grant Report, San Francisco, California, 1978.

17. *Building Configuration and Seismic Design,* C. C. Arnold, R. Reitherman, John Wiley & Sons, New York, New York 1982.

18. *Behavior of Structural Systems Under Dynamic Loads,* R. L. Sharpe, E.G. Kost, J. Lord, Building Practices for Disaster Mitigation, Building Science Series 46, National Bureau of Standards, Washington, D.C., February 1973.

19. *Manual of Steel Construction, Load & Resistance Factor Design (LRFD) for Structural Steel Buildings,* First Edition, 1986 and Supplement No. 1, January 1989, American Institute of Steel Construction, Chicago, Illinois.

20. *Allowable Stress Design and Plastic Design Specifications for Structural Steel Buildings (ASD),* American Institute of Steel Construction, Chicago, Illinois, June 1989.

21. *Evolution of Codes and Standards for Earthquake-Resistant Reinforced Concrete Building Construction,* R.L. Sharpe, Proceedings, Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, California, 1977.

22. *Seismic Design of Concrete,* R.L. Sharpe, E. Elsesser, ACI Workshop, American Concrete Institute, Annual Convention, San Diego, California, April 1977.

23. *Structural Concepts and Details for Seismic Design,* UCRL-CR-106554, Lawrence Livermore National Laboratory, Livermore, California, September 1991.

24. *An Evaluation of Seismic Design Provisions for Masonry in the United States,* B.I. Sveinsson, R.L. Mayes, H.D. McNiven, Report 81-10, Earthquake Engineering Research Center, University of California, Berkeley, California, August 1981.

25. *National Design Specification for Wood Construction,* including supplement, National Forest Products Association, Washington, D.C., 1991.

26. *Needed: Seismic Design of Mechanical and Electrical Equipment,* R. L. Sharpe, R. Gallagher, Building Standards, International Conference of Building Officials, Whittier, California, September-October, 1972.

27. *Practical Equipment Seismic Upgrade and Strengthening Guidelines,* UCRL 15815, Lawrence Livermore National Laboratory, Livermore, California, September 1986.

28. *Guidelines for Project Design Peer Review,* Structural Engineers Association of Northern California, San Francisco, Draft, February 1991.

29. *Project Peer Review Guidelines,* American Consulting Engineers Council/American Society of Civil Engineers, New york, New York, 1990.

30. *Independent Engineer Review of Base Isolation Projects — Buildings,* R.L. Sharpe, Proceedings, Seminar and Workshop on Base Isolation and Passive Energy Dissipation, Applied Technology Council, Redwood City, California, March 12-14, 1986.

31. *The Status of Peer Review*, R.L. Sharpe, E. Elsesser, Proceedings, Seminar and Workshop on Base Isolation, Passive Energy Dissipation, and Active Control, Applied Technology Council, Redwood City, California, March 11-12, 1993.

Appendices to Chapter 6a

Contents

A. *Interim Recommendation Number 2, January 1995; Interpretation of Moment Frame Connections, 1994 UBC Emergency Code Change*, SEAOC Seismology Committee.

B. Selected tables and figures from the 1994 *Uniform Building Code* (Ref. 2).

C. *Discontinuities in Strength of Vertical Resisting System*, Excerpts from Ref. 6, Sec2.2.5.4.2, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, FEMA and the Building Seismic Safety Council, Washington, D.C. 1994.

Appendix A - Chapter 6a

Interim Recommendation Number 2
January 1995

INTERPRETATION OF MOMENT FRAME CONNECTIONS 1994 UBC EMERGENCY CODE CHANGE

INTRODUCTION

In response to concerns about the performance of prescriptively designed steel moment frame beam-column connections during the 1994 Northridge Earthquake, the Board of Directors of the International Conference of Building Officials passed an emergency code change that deleted provisions prescribing the requirements of beam-column joint connections for use in Seismic Zones 3 and 4. In place of the deleted sections, the following paragraphs were inserted and comprise the amended 1994 *UBC* requirements for steel moment frame connections.

> **2211.7.1.1 Required Strength**
> The girder-to-column connections shall be adequate to develop the lesser of the following:
>
> 1. The strength of the girder in flexure.
>
> 2. The moment corresponding to development of the panel zone shear strength as determined by Formula (11-1).
>
> **2211.7.1.3 -2 Connection Strength**
> Connection configurations utilizing welds and high strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotations and to develop the strength criteria in Section 2211.7.1.1 considering the effects of steel overstrength and strain hardening.

The uncertainty regarding the nature of the causes and solutions to the connection problem suggested the use of broad, general language in the emergency code change, allowing latitude in interpretation by structural engineers and building officials. These recommendations, produced by the Seismology Committee of the Structural Engineers Association of California, supersede those published in the October 1994 issue of *Building Standards Special Newsletter*, and are intended to provide background information to assist designers and building officials in interpreting and applying the revised provisions. These are *interim* recommendations and may be revised as more information is obtained. Designers and building officials are urged to keep abreast of continuing developments in this field.

A research program developed under the auspices of SEAOC, ATC, and CUREe, otherwise known as the SAC Joint Venture, is underway and information will be published frequently in the coming months. The bibliography at the end of these Interim Recommendations contains additional reference material that will aid the engineer and building official in interpreting the emergency code change. Interested parties may obtain material published by SAC by contacting:

Structural Engineers Association of California
555 University Avenue, Suite 126
Sacramento, California 95825
(916) 427-3647 FAX (916) 568-0677

It is unlikely that universal prescriptive design standards will produce acceptable seismic frame connections. Those that use these Interim Recommendations should recognize that there is considerable professional debate regarding the specific steps needed to achieve acceptable seismic performance in seismic frame beam-column connections. Therefore, it is recommended that these Interim Recommendations be applied with sound professional judgment to reflect the specific conditions present in the subject project and not adopted as unvarying design requirements. In addition, engineers must insure that the design decisions they make in an attempt to avoid the connection damage noted following the Northridge Earthquake do not develop connections that possess excessive strength at the expense of an acceptable level of ductility. Achieving this balance will represent a significant challenge.

These Interim Recommendations are intended for new construction. Application of these recommendations to existing or damaged structures should be done with judgment until such time as specific recommendations for existing buildings are available.

RECOMMENDATIONS

While the precise causes of the steel frame connection damage are not known with certainty, the damage has been generally attributed to the following two basic reasons:

1. Concentrating plastic strains in the welded portion of the connection by locating the "plastic hinge" at the column face may have caused the non-ductile connection behavior observed. The reader is directed to the bibliography, particularly AISC Update 1 and SAC Advisory 3 for a comprehensive discussion of recent testing of mechanisms to shift the plastic hinge from the column face. *Designers are advised, where plastic rotational demands will exceed 0.01 radian in the frame girder, to utilize connection details that locate the plastic hinge away from the column face. Stresses at the column face should be minimized, as discussed further below.*

2. Ineffective welding, unacceptable welding practices, inappropriate weld detailing, and/or inadequate welding inspection may be responsible for some premature fractures. Improved welding procedures and inspection practices are discussed in detail in SAC Advisory No. 3, dated January 1995. *Designers are advised to*

implement improved weld details, procedures and inspection practices to reduce the possibility of welding-related failures.

Engineers must estimate both demand and capacity. These Interim Recommendations discuss how anticipated seismic demand may influence the performance of beam-column joints and provide a context for interpreting the required capacity of the connection. Capacity must address both the underlying strength of a connection as well as its ability to develop the deformations that are anticipated during a large earthquake.

The following definitions will help clarify the discussion that follows:

Drift: The relative displacement or deformation between two reference points such as two floors or the base of the building and the roof. It is often expressed as a ratio of the displacement to the distance between the reference points in the form of a percentage.

Plastic rotation, θ_p: The angle of rotation, measured in radian, occurring as a result of plastic hinging. It may be a combination of beam hinging and column panel zone yielding. For a frame girder assumed to have a point of inflection at its midpoint, modeled as a cantilever beam with insignificant column panel zone yielding, it is computed as follows:

$$\theta_p = 2\Delta_p/\ell_c$$

where

Δ_p = plastic deflection = $\Delta - \Delta_e$
Δ = Total deflection measured at the point of inflection
Δ_e = Elastic deflection
ℓ_c = Clear span of frame girder (measured at face of column)

The definition of plastic rotation discussed above corresponds to that used in most of the relevant testing of beam-column joints. Designers must adjust the definition to reflect the difference between an experimental set-up measuring the simulated deflection of frame (by measuring the tip deflection of a cantilever beam) and the story drift that reflects the demand on a frame. These calculations are beyond the scope of these recommendations; however, relevant examples are found in steel design texts such as Englekirk [1994].

DEMAND

Traditionally, engineers have calculated demand in moment frames by sizing the members for strength and drift using code forces (either equivalent static or reduced dynamic forces) and then "developing the strength of the members." Since 1988, "developing the strength" has been accomplished by prescriptive means. It was assumed that the prescribed connections would be strong enough to yield the beam or girder (in bending) which would then hinge in a nearly

perfectly plastic manner producing the plastic rotations necessary to dissipate the energy of the earthquake.

The implicit total frame deformation under real earthquake loads, including both elastic and plastic deformation, is assumed to be in the range of $3R_w/8$ to $3R_w/4$ times the drifts corresponding to code forces. This range of estimated total deformation has been corroborated by inelastic time history analyses for a small number of code-designed steel structures. If we assume that the code-based drift ratio is 0.04 and $R_w = 12$, the approximate range of total drift is:

$$\text{Total Drift} = \text{Code-based Drift} \times 3R_w/8 \text{ to } 3R_w/4$$
$$= 0.04/12 \times (4.5 \text{ to } 9) = 0.015 \text{ to } 0.03$$

Therefore, an estimate of the drift demand is approximately 0.015 to 0.03 times the story height for special moment frames designed to code allowable drift limits. In such frames, a portion of the drift will be due to elastic deformations of the frame, while the balance must be provided by plastic rotations of the beam plastic hinges, by yielding of the column panel zone, or by a combination of the two.

In the 1994 Northridge Earthquake, many moment frame connections fractured with little evidence of plastic hinging of the beams or yielding of the column panel zones. Testing of moment frame connections both prior to and subsequent to the earthquake suggests that the standard welded flange-bolted web connection is unable to reliably provide plastic rotations beyond about 0.01 radian for all ranges of beam depths and often fails below that level. Prior tests on sections shallower than those found in modern steel frames did report some successful results that achieved plastic rotations in excess of 5%. Since the elastic contribution to drift may approach 0.01 radian, the necessary inelastic contributions will exceed the capability of the standard connection in many cases. Thus, for frames designed for code forces and for the code drift, the necessary plastic rotational demand may be on the order of 0.02 radian or more and new connection configurations must be developed to accommodate such rotation without brittle fracture.

In lieu of designing connections to accommodate large assumed plastic rotations, it may be possible in some frames to substantiate a lower rotational demand by a more rigorous analysis. Methods such as an inelastic time history analysis using an appropriate suite of time histories, or through less sophisticated methods involving the calculation of plastic rotations required at a drift of $3R_w/4$ times the code-based drift may provide sufficient insight into the probable plastic rotation demand. Perhaps, if a frame were designed such that it were strong enough to limit plastic rotations in the frame girder to 0.01 radian or less, the use of the standard (prescriptive) connection with welding enhancements might be justified.

Moment frames in dual systems in low- to mid-rise buildings, and those dual systems in the lower floors of taller buildings, are likely to be subjected to significantly smaller plastic rotation demands

than those in pure moment frame structures and may meet the above recommended plastic rotation limit of 0.01 radian. Similarly, taller buildings framed as "tubes" may have low rotational demands in some of the moment connection locations.

In summary, the following assumed demands are recommended in the absence of project-specific information to the contrary:

1. Unless rotational demands are specifically calculated and shown to be lower, connections should be shown to be capable of developing a minimum plastic rotation of 0.020 to 0.025 radian.

2. When plastic rotation demand on the frame girder can be demonstrated to be 0.0075 radian or less, under the excitation of a design basis earthquake, or at R_w times code forces, the standard connection as prescribed in the 1991 UBC may be considered for use for Group 1 and 2 sections not deeper than W24x.

CAPACITY

On the capacity side, the emergency code change provision requires that the connections "demonstrate by approved cyclic test results or calculation, the ability to sustain (the necessary) inelastic rotations and to develop the strength criteria..." What are "approved cyclic tests?"

Approved Cyclic Test Results

The SAC program and additional NSF-funded research will attempt to clarify what should constitute an "approved cyclic test." Issues to be addressed include the effects of:

1. Dynamic loading (high cycle strain rates)
2. Time history loading
3. Composite slabs
4. Axial load on columns
5. Girders on both sides of columns

In the meantime, the answer to what should constitute "approved cyclic tests" requires that three questions be answered. First, what should each test consist of? Second, how many tests should be required? Third, how may tests be extrapolated?

Much of the limited research devoted to behavior of seismic frame beam-column connections is referenced in the Bibliography, and only a few test results related to representative beam-column connections are available. One recent program executed at the University of Texas [Engelhardt, 1994] under the sponsorship of the American Institute of Steel Construction consisted of pseudo-static tests of full-scale bare steel specimens. The girder was on one side of the column only and no axial load was applied to the column. The loading cycles consisted of three full cycles slowly applied at increasing load increments or plastic rotation angles until failure occurred or the limits

of the testing apparatus was achieved. This is roughly comparable to the recommendations of ATC-24, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures*. These tests replicated many of the failures noted in frame beams and welds examined following the Northridge Earthquake, although no cracks in the columns were reproduced. Unless future testing programs reveal significant effects of dynamic loading rate or time history loading, testing protocol similar to ATC-24 will likely become the accepted standard, and unless the effects of the other factors noted above (e.g. composite slabs) are found to be compelling, tests emulating the University of Texas tests should be considered as acceptable for "approved tests."

Since the level of confidence in connections developed strictly on the basis of calculations may not be as high as those based on tests, the use of testing is encouraged. Tests are, however, relatively expensive and a reasonable degree of flexibility in interpreting the results of limited testing programs must be acknowledged.

The testing program should replicate as closely as practical the anticipated conditions in the field, including such things as:

1. Member sizes and material specifications.
2. Welding process, details and construction conditions.
3. Coverplates, continuity plates, web tabs, bolts, and doubler plates.
4. Joint configuration (e.g. beams on both sides).

While the testing of all connection geometries and member combinations in any given building is not practical, the number of tests must be large enough to be meaningful yet small enough to not be unreasonably costly. It is suggested that three specimens accurately reflecting actual conditions, with acceptable performance in *all three* be considered the minimum number of specimens. If any of the specimens fail prematurely, a redesign should be considered before retesting, and three redesigned specimens should be tested.

How much extrapolation should be accepted is a difficult decision. As additional testing is done, more information may be available on what constitutes "conservative" testing conditions, thereby allowing easier decisions relative to extrapolating tests to actual conditions which are likely to be less demanding than the tests. For example, it is hypothesized that connections of shallower, thinner flanged members are likely to be more reliable than similar connections consisting of deeper, thicker flanged members. Thus, it may be possible to test the largest assemblages of similar details and extrapolate to the smaller member sizes -- at least in comparable member group families.

Calculations

An approved program for steel moment frame connections may consists of a testing program with calculations used to assist in extrapolation of results or may be based on calculations alone. For calculations alone to be acceptable, they should be correlated to tested material properties for

base metals and welds. The properties should be those corresponding to the axes of loading of the base metal or weld in the connections and to the welding processes and materials intended for use. The tested properties may be specific to the materials and processes to be used in the project, or based on a statistically-based testing program. Use of properties inferred from other testing programs must be done with appropriate care and, where such inferred properties are used, designs should reflect the uncertainty inherent in such an indirect approach.

It should be noted that joint strengthening methods that involve shifting the plastic hinge away from the column face through the use of coverplates, haunches, or other means will magnify the demand at the beam-column joint and these effects must be considered in any analytical model.

Properties of materials which can be inferred from the University of Texas tests, welding industry information, work of the AISC Task Force and surveys of steel producers are summarized below in Table 1. The purpose of this listing is to suggest that the use of specified properties (e.g. F_y = 36 ksi for A36 steel) is not appropriate for the validation of a connection design rather than to suggest the precise values an engineer should use. Further, the mechanical behavior of the joint is very complex with a number of factors influencing how the demand is concentrated within individual elements of the joint. The values included in Table 1 are an attempt to reflect the empirical observations of joint behavior rather than to suggest a complete theoretical model of behavior. As there is a high degree of variability, the values in Table 1should be used carefully and verified for each project to the extent possible.

The weld filler metal strength assumed equal to 60% of nominal axial strength reflect the empirical results of recent tests, and should not be interpreted as suggesting that filler metal strength did not meet minimum ASTM requirements. For example, tests of specimens similar to the standard moment connection failed through the flange welds at a moment of about 25,000 in-kips. For the W36 x 150 beam, this corresponds to a weld stress of about 50 ksi *assuming a uniform distribution of stress across the weld.* It is unlikely that the stress in either the flange or the weld is uniform, particularly for joints without column flange stiffeners, and stresses far higher than those suggested by a uniform assumption may exist in selected elements of the weld (e.g. in the vicinity of the column web).

Since failures that match the University of Texas test have been found in buildings with column flange stiffeners and these failures can be classified as brittle, it seems prudent to investigate the average stress on the weld. The average weld stress in the successful coverplated tests was on the order of 40 ksi while average weld stresses in some specimens experiencing weld damage was higher than 40 ksi but less than the assumed tensile strength of the filler metal. However, tests on segments from damaged joints suggest that uniaxial tensile strength properties of the weld, heat affected zone, and base metal met or exceeded the minimum ASTM requirements. The apparent reduction in developed weld strength appears to be a reflection of the analytical simplifications used to analyze the joint rather than a lack of strength in the filler metal.

The through-thickness strength of the column flange is perhaps the most difficult material property to determine and may be the least reliable. As noted above, it is unlikely that uniform stress distributions exist in either the flanges or the weld; however, the precise affect on the "through-thickness" failures of this non-uniform distribution is not known with certainty. Because the shapes of the failure plane are unpredictable and it is unlikely that reliable through-thickness properties can be determined by testing, it is expedient at this time to calculate the stress at the face of the column assuming a uniform stress distribution and use a conservative estimate of corresponding through-thickness strength.

Table 1
STEEL PROPERTIES FOR USE IN CONNECTION DESIGN

Material	F_y min.	F_y mean	F_u
A36 Beam	36 ksi	49[1] ksi	58-80 ksi
Dual Certified Beam	50	55[1]	65 min.
A572-50 Column/Beam			
Axial	50	55[1]	65 min.
Through-Thickness			40[2,3]

Notes
1. Based on coupons from web. May be lower for thick flanges, such that $F_{y\,flange}$ is approximately equal to $0.95F_{y\,web}$.
2. Assumed at $0.75F_u$ of nominal strength.
3. Stress at face of column flange.

Depending on how one decides to calculate this stress, it can be as high as the weld stresses noted above, (if it is assumed to be the stress at the face of the column flange calculated using the elastic section modulus) or it may be significantly lower than that depending on the depth of the potential failure plane. For the University of Texas specimen that failed when a divot was pulled from the column flange, the stress at failure was approximately 38 ksi (at the face of the column flange). The tear-out extended nearly one inch into the column material at some locations. If one assumes that the surface area of the failure plane in the case of a "divot" failure will be larger than the contact area between the column and beam flanges at the face of the column, the average stress causing the "divot" failure will be lower than the apparent stress at the face. For these reasons, an estimated effective through-thickness strength of 60% of nominal strength is suggested.

If a statistically significant number of through-thickness tests of column flanges are performed as a part of the SAC or other test program, a level of reliability may be established and the use of a ϕ-factor may be the appropriate formulation. The required ϕ-factor to be used could be related to the redundancy of the system with a higher factor being permitted for systems with a higher degree of redundancy. It is possible that the reliability of the through-thickness properties is not

random, but related to metallurgy, rolling practices, geometric properties of the column flange, and presence of column axial force. In the future, it may be possible to mitigate some of these effects though use of steels with more favorable physical properties.

In computing the total force demand on the connection, several factors need to be considered when computing the forces (moments and shears) acting on the connection.

1. The value of the moment at the plastic hinge location, denoted as M_{pr}, to be consistent with nomenclature in the UBC, must be increased beyond the specified value of M_p, considering the following:

   a. Actual yield stress f_y greater than the specified f_y.
   b. Strain hardening.
   c. Uncertainty of calculations in the absence of physical testing.

2. The value of the moment (M_c) and shear (V_c) acting at the connection cross section should be based on the value of M_{pr} noted above and must account for gravity loads on the beam itself. In all cases, the free body diagram, moment diagram and shear diagram of the beam (from face of column to face of column) must satisfy all principles of statics.

The following equation relates the connection moment (M_c) to the specified plastic moment (M_p):

$$M_c = \alpha \, \beta \, M_p$$

where α amplifies $\beta M_p = M_{pr}$, to account for the connection moment being greater than the plastic hinge moment and

β amplifies M_p, to account for overstrength because of actual vs. specified yield strength and strain hardening as well as to account for uncertainty in analytical models of joint behavior.

To compute β, actual yield, strain hardening and appropriate uncertainty factor need to be determined. SEAOC recommends the following:

Actual Yield: The actual yield strength, for purposes of computing M_{pr}, may be taken as:

$$F_{y\,actual} = 0.95\, F_{y\,mean}$$

$F_{y\,mean}$ for A36 and Grade 50 is shown in Table 1, and is based on a survey of web coupon tensile tests. The 0.95 factor is believed to be an appropriate factor to convert from web to flange material.

Strain Hardening: A factor of 1.1 is recommended for use with the mean yield stress in the foregoing table when calculating the demands on the connection and reinforcement. The 1.1 factor for strain hardening, or other sources of strength above yield, agrees fairly well with the University of Texas test results. It should be noted that the 1.1 factor could underestimate the overstrength where significant flange buckling does not act as the gradual limit on the connection demand as was seen in the University of Texas tests. Nevertheless, the 1.1 factor seems a reasonable expectation of overstrength considering the complexities involved.

Modeling Uncertainty: Where a design is based on approved cyclic testing, the modeling uncertainty may be taken as 1.0, otherwise SEAOC recommends a value of 1.2.

In summary, for A36 and Gr. 50 steel, we have:

A36 steel: $\beta = [0.95\ (49\text{ksi})/36\text{ ksi}]\ (1.1)\ (1.2) = 1.71$, say 1.7

A572, Grade 50: $\beta = [0.95\ (55\text{ ksi})/50\text{ ksi}]\ (1.1)\ (1.2) = 1.38$, say 1.4

From the above, the probable plastic hinge moment value M_{pr} is found as:

$$M_{pr} = \beta M_p = \beta\ (Z_x F_{y\,specified})$$

To find the probable value of the moment acting on the connection, the plastic hinge location must be determined and gravity loadings must be considered. The value of α must be determined by a static analysis of the beam span, using plastic hinge locations that result in a statically admissible diagram. Note that α will increase as gravity load increases.

SEAOC recommends the following factor load combination for this analysis:

$$U = 1.2D + 0.5L + "\alpha M_p"$$

Use of calculations alone based on inferences made from the University of Texas tests presupposes that the actual conditions will mirror those of the tests as closely as possible. Such conditions include member materials, welding materials, processes, and procedures, and construction sequence (residual stresses). Acceptability of a calculations-only process of

connection design is a matter for careful consideration by the structural engineer and the building official. Subjective factors affecting the acceptability of this approach should include:

1. *The importance of the structure:* Higher levels of conservatism in applying a calculation-only approach should be considered for more important facilities.

2. *Confidence in the lateral forces used in design:* Projects with extensively researched, site-specific seismic hazard studies carefully applied may warrant less conservatism than those that do not use this type of information. Most structures are designed to satisfy the code minimum seismic forces. Structures that are designed assuming higher levels of seismic demand than found in typical projects, may warrant greater latitude in applying a calculation-only approach.

3. *The degree of redundancy and potential over-strength in the structure:* Greater care in applying a calculation-only approach should be considered in structures with a limited number of lateral force resisting elements in each direction or unusual building geometries. Structures with a high degree of redundancy may be able to tolerate better limited instances of marginal joint performance. Frames designed to limit the rotational demand by relying on elastic or near-elastic behavior may also be more amenable to a calculation-only approach than those that depend on high levels of plastic rotation to dissipate anticipated seismic demands. However, it has not been shown that superior seismic performance results when strength is substituted for ductility, and overly strong, non-ductile connections are not intent of these Interim Recommendations.

For essential structures, structures that will contain hazardous materials, and structures that are designed with a low degree of conservatism or redundancy, use of connections based on previously tested connections or project-specific testing of connections is strongly recommended. This recommendation should be considered until such time as SAC or other programs develop sufficient data to allow formulation of analytical design guidelines of general application.

For non-essential structures designed with a reasonable degree of redundancy or overstrength and incorporating enhanced welding requirements and quality control, calculations as described above, using proportioning and stress levels compatible with the University of Texas tests, may provide sufficient assurance of reliability.

Welding Enhancement

Review of welding practices and observed field conditions in damaged steel frame buildings suggest that proper welding procedures were not followed uniformly prior to the Northridge Earthquake. While welding issues are not addressed explicitly by the emergency code change, the tests performed to date suggest that design considerations alone will not produce acceptable connection performance and that strict adherence to proper welding procedures is an essential ingredient to insure acceptable performance.

1. Strictly enforce the welding requirements in AWS D1.1 as modified in 1994 *UBC* Chapter 22, Division VIII or IX.

2. Implement the special inspection requirements in 1994 UBC section 1701 and AWS D1.1. Visual inspection means that the inspector inspects periodically the welding for adherence to the approved welding procedure specification starting with fit-up and proceeding through the welding process. Reliance on the use of nondestructive examination (NDE) at the end of the welding process alone should be avoided. Use visual inspection in conjunction with NDE to improve the chances of achieving a sound weld.

3. Require the fabricator to prepare and submit a welding procedure specification (WPS) with at least the information required by AWS D1.1. The WPS shall be acceptable to the engineer of record, it should be furnished to the building official, and it shall be strictly enforced. The WPS shall be used in providing the required visual inspection. The WPS should list the position, electrode type and size, travel speed, electrode stickout, voltage and amperage with acceptable ranges, bead size, weld sequence, stress relieving requirements, and other pertinent data. A copy of the filler metal manufacturer's technical data sheet should be submitted with each WPS to confirm the pertinent welding parameters.

4. Filler metal should have a notch toughness not less than 20 ft-lbs as measured by a standard Charpy V-notch test, ASTM E23, in accordance with the applicable filler metal specifications referenced in AWS D1.1. The notch toughness shall be measured at least 30 degrees F. below the lowest ambient service temperature (LAST) but not warmer than 0 degrees F.

5. All welds for the frame beam-column joints shall be started and ended on weld tabs were practical. All weld tabs should be removed, the affected area ground smooth and tested for defects using the magnetic particle method.

6. Weld dams do not meet the intent of weld tabs and are not permitted in lieu of weld tabs.

7. Backing bars, if used, should be removed from the girder bottom flange, the weld root back-gouged by air arcing and the area tested for defects using the magnetic particle method. The weld shall be repaired and reinforced with a fillet weld per AWS D1.1. Removal of the backing bar at the top girder flange is at the discretion of the engineer of record.

BIBLIOGRAPHY

The evolving nature of this challenge requires that structural engineers make every effort to say informed. Most of the following publications are widely available and provide important technical background to further guide the structural engineer and building official.

American Institute of Steel Construction (AISC), "Seismic Provisions for Structural Steel Buildings," *Manual of Steel Construction, Load & Resistance Factor Design (Volume I)*, Chicago, IL: American Institute of Steel Construction, 1994.

AISC Advisory Committee on SMRF Research, *Northridge Steel Update 1*,, Chicago: American Institute of Steel Construction, October, 1994.

AWS, *Structural Welding Code, ANSI/AWS D1.1*, 1994.

Anderson, J.C. and R.R. Linderman, "Steel Beam to Box Column Connections," Report No. CE 91-03, Los Angeles: University of Southern California, 1991.

Bertero, V., J. Anderson, and H. Krawinkler, *Performance of Steel Building Structures During the Northridge Earthquake*, Report No. UCB/EERC-94/09, Berkeley: Earthquake Engineering Research Center, August, 1994.

Engelhardt, M.D. and A.S. Husain, "Cyclic-Loading Performance of Welded Flange-Bolted Web Connections," *Journal of Structural Engineering*, New York: American Society of Civil Engineers, 119:12, December 1993.

Engelhardt, M.D. and T.A. Sabol, "Testing of Welded Moment Connections in Response to the Northridge Earthquake, Progress Report to AISC Advisory Committee on SMRF Research," *Northridge Update Number 1*, American Institute of Steel Construction, October, 1994.

Englekirk, R.E., *Steel Structures: Controlling Behavior Through Design*, New York: John Wiley & Sons, 1994.

International Conference of Building Officials [ICBO], *Uniform Building Code, 1994 Edition*, Whittier, CA: International Conference of Building Officials, 1994.

Popov, E.P., and R.M. Stephen, "Cyclic Loading of Full Size Steel Connections," *Bulletin No. 21*, Washington, D.C.: American Iron and Steel Institute, 1972.

Popov, E.P. and V. Bertero, Cyclic Testing of Steel Beams and Connections," *Journal of the Structural Division*, New York: American Society of Civil Engineers, June 1973.

Popov, E.P., N.R. Amin, J. Louie, and R.M. Stephen, "Cyclic Behavior of Large Beam-Column Assemblies," *Engineering Journal*, Chicago: American Institute of Steel Construction, 23:1, 1986.

SAC, Steel Moment Frame Advisory No. 1, Sacramento, CA: SAC Joint Venture, September 26, 1994.

SAC, Internal Working Document, Steel Moment Frame Connection Advisory No.2, Sacramento, CA: SAC Joint Venture, October 19, 1994.

Structural Engineers Association of California (SEAOC), *Recommended Lateral Force Requirements and Commentary*, Sacramento, CA: Structural Engineers Association of California, 1990.

SEAOC, Ductile Steel Frame Beam-Column Joints, A Discussion of Preliminary Observations, Conclusions, and Recommendations, Seismology Committee, Sacramento: Structural Engineers Association of California, November, 1994.

Tsai, K.C. and E.P. Popov, "Steel Beam-Column Joints in Seismic Moment Resisting Frames," *Report No. UCB/EERC-88/19*, Berkeley: Earthquake Engineering Research Center, 1988.

Wakabayashi, M., *Design of Earthquake Resistant Buildings*, New York: McGraw-Hill Book Co.,1986.

Appendix B - Chapter 6a

Selected Tables and Figures from the 1994 Uniform Building Code (Ref. 2)

TABLE 16-I—SEISMIC ZONE FACTOR *Z*

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

The zone shall be determined from the seismic zone map in Figure 16-2.

TABLE 16-J—SITE COEFFICIENTS[1]

TYPE	DESCRIPTION	*S* FACTOR
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second (762 m/s) or by other suitable means of classification, or (b) Medium-dense to dense or medium-stiff to stiff soil conditions, where soil depth is less than 200 feet (60 960 mm).	1.0
S_2	A soil profile with predominantly medium-dense to dense or medium-stiff to stiff soil conditions, where the soil depth exceeds 200 feet (60 960 mm).	1.2
S_3	A soil profile containing more than 20 feet (6096 mm) of soft to medium-stiff clay but not more than 40 feet (12 192 mm) of soft clay.	1.5
S_4	A soil profile containing more than 40 feet (12 192 mm) of soft clay characterized by a shear wave velocity less than 500 feet per second (152.4 m/s).	2.0

[1]The site factor shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless the building official determines that soil profile S_4 may be present at the site, or in the event that soil profile S_4 is established by geotechnical data.

TABLE 16-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category I facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category I, II or III structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances which, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category I Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations; and other public utility facilities not included in Category I or Category II above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures[4]	All structures housing occupancies or having functions not listed in Category I, II or III and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1631.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Sections 108, 1701 and 1702.
[3]For anchorage of machinery and equipment required for life-safety systems the value of I_p shall be taken as 1.5.

TABLE 16-L—VERTICAL STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Stiffness irregularity—soft story** A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1627.8.3, Item 2
2. **Weight (mass) irregularity** Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof which is lighter than the floor below need not be considered.	1627.8.3, Item 2
3. **Vertical geometric irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	1627.8.3, Item 2
4. **In-plane discontinuity in vertical lateral-force-resisting element** An in-plane offset of the lateral load-resisting elements greater than the length of those elements.	1628.7
5. **Discontinuity in capacity—weak story** A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	1627.9.1

TABLE 16-M—PLAN STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Torsional irregularity—to be considered when diaphragms are not flexible** Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	1631.2.9, Item 6
2. **Reentrant corners** Plan configurations of a structure and its lateral force-resisting system contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1631.2.9, Items 6 and 7
3. **Diaphragm discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1631.2.9, Item 6
4. **Out-of-plane offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	1628.7; 1631.2.9, Item 6; 2211.8
5. **Nonparallel systems** The vertical lateral load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	1631.1

TABLE 16-N—STRUCTURAL SYSTEMS

BASIC STRUCTURAL SYSTEM[1]	LATERAL-FORCE-RESISTING SYSTEM—DESCRIPTION	R_W[2]	H^3 × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels		
	a. Wood structural panel walls for structures three stories or less	8	65
	b. All other light-framed walls	6	65
	2. Shear walls		
	a. Concrete	6	160
	b. Masonry	6	160
	3. Light steel-framed bearing walls with tension-only bracing	4	65
	4. Braced frames where bracing carries gravity loads		
	a. Steel	6	160
	b. Concrete[4]	4	—
	c. Heavy timber	4	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	10	240
	2. Light-framed walls with shear panels		
	a. Wood structural panel walls for structures three stories or less	9	65
	b. All other light-framed walls	7	65
	3. Shear walls		
	a. Concrete	8	240
	b. Masonry	8	160
	4. Ordinary braced frames		
	a. Steel	8	160
	b. Concrete[4]	8	—
	c. Heavy timber	8	65
	5. Special concentrically braced frames		
	a. Steel	9	240
3. Moment-resisting frame system	1. Special moment-resisting frames (SMRF)		
	a. Steel	12	N.L.
	b. Concrete	12	N.L.
	2. Masonry moment-resisting wall frame	9	160
	3. Concrete intermediate moment-resisting frames (IMRF)[5]	8	—
	4. Ordinary moment-resisting frames (OMRF)		
	a. Steel[6]	6	160
	b. Concrete[7,8]	5	—
4. Dual systems	1. Shear walls		
	a. Concrete with SMRF	12	N.L.
	b. Concrete with steel OMRF	6	160
	c. Concrete with concrete IMRF[5]	9	160
	d. Masonry with SMRF	8	160
	e. Masonry with steel OMRF	6	160
	f. Masonry with concrete IMRF[4]	7	—
	2. Steel EBF		
	a. With steel SMRF	12	N.L.
	b. With steel OMRF	6	160
	3. Ordinary braced frames		
	a. Steel with steel SMRF	10	N.L.
	b. Steel with steel OMRF	6	160
	c. Concrete with concrete SMRF[4]	9	—
	d. Concrete with concrete IMRF[4]	6	—
	4. Special concentrically braced frames		
	a. Steel with steel SMRF	11	N.L.
	b. Steel with steel OMRF	6	160
5. Undefined systems	See Sections 1627.8.3 and 1627.9.2	—	—

N.L.—No limit.
[1]Basic structural systems are defined in Section 1627.6.
[2]See Section 1628.3 for combination of structural system.
[3]*H*—Height limit applicable to Seismic Zones 3 and 4. See Section 1627.7.
[4]Prohibited in Seismic Zones 3 and 4.
[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1632.2.

(Continued)

FOOTNOTES TO TABLE 16-N—(Continued)

[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2211.6 may use an R_w value of 12.
[7]Prohibited in Seismic Zones 2, 3 and 4.
[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1631.2.7.

TABLE 16-O—HORIZONTAL FORCE FACTOR, C_p

ELEMENTS OF STRUCTURES, NONSTRUCTURAL COMPONENTS AND EQUIPMENT[1]	VALUE OF C_p	FOOTNOTE
1. **Elements of structures**		
1. Walls including the following:		
a. Unbraced (cantilevered) parapets	2.00	
b. Other exterior walls above the ground floor	0.75	2,3
c. All interior bearing and nonbearing walls and partitions	0.75	3
d. Masonry or concrete fences over 6 feet (1829 mm) high	0.75	
2. Penthouse (except when framed by an extension of the structural frame)	0.75	
3. Connections for prefabricated structural elements other than walls, with force applied at center of gravity	0.75	4
4. Diaphragms	—	5
2. **Nonstructural components**		
1. Exterior and interior ornamentations and appendages	2.00	
2. Chimneys, stacks, trussed towers and tanks on legs:		
a. Supported on or projecting as an unbraced cantilever above the roof more than one half their total height	2.00	
b. All others, including those supported below the roof with unbraced projection above the roof less than one half its height, or braced or guyed to the structural frame at or above their centers of mass	0.75 2.00 0.75	10
3. Signs and billboards	0.75	
4. Storage racks (include contents)	0.75	4,6,7,11
5. Anchorage for permanent floor-supported cabinets and book stacks more than 5 feet (1524 mm) in height (include contents)	0.75	4,9
6. Anchorage for suspended ceilings and light fixtures		
7. Access floor systems		
3. **Equipment**		
1. Tanks and vessels (include contents), including support systems and anchorage	0.75	
2. Electrical, mechanical and plumbing equipment and associated conduit, ductwork and piping, and machinery	0.75	8

[1]See Section 1630.2 for items supported at or below grade.
[2]See Section 1631.2.4 and Section 1630.2.
[3]Where flexible diaphragms, as defined in Section 1628.6, provide lateral support for walls and partitions, the value of C_p for anchorage shall be increased 50 percent for the center one half of the diaphragm span.
[4]Applies to Seismic Zones 2, 3 and 4 only.
[5]See Section 1631.2.9.
[6]Ceiling weight shall include all light fixtures and other equipment or partitions which are laterally supported by the ceiling. For purposes of determining the seismic force, a ceiling weight of not less than 4 pounds per square foot (19.5 kg/m^2) shall be used.
[7]Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed provided the walls are not over 50 feet (15 240 mm) apart.
[8]Equipment includes, but is not limited to, boiler, chiller, heat exchanger, pump, air-handling unit, cooling tower, control panel, motor, switch gear, transformer and life-safety equipment. It includes major conduit, ducting and piping serving such machinery and equipment and fire sprinkler systems. See Section 1630.2 for additional requirements for determining C_p for nonrigid or flexibly mounted equipment.
[9]W_p for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 10 psf (0.479 kN/m^2) partition load allowance.
[10]In lieu of the tabulated values, steel storage racks may be designed in accordance with Chapter 22, Division VI.
[11]Light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings shall be independently supported from the structure above as specified in U.B.C. Standard 25-2, Part III.

TABLE 16-P—R_W FACTORS FOR NONBUILDING STRUCTURES

	STRUCTURE TYPE	R_W
1.	Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	3
2.	Cast-in-place concrete silos and chimneys having walls continuous to the foundation.	5
3.	Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	4
4.	Trussed towers (freestanding or guyed), guyed stacks and chimneys.	4
5.	Inverted pendulum-type structures.	3
6.	Cooling towers.	5
7.	Bins and hoppers on braced or unbraced legs.	4
8.	Storage racks.	5
9.	Signs and billboards.	5
10.	Amusement structures and monuments.	3
11.	All other self-supporting structures not otherwise covered.	4

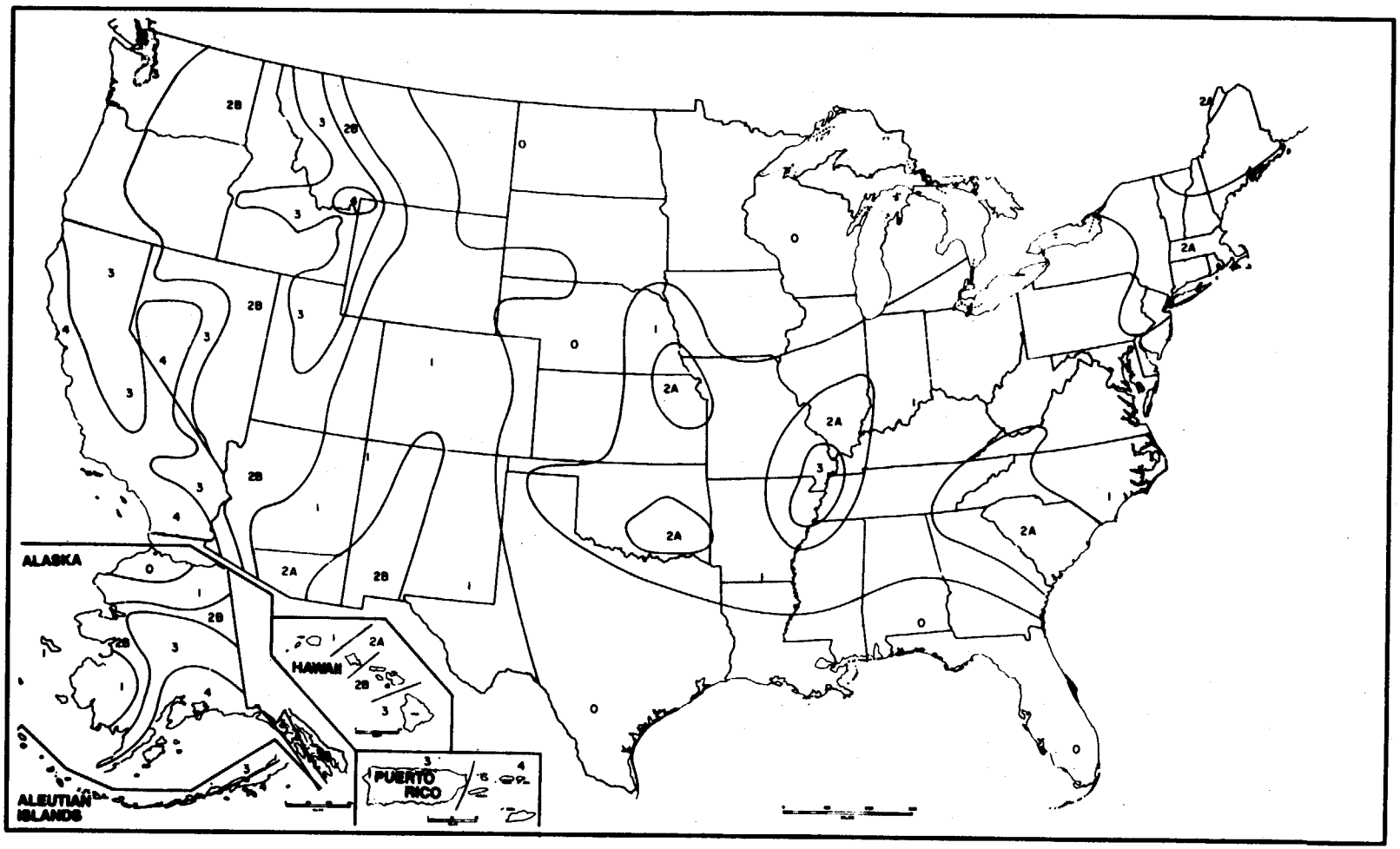


FIGURE 16-2—SEISMIC ZONE MAP OF THE UNITED STATES

For areas outside of the United States, see Appendix Chapter 16.

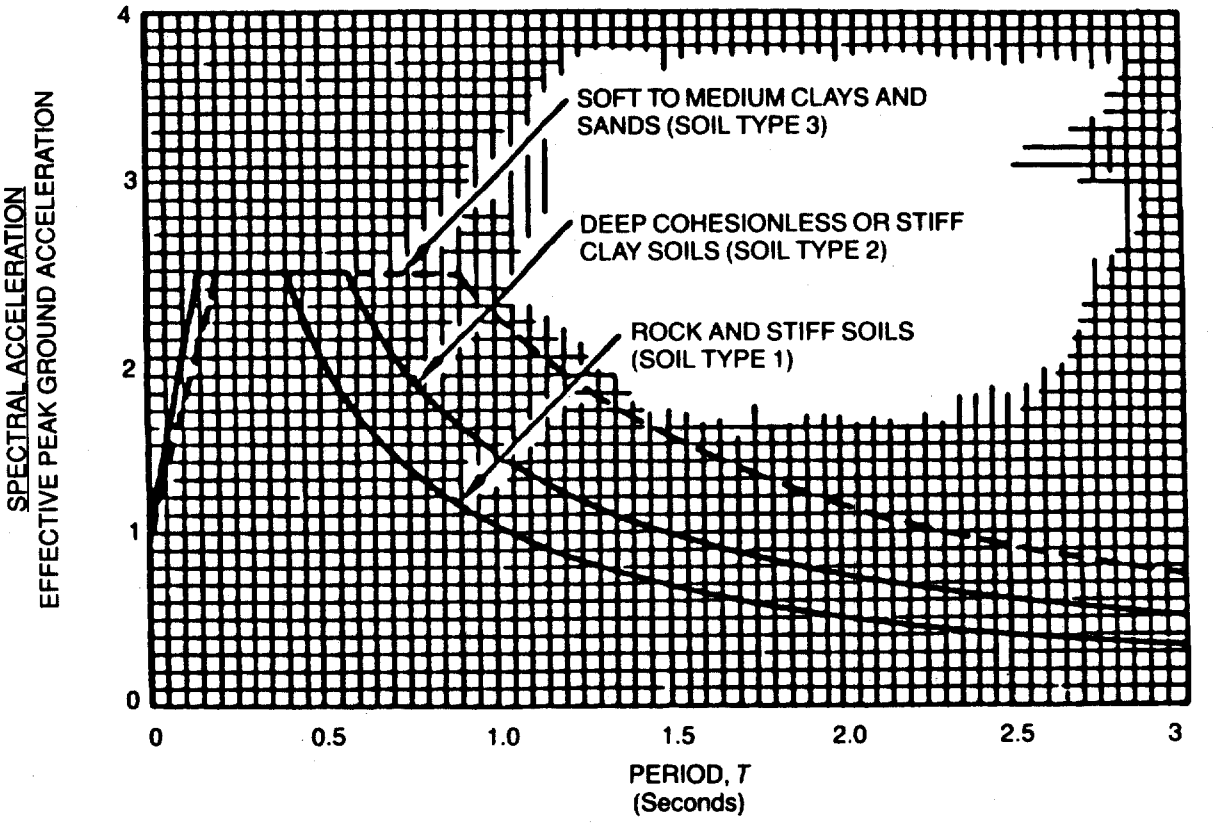


FIGURE 16-3—NORMALIZED RESPONSE SPECTRA SHAPES

Appendix C - Chapter 6a

Discontinuities in Strength of Vertical Resisting System

(Excerpts From Reference 6)

NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1994

Federal Emergency Management Agency and Building Seismic Safety Council

Section 2.2.5.4.2 Provisions and Commentary

Provisions

PLAN OR VERTICAL IRREGULARITIES

The design shall consider the potential adverse effects when the ratio of the strength provided in any story to the strength required is significantly less than that ratio for the story immediately above and the strength shall be adjusted to compensate for this effect.

Commentary

DISCONTINUITIES IN STRENGTH OF VERTICAL RESISTING SYSTEMS

This Section requires consideration of discontinuities in strength. It is not generally recognized that large discontinuities in story strength can cause adverse response effects in a building. Usual practice is to determine what size, length, or strength of resisting elements is required; if more than the required strength is provided, so much the better. Unfortunately, the extra strength in a story, if significantly different than that in adjacent stories, can produce responses that vary greatly from those calculated by using the procedures in Section 2.3 or 2.4 due to the concentration of inelastic deformations in a weak story.

The early developers of the *NEHRP Provisions* considered the following approach to this problem:

1. Compute the ratio of shear capacity to the design shear for each story. Denote this ratio for story n by r_n.

2. Compute r, the average of r_n over all stories.

3. If for any story r_n is less than $\frac{2}{3r}$, modify R and C_d for the building as given by Table 2.2.2 to $\tilde{R}$ and $\tilde{C}_d$

   where

$$\tilde{C}_d = \frac{1+(C_d-1)}{2}$$

$$\tilde{R} = \left(\frac{\tilde{C}_d}{C_d}\right) R$$

4. Use $\tilde{R}$ instead of R to recompute the lateral forces, and $\tilde{C}_d$ instead of C_d in computing story drifts. It is believed that further study should be given in to this problem.

Authors Note:

The collapse of intermediate stories in many buildings during the 1995 Hyogo-Ken Nanbu earthquake in Japan seems to indicate that the subject of vertical irregularities in strength and stiffness should be given considerable study.

Chapter

6b

Dynamic Analysis Applications

Roland L. Sharpe

Introduction

The advent of improved computer hardware and structural analysis software has almost made the previously forbidding task of performing dynamic analysis for building design an everyday chore for most structural engineering firms. The difference between static and dynamic analyses is sometimes little more than a few changes of input parameters with some computer programs. The addition of dynamic analysis provisions in the 1988 *Uniform Building Code* (UBC) and later editions reflect this general trend (Ref. 1). Previously, ATC-3-06 (Ref. 22) devoted a chapter to dynamic analysis. DOE-STD-1020 (Ref. 26) requires dynamic analysis for facilities in Performance Categories 3 and 4. Dynamic analysis applications in the context of building code requirements (Refs. 2 and 3) and when dynamic analysis is needed are presented in the following subsections.

- Types of Seismic Analysis Procedures
- Advantages of Dynamic Analysis
- Mathematical Modeling
- Building Analysis Software
- Summary

Types Of Seismic Analysis Procedures

The responses of structures to significant seismic motions are dynamic and inelastic. In recognition of this fact, building codes recommend analytical methods of varying degrees of complexity to obtain equivalent seismic forces for design of building structures.

Current codes note that the vertical component of ground motion may be defined by scaling from the corresponding horizontal acceleration by a factor of two thirds. Alternative factors may be used when substantiated by site-specific data. In seismic Zones 3 and 4 the UBC requires horizontal cantilever components to be designed for a net upward force of 0.5ZWp and prestressed components to be designed using not more than 50% of the dead load for the gravity load, alone or in combination with lateral force effects.

The Northridge, California and Hyogo-Ken Nanbu, Japan earthquakes have produced high values of vertical motion. Thus, for structures within a few kilometers of an active fault, the potential effects of simultaneous occurrence of high amplitude motions in the three orthogonal directions should be considered. Various methods exist for combining such loadings as noted in the following text.

Seismic analysis methods recommended by various building codes and/or design guidelines follow (Refs. 2-7).

Static Lateral Force (SLF) (or Equivalent Lateral Force) Procedure

In this procedure, it is assumed that the dynamic cyclic response to earthquake ground motions of a reasonably regular structure (configuration-wise) can be adequately calculated using *equivalent* static lateral forces. The design base shear is calculated for a given structural system based on the assumption that the structure will undergo several cycles of inelastic deformation during major seismic motion. The force level is related to the type of structural system and its assumed capability to undergo these inelastic deformations and dissipate the seismic energy without collapse. The base shear is then distributed over the height of structure based on an assumed linear mode shape (inverted triangle).

The basic differences between a static lateral force (SLF) analysis and a dynamic modal analysis are in the calculation of the fundamental vibrational period of the structure, the calculation of forces in one plane at a time, and the distribution of the lateral force (base shear) over the height of the structure. For SLF, the period T is generally calculated using a semi-empirical formula based on the type of framing and the construction material used. In modal analysis, the natural vibrational modes are calculated using principles of mechanics. With SLF, the lateral forces are calculated in one direction at a time. Two-dimensional (2D) modal analysis can calculate one horizontal direction at a time. However, three-dimensional (3D) analysis using a 3D model also can be performed.

As noted previously, with SLF, the lateral force is distributed vertically based on an inverted triangular-shaped distribution. With modal analysis, the vertical distribution of lateral forces is based on the natural vibration modes, which are determined from the actual mass and stiffness distributions over the height of the structure.

Dynamic analysis can provide a more accurate representation of the distribution of lateral force in a structure. However, the accuracy of results is dependent on the realism of the mathematical model. Thus the SLF base shear is used as a lower bound for the dynamic analysis base shear.

Dynamic Lateral Force Procedures

Dynamic lateral force procedures differ from static procedures in the following aspects:

- The contributions of higher modes other than the fundamental mode of the building structure can be considered
- 3D models can be used
- The time-dependent response of the structure during the earthquake motion can be considered
- The nonlinear structural behavior during the time-history of the earthquake motion can be explicitly considered.

Several methods are recommended for dynamic analysis by building codes to partially or completely incorporate the above features.

- *Response Spectrum Analysis*

This is the most commonly used elastic dynamic analysis. A response spectrum is a plot of the maximum responses (acceleration, velocity, or displacement) of a series of single-degree-of-freedom oscillators with varying periods of vibration to a time history of earthquake motions. It does not indicate the duration of the earthquake, nor does it indicate the time when the peak response occurs. However, it provides an indication of the predominant period of the input earthquake

motions. Each input earthquake-time-history generates a different response spectrum shape for a specific damping ratio. The spectrum shapes given in the 1994 UBC are a smoothed average of normalized 5%-damped spectra. They were obtained from actual ground motion records grouped by subsurface soil conditions, and generally are applicable for earthquakes in California and the western United States.

In response spectrum analysis, peak modal responses of all significant modes are first calculated, then approximate total structural response is obtained by combining the modal responses in a statistical manner. Methods used are Square Root of the Sum of the Squares (SRSS), Complete Quadratic Combination (CQC), and Absolute Sum (ABS). The SRSS method is accurate for small modal damping ratios of 5% and less, and well-spaced modal periods. The CQC method considers the modal coupling effects with larger damping ratios and close modal periods. However, both the SRSS and CQC methods will produce acceptable accuracy under proper conditions. The ABS method produces the most conservative results, but is generally considered too conservative for most uses.

- *Elastic Time-History Analysis*

This method, which assumes linear elastic structural behavior, calculates the time-dependent dynamic response of the structure using either modal analysis or the direct numerical integration method. This method is generally more complex to implement than response spectrum analysis.

- *Capacity Spectrum Method*

This is a step-by-step approach to approximating the inelastic capacity of the structure by using response spectrum analysis (Ref. 4). It is an intermediate step toward performing a complete nonlinear dynamic time-history analysis.

- *Nonlinear Time-History Analysis*

This method calculates the time-dependent dynamic response of structures considering nonlinear structural behavior. Currently, the analysis is time consuming and complex, and usually does not consider a whole structure in three dimensions. However, it is the most accurate method for analyzing specific nonlinearity of a structure.

Advantages of Dynamic Analysis

It is commonly thought that dynamic analysis is always needed for irregular and/or complex structures. The following text discusses when it is appropriate or necessary to use dynamic analysis and advantages it provides beyond those provided by the static lateral force method.

The static lateral force (SLF) method assumes a structural system with the following characteristics:

- Story drift ratios and masses are reasonably uniform over the height of the structure
- The governing modes of vibration are primarily translational, not torsional.

A structural system with these characteristics has a linear mode shape. The UBC adds a concentrated force at the top of the structure when the building period exceeds 0.7 seconds that accounts for the higher mode effects at the top of the structure.

Table 6b-1 lists five categorical cases for which the use of SLF is permitted. These structural systems are deemed to have seismic responses that have the above characteristics.

When a structure's response differs significantly from the assumed linear mode shape or a certain response needs to be analyzed more accurately, then one of the dynamic methods needs to be used. Building codes advocate the static method as a basis, and scaling of results from dynamic analysis to static method is required because:

- The static method has gained acceptance because regular buildings designed by this approach have, in general, performed acceptably in earthquakes. There is a good correlation between design and results.

Table 6b-1. Application of seismic analysis procedures.

Seismic response	Static lateral force analysis	Dynamic Analysis			
		Response spectrum	Elastic time-history	Capacity spectrum	Nonlinear time-history
1. Zone 1: All structures	Adequate[1]				
2. Zone 2: Occupancy Category IV[2]	Adequate[1]				
3. Regular structure less than 240 feet in height	Adequate[1]				
4. Irregular structure less than 5 stories or 65 feet	Adequate[1]				
5. Flexible structure supported on rigid lower portion	Adequate[1]				
6. Tall structure, 240 feet or more in height		Adequate[1]			
7. Structure with vertical irregularity		Adequate[1]			
8. Structure with plan irregularity		Adequate[1]			
9. Mixed vertical framing system		Adequate[1]			
10. Soil type S_4 site (UBC)		Adequate[1]			Preferable
11. Building separation and pounding effects portion	Approximate for separation	Approximate for separation	Approximate for separation	Approximate for separation	Preferable
12. Seismic-isolated structure	Approximate	Approximate	Approximate	Approximate	Preferable
13. Post-yield analysis				Adequate	Adequate

1. Per UBC, NEHRP Provisions, UCRL 15910 and DOE-STD-1020 (Refs. 1-3, 25,26).
2. Table 16-K, 1994 UBC (Ref. 2).

- It is difficult to incorporate all parameters completely in a dynamic analysis, and the modeling is subject to analysts' interpretation and judgment. Scaling is required to make forces from dynamic analysis results consistent with the static method and to ensure that the dynamic design base shear is sufficiently conservative. Seismic displacements (drift) from dynamic analysis should be viewed as realistic and should not be scaled.

The SLF method or the 2D modal analysis may not be adequate for cases where:

- The building configuration is such that the lateral motions in two directions and the torsional motions are strongly coupled
- The building has irregular mass and stiffness properties
- The building has an irregular distribution of story strengths, which could induce ductility-demand concentrations in some stories.

For such cases, a 3D response spectrum analysis should be employed.

Seismic responses that cannot be obtained accurately using the static method and the applicability of each dynamic analysis procedure are assessed in relation to building code requirements in the following text. Qualitative terms of *adequate, approximate,* and *preferable* are used in the Table 6b-1 summary.

Effects of Higher Modes

The static method may not give sufficiently accurate results for structures for which the effects of higher modes may contribute substantially to the response. The fundamental period of a tall structure is long and generally beyond the range of predominant periods of input earthquake ground motions, and thus, the contribution from higher modes becomes substantial. Dynamic analysis considers the proper mode shapes and contributions from each mode and is the proper method to be used for tall structures. Therefore, building codes require dynamic analysis for structures 240 feet or more in height. Response spectrum analysis is generally adequate for this purpose (see Table 6b-1).

Effects of Vertical Irregularity

The distribution of lateral force along the height of a structure with stiffness, weight, or geometric vertical irregularity can differ materially from the linear distribution assumed by the static method. Currently, codes specify vertical irregularity in terms of percentage change in adjacent floors and require that dynamic analysis be used when vertical irregularity limitations are exceeded. The response spectrum method is generally adequate for this purpose (see Table 6b-1).

Effects of Plan Irregularity

Structures with plan irregularity, such as torsional irregularity (location of center of mass different from center of resistance), reentrant corners, diaphragm discontinuity, out-of-plane offsets, or nonparallel lateral-force-resisting systems, often will be subject to increased axial loads on corner columns caused by coupling of torsional with translational and vertical motions. This can lead to increased dynamic loads in the structure. Building codes define these various types of irregularity and require the use of dynamic analysis when irregularity exceeds specified limits. The response spectrum method is adequate for this purpose (see Table 6b-1).

Effects of Mixed Structural Systems with Height

When a structure has mixed lateral-load-resisting systems along its height, the irregularity invalidates the assumptions of the static method. Building codes require that structures over five stories or 65 feet in height with this type of system, that are located in Seismic Zones 3 or 4, be analyzed using dynamic analysis. The response spectrum method is adequate for this purpose (see Table 6b-1); however, if concentrations of inelastic behavior in certain stories are possible, a complete evaluation is best provided by nonlinear time-history analysis. For this situation, a

simplified model considering only the localized nonlinearity may be used (see Table 6b-1).

Effects of Soft Foundation Soil

The 1985 Mexico City earthquake caused significant damage in a region with deep soft clay deposits. The main cause of damage was resonance build-up between the structures and the long-duration, low-intensity ground motion. For structures located at a soft soil site, the following factors should be considered:

- Site-specific response spectra should be developed
- Possible inelastic response
- Soil flexibility should be incorporated in the model in lieu of the fixed base condition often used in modeling.

Building codes allow the response spectrum method to be used, but designers should take into account possible lengthening of period caused by inelastic response and occurrence of resonance.

Building Separation and Pounding Effects

When two buildings are not separated by a distance sufficient to avoid contact under seismic motion, pounding will occur. For the design of a new building, the deflection may be estimated using either static or dynamic analysis, and a sufficient gap provided to minimize the problem. It is essential that unreduced spectra and reasonable stiffness values be used. For existing structures, pounding effects can be adequately analyzed only by dynamic analysis. For this purpose, a simplified model can be used with gap elements as its only nonlinear elements. Nonlinear time-history analysis is the proper method to be used (see Table 6b-1). However, to estimate the relative deflection of two adjacent buildings considering only the phasing of the response, a linear (elastic) time-history analysis can be used (see Table 6b-1).

Seismic-Isolated Structures:

The use of seismic isolation in structures has gained acceptance in recent years, although it is still in the infant stage. The 1991 UBC was the first code to include seismic isolation design provisions (as an appendix). This reflects the trend of seismic isolation design in the United States. Paralleling earthquake design regulations in the main text, the UBC includes static response spectrum, and time-history analysis methods for the seismic-isolated structure (see Table 6b-1). The static method is essentially for low-rise structures (less than 4 stories or 65 feet high) located far away (over 15 km) from all active faults, on firm soil (UBC Site Factor S_1 or S_2), with low seismicity. The response spectrum method can be used with more relaxed restrictions and includes UBC type S_3 soil site factor and taller than 65 feet structures. However, the static and response spectrum analysis methods are mostly used only for the preliminary design stage. In the final design, nonlinear time-history analysis is used for the following reasons:

- Most seismic-isolated structures are important structures, and therefore the extra cost for this analysis is generally justified
- All current isolation systems are vendor proprietary. To ensure adequate final design, nonlinear analysis is required
- An isolation system absorbs energy primarily through hysteretic damping; thus, an accurate determination of damping is essential. Nonlinear time-history analysis can more accurately consider hysteretic damping than do simplified formulas using *effective* damping.

Post-yield Analysis

Building codes imply that structures designed in accordance with code requirements should be able to resist a major earthquake without collapse, but probably with some

structural as well as nonstructural damage. Possible weak links and areas of inelastic deformation are not explicitly quantified because code provisions are generally based on reduced forces and allowable stresses for the design of structural members and connections. To quantify the effects of nonlinearity, post-yield analysis is required by the *Tri-Services Manual* (Ref. 4) and the *Natural Earthquake Hazard Reduction Program* (NEHRP) (Ref. 3). Some problems that can be specifically identified by nonlinear analysis are as follows:

- *Effects of Lengthening Structural Periods*

Structures designed according to building codes are expected to deform inelastically during major earthquakes. This causes structural periods to increase. If a structure is founded on firm soil, the period lengthening may decrease seismic forces and limit further damage to the structure, provided that the structure has sufficient ductility. However, if a structure is founded on soft soil, lengthening of the period may induce resonance between the structure and the foundation soil and cause further damage to the structure. A similar phenomenon may occur with some lower buildings founded on firm soil.

- *Torsional Instability Caused by Inelastic Deformation*

Most linear procedures, such as static and response-spectrum methods, are based on the assumption that the displacements of a nonlinear structure are similar to those of the same linear structure. This linearization procedure is useful for design purposes because it simplifies a complex problem. However, it may not be adequate for analyzing asymmetrical yielding on a horizontal plane; a situation that can occur when columns along one side of a structure yield first because of torsion. The yielding shifts the center of rigidity to the unyielded side and increases the torsional moment applied and thus also, the potential for torsional instability as the motions continue.

- *Concentration of Ductility Demand*

If yielding of columns or other lateral-force-resisting elements occurs at a certain story, most of the inelastic energy is absorbed at this story. If the deformation becomes excessive, instability may occur.

An assessment of the impact of the three above problems can be best calculated using either capacity spectrum or nonlinear time-history analysis methods (see Table 6b-1).

Mathematical Modeling

Structural analysis is performed using a mathematical model. Analytical results are meaningful only if the model closely represents the structural behavior of the real structure. An actual building structure can be rather complicated. Depending upon the problem to be solved and the main purpose of the analysis, models with varying degrees of complexity are used. The essential parameters that need to be translated to a mathematical model from an actual structure are discussed hereinafter, followed by various mathematical models used by practicing structural engineers.

A dynamic analysis equation includes four basic terms: the inertia force (mass related), the damping force, the elastic force (stiffness related), and input earthquake motion. To create an accurate model, these four terms must be correctly translated from the physical structure to mathematical quantities. The input earthquake motions are represented by either static force, response-spectrum curves, or acceleration time-history as discussed previously. The terms are further discussed hereafter.

Stiffness of Structural Members and Connections

The induced elastic force is related to structural stiffness and displacements. Therefore, to accurately calculate elastic force, the stiffness and connections of structural members must be accurately modeled. Structural stiffness includes three general categories: vertical members, horizontal members, and their connections. Vertical members include columns, bracing, and walls. Horizontal members include beams, floor slabs, and horizontal truss bracing. Connections include beam-to-column connections, the panel zones (in structural steel) at the intersection of

beams to columns, and the connections of columns to foundations.

In general, cross sections of column/beam/wall sections are used for linear elastic analysis, whether of steel or concrete. Concrete floor slabs are usually assumed to be rigid in plane. Connections are usually assumed to be either a hinged or moment connection. However, if the load path is not continuous from one portion of the structure to the other, the effects of discontinuity need to be considered. The panel zone is usually assumed to have no deformation, although some structural analysis programs can accommodate various degrees of rigidity to approximate the effects of panel zone deformation.

When modeling a concrete structure, the question of whether to use the gross cross section or cracked section is important. Most concrete structures tend to crack with age because of shrinkage, minor differential settlements, temperature movements, and creep. The cracks tend to reduce the lateral stiffness of the structure. Use of gross cross sections for calculating the period of a structure results in a stiffer structure and thus, for most buildings, a higher seismic base shear. However, cracked sections should be used in calculating probable story drifts.

Damping

Damping ratio reflects the energy-dissipating capacity of a structure. Damping is assumed to be viscous or velocity proportional. An equivalent damping ratio is usually used for the analysis. Depending upon the structural material and level of yielding, the damping ratio may vary from 3% to 15%, as shown in Table 6b-2 (Ref. 4).

Table 6b-2. Damping values for structural systems (Ref. 4).

Structural system	Elastic-linear	Post-yield
Structural steel	3%	7%
Reinforced concrete	5%	10%
Masonry shear walls	7%	12%
Wood	10%	15%
Dual systems	(1)	(2)

(1) Use the value of the primary, or more rigid, system. If both systems are participating significantly, a weighted value, proportional to the relative participation of each system, may be used.

(2) The value for the system with the higher damping value may be used.

Mass

The masses used in the calculations include translational mass and rotational mass-moment-of-inertia. The translational mass is commonly lumped at one or a few selected joints at each floor. The rotation mass-moment-of-inertia is calculated about the vertical axis, and is usually approximated assuming a uniformly distributed mass at each floor. This quantity affects the torsional period. In some cases, such as a building with unusually heavy exterior walls, the quantity may be calculated incorporating this special mass distribution. This will somewhat increase the torsional period.

The following describes various mathematical models and their common usage by practicing engineers. The models are classified in terms of geometric complexity. For each geometry, the boundary condition at base and material nonlinearity can be varied, as discussed later.

- *Single-Degree-of-Freedom (SDOF) System*

A SDOF system is the most fundamental representation of a dynamic analysis model. It represents three kinds of real buildings:

  - A single column structure with a relatively large mass at top, such as a water tower, or a single-story shear-wall building, Fig. 6b-1(a)

  - A single-story frame structure with flexible columns and rigid girder system, Fig. 6b-1(b)

  - A simplified structure for analyzing the response of the fundamental mode only, such as the superstructure of a seismic-isolated system, Fig. 6b-1(c).

- *Lumped Multi-Degree-of-Freedom System (MDOF)*

Multistory buildings can be analyzed as MDOF systems with masses lumped at each floor level. The lateral stiffness can be represented either by a single cantilever column as a stick model, Fig. 6b-2(a), or by two columns attached to rigid floor girders as a shear-beam model, Fig. 6b-2(b). These models are usually considered two-dimensional, moving only in a planar direction; thus, torsional effects cannot be directly determined. To properly use this model, the stiffness at each floor needs to be properly evaluated. A stick model can be used to model a multistory shear-wall building. The shear-beam model as shown in Fig. 6b-2(b) can be used to model a multistory frame structure with large spandrel girders.

- *Two-Dimensional (2D) Model*

A 2D model can be used to simulate structures in some detail, including for example, frame column/beam members or shear walls with or without openings (Fig. 6b-3). A 2D model can represent a building assuming rigid floor diaphragms, or a portion of a building with a flexible diaphragm. Torsional effects cannot be included directly in the model. This type of model is often used because of its simplicity.

- *Three-Dimensional (3D) Model*

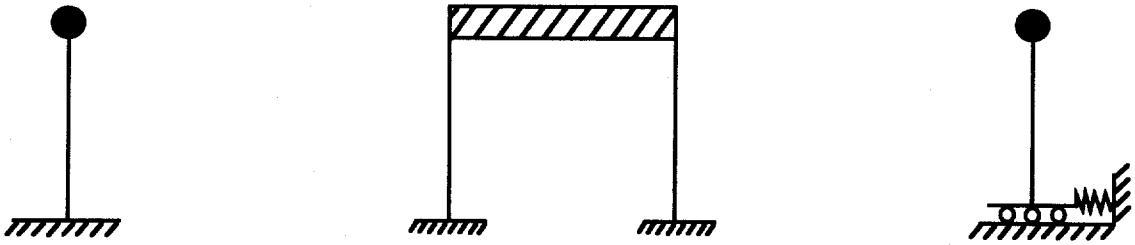
A 3D model can be used to simulate structures in considerable detail, including, for example, frame column/beam members, shear walls with or without openings, and either rigid or flexible floor diaphragms (Fig. 6b-4). The assumption of rigid floor diaphragms, which reduces computations drastically, is common practice. Although, the 3D model with rigid floor diaphragms cannot consider the effects of floor openings properly, torsional effects can be included.

- *Fixed vs Flexible Base*

In most analyses, the building structure model is assumed fixed at the base, without considering the effects of foundation flexibility. When the foundation soils are soft, such as UBC (Ref. 2) Type S_4 , soil flexibility may have significant effects on the structural response (see Chapter 6a and the *Commentary* in ATC-3-06, Ref. 22). Soil springs are often used to model foundation flexibility (Fig. 6b-5).

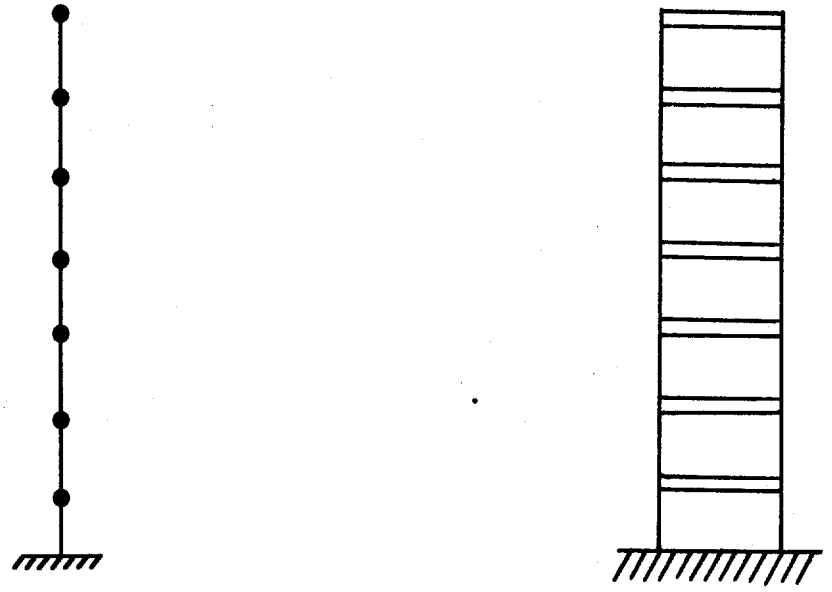
- *Linear vs Nonlinear Model*

Generally the building or structure model is assumed to be linear elastic. However, for



(a) (b) (c)

Fig. 6b-1. Single-degree-of-freedom system (SDOF).

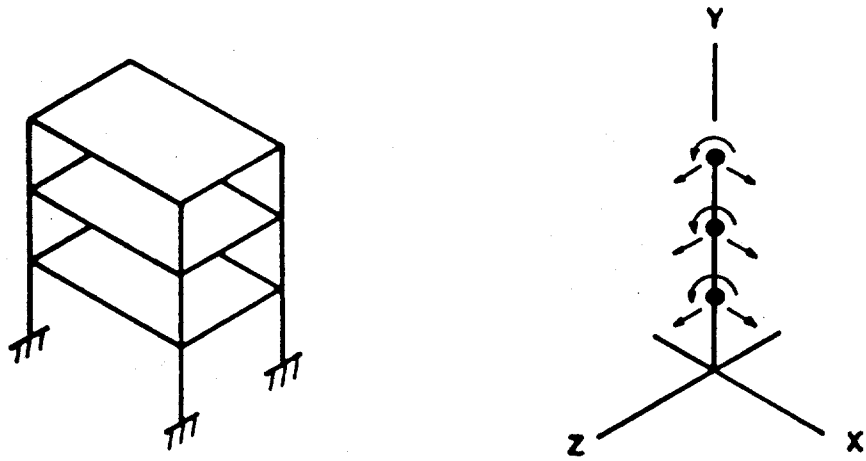


(a) Stick Model (b) Shear-Beam Model

Fig. 6b-2. Lumped multi-degree-of-freedom system (MDOF).

Planar structure **Mathematical model**

Fig. 6b-3. Two-dimensional (2D) model.



3-D structure **Mathematical model**

Fig. 6b-4. Three-dimensional (3D) model.

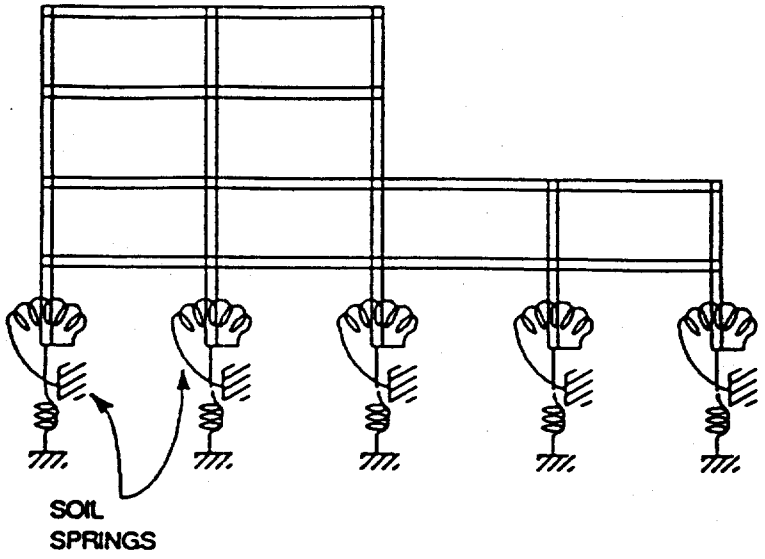


Fig. 6b-5. Flexible base model.

certain cases such as a seismic-isolated structure or a post-yield evaluation, nonlinear analysis becomes a necessity. Nonlinear analysis differs from linear analysis because of the varying force-deflection relationships of structural members or assemblies.

Two of the simplest models, elastoplastic and shear-beam, are shown in Fig. 6b-6.

Building Analysis Software

There are many structural analysis software programs. Some are for mainframe computers only, but most are available for personal computers (PCs). Some are tailored for building analysis only; some for bridges only; and some general programs can be used for various types of structures. The following discussion covers some PC software being used for building analysis, linear or nonlinear. The linear programs are described first, followed by five nonlinear programs. Table 6b-3 summarizes the major features of the programs.

ETABS (Extended Three-Dimensional Analysis of Building Systems)

This program is specially tailored for the elastic analysis of multistory buildings. It was originally developed at the University of California, Berkeley (Ref. 8), and now is a proprietary program of Computer & Structures, Inc. (CSI) of Berkeley, California (Ref. 9). The building is idealized as a 3D assemblage of vertical frames and shear-wall systems interconnected by horizontal rigid floor diaphragms. ETABS is designed to perform linear elastic analysis for both static and earthquake dynamic loadings. The assumption of rigid floor diaphragms makes the program very efficient and convenient for analysis of buildings.

COMBAT (Comprehensive Building Analysis Tool)

This program is a public-domain program for the linear analysis of buildings subject to seismic and wind excitations, and is distributed

(a) Elastoplastic

(b) Shear-beam model

Fig. 6b-6. Nonlinearity – two simplest models.

by the National Information Services for Earthquake Engineering (NISEE) (Ref. 10). The most noteworthy analytical features of the program are:

- Ability to accurately model three-dimensional building behavior
- Capability for modeling various floor systems such as flexible, perforated, or rigid diaphragms
- Capability to allow accurate modeling of shear walls with openings via a substructure technique
- Explicit modeling of foundation flexibility.

The program includes a number of pre- and postprocessors allowing enhanced user control over input and output. Element-specific postprocessors are included, and in-structure floor response spectra may be generated.

SAP90 (Structural Analysis Program for 1990's)

SAP90 is a general-purpose finite-element structural analysis program for static and dynamic response of linear 3D systems. The program is written to analyze structures that are idealized by combinations of structural element types. It was originally developed by Professor E.L. Wilson at the University of California, Berkeley (Ref. 11), and is now a proprietary program of CSI of Berkeley (Ref. 12).

STAAD-III (Structural Analysis and Design/Drafting)

This is a general finite-element structural analysis program with postprocessing for design and drafting. It is for linear elastic analysis of structures subjected to static and dynamic loadings, and is a proprietary program owned by Research Engineers, Inc. of Orange, California (Ref. 13).

Table 6b-3. Programs for Structural Analysis.

Program	Model	Linear Elastic	Nonlinear	Loadings	General Comments
ETABS	3D	Yes	Yes	Static and dynamic seismic	Rigid or flexible diaphragms
COMBAT	3D	Yes	Yes	"	Rigid or flexible diaphragms Can model shear walls with openings, foundation flexibility. Generate floor response spectra.
SAP90	3D finite element	Yes	—	"	
STAAD-III	Finite element	Yes	—	"	Post-processor for design and drafting
GTSTRUDL	Finite element	Yes	Yes (static)	"	Analysis and design
RISA-2D	2D finite element	Yes	—	"	"
RISA-3D	3D finite element	Yes	—	"	"
IMAGES-3D	3D finite element	Yes	—	"	Graphic options for micro-CAD
ANSR-II	3D finite element	Yes	Yes	"	
DRAIN-2DX	2D plane structure	Yes	Yes	"	
3D-BASIS	3D	Yes	Yes	"	Base-isolated structures
SLAM-2	3D	Yes	Yes	Pounding between buildings	
ADINA	3D finite element	Yes	Yes	Static and dynamic seismic	

GTSTRUDL

This is a general finite-element program for structural analysis and design. Analysis includes both linear and nonlinear static, and linear dynamic. It is a proprietary program of Georgia Institute of Technology, Atlanta, Georgia (Ref. 14).

RISA-2D (Rapid Interactive Structural Analysis - 2 Dimensional)

This program is a general-purpose, finite-element-based 2D analysis and design tool. RISA-2D can perform linear elastic analyses for both static and dynamic (seismic) loadings. A companion program, RSPECTRA, may be used to calculate and plot the response spectra used by RISA-2D. RISA-2D and RSPECTRA were developed by and are owned by RISA Technologies, Lake Forest, California (Ref. 23). New 3D versions of these programs are now available (see Table 6b-3).

IMAGES-3D

This is a 3D finite-element analysis program for the PC that is fully interactive. It can perform static, response spectra, and time-history linear elastic analyses. It has graphic options translators for micro-CAD software programs. The graphic options include animation of deformed shapes. Images-3D is a proprietary program owned by Celestial Software, Inc., Berkeley, California (Ref. 24).

ANSR-II (Analysis of Nonlinear Structural Response)

Nonlinear analysis programs are not often used by building design engineers. However, nonlinear analysis is needed for special projects such as a seismic-isolated structure, ultimate capacity of a structure, or pounding analysis. Some programs are available for PC users.

ANSR-II is a general-purpose, 3D, finite-element program for static and dynamic analyses of nonlinear structures. It was originally developed at the University of California, Berkeley (Ref. 15), and is a public-domain software, available from NISEE (Ref. 18). A version modified by Dynamic Isolator Systems, Inc. (DIS) is specially tailored for analysis of seismic-isolated systems (Ref. 16).

DRAIN-2DX (Dynamic Response Analysis of Inelastic 2-Dimensional Structure)

DRAIN-2DX is a general-purpose computer program for static and dynamic analysis of inelastic plane structures. It was developed at the University of California, Berkeley (Ref. 17), and is public-domain software available through NISEE (Ref. 18).

3D-Basis (3-Dimensional Base Isolation Structure)

3D-Basis is a nonlinear dynamic analysis program for base-isolated structures. It was developed at the State University of New York at Buffalo, and is public-domain software available through NISEE (Ref. 19).

SLAM-2 (The analysis of structural pounding)

SLAM-2 is a program for analyzing potential pounding between two flexible 3D multistory buildings. It was developed at the University of California, Berkeley (Ref. 20), and is a public-domain program available through NISEE (Ref. 18).

ADINA

ADINA is a general-purpose, finite-element program for linear and nonlinear analyses of 3D structures. It is a proprietary program of ADINA R&D, Inc., Watertown, Massachusetts (Ref. 21).

Summary

The SEAOC Seismology Committee, recognizing the affordability of both computer hardware and structural analysis software, developed provisions for dynamic analyses. UBC incorporated these procedures in the 1988 edition (Ref. 1) and editions thereafter. The *Tri-Services Manual* describes in detail the procedures for dynamic and post-yield analyses in the 1986 Edition (Ref. 4) and the *NEHRP Handbook* includes dynamic analyses (Ref. 3). UCRL-15910 (Ref. 25), now DOE-STD-1020 (Ref. 26), requires dynamic analysis procedures

for Performance Category 3 and 4 facilities including their structures, systems, and components.

The analysis procedures are numbered in the order of increasing complexity:

1. Static lateral load procedures (elastic)
2. Static lateral load (nonlinear)
3. Dynamic lateral load procedures
   a. Response spectrum analysis
   b. Elastic time-history analysis
   c. Capacity spectrum method
   d. Nonlinear time-history analysis.

Building codes advocate the static method as a basis, and scaling of results from dynamic analysis to static method is required for two reasons:

- The static method has gained acceptance because regular buildings properly designed by this approach have, in general, performed acceptably in earthquakes
- It is difficult to incorporate all parameters completely in a dynamic analysis, and the modeling is subject to analysts' interpretation and judgment. Scaling is required to make dynamic analysis force results consistent with the static method and to ensure that the design base shear is sufficiently conservative. The SEAOC *Commentary* for Section 1F.5.c. (Ref. 7) presents a good explanation of the scaling process.

If an elastic time-history is to be performed, both static and response spectrum methods should be performed to validate and compare analytical results. Similarly, if a nonlinear analysis is to be performed, both static and linear analyses should be performed. For the analysis of any structure, it is recommended that a simple analysis be made first and the results interpreted, and then a decision made as to whether a more complex analysis is necessary. Only by following this careful process can an engineer interpret the results meaningfully.

References

1. *Uniform Building Code,* International Congress of Building Officials (ICBO), Whittier, California, 1988.
2. *Uniform Building Code,* International Congress of Building Officials (ICBO), Whittier, California, 1994.
3. *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* , FEMA 178, Building Seismic Safety Council (BSSC), Washington, D.C., 1992.
4. *Seismic Design Guidelines for Essential Buildings (SDG),* TM5-809-10-1, Department of the Army, the Navy, and the Air Force (Tri-Services), February 1986.
5. *Seismic Design Guidelines for Upgrading Existing Buildings (SDGU)* , TM 5-809-10-2, (Tri-Services), Final Manuscript, December 5, 1986.
6. *Seismic Design for Buildings (SDM),* TM 5-809-10, (Tri-Services), October 1992.
7. *Recommended Lateral Force Requirements and Commentary,* Structural Engineers Association of California (SEAOC), Sacramento, California, 1990.
8. *ETABS, Three-Dimensional Analysis of Building Systems (Extended Version),* E. L. Wilson, J. P. Hollings, H. H. Dovey, EERC, Report No. 75-13, University of California, Berkeley, California, April 1975.
9. *ETABS, Three-Dimensional Analysis of Building Systems,* Computers & Structures, Inc., (CSI), Berkeley, California, Revised February, 1989.
10. *COMBAT-Comprehensive Building Analysis Tool,* Computech Engineering Services, Inc., Berkeley, California; Modified by: Zeus Engineering, Inc., Falls Church, Virginia; Distributed by National Information Services for Earthquake

Engineering (NISEE), Earthquake Engineering Research Center, University of California, Berkeley, California.

11. *SAP - A General Structure Analysis Program,* E. L. Wilson, Structural Engineering Laboratory Report No. UC SESM 70-20, University of California, Berkeley, California, September 1970.

12. *SAP90, A Series of Computer Programs for the Static and Dynamic Finite Element Analysis of Structures,* Computers & Structures, Inc. (CSI), Berkeley, California, Revised July, 1989.

13. *STAAD-II, Structural Analysis, Design and Drafting Software,* Research Engineers, Inc., Orange, California.

14. *GTSTUDL,* GITCES System Laboratory, Georgia Institute of Technology (GIT), Atlanta, Georgia.

15. *ANSR-II, Analysis of Nonlinear Structural Response,* D. P. Mondkar, G. H. Powell, Earthquake Engineering Research Center, Report No. UBC/EERC 79-17, University of California, Berkeley, California, July 1979.

16. *Dynamic Isolation System (DIS),ANSR-II,* Berkeley, California.

17. *Drain-2DX, USER GUIDE,* R. Allahabadi G.H. Powell, Earthquake Engineering Research Center, Report No. UBC/EERC-88/06, University of California, Berkeley, California, March 1988.

18. *Computer Software for Earthquake Engineering,* Distributed by National Information Service for Earthquake Engineering (NISEE), Earthquake Engineering Research Center, University of California, Berkeley, California. Funded by National Science Foundation, June 1992.

19. *3D-Basis, Nonlinear Dynamic Analysis of 3-Dimensional Base Isolated Structure,* S. Nagarajaiah, A. M. Reinhorn, M. Constantinou, National Center for Earthquake Engineering Research Center (NCEER), NCEER-91-005, Buffalo, New York, February 1991.

20. *SLAM-2, USER GUIDE-A Computer Program for the Analysis of Structural Pounding* , B. F. Maison. K. Kasai, NISEE, Earthquake Engineering Research Center, University of California, Berkeley, California, August 1990.

21. ADINA, ADINA R&D, Inc., Watertown, Massachusetts.

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

23. *Risa-2D, Rapid Interactive Structural Analysis - 2 Dimensional,* Version 3.02 and *3D Rspectra, Response Spectra Calculations, Editing and Plotting, Version 1.00,* Risa Technologies, Lake Forest, California.

24. *Images-3D, Finite Element Analysis Program, Version 2.0 with Graphic Options,* Celestial Software, Inc., Berkeley, California.

25. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities,* UCRL-15910, Rev. 2, U.S. Department of Energy, Washington, D.C., June 1990.

26. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities,* DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., 1994.

Chapter

6c

Soil-Structure Interaction

James J. Johnson

Introduction

The fact that a structure rests on a soil medium affects the seismic response of the structure in three ways:

1. It affects the ground motion, as illustrated in the ground response spectra shown in Fig. 6c-1 taken from the 1994 *Uniform Building Code* (UBC)

2. It may affect the foundation stiffness and damping of the structure resting on it

3. The presence of a structure on or in soil can also affect the overall response of the soil-structure system.

The latter two effects are lumped together into what is termed *soil-structure interaction* (SSI). The first effect can be accounted for without consideration of a structure being present, it merely represents the ways that seismic waves propagate vertically from bedrock through various stiffnesses of soils.

The second effect can be viewed as changing the vibrational properties of the structure by providing a flexible medium for the foundation (like having a spring between the foundation and the point where the input motion is applied). Conventional structures designed or analyzed by the UBC provisions do not account for this effect because they are assumed to have a fixed base. Under this assumption foundations cannot rock or translate; consequently, the vibrational properties of the structure are not affected by the stiffness of the soil underneath. However, when massive, stiff structures are founded on relatively soft soil, as in the case of some nuclear power plants and industrial structures, the fixed-base assumption is not realistic and SSI should be accounted for.

The third effect occurs when a massive, stiff foundation interferes with the natural passage of seismic waves through the soil. An example is the case of a very stiff embedded foundation, which acts as a rigid inclusion in a flexible soil medium with the result of reducing the foundation motion over that of a surface foundation. This effect is usually referred to as *kinematic interaction*.

In general, the motion recorded in the free-field (away from all structures) differs from that measured on the foundation of a structure due to SSI. One exception is a structure founded on rock or very stiff soil and subjected to vertically propagating waves only. Fig. 6c-2 compares response spectra for adjacent buildings, one with and one without a basement. The one with a basement has a lower amplitude motion and a longer period as a result of SSI.

The behavior of a structure depends on the characteristics of the ground motion, the surrounding soil, and the structure itself. In the

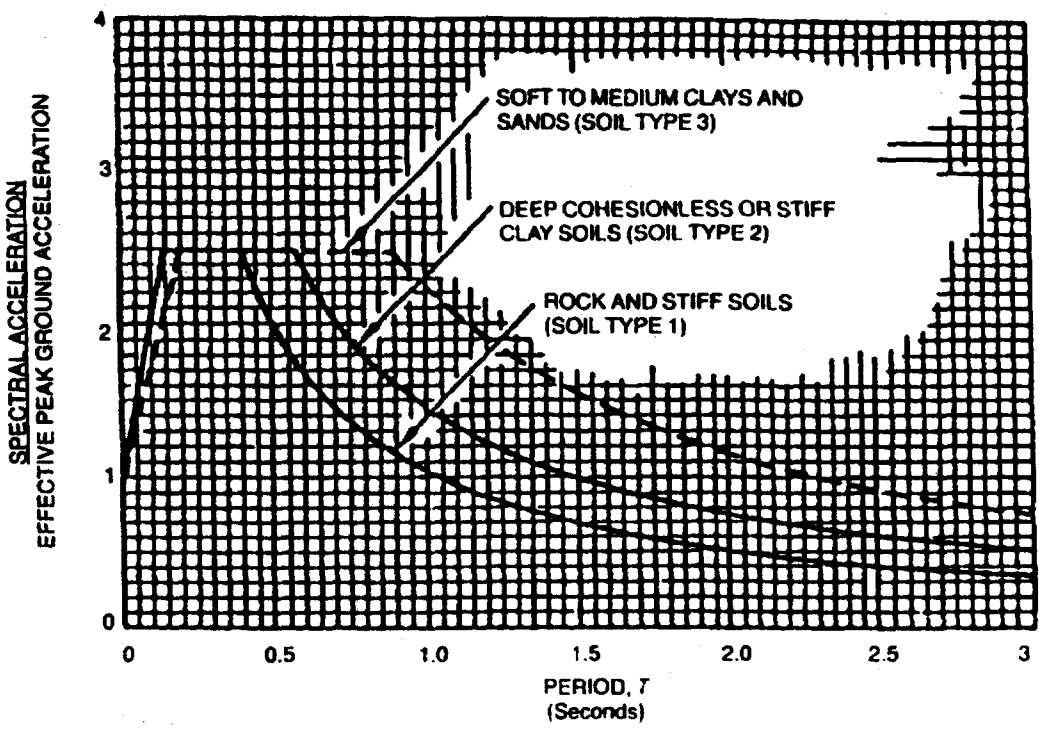


Fig. 6c-1. Effect of soil on normalized ground response spectra shapes.

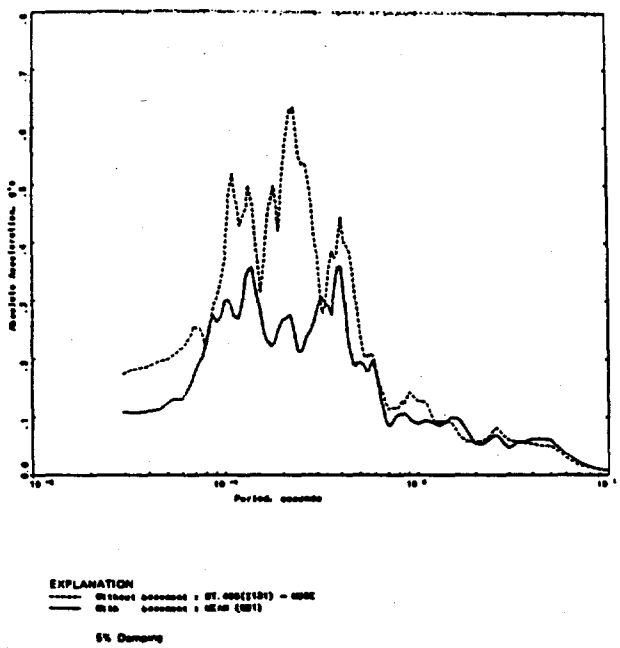


Fig. 6c-2. Comparison of response spectra, adjacent buildings with and without basement, San Fernando, California earthquake.

text that follows, free-field ground motion and SSI are discussed conceptually in a more technical sense. Included are a definition of the elements of SSI, data needs and acquisition, and importance of SSI depending on soil, foundation, structure characteristics, and other aspects.

Free-Field Ground Motion

Free-field ground motion denotes the motion that would occur in soil or rock in the absence of the structure or any excavation. This motion is described by accelerations, velocities and displacements that vary as a function of time and of location in the soil or rock. Motion at any point is typically described by three orthogonal translational components, two horizontal and one vertical. Rotations at a point are generally ignored as being inconsequential to the structure and unquantifiable from existing data.

Motion on the soil-or rock-free surface differs from motion at depth. Typically, motions at depth in soil or rock have lower amplitudes than those on the surface. The frequency content of the motion also varies with depth and as a function of the soil or rock material properties. Motion of different points on a horizontal plane, such as the soil or rock surface, also may differ as a function of time due to the earthquake wave propagation mechanism or due to inhomogeneities, which introduce slight variations from point-to-point. The variation of free-field motion with depth in the soil is a first order effect with respect to SSI; whereas, the variation of free-field motion on a horizontal plane, such as the free surface, is considered second order. Accounting for these variations (where appropriate) tends to produce lower estimates of structural response.

Elements of SSI

The SSI phenomenon can be thought of as two elements; *kinematic* and *inertial interaction*.

Kinematic interaction is the integrating effect that occurs as portions of the structure and foundation that interface with the soil or rock are subjected to differing free-field ground motion. Variations in translational free-field ground motion result in net translations and accompanying rotations due to the integration or averaging process. *Kinematic interaction* is typically treated separately from *inertial interaction* from a conceptual standpoint and frequently from a calculational standpoint. The result of accounting for *kinematic interaction* is to generate an effective input motion, which is denoted *foundation input motion.*

Inertial interaction denotes the phenomenon of dynamic behavior of the coupled soil-structure system. The base excitation is defined by the *foundation input motion* accounting for *kinematic interaction*. The behavior of the foundation on the soil is modeled by foundation impedances (generalized soil springs) that describe the force-displacement and radiation damping characteristics of the soil. The structure is modeled by lumped mass and distributed stiffness models representing its dynamic response characteristics. *Radiation damping* is the dissipation of vibrational energy into the surrounding soil, much like the spreading of water waves away from the point at which an object is dropped into or excited in the water.

SSI Effects on the Response of a Structure

SSI can have an important effect on some characteristics of ground motion, foundation and structural response. Consideration of SSI can decrease calculated responses as discussed below.

The type of structure and its foundation has a significant impact on the importance of SSI. Usually, SSI effects can be ignored for conventional structures without significant embedment. Even for embedded structures, ignoring SSI is usually conservative. However, if large seismic loads are predicted for a large, expensive structure, or, a major building containing expensive equipment, considering embedment effects in an SSI analysis can often avoid prediction of overly conservative loads or underestimating structural displacements. If a major structure contains hazardous materials and/or important life safety-related equipment,

one should at least investigate the effects that SSI would have on the frequency of the structural system and its consequent effect on spectral accelerations. If the frequency change results in increased loads by shifting to a higher point on the response spectrum, SSI should be considered. In this case, it would be conservative to simply account for the frequency shift and ignore the effect of increased damping; however, it may be overly conservative to do so.

SSI is most important for massive, stiff structures with mat foundations or with foundation systems significantly stiffened by the structure's load-resisting system. An example of the latter case would be a structure with substantial shear walls on spread or strip footings. For structures with embedded or partially embedded foundations, SSI is especially important. Structures on soft soil are affected more than those on rock. However, structures of large plan dimensions founded on rock can be influenced significantly by *kinematic interaction*, i.e., filtering of the ground motion into the structure.

SSI is an important phenomenon to take into account when the ground motion contains significant high-frequency content, i.e., greater than 10Hz. (periods less than 0.1 seconds) because it will significantly reduce the calculated response at higher frequencies.

Whether SSI analysis will show an increase or decrease the maximum response of a structure depends on the response quantity in question, the characteristics of the ground motion, and the soil-structure system. SSI may increase, decrease, or have no effect on peak response depending on the frequency characteristics of the ground motion and the relationship between the fixed-base frequencies of the structure (ignoring SSI) and the frequencies of the interacting soil system. The soil-structure system frequencies will be less than those for the fixed-base structure because of the flexibility of the soil under the foundation. If this reduction moves the frequencies into a range of amplification of the ground motion, response increases. The amount of increase is dependent on remaining SSI parameters, such as *kinematic interaction*, which tends to reduce response, or an increase in energy dissipation due to *radiation damping*, which also tends to reduce response.

These observations apply to peak structure response parameters; loads, floor and structure *displacements, and accelerations*. For structures that house essential or life safety-related equipment, in-structure response spectra are frequently required. In this case, the frequency range of interest for equipment qualification and design must be known before one can assess whether considering SSI will show increase or decrease in the calculated response. An SSI analysis may reduce force levels in the structure, but increase displacements. Both effects need consideration.

SSI Analyses

The analysis of SSI depends on specification of the free-field motion and the analyst's modeling idealization of the soil and structure. Modeling the soil entails estimating its configuration and material properties. Modeling the structural system includes the structure and its foundation. Fig. 6c-3 illustrates the elements of the analysis necessary to calculate seismic response including SSI effects.

Specifying of the free-field ground motion

Specifying the free-field ground motion at a site is the most uncertain step in the process of SSI analysis. It entails specifying the point at which the motion is applied, the amplitude and or frequency characteristics of the motion (referred to as the control motion and typically defined as ground response spectra, power spectral density functions, and/or time histories), the spatial variation of the motion, and, in some cases, duration, magnitude, and other earthquake characteristics. In terms of SSI, the variation of motion over the depth and width of the foundation is relevant. For surface foundations, the variation of motion on the surface of the soil is important. For embedded foundations, the variation of motion over both the width and depth of the foundation should be determined.

The point at which the analyst applies the input earthquake motion should always be

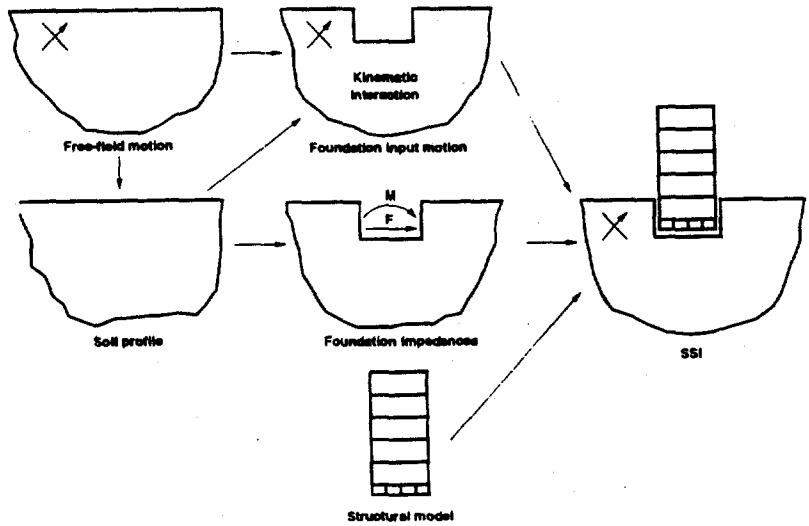


Fig. 6c-3. Schematic representation of the elements of soil-structure interaction.

assumed on a free surface of soil or rock. Ignoring the variation of ground motion with depth of soil or in a horizontal plane is almost always conservative. The decision to ignore it should be based on economic considerations if accurate predictions are not expected or required.

Modeling the soil-structure system

Modeling the soil-structure system entails modeling the soil profile, nonlinear material behavior of soil, the foundation, and the structure.

Soil modeling

For soil sites, describing the soil configuration (layering or stratigraphy) and the dynamic material properties of soil layers is necessary. Modeling the soil can be visualized in two stages: (1) determining the soil profile and associated material properties, and (2) defining the dynamic material behavior of the soil as a function of the induced strains from the earthquake and soil-structure response. Currently, soil is modeled as a linear or equivalent linear viscoelastic material in SSI analyses of practical application. Three material parameters define the viscoelastic model; two elastic constants, such as shear modulus and Poisson's ratio, and material damping. The equivalent linear method approximates the nonlinear stress-strain relation with a secant modulus and material damping values selected to be compatible with the average shear strain induced during the motion using an iterative procedure. Requirements for the model are low strain shear modulus and Poisson's ratio (or other equivalent parameter), material damping values, and their variation with strain.

The soil model is developed from correlating the field exploration and laboratory test results. Field exploration relies heavily on soil boring programs, which provide information on the spatial distribution of soil (horizontally and with depth) and produce samples for laboratory analysis. In addition, some dynamic properties are measured in situ; for example, shear wave velocity, which leads to the value of shear modulus at low strains.

Laboratory tests are principally used to measure dynamic soil properties and their

variation with strain, soil shear modulus, and material damping.

Correlating field and laboratory data entails comparing low-strain values and adjusting the laboratory-determined variations to the in situ measured values.

Equivalent linear soil properties are calculated by performing a site response analysis using the above-described data and an appropriate computer program.

The most important consideration in modeling the soil, in addition to those aspects discussed above, is the recognition of the inherent uncertainty in defining soil behavior when subjected to dynamic ground motion. There is uncertainty in each aspect of defining and modeling the site soil conditions for SSI analysis purposes. To account for this uncertainty, a range of soil conditions should be considered in any SSI analysis. Effective soil stiffness should be assumed to vary from 67% to 150% of the best estimate values as a minimum range.

Foundation modeling

Three aspects of modeling the foundation are most important; stiffness, embedment, and geometry. Assessing and modeling the effective stiffness of a foundation will directly determine how important the effects of SSI will be. Stiff foundation systems supporting massive structures are those situations where the effect of SSI can be most important. Most foundations of major building structures cannot be considered rigid by themselves. Structural load-resisting systems, however, such as shear walls, significantly stiffen their foundations. Hence, in many instances, the effective stiffness of the foundation is very high and it may be assumed to behave rigidly. When such rigid behavior is relatively certain, SSI can be important. If this is not the case, local flexibility of footing/soil interfaces may require modeling because it can affect frequencies of the overall soil-structure system.

Less important for surface-mounted foundations are other SSI effects, such as *kinematic interaction* and *radiation damping*, which have minimal effect on surface-structure response.

Foundation embedment is important because it has a first-order effect on SSI. *Kinematic interaction* effects are significant for embedded foundations. Also, foundation impedances (modeled by soil springs) are significantly affected by embedment.

Detailed modeling of the foundation geometry is thought to have a second order effect on structural response.

The amount of effort put into structure model refinement should be compatible with desired end results of the analysis. A too detailed model may be a waste of resources if the detail is not required.

Parameters to be evaluated for the structure model are damping, idealization of stiffness, and treatment of boundary conditions.

SSI analysis

Combining all of the previous steps leads to an SSI analysis of the structural system of interest.

When evaluating the SSI model and the analytical results, one should evaluate intermediate and end results including: fixed-base versus soil-structure system frequencies, the amount of soil softening due to earthquake excitation compared to the low strain values, the effect of soil property variations on dynamic model parameters such as system frequency, important response parameters, and structural damping values used. For partially embedded structures, response at grade level floors should, as a general rule, be less than the free-field ground surface motion.

Obviously, the use of SSI is a rather expensive and time-consuming process. In cases where it lowers predicted response, it should be justified both by construction cost savings and by the certainty that it will not unduly reduce conservatism needed for safety.

Table 6c-1 is provided to indicate the sources of professional expertise necessary to supply the parameters required for SSI analysis.

Table 6c-1. Consultants who provide services or parameters required for SSI analysis.

Consultants Services or Parameters	Specified by code or external body	Seismologist/ seismic hazard model	Geotechnical engineer	Civil/structural engineer	Civil/Structural/ geotechnical engineer with SSI experience
Free-field ground motion					
• Control or input motion	X	X			
• Variation horizontally and with depth	X				
• Earthquake motion characteristics	X	X			
Soil Models					
• Field tests			X		
• Lab tests			X		
• Correlation			X		
• Equivalent linear values			X		X
Structural system model				X	X
SSI model and analysis					X

Chapter

6d

Seismic Isolation and Passive Energy-Dissipation Systems

Eric Elsesser
Roland L. Sharpe

Introduction

Earthquake ground motions induce dynamic cyclical inertial forces in all elements of a building or structural system. The amplitude of these inertial forces is primarily a function of:

- The intensity, frequency content, and duration of the ground motion
- The characteristics and depth of the foundation materials (soft or medium soils or rock)
- The mass, stiffness, damping, and configuration of the structure.

The first two factors are givens for a particular site, whereas the third can be controlled to some extent by designers.

There have been many attempts to reduce the induced seismic forces in buildings, including making the structure more flexible (less stiff), reducing the mass by using lighter weight materials, providing a partial isolation system between the foundation and superstructure, or adding damping and energy dissipation to the structural system.

This chapter discusses the concepts of seismic isolation and supplemental damping/passive energy dissipation systems including the following topics:

- Use of new concepts to concentrate seismic energy dissipation
- Use of energy dissipation mechanisms to control behavior and modify seismic response
- Use of new mechanisms in conjunction with conventional construction
- Extensive dynamic testing of these systems, and limited earthquake exposure.

Seismic Isolation Systems

What is Isolation?

Seismic isolation is *de-coupling* of the building from the ground, severing the rigid structure-to-ground contact with a mechanical device usually located between the structure and its foundation. The device is designed to dissipate most of the destructive earthquake

energy. Isolation bearings allow large lateral displacements to occur within each isolator, thereby protecting the building's superstructure from the usual harmful seismic distortions.

Why Isolation?

Seismic isolation devices dramatically reduce the seismic impact on buildings and structures. A reduction in seismic forces reduces demands placed on the building structure and assemblies, enhances seismic performance, and allows more economical solutions to be used for basic construction; isolation can be cost-effective.

Isolation designs are possible because of recent advances in analytic capability, the advent of new isolation bearing technology, and the capability to perform dynamic shake-table testing of isolation systems.

Seismic Isolation Mechanisms

The concept of seismic isolation has been the subject of much research since the turn of the century. An early patent for isolation was applied for by a dentist (Ref. 1). The basic idea is to decouple the structure (building) from the ground motion. This can be done as shown in Fig. 6d-1.

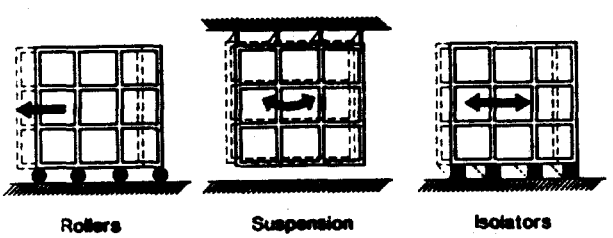


Fig. 6d-1. Seismic isolation concepts.

Of these three concepts, rollers by themselves provide no limit to displacement, and hence, are normally not practical. Neither is overhead suspension. Seismic isolation using elastometric bearings, friction pendulum systems, or sliders is practical. However, the appropriateness and limitations of each system for a particular application must be carefully evaluated.

Decoupling is achieved by significantly altering the fundamental period of vibration and increasing the damping of the isolator/structural system. The response of the building (structure) can be reduced to 1/4 or 1/5 of that experienced by a fixed-base structure. A typical response spectrum, (Fig. 6d-2) showing the lateral load factor plotted vertically and the fundamental period of vibration horizontally, shows that the period shift for the isolated structure from 0.5 to >2.0 greatly reduces the required seismic force (base shear).

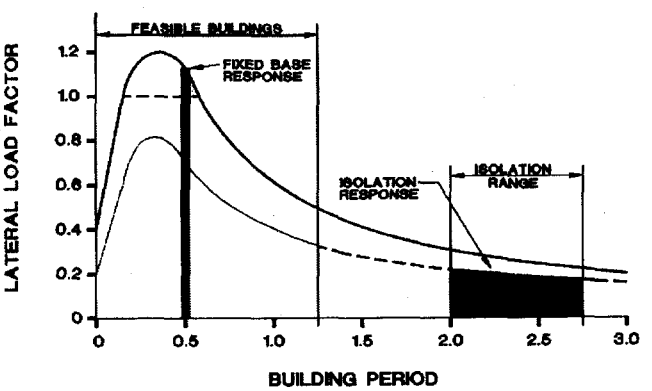


Fig. 6d-2. Period shift for isolated structure.

However, as discussed later under *Isolation Response*, overall displacement at the structure/isolator interface is increased, but the interstory displacement (drift) is less as shown in Fig. 6d-3.

Isolation Bearings

There are three commercially available systems in the United States (Fig. 6d-4).

- Elastomeric with high damping rubber
- Elastomeric with lead core
- Friction pendulum with concave steel plates.

Systems using sliders (sliding surfaces in contact) have been proposed and are being researched.

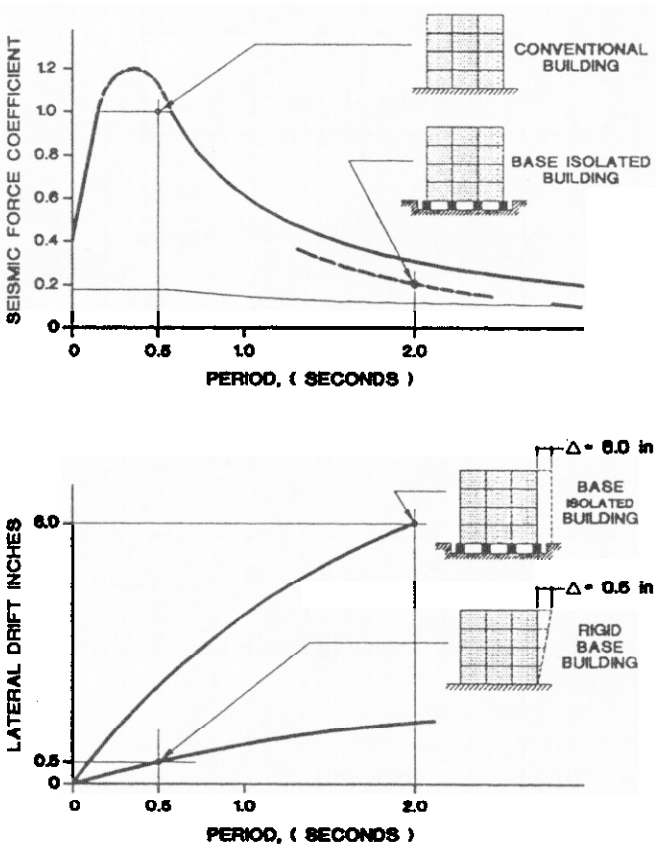


Fig. 6d-3. Comparison of base shear and drift.

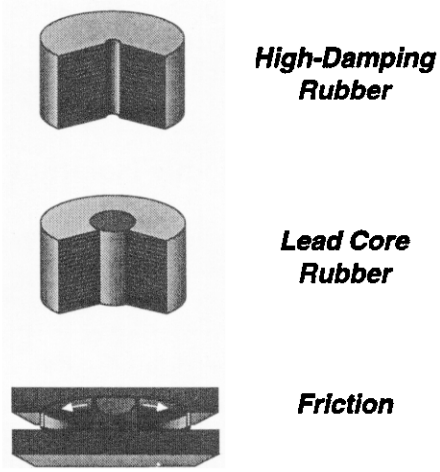


Fig. 6d-4. Seismic isolation systems commercially available in the United States.

System Characteristics

All seismic isolation systems depend on stable hysteretic behavior to provide energy dissipation. Proper selection of materials is essential to provide a high level of damping to reduce the overall systems dynamic response to seismic forces.

Three behavior characteristics are essential:

- Rigid vertically but flexible laterally
- Self centering, system tends to return to center when ground motion ceases
- Durable materials used whose characteristics do not change significantly over time.

Isolation Response

Isolation-bearing response is governed by tuning the bearing by selecting appropriate dimensions, proportions, and materials. Large lateral deformations are anticipated within isolation bearings, and are a function of isolator stiffness, internal damping, and the intensity of the particular earthquake ground motion. The seismic forces transmitted to the building can be controlled by bearing selection. Dynamic characteristics also can be controlled by bearing type and location.

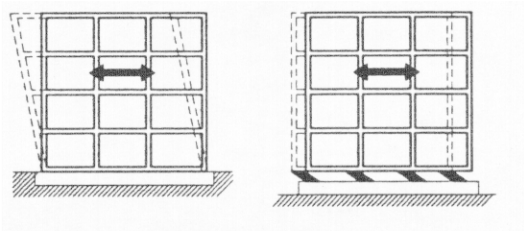


Fig. 6d-5 . Building movement.

In Figs. 6d-5 and 6d-6, the movement of conventionally framed or fixed-base buildings is compared to movement of buildings with seismic isolation. Isolated buildings move as a unit and have very small interstory distortions.

Location of Isolators

Isolation bearings are most effectively located at the lowest point in the superstructure immediately above the foundation. To achieve

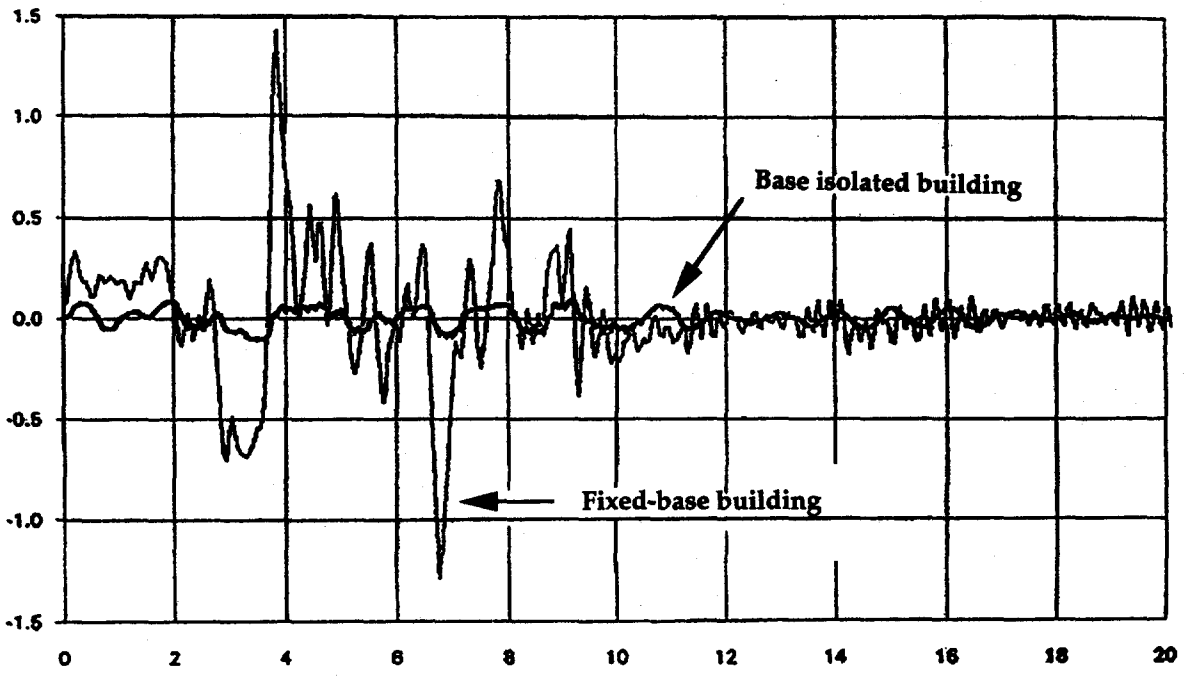


Fig. 6d-6. Lateral movement of the top of a fixed-base building compared with that of a base isolated building dramatically illustrates the reduction of seismic response.

low-cost solutions isolators must be located appropriately as illustrated in Fig. 6d-7.

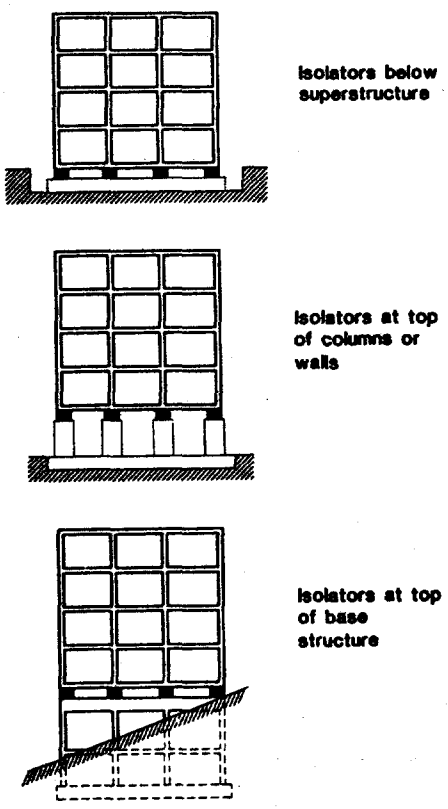


Fig. 6d-7. Locations of isolators.

Selecting Isolation Systems

Isolation bearings must be compatible with the intended and assumed structural response.

Bearings can be specified as either predesigned or vendor designed, as sole source or as multiple-system bids (Ref. 2).

Elastomeric (high-damping and lead core rubber) or sliding systems (friction pendulum and others) are presently available commercially.

Appropriate Use of Seismic Isolation

Seismic isolation is not a panacea for all buildings. However, the concept often is appropriate for use in design of new buildings and for retrofit or seismic upgrading of existing buildings where there is a need to limit damage or prevent disruption of service.

Structures proposed for seismic isolation must have appropriate dynamic properties. The building superstructure must be relatively rigid (with short periods of vibration) to accommodate the longer periods generated by the isolators without dynamic resonance.

The use of seismic isolation may be the optimum solution for upgrading historic buildings, where more conventional strengthening procedures/methods, such as adding steel bracing, poured concrete walls, or pneumatically placed concrete walls, would adversely impact the historic character of the building. Seismic isolation for retrofitting buildings is discussed in detail in Chapter 8c.

New projects housing critical or essential facilities, buildings with expensive and valuable contents, and structures where superior seismic performance is desired are good candidates for seismic isolation. They include:

- Hospitals
- Emergency Operations Centers
- Police and fire stations
- Hi-tech buildings and computer centers
- Communication centers
- Highway bridges and viaducts
- Critical data processing areas or centers
- Research laboratories with motion-sensitive or long-term research experiments
- Important manufacturing systems or processes that require continuing operation
- Critical materials storage.

The nominal extra cost of seismic isolation may be a good investment for buildings or facilities with critical occupancies, such as listed above.

Technical limitations that should be considered when judging the feasibility of seismic isolation involve buildings with a fundamental period of isolation of 1.5 seconds or less, or as much as 10 to 15 stories in height. Isolators, with normal periods of 2.0 to 3.0 seconds, remarkably reduce earthquake-induced loads within buildings. Economics tends to dictate that it is only feasible to use seismic isolation for one- or two-story buildings if they house special occupancies or critical processes.

Critical Issues

If one of the above situations exists and seismic isolation appears to be appropriate, then the concept should be tested against each of the following critical technical issues:

- **Building Resonance**

  The basic fixed-base building period must not induce resonance with the isolation system. It must have a period equal to not more than one-half of that for the isolated structure system. For example, if the isolated structure system will have a period of 2.4 seconds, the building period must be less than 1.2 seconds. If this condition is met, isolation is feasible.

- **Ground Resonance**

  If soft soil underlies the building, it may considerably lengthen the period of the building-soil system and make isolation undesirable or unwarranted. The period of the isolated building, which may range from 2.0 to 2.5 seconds, must be at least two times the period of the site, (i.e., a maximum site period of 1.0 to 1.25 seconds) to avoid dynamic resonance with the site. If this condition is met, isolation is feasible.

- **Uplift Forces**

  The load of the building superstructure to be supported by isolators must be proportioned to avoid uplift forces on the isolators, because isolators cannot realistically withstand uplift (tensile) forces. The lateral-load-resisting system of the superstructure must be distributed (spread) at the base to minimize overturning and eliminate uplift.

Either dynamic resonance or structural uplift can make seismic isolation an unwise choice.

Additional Issues

Other issues that must be addressed when evaluating the feasibility of seismic isolation are:

- **Constructability**

  The building framing must be conceived so that the construction sequences will allow installation of isolators. For new buildings, isolators are placed directly on foundations or lower-story construction using an appropriate mounting that will allow future removal and replacement of the isolator.

- **Distribution Diaphragm**

  All isolators in a system must move together in unison. This can be achieved by providing a rigid distribution diaphragm or grid/truss both below and above the isolator plane. A foundation mat or intertie system provides the lower system, while a heavy rigid concrete slab or steel bracing grid suffices above the isolator plane.

- **Superstructure Rigidity**

  Ideally, the superstructure resting on isolation bearings should be a rigid box. Structural systems with rigid bracing or walls which minimize dynamic amplifications are better than flexible systems using moment frames.

- **Accessibility**

  A service space is required to allow for examination of isolators and/or their replacement. A deep crawl space with a level surface on which to move isolators would be appropriate. An isolator may weigh from 1,000 to 3,000 lbs or more.

- **Isolation Gap**

  A gap or space is required around the entire isolated structure to accommodate horizontal movement of the structure/isolators. The gap should be at least 12 to 30 inches, depending on the characteristics of the isolation system used and its stiffness. Utility interfaces with the building must be designed with flexible joints or other means to accommodate the building movement with respect to adjacent soil. Entrances to the building, stairs, and elevators also must be specially designed for these displacements.

- **Fire Rating**

  Isolators should have a fire rating equivalent to that of the building framing above.

- **Durability**

  Isolators fabricated from elastomerics, rubber, or steel disks with rubber are expected to last 50 years or more. The isolator assemblies, if properly fabricated, should have equivalent longevity.

- **Testing**

  Isolators should be tested at three stages:

  1. Prototype testing prior to manufacture to confirm the isolator design

  2. During manufacture to confirm the manufacturing process

  3. Periodically during their life (every 5 to 10 years) to confirm that properties do not change.

  Prototype and quality control tests for seismic isolation systems are required by the 1994 *Uniform Building Code* (Ref. 3) and the 1991 specifications of the *American Association of State Highway and Transportation Officials* (AASHTO) (Ref. 4). *The National Institute of Standards and Technology* (NIST) has issued three draft guideline documents for public review and comment. The guidelines cover *Pre-Qualification, Prototype,* and *Quality Control Testing* for elastomeric and sliding isolation systems, discuss the need for the various tests, and present proposed requirements (Refs. 5, 6, 7).

  Prequalification tests, not currently required by codes, are essential during development of a new system.

- **Maintenance/Replacement**

  All isolators should be inspected periodically. Should the periodic testing or inspection indicate degradation of the materials or properties, replacement is necessary. Provisions must be made during construction to permit jacking the building to allow removal and replacement of isolators.

Analysis Procedures

The use of base isolation for buildings was initiated in the United States in 1984-86 with the design and construction of the Foothill Communities Law and Justice Center in Rancho Cucamonga, California. This application was followed in 1985-89 by the renovation and seismic strengthening of the Salt Lake City and County Building, a 90-year old unreinforced bearing wall structure. Both of these buildings were designed without benefit of formal design codes for base isolation systems.

The Structural Engineers Association of Northern California (SEAONC) established a working group in 1980 to study the use of base isolation and draft design guidelines. A brief document was developed that was used by the *Base Isolation Subcommittee*, a successor committee, formed in early 1985. The SEAONC *Tentative Isolation Requirements* (Ref. 8) published in September 1986 were used as guidance for the isolation design of the Salt Lake City and County Building.

After publication of SEAONC Draft Guidelines, the *Structural Engineers Association of California* (SEAOC) formed a statewide committee on the use of base isolation with the charge to develop a consensus document that could be submitted for adoption to the *International Conference of Building Officials* (ICBO) publisher of the *Uniform Building Code* (UBC). After further study and review, *Earthquake Regulations for Seismic Isolated Structures* was adopted by the ICBO for the 1991 UBC. *The California Office of Statewide Health Planning and Development* (OSHPD) also adopted base isolation regulations for use in design and construction of hospitals. In certain areas, the state requirements are more restrictive than those in the UBC.

UBC and OSHPD Requirements

Table 6d-1 presents a comparison of the major UBC and OSHPD base isolation requirements. The seismic input requirements are nearly identical except for sites within 15km of an active fault, for which the UBC requires consideration of near-field ground-motion phenomena. Both codes require the use of a 1,000-year return period earthquake (10% probability of being exceeded in 100 years) for calculating maximum displacement.

OSHPD does not permit the use of simplified static formulas, which are allowed in the UBC. A response-spectrum or time-history analysis must be made using a 3D model. A response-spectrum analysis may be used only for sites more than 15 km from an active fault and is permitted for nearly elastic systems, i.e., systems where a few elements may go into yield, but the structure remains stable. A more detailed explanation of *nearly elastic* is given in an appendix to the OSHPD regulations (Ref. 9). Time-history analysis for nonlinear systems is mandated by both codes and must be used for all S_4 soil sites (UBC) and all sites less than 15 km from an active fault (OSHPD).

Design forces may be determined from simplified formulas for sites, structures, and isolation systems that meet specified restrictions for the UBC. OSHPD requires a dynamic analysis for all sites and structures. The UBC lists R_{WI} reduction factors for many types of building structural systems as shown in Table 6d-2.

Design displacements may be determined from simplified formulas (UBC) or may be reduced from those resulting from a dynamic analysis. OSHPD requires the use of 1.5 times displacements calculated using the *Maximum Probable Earthquake* (MPE) or 1.1 times those from the *Maximum Credible Earthquake* (MCE).

Load combinations are the same for both codes. OSHPD specifies the use of alternative vertical loads or support elements as a safeguard in case isolator instability should occur in a major earthquake. UBC does not have this provision, however, many designers provide some type of support. UBC requires a slightly greater separation gap.

Table 6d-1.

UBC[1] and OSHPD[2] Seismic Isolation Requirements.

REQUIREMENT	UBC	OSHPD
SEISMIC INPUT		
Design basis (MPE) (maximum probable earthquake)	475-year return period	Same as UBC
Maximum (MCE) (maximum capable earthquake)	1000-year return period ($\geq$ 1.25 MPE)	Same as UBC
Lower bound spectrum	0.8 UBC Spectrum	Same as UBC
Time histories	Within 10% of MCE response spectra, with near-field for site $\leq$ 15km to fault	Same as UBC
ANALYSIS		
3-D Modelling	Not Specified	Required
Static analysis ok if	> 15km, S_1 or S_2 Soil, Zone 3 or 4, $\leq$ 4 stories or 65 feet, $T_1 \leq 3.0$ sec., $T_1 > 3T$, regular config. & near linear system	Not applicable
Response-spectrum analysis required if	< 15km S_1, S_2 or S_3 Soil, $T_1 > 3.0$ sec, nearly elastic system, Zone 1, 2A, 2B	Same as UBC
Time history required if	Nonlinear isolation system or S_4 Soil	Same as UBC
DESIGN FORCES		
MPE (for superstructure design)	From simplified formulas or dynamic analysis • $V_S = K_{MAX} D/R_{WI} \geq VR_{WI}/R_W$ • $V = V_{DYNAMIC}/R_{WI}$ or .6 to .8 $K_{MAX}D/R_{WI}$ if regular	Same as UBC
MCE	Isolation system must remain stable	Same as UBC
DESIGN DISPLACEMENTS - ISOLATION SYSTEM		
MPE	D_T simplified formula or 0.9 D_T from dyn analysis or $D_{TM} = M_M D_T$	Same as UBC
MCE	D_{TM} from 1.0 MCE	Same as UBC
Separation gap	MCE	Same as UBC
LOAD COMBINATIONS		
Superstructure Design	1.2DL + 1.0LL $\pm$ E 0.8DL $\pm$ E	Same as UBC
OTHER REQUIREMENTS		
Redundant vertical load element	Not required	Required

1 Uniform Building Code, 1994 Edition
2 Office of Statewide Health Planning and Development, State of California, 1994 Draft Edition

Table 6d-2. Reduction Factors for Base Isolated Structures.

BASIC STRUCTURAL SYSTEM[1]	LATERAL LOAD-RESISTING SYSTEM DESCRIPTION	$R_W I$[2]	H^3 x 304.8 for mm
1. Bearing wall system	1.1 Light-framed walls with shear panels		
	1.1.1 Plywood walls for structures three stories or less	2.6	65
	1.1.2 All other light-framed walls	2.2	65
	1.2 Shear walls		
	1.2.1 Concrete	2.6	160
	1.2.2 Masonry	2.6	160
	1.3 Light-steel-framed bearing walls with tension-only bracing	1.6	65
	1.4 Braced frames where bracing carries gravity loads		
	1.4.1 Steel	1.8	160
	1.4.2 Concrete[4]	1.5	
	1.4.3 Heavy timber	1.5	65
2. Building frame system	2.1 Steel eccentric-braced frame (EBF)	3.0	240
	2.2 Light-framed walls with shear panels		
	2.2.1 Plywood walls for structures three stories or less	2.6	65
	2.2.2 All other light-framed walls	2.2	65
	2.3 Shear walls		
	2.3.1 Concrete	3.0	240
	2.3.2 Masonry	3.0	160
	2.4 Concentric-braced frames		
	2.4.1 Steel	2.2	160
	2.4.2 Concrete[4]	2.6	
	2.4.3 Heavy timber	2.6	65
3. Moment-resisting frame system	3.1 Special moment-resisting frames (SMRF)		
	3.1.1 Steel	3.0	NL[5]
	3.1.2 Concrete	3.0	NL
	3.2 Concrete intermediate moment-resisting frames (IMRF)[6]	2.2	
	3.3 Ordinary moment-resisting frames		
	3.3.1 Steel	1.8	160
	3.3.2 Concrete[4]	1.5	
4. Dual system	4.1 Shear walls		
	4.1.1 Concrete with SMRF	3.0	NL
	4.1.2 Concrete with concrete IMRF[5,6]	2.6	160
	4.1.3 Masonry with SMRF	3.0	160
	4.1.4 Masonry with concrete IMRF[4]	2.6	
	4.2 Steel EBF with steel SMRF	2.6	NL
	4.3 Concrete-braced frames		
	4.3.1 Steel with steel SMRF	2.2	NL
	4.3.2 Concrete with concrete SMRF[4]	2.2	
	4.3.3 Concrete with concrete IMRF[4]	1.8	
5. Undefined systems	See Section 1627.9.2		

1 Basic structural systems are defined in Section 1627.6.
2 See Section 1628.3 for combination of structural system.
3 H — Height limit applicable to Seismic Zones 3 and 4. See Section 1627.7 for exceptions.
4 Prohibited in Seismic Zones 3 and 4.
5 NL — No limit.
6 Prohibited in Seismic Zones 3 and 4 , expect as permitted in Section 1632.2.

The UBC specifies larger site coefficients for base isolated structures than for fixed-base designs. Table 6d-3 shows a comparison of site coefficients for each.

Testing and Earthquake Performance

Commercially available isolation bearings have been extensively tested on shaking tables (at the University of California, Berkeley) and under full scale cyclic loads (at numerous facilities). These types of tests are essential for verification of isolation bearing systems.

The 1994 Northridge, California earthquake proof-tested the lead-core-rubber isolation bearing system that had been installed during recent construction of the University of Southern California (USC) Hospital Building in Los Angeles. The USC Building, which was fully instrumented, successfully demonstrated that the isolation concept not only performs as predicted, but also protects building contents. Ground accelerations of 0.37g measured immediately below the isolators were reduced to 0.13g above the isolators; a reduction of 1/3 of the acceleration expected in a fixed-base conventional building. Roof-top accelerations were about 1/5 of that expected in a conventional building.

Passive Energy-Dissipation Systems

Passive energy-dissipation systems can be a viable alternative to seismic isolation. The basic premise is to reduce the building response to seismic ground motions by increasing the structural damping and dissipating earthquake-induced energy by supplementing the energy-dissipation capability of the building's structural framing system. In conventional earthquake-resistant design, energy dissipation (damage) is expected to occur by plastic hinging at beam-column joints or by cracking or yielding of shear walls or other shear-resisting elements. The function of energy-dissipation devices is to increase the energy-dissipation capability of the system without increasing the displacement/deformation of the building system. These devices are not part of the gravity-load-resisting system and can be replaced after an earthquake, if necessary. The intent of such devices is to confine inelastic activity primarily to the dissipation devices per se and thus, minimize damage to the gravity-load-resisting and nonstructural systems during a design-level earthquake.

Table 6d-3.

Site Coefficients for Seismic Isolated and Fixed Base Structures.

SOIL PROFILE TYPE[1]	SITE COEFFICIENTS	
	S_1 FACTOR BASE ISOLATED	S FACTOR FIXED BASE
S_1	1.0	1.0
S_2	1.4	1.2
S_3	2.3	1.5
S_4	2.7	2.0

[1] See UBC 1994, Table 16-J for description of soil profile types and related requirements.

Effect of Damping on the Seismic Response of Structures

Fig. 6d-8 is a simple illustration of the effects of damping on building response. Fig. 6d-8(a) illustrates how a structure, theoretically, with no damping, would continue to vibrate. Fig. 6d-8(b) shows how the introduction of damping causes the response (vibration) to diminish rapidly.

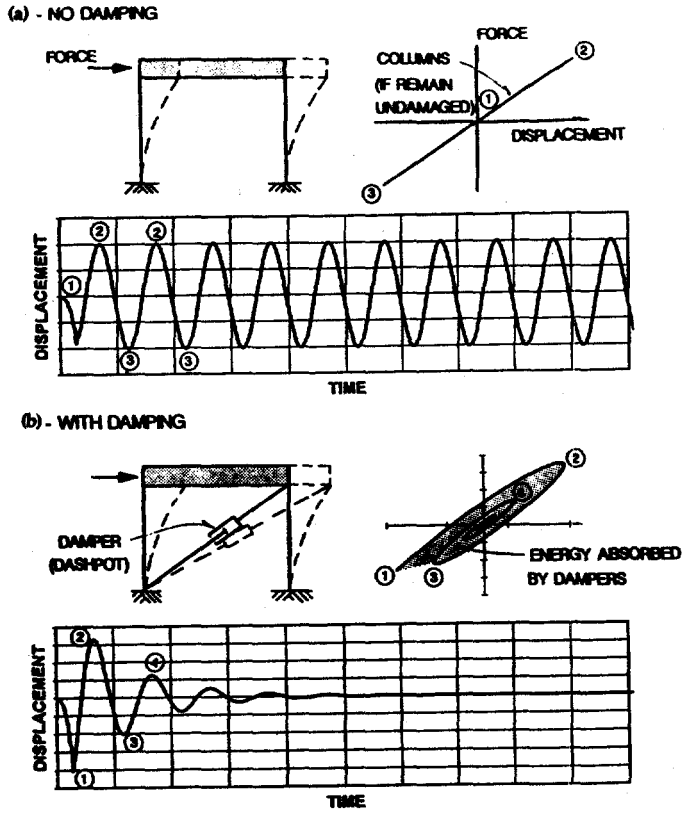


Fig. 6d-8. Effect of damping on structure seismic response.

Natural Structural Damping and Damping Levels

Damping in buildings is a result of energy dissipation that occurs in joint movements, frictional losses from the differential movements of adjacent building elements, differential movements of exterior cladding elements in relation to each other, and differential movements of exterior cladding elements in relation to each other and the building structure.

Fig. 6d-9 shows the effect of various levels of damping on displacement. It can be seen that adding damping can materially reduce the displacement (story drift) of the structural framing.

Types of Damping

There are generically four types of damping:

- Viscous damping
- Viscoelastic damping
- Hysteretic damping
- Friction damping.

Examples of devices to achieve supplemental damping are shown in Fig. 6d-10.

Viscous Damping

Viscous damping devices operate on the principle of fluid flow through orifices (similar to automobile shock absorbers). They originated in the early 1960s as energy-absorbing buffers for individual machinery. Damping is activated by velocity of motion. Pure viscous damping does not change stiffness, period of vibration, or strength of a building. Its application to seismic design of buildings has been studied since 1990. Fig. 6d-10(a) illustrates a commercially available viscous device. Both individual dampers and building shaking table tests have been performed at the State University of New York, Buffalo (Ref. 10).

Viscoelastic Damping

Viscoelastic damping devices generally use a form of acrylic polymer (similar to synthetic rubber) as a shock absorber. The lateral stiffness of the damper contributes to the overall stiffness of the building. This type of damper often is used to minimize vibrations caused by wind pressures on a building. Extensive testing of individual dampers, as well as building shaking table tests were performed at the University of Michigan, the State University of New York at Buffalo, the Army Corp of Engineers - Construction Engineering Research Laboratory (CERL), and the University of California at Berkeley (Refs. 11, 12, 13, 14). A viscoelastic damper is shown in Fig. 6d-10(b).

Hysteretic Damping

Hysteretic damping dissipates the seismic energy by yielding of steel elements and/or

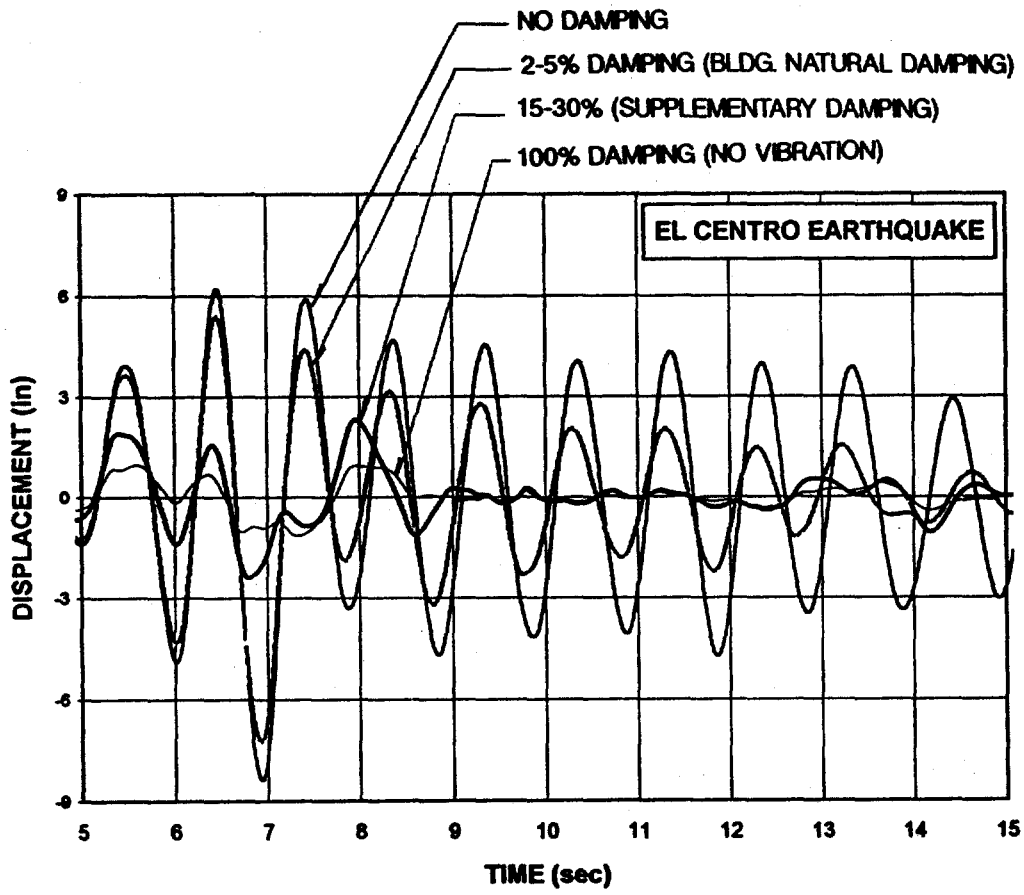


Fig. 6d-9. Effect of damping on structural displacement.

cracking of structural concrete elements. It is generally associated with damage to main structural elements (beams, columns, walls) and may not be reliable unless the structural elements have ductility (including connections).

Supplementary hysteretic damping can be provided by adding special yielding elements activated by building displacement (story drift). Fig. 6d-10(c) shows a supplementary damping device with *added damping and stiffness* (ADAS). Tests of individual damping devices as well as building shaking-table tests were performed at the University of California at Berkeley (Ref. 15).

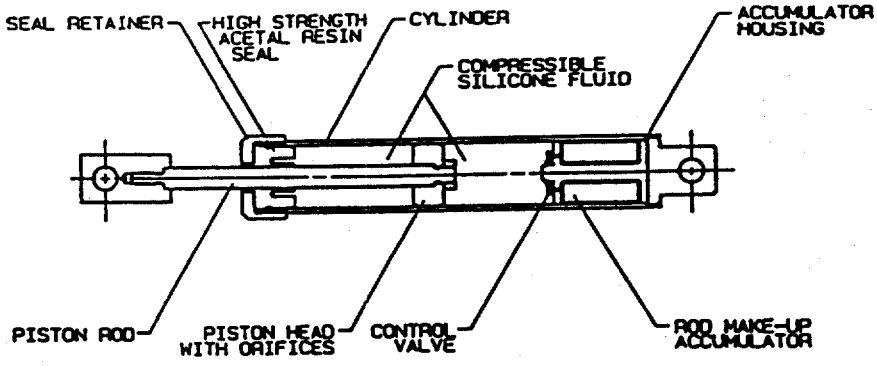
Friction Damping

Friction damping devices are based on the friction force developed between metal surfaces as they displace relative to each other. This type of damper largely remains inactive under small seismic loading because the frictional interfaces tend to stick together. Fig. 6d-10(d) shows a friction damper device that has been tested at the University of California at Berkeley (Ref. 15).

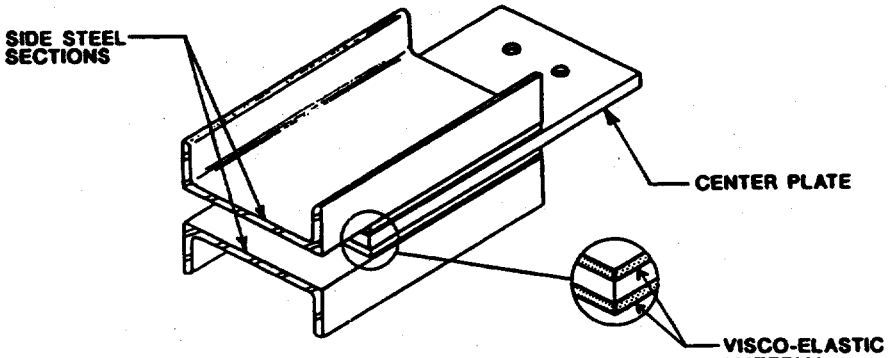
Testing of Passive Energy-Dissipation Devices

Testing is required to confirm engineering characteristics of the devices for use in analysis and design, and to verify the adequacy of systems. Prototype testing should be conducted on at least two full-size devices of the type and size to be used in the structure. Consideration should be given to any effects of vertical loading in the structure that might be imposed on the engineered dissipating device. Fig. 6d-11 illustrates typical force displacement data from testing of damping devices.

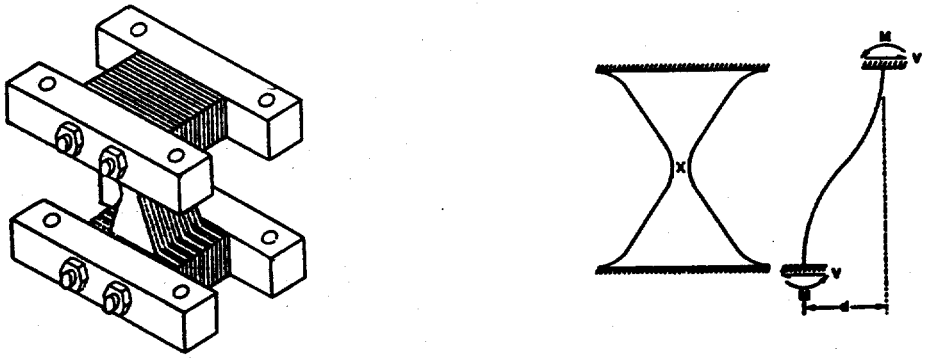
Three levels of testing are recommended in an appendix to Chapter 2 of the *NEHRP*



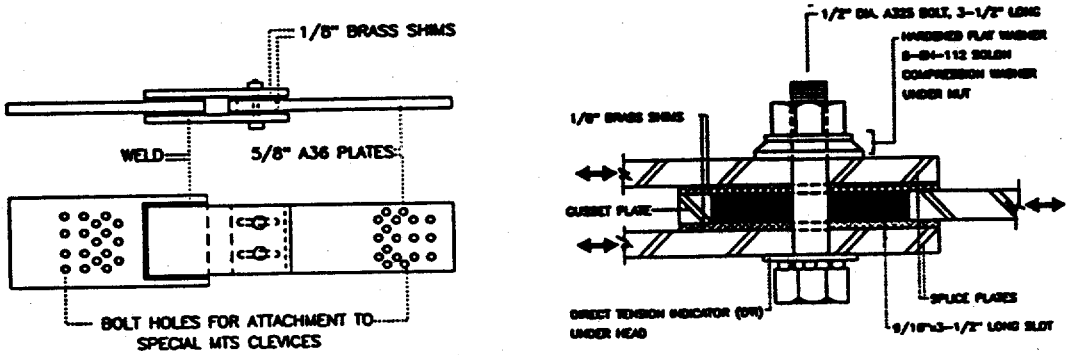
(a) Taylor Device (Fluid Viscous Damper)



(b) 3M Visco-elastic Damper

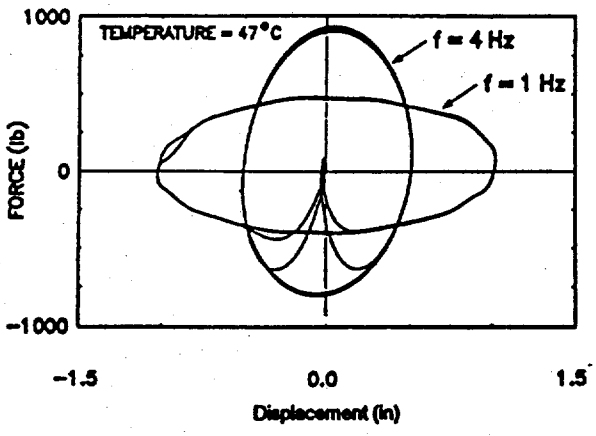


(c) Added Damping and Stiffness (ADAS) Device

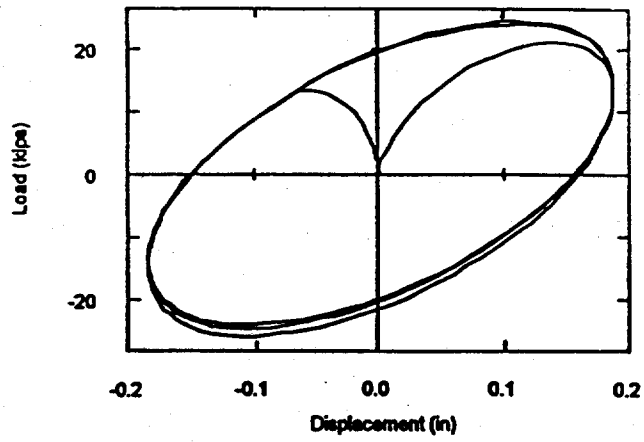


(d) Slotted-Bolted Connection (SBC)

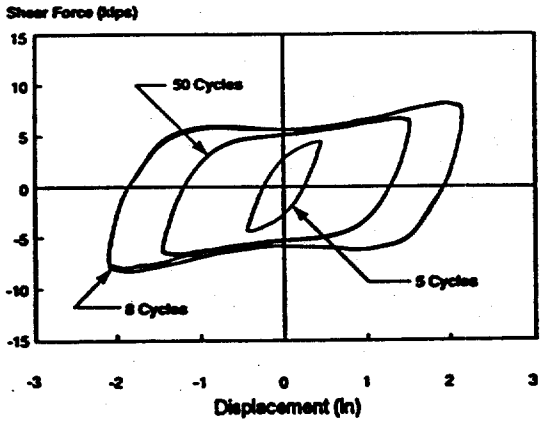
Fig. 6d-10. Typical supplemental damping devices.



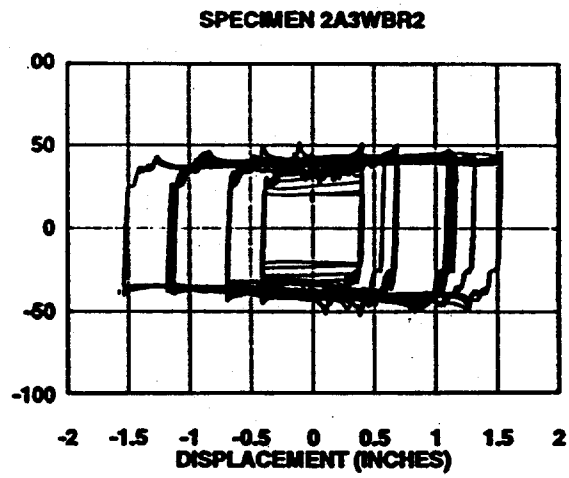
(a) Fluid Viscous Damper
(Ref. Constantinou and Syman, 1993)



(b) Visco-elastic Damper
(Ref. Blondet, 1993)



(c) ADAS Device
(Ref. Scholl, 1991)



(d) Slotted-Bolted Connection
(Ref. Grigorian and Popov, 1994)

Fig. 6d-11. Typical force displacement response of added damping devices.

Provisions, 1994 edition (Ref. 16) for the prototype devices:

- *"Two hundred fully-reversed cycles of loading at the level of force in the energy dissipation device computed for the design wind storm,*
- *Fifty fully-reversed cycles of loading at the design displacement, that is, the displacement in the devices corresponding to the design-level earthquake, and*
- *Ten fully-reversed cycles at a displacement corresponding to the maximum capable earthquake."*

If the response of the devices is dependent on loading frequency, tests should be conducted as a minimum at loading rates equal to 0.5, 1.0, and 2.0 times the fundamental frequency of the building/structure system calculated with the devices in place. If the devices are affected by or dependent on bilateral loading, additional testing should be performed.

Peer Review for Seismic Isolation and Passive Energy-Dissipation Projects

Seismic isolation and energy-dissipation systems (as evolving state-of-the-art concepts) are gaining acceptance by owners for reducing the seismic response of buildings, as well as for protecting equipment housed therein. Independent engineer peer reviewer(s) have been retained for seismic isolation projects since the first United States application of seismic isolation for buildings; the Foothills Communities Law and Justice Center, Rancho Cucamonga, California.

Most of the subsequent seismic isolation and passive energy-dissipation projects have been for the seismic retrofit of existing buildings. However, a number of new facilities have been and are being designed, and all of these are being peer reviewed.

Guidelines for independent engineer peer review of conventional projects have been proposed (ASCE, Ref. 17, ACEC/ASCE, Ref. 18; SEAONC, Ref. 19), but specific rules for seismic isolation or energy dissipation projects have not been developed.

Status and Purpose of Peer Review

Independent engineering peer review of seismic-isolation projects is required by the UBC and local jurisdictions such as San Francisco and Los Angeles. UBC Section 1660.1 (Appendix Chapter 16) specifies that a design review shall be made of the isolation system and related test programs. Section 1660.2 outlines the scope of the design review:

"Isolation system design review shall include, but not be limited to, the following:

1. *Review of site-specific seismic criteria, including the development of site-specific spectra and ground motion time histories, and all other design criteria developed specifically for the project.*
2. *Review of the preliminary design, including the determination of the total design displacement of the isolation system and lateral force design level.*
3. *Overview and observation of prototype testing (Section 1661).*
4. *Review of the final design of the entire structural system and all supporting analyses.*
5. *Review of the isolation system quality control testing program (Section 1657.2.9)."*

A similar review requirement will probably be developed for energy-dissipation systems.

The purpose of an independent engineer peer review as stated by SEAONC and ACEC/ASCE is to *"...enhance the quality and provide a measure of additional assurance regarding the performance and safety of the complete project."* It is not a substitute for a building official's plan check or value engineering to suggest alternative systems, materials, and methods for reducing construction costs. For seismic isolation projects, the purpose should be to better ensure:

- Quality of design and design approach
- Quality of project documentation
- Structural performance
- Project objectives are met.

Timing of Peer Review

Peer review comments or inquiries early in a project have the most effect on design-engineering decisions. If the peer review does not start until design development is well underway or near completion of construction documents, details that could have been modified rather easily during conceptual design can become major changes and consequently have an adverse impact on project cost and schedule.

Scope of Review

The scope of peer review and how it is interpreted by the peer review members and the *Structural Engineer of Record* (SER) can become a major bone of contention. The scope must be carefully delineated prior to start of review and care must be taken by peer reviewers to ensure that there is no encroachment on the design responsibility of the SER.

It is essential early on in the project that the client's requirements be defined, i.e., what are expected performance criteria? Are they minimum structural damage, continued functioning during and after a major earthquake, minimum motion effect on equipment, or other? The feasibility of attaining the expected performance should be assessed by the peer review team.

Qualifications of Peer Reviewers

The success of the peer review effort is directly related to the experience, qualifications, and ability of the individual reviewers and the peer review team. The SER should be consulted during the selection process because it is essential that peer review members have the respect of the SER. *SEAONC Guidelines* (Ref. 19) suggests several qualifications:

- Independence from design team and no involvement in the project before or after the review
- Ability to cooperate with others
- Ability to conduct the review in an unbiased and constructive manner
- Equal or higher level of technical expertise than the SER
- California structural engineer as lead
- Familiarity with governing regulations for the project.

ACEC/ASCE (Ref. 18) further suggests that the team be composed of registered professionals with at least 15 years of relevant experience. The authors recommend that *each* structural reviewer have substantial experience in the analysis *and* design of major buildings. An understanding of how a building will perform during an earthquake is essential.

There is often a tendency to believe the results of complex analyses, but they must be related to reality. The results of dynamic analyses depend in large part on the assumptions made while preparing the mathematical model(s). The owner, by retaining a peer review team, is looking for added engineering judgment and intuition. Engineering intuition is an off-shoot of seasoned judgment. Design calculations or computer printouts may look beautiful, nevertheless intuition may indicate something is wrong. Questioning by an experienced peer reviewer could reveal the problem.

Analysts view a building from a numerical perspective; structural designers view it as a physical structure composed of many integral elements. Experienced structural engineers view the results of analyses and calculations as a guide to their judgment.

Recommended Procedures for Peer Review

1. The selection of peer reviewers should follow the *SEAONC Guidelines* (1991) for qualifications and, in addition, require each reviewer to have at least 15 years of relevant analysis and design experience with major projects. The SER should be consulted during the selection process and his views incorporated where feasible.

2. The scope of the peer review should be carefully delineated in writing prior to start of work, and a written agreement for services, including an indemnification clause, executed.

3. The project performance criteria should be discussed and agreed upon among the owner, SER, and peer review team.

4. The data provided should be assessed by the peer review and, if deemed to be inadequate, the Owner and SER should be so informed.

5. The data presented on the geotechnical characteristics of the site should be evaluated. Are the types of ground motions that might be expected included, and are the response spectra or time histories proposed for the site representative of possible earthquake ground motions in the immediate area?

6. The dynamic response characteristics of the isolator or energy-dissipation systems should be assessed by the SER as:

   a. A separate system; what are its characteristics after 10 or 20 cycles of maximum displacements when under design loading?

   b. In combination with the proposed structure; what are the stability safety factors at maximum displacements?

   c. A total system consisting of the structure, isolator system, and underlying soils.

7. The design for seismic isolation should make provisions for the effects of differential displacements in any horizontal or vertical direction for:

   a. Interfaces between structure, supporting foundations, and surrounding soils

   b. Ingress and egress to the building

   c. Architectural elements such as fascia, flashing, exterior walls, doors, and elevators

   d. Mechanical and plumbing systems such as fire protection, piping, ducts, and equipment

   e. Electrical systems such as conduit, wiring, raceways, and substations

   f. Utilities such as sanitary and storm (rain-water) sewers, natural gas piping, communications, and water supply.

8. The uses and experience records for seismic isolation and energy-dissipation systems are limited. Therefore, the SER must approach each item with care and give detailed consideration to its effect on the overall response of the building or structure, especially if the displacements prescribed in the design criteria are exceeded in a major earthquake.

9. The specifications should include procedures for inspection and maintenance of the isolator systems. The design should provide for inspection and maintenance plus possible removal and replacement or repair of components.

10. The isolation or energy dissipation system is only as good as it is fabricated and installed. The SER must specify detailed quality assurance procedures and requirements for isolator design, fabrication, and installation. Details must be carefully developed.

DOE Guidance

The U.S. Department of Energy (DOE) sponsored a *Short Course on Seismic Base Isolation*, August 10-14, 1992, which included:

- Overview of base isolation and its applications
- Development and implementation of base isolation in the United States and abroad
- Theory technical background
- Modeling and analysis issues
- Application to design with SEAOC/UBC provisions
- Design examples.

Extensive course notes (Ref. 20) provide considerable detail about the state of the art in base isolation.

References

1. *Introduction to Base Isolation*, E. G. Kost, American Society of Mechanical Engineers, Third National Conference on Pressure

Vessels and Piping, 79-PVG, June 25-29, 1979.

2. *Procurement Strategies for Seismic Isolation Systems,* M. T. Walter, Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control, ATC-17-1, Applied Technology Council, Redwood City, California, March 1993.

3. *Uniform Building Code,* International Conference of Building Officials, Whittier, California, 1994.

4. *Guide Specifications for Seismic Isolation Design,* American Association of State Highway and Transportation Officials, Washington, D.C., 1991.

5. *Draft Guidelines for Pre-Qualification and Prototype Testing of Seismic Isolation Systems,* National Institute of Standards and Technology, NISTIR 5359, Washington, D.C., March 1994.

6. *Draft Guidelines for Quality Control Testing of Elastomeric Seismic Isolation Systems,* National Institute of Standards and Technology, NISTIR 5345, Washington, D.C., February 1994.

7. *Draft Guidelines for Quality Control Testing of Sliding Seismic Isolation Systems,* National Institute of Standards and Testing, NISTIR 5371, Washington, D.C., March 1994.

8. *Tentative Isolation Requirements,* Structural Engineers Association of Northern California, San Francisco, California, 1986.

9. *An Acceptable Method for design and Review of Hospital Buildings Utilizing Base Isolation,* Office of Statewide Health Planning and Development, State of California, Sacramento, California, 1989.

10. *Experimental Study of Seismic Response of Buildings with Supplemental Fluid Dampers.,* M.C. Constantinou, M.D. Symans, The Structural Design of Tall Buildings, Vol. 2, pp. 93-132, State University of New York, Buffalo, New York, 1993.

11. *Inelastic Response Spectra with High Damping,* J.P. Wu, R.D. Hanson, American Society of Civil Engineers, Journal of the Structural Division, 115(6):1412-1431, University of Michigan, Ann Arbor, Michigan, 1989.

12. *Seismic Behaivor and Design Guidelines for Steel Frame Structures with Added Viscoelastic Dampers,* K.C. Chang, M.L. Lai, T.T. Soong, D.S. Hao, Y.C. Yeh, Technical Report No. NCEER-93-0009, State University of New York, Buffalo, New York, May 1993.

13. *Earthquake Simulator Testing and Analytical Studies of Two Energy-Absorbing Systems for Multi-story Structures,* I.A. Aiken, J.M. Kelly, Report No. UCB/EERC-90/03, Berkeley, California, 1990.

14. *Energy Dissipation with Slotted Bolted Connections,* C.E. Grigorian, E.P. Popov, UCB/EERC-94/02, Berkeley, California, 1994.

15. *Earthquake Simulator Testing of Steel Plate Added Damping and Stiffness Elements,* A. Whittaker, V.V. Bertero, J. Alanso, C.L. Thompson, UCB/EERC-89/02, Berkeley, California, 1989.

16. *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings,* FEMA 222, Federal Emergency Management Agency, Washington, D.C., 1994.

17. *Quality in the Constructed Project, A Guide for Owners, Designers and Constructors,* Vol. 1, Manual of Engineering Practice No. 73, Chapter 21, Peer Review, American Society of Civil Engineers, New York, New York, 1990.

18. *Project Peer Review Guidelines,* American Consulting Engineers Council/American Society of Civil Engineers, New York, New York, 1990.

19. *Guidelines for Project Design Peer Review,* Structural Engineers Association of Northern California, San Francisco, California, Draft February 1991.

20. *Short Course on Seismic Base Isolation,* U.S. Department of Energy, Office of Risk Analysis and Technology, Office of Nuclear Safety Policy and Standards, Course Notes, August 10-14, 1992, Berkeley, California, 1992.

Chapter

7

Evaluation of Existing Buildings

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

Introduction

When existing facilities are reviewed for earthquake safety, it is not unusual to find serious structural deficiencies in a significant percentage of the buildings reviewed. This has been the experience in both public and private sectors. Unfortunately, even relatively new buildings, presumably designed to comply with modern seismic design codes, are sometimes found to be hazardous. In many areas in the United States, earlier codes have been more or less permissive with respect to seismic design. Also, to keep construction costs as low as possible, many buildings were legalistically designed to barely meet the minimum requirements of the seismic code. Usually these buildings lack the ductility or redundancy that is generally obtained if designers realistically carry out the fundamental intent of the code. Sometimes important elements of the lateral-force-resisting system were constructed of brittle material. A good example is the non-ductile moment-resisting reinforced concrete frame that was permitted by seismic codes for buildings up to 160 feet high (about 10 stories) until its poor performance in the 1971 San Fernando, California earthquake.

As previously stated, it is important for facility managers to recognize that a building of recent vintage may be unsafe even if it was "designed to code." This is always an important consideration, particularly when a number of buildings must be reviewed for earthquake safety and a priority system is to be established for the sequence of review.

The term *designed to code* is a relative matter depending upon the year the building was designed and constructed. Structural engineers learn from every new earthquake, and from time-to-time this information is reflected in significant changes in the earthquake provisions of the *Uniform Building Code*. In the same manner, research results can lead to code changes. These changes almost invariably lead to code *improvements*. Looked at from the opposite point of view, each seismic code change increases the inventory of *nonconforming* buildings, which some view as potentially *unsafe* buildings. At times, structural provisions of the code have been weakened and sometimes new types of construction have been allowed for short periods before the design was fully codified. *Designed to code* should not be taken as assurance that a building is well-designed for earthquake safety.

Also, it should not be taken for granted that just because a building was designed at a particular time, it was actually designed to comply with the code in effect at that time. A determination should be made whether or not it

was in fact *designed to code.* Also, it should be determined whether an existing building was actually *constructed to code* in accordance with its construction contract documents. The foregoing are good reasons to utilize structural engineering consultants who have had significant experience in observing earthquake damage to buildings and in evaluating their anticipated seismic performance, to make a seismic evaluation of existing facilities.

Historically, the subject of modern hazardous buildings has been a sensitive one for the engineering profession to deal with publicly. The legal aspects of the problem — the potential liabilities involved, pressures from materials suppliers, heavy owner investments, occupancy considerations, and technical limitations of code enforcement—all tend to cloud the issue and suppress public discussion. Even in cases when public buildings have suffered serious structural seismic damage because of poor design, the profession often has been unable to deal with the problem as clearly and effectively as one would hope. Many of these buildings met the legal minimum code requirements only marginally, while missing the broader, more basic *motherhood* provisions of the code that define the intent to provide a safe building. The Olive View Hospital which failed in the 1971 San Fernando earthquake is an example. Many knowledgeable earthquake engineers, who were convinced of the hazards inherent in such buildings, would not speak out publicly because of concerns for personal liability or loss of clientele. The easier approach therefore was to modify the code to solve future problems rather than to press for mitigation of existing hazards.

Fortunately, the State of California Seismic Safety Commission was more forthright in bringing specific types of hazardous buildings to public attention. In November 1985, a Subcommittee of the California Seismic Safety Commission (SSC) presented report No. SSC 85-04, *Potentially Hazardous Buildings,* which identified certain building types it considered to be potentially hazardous, excluding unreinforced masonry structures. The special problem of unreinforced masonry buildings had been extensively studied by others, and steps have been initiated by several California cities to develop regulatory provisions to mitigate the safety hazard they represent.

The subcommittee's report proved accurate in events such as the 1989 Loma Prieta and the 1994 Northridge, California earthquakes. Portions of the Subcommittee's report that describe potentially hazardous buildings, both old and modern, are reproduced hereafter for the benefit of facility managers.

"POTENTIALLY HAZARDOUS BUILDINGS"

"Non-Ductile Concrete Frame Buildings"

"Throughout the State of California, a large number of *non-ductile concrete frame buildings* exist. Included are buildings from the very early days of reinforced concrete structures, up to the time when building codes required that any moment-resisting frame that was part of the earthquake-resisting system had to be a ductile moment-resisting frame (1968 San Francisco Code, 1973 *Uniform Building Code).* Non-ductile concrete frame buildings have suffered badly in all recent earthquakes. In general, this type of building does not have a shear-wall system, and the lateral force resistance is expected to be provided through frame action. Many of these buildings were designed and constructed prior to the adoption of earthquake codes, and they have some, but very little, calculable seismic resistance. Buildings that were designed and built under a building code requiring earthquake-resistant design can probably show more calculated resistance, but that resistance is generally not sufficient to withstand earthquake forces that will stress the building beyond its elastic limit. The details of construction of the non-ductile concrete frame building are such that severe damage and possible collapse can be expected in areas of exposure to strong earthquakes.

"Non-ductile concrete building frames can be found in one-story parking garages or similar structures, with relatively heavy concrete roof systems, ostensibly braced by slender non-ductile concrete columns that often cantilever out of the foundations.

"Current building (seismic) codes do not allow further construction of this type of

building, although this presumes adequate code enforcement.

"Strengthening of existing buildings of this type can best be accomplished through the use of concrete shear walls, braced frame systems, or exterior buttresses. The main object is to limit the deflection of the non-ductile frame by incorporating a very stiff bracing system. Some examples of failure and damage to non-ductile frames are: VA Hospital, San Fernando, California; the ambulance structure, Olive View Hospital, San Fernando; Petunia Building, Caracas, Venezuela; and Cypress Gardens, Caracas."

"Precast Concrete Buildings"

"While most of these buildings are of tilt-up concrete construction, they fall into two basic subcategories: Pre-1973 tilt-up buildings, and recent and current tilt-up precast buildings.

"Pre-1973 Tilt-up Buildings: Following the extensive damage to tilt-up buildings in the 1971 San Fernando earthquake, changes were made in the building codes requiring positive anchorage of the precast wall panels to floor and roof diaphragms, as well as other specific diaphragm details. It is presumed that buildings constructed since that time are adequate in this respect; however, there are many buildings throughout California that were constructed with details similar to those used in damaged San Fernando buildings. (Editor's note. Both pre- and post-1973 UBC tilt-up structures performed poorly in the 1994 Northridge earthquake because of ineffective anchorage to wood roof and floor diaphrams.) Buildings in this category were of masonry wall construction as well as tilt-up concrete. (An example of this type of construction is the Vector Electronics Building, which suffered severe damage in the 1971 San Fernando earthquake.)

"Pre-1973 tilt-up concrete buildings could be reinforced at relatively low cost to provide anchorage sufficient to prevent wall collapse during a severe earthquake, because the roof-to-wall interface is usually exposed and accessible.

"Recent and Current Tilt-up Precast Buildings: While wall-diaphragm connections and diaphragm details have improved since 1973, other aspects of tilt-up and precast concrete construction have deteriorated. The use of tilt-up construction has expanded beyond the original industrial warehouse type of building with mostly solid wall panels, into light industrial, commercial, retail, and housing uses, with much more attention to 'architecture.' This trend has led to precast tilt-up buildings with many large openings, that in the extreme become simply spandrels with very narrow integral columns. In general, adjacent panels are no longer connected together except for chord reinforcing at the diaphragm, and the panels are not adequately connected to the foundation system. Whereas tilt-up construction was originally used primarily for one-story construction, two-story buildings are now quite common and taller buildings are occasionally observed. This new generation of tilt-up building does not have the characteristics of ductility, toughness, and redundancy that are necessary for good earthquake performance. These buildings have not yet been tested by a real earthquake, but it is the opinion of this Subcommittee that they will prove to be collapse hazards.

"Casual observation of a number of these precast concrete buildings currently being constructed makes it quite clear that the designers of these buildings either do not understand the primary elements of earthquake-resistant design, or they choose to ignore them. Of equal significance is the fact that these designs are being approved by local building departments.

"These precast buildings are being constructed in large numbers throughout California, particularly in areas of rapid growth of light industry, which in many cases also happen to be in areas of high seismicity.

Strengthening this type of existing building would probably vary depending on the nature of the building, but should probably include a well-distributed system of concrete shear walls or braced frames (or buttresses)."

"Soft-Story Buildings"

"The soft-story building is much more difficult to identify because the structural system is often obscured by cladding. There are many such buildings, but we have no idea how many or where they are located. We think the scope of the problem is significantly less than for non-ductile concrete frame buildings or precast concrete buildings. Nevertheless, these buildings should be identified, as they can present a very great

hazard. The most hazardous of these buildings also fall under the non-ductile concrete frame category. (Examples given of soft-story buildings are the Olive View Hospital, damaged in the 1971 San Fernando earthquake, and the Imperial County Service Building damaged in the 1979 Imperial Valley earthquake.)

"Future hazardous buildings of the soft-story variety might be avoided by enacting code changes outlawing this type of construction. There are numerous building configurations, however, that can have a negative influence on building performance under seismic loading. Requiring *special analysis* and ductile performance for *irregular* buildings is considered to be a better solution, but it depends on knowledgeable engineers and architects, and good plan-review procedures.

"The correction of existing hazards posed by the soft-story building can probably be done most effectively through the addition of concrete shear walls or braced frames (or buttresses)."

"Other Identified Hazards"

"Prestressed Concrete Buildings: Buildings with prestressed elements, some cast-in-place and some precast, have suffered significant distress from high-stress concentrations resulting from creep, shrinkage, and temperature shortening. Volume change of this type is not compatible with the philosophy of providing ductility, toughness, and redundancy by tying together an earthquake-resistant building.

"The long-term creep-shortening of the prestressed double tees was the basic problem behind the Antioch High School (California) roof failure, which resulted in evaluations of similar systems in other schools throughout California.

"Many post-tensioned structures have been observed to suffer significant cracks and structural deterioration caused by the creep-shortening effect of the post-tensioning forces. Many of these structures are undergoing scrutiny by knowledgeable owners, and this type of construction will certainly bear watching.

"Pre-1934 Unreinforced Masonry Buildings having less than five dwelling units: These buildings are now exempted by the Los Angeles ordinance, but there is still a potential for a large loss of life throughout California in this type of building. Research is required to better analyze, evaluate, and reduce earthquake hazards in these buildings.

"Theaters and Auditoriums with Long-Span Roof Structures built prior to 1933 in California: This class of structure presents a potential hazard to a relatively large number of occupants because of the wide span of the roof structure and the lack of a lateral force-resisting system. Further hazards are created by high stage walls and scenery as well as other materials in the fly lofts.

"**Exterior Cladding and Glazing:** Many buildings are clad with heavy precast concrete or masonry panels that are attached to the basic frame. The strength of these connections is vitally important to prevent the panels from breaking loose as a result of building distortions in a severe earthquake. Buildings constructed before the advent of current (seismic) code anchorage requirements are particularly suspect.

"These systems should be the subject of continuing research, as should glazing practices and details to prevent shattering of glass.

"Parapets: This list would not be complete without parapets. Although Los Angeles long ago completed a hazardous parapet abatement program and San Francisco is doing so now, other cities with similar seismic exposure should follow suit." (end of reproduced portion of report).

Author's Summary

The California Seismic Safety Commission's work marked an important step forward in public recognition of problems inherent in many modern hazardous buildings. Since then, public attitudes in California about earthquake hazards and financial risks of liability have changed considerably. Engineers became more outspoken about building types that are substandard and hazardous to occupants. The public became more knowledgeable about the presence of dispersible, highly toxic materials in both private and government-owned buildings and more concerned about the consequences of seismic failures. Owners and managers in turn became more aware of their corporate and sometimes personal liability for the safety of occupants and the public, particularly when a building is known to be seismically unsafe.

•

The 1989 Loma Prieta earthquake triggered a backlog of growing concern about liability that had been growing with each of the numerous damaging earthquakes experienced in California during the preceding ten years or so. Allowing people to continue to occupy a building, after the Loma Prieta earthquake had revealed it to be a potential collapse hazard, was unacceptable to most responsible owners. Consequently, many damaged buildings, exposed as high seismic risks, were evacuated until they could be realistically strengthened to resist a major earthquake. This was in sharp contrast to experience in the aftermath of the destructive San Fernando earthquake of 1971. Then, numerous damaged buildings that sustained serious damage remained occupied while relatively cosmetic repairs were made, which left them susceptible to future destruction.

Although building types that are susceptible to earthquake damage are well known to most experienced engineers, evaluation of buildings presumably designed and constructed to modern seismic codes requires a disciplined process. Chapters 7a and 7b provide a systematic approach, including guidelines for fulfilling DOE evaluation requirements, the use of complementary Advanced Technology Corporation (ATC) and National Earthquake Hazard Reduction Program (NEHRP) methodologies, and investigative technologies and inspection tips covering various types of construction.

Chapter

7a

Evaluation Guidelines

Frank E. McClure

Evaluation Requirements

The need to evaluate existing Department of Energy (DOE) facilities is described in DOE Order 5480.28, *Natural Phenomena Hazards Mitigation* (Ref. 1) *and* DOE-STD-1020, *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities.* (Ref. 2). DOE Order 6430.1A, *General Design Criteria* (Ref. 3), requires that earthquake resistant design for DOE buildings and other structures be determined in accordance with the procedures contained in DOE-STD-1020 and the *Uniform Building Code* (UBC) (Ref. 4). DOE requires that structures survive natural phenomena events commensurate with their importance and the hazard they present.

DOE Order 5480.28 requires DOE facilities including *Structures, Systems and Components* (SSCs) to be designed and constructed to withstand the effects of natural phenomena hazards (NPH) including earthquakes. An objective for all SSCs is to prevent loss of structural integrity endangering life safety. An additional objective is to prevent loss of capability to perform functions consistent with:

- Importance to safety for workers and the public
- Impact on the environment
- Repair and replacement costs
- Programmatic mission.

Existing DOE facilities (SSCs) are to be reevaluated under the following circumstances:

- The SSC was originally designed without adequate NPH design and construction standards
- The function of the SSC has changed to a performance category (PC) with more stringent requirements
- There has been a significant change in understanding that results in an increase in the site hazard
- When an SSC has been subjected to an unresolved safety question
- A significant physical change in the SSC has been caused by an addition, a modification, deterioration, or a damaging NPH event.

Reevaluation in keeping with these circumstances can be *triggered* by the preparation or upgrading of a Safety Analysis Report (SAR).

DOE Order 5481.1B, *Safety Analysis and Review System* (Ref. 5), covering non-nuclear facilities and DOE Order 5480.23, *Nuclear Safety Analysis Reports* (SARs) (Ref. 6), covering nuclear facilities, require seismic evaluations of facilities in preparation of SARs when needed for

authorization to operate a DOE facility. The definition of a nuclear facility as stated in DOE Order 5480.23 is a reactor or a facility with activities or operations which involve radioactive and/or fissionable materials in such form that a nuclear hazard potentially exists to the employees or general public. The threshold quantities which determine whether a facility is nuclear or non-nuclear are covered by DOE standards.

SSCs must be evaluated for the effects of earthquakes, including ground shaking, differential ground motion induced by fault displacement, liquefaction, slope instability, ground lateral spreading, compaction, and subsidence. For existing SSCs, an assessment must be made for the *as-is* condition. This is required to determine compliance or noncompliance with seismic design provisions and to assess the need for strengthening.

DOE-STD-1020 specifies five performance categories (PCs) for SSCs ranging from 0 through 4, with a graded performance goal assigned to each category based upon the consequences of failure or damage. Facilities assigned to PC-0 must have no need for seismic protection for safety, mission, or cost concerns. *There must be no life safety issues with an SSC classified in PC-0.*

Performance Categories 1 and 2 generally correspond to *Uniform Building Code* (UBC) occupancy categories for *Standard Occupancy Structures* and *Essential Facilities*, respectively. Essential Facilities generally include hospitals, fire and police stations, and other facilities where continuity of function is necessary for emergency operations.

SSCs that involve activities in non-nuclear facilities that have the potential for uncontrolled release of hazardous materials to the environment or SSCs that have a significant programmatic impact (highly valuable) may be included in PC-3. This category also may be used to achieve superior performance dictated by mission objectives or cost considerations. PC-3 is consistent with current practice for reevaluation of commercial plutonium facilities. It is more conservative than the UBC requirements for *Essential Buildings* and less conservative than requirements for civilian nuclear power plants.

The seismic provisions for PC-4 are consistent with those used for reevaluation of civilian nuclear power plants.

It is important to remember that SSCs must be considered and evaluated individually. For example, a structure (building) may house a process or experiment (system and components) that utilizes highly toxic gases. The building structure may be in PC-2 and the process or experiment in PC-3.

For Performance Categories 3 and above, the seismic loadings must be based upon site-specific hazard studies (Refs. 1 and 2). For Performance Categories 1 and 2, earthquake loadings may be established by common geotechnical methodologies acceptable under UBC provisions.

Performance Categories 1 and 2 (UBC occupancy categories for *Standard Occupancy Structures* and *Essential Facilities*, respectively) are discussed here to illustrate recommended evaluation methodologies. These performance categories include 95% or more of the total number of facilities owned or leased by DOE. The objectives of the *Seismic Safety Manual* are restricted to facilities that would normally be placed in PC-1 or PC-2. However, many principles and recommendations that are discussed are good practice applicable to all performance categories. For detailed guidance with respect to PC-3 and 4 facilities, refer to Reference 2.

Evaluation Guidance

Background

Prior to 1987, there were no national standards or guidelines used to evaluate existing buildings for seismic resistance. Consequently, structural engineers experienced in earthquake engineering developed individualized evaluation methodologies and reporting formats based upon personal assessment of damage to buildings caused by earthquakes. The first standard evaluation report forms were developed and used by structural engineering consultants to provide data to set earthquake insurance premiums for the insurance industry. Generally, buildings were reviewed and evaluated for compliance with seismic criteria and design provisions in the building code in effect at the time of the investigation. Some

engineers utilized more specific criteria and checklists for items to be inspected based upon personal observations of earthquake damage.

DOE Standard 1020

DOE-STD-1020 provides evaluation criteria for the seismic design and evaluation of PC-2 and lower SSCs based upon the provisions of the UBC. The evaluation methodology for these facilities is set forth in step-by-step procedures. The total seismic base shear to be used in the evaluation is defined therein, and is normally not less than prescribed by the UBC for new buildings. There are, however, alternative provisions for lowering evaluation criteria, which include:

- Using a hazard exceedance probability of twice the value recommended for the performance category specified for the SSC being considered

- Changing the usage or occupancy of the facility or SSC so that a less conservative performance category is required

- Conducting a more sophisticated evaluation to justify that less conservative criteria will adequately fulfill required performance goals.

If the evaluation shows that the required performance goals for the SSC cannot be achieved in the *as-is* condition, then the SSC must be strengthened to resist the full criteria required for the design of new facilities.

Generally, the base shear prescribed in DOE-STD-1020 for evaluation is larger than the base shear specified in ATC 14 (Ref. 7), ATC 22 (Ref. 8), and the *NEHRP Handbook* (Ref. 9) *for Seismic Evaluation of Existing Buildings;* thus, the DOE-STD-1020 evaluation criteria are generally more conservative. The ATC and NEHRP documents utilize a lower base shear for evaluation of existing buildings than for the design of new buildings. The reasoning for this is somewhat intuitive and arbitrary. It is based upon the idea that less conservatism can be tolerated in an existing building because it can be strengthened only at substantial cost in money and disruption of use, whereas extra conservatism can be justified in new buildings because it can be incorporated for little increase in cost. This concept has gained acceptance in the private sector because it is believed that a building should be substantially below the current code standard before triggering the requirement for a costly upgrade and because a higher level of earthquake damage in an existing building is usually more acceptable to the public.

Before one accepts this premise, one should appraise the cost benefits of the reduction of damage provided by seismic strengthening and reasonably ensure that life-safety threats are reduced. Usually an evaluation of the consequences of failure provides the most reasonable basis for design criteria for strengthening an existing building as well as for designing a new one. It should be noted that the reduced base shear used by ATC and NEHRP to evaluate existing buildings is not the level of force to be used if retrofit is necessary. As well, criteria required for strengthening DOE facilities must be the same as those required for new structures. This is generally true for other jurisdictions outside DOE that utilize the UBC.

Unlike ATC and NEHRP, DOE 5480.28 and DOE-STD-1020 require that existing facilities be evaluated and strengthened, if necessary, based upon the seismic loadings for new facilities. Some options are provided for dealing with marginal facilities that do not meet the evaluation provisions, but generally these alternatives still require that the performance goals be met.

ATC 14 Evaluation Methodology

The *Applied Technology Council* (ATC) is a nonprofit corporation established in 1971 through the efforts of the *Structural Engineers Association of California* (SEAOC) to assist design practitioners in structural engineering. In 1987 ATC published ATC 14 for *Evaluating the Seismic Resistance of Existing Buildings,* (Ref. 7). It was one of the first evaluation procedures to be based directly on the performance of buildings in past earthquakes. This procedure addresses only life-safety concerns; it does not evaluate potential damage. It is based on the seismic provisions in the 1985 edition of the *Uniform Building Code* using *working-stress* evaluation criteria. Because DOE 6430.1A specifies the UBC for seismic design, ATC 14 has been used to evaluate DOE facilities. The evaluation statements in ATC 14 are consistent with the working-stress approach in the UBC.

ATC 14 also includes a 1987 state-of-the-art review of earlier evaluation methodologies and is an excellent reference for observations about the performance of 15 common structural building types during past earthquakes. These building types are:

1. Wood, light frame
2. Wood, commercial and industrial
3. Steel moment frame
4. Steel braced frame
5. Steel light frame
6. Steel frame with concrete shear walls
7. Steel frame with infill shear walls
8. Concrete moment frame
9. Concrete shear walls
10. Concrete frame with infill shear walls
11. Precast/tilt-up concrete walls with lightweight flexible diaphragms
12. Precast concrete frames with concrete shear walls
13. Reinforced masonry bearing walls with wood or metal deck
14. Reinforced masonry bearing walls with precast concrete diaphragms
15. Unreinforced masonry bearing wall buildings.

The fundamental approach in ATC 14 is to ascertain whether there is a complete lateral-force-resisting system with a coherent load path and whether appendages and veneers are properly attached. The adequacy of seismic performance of the structural system and components, and exterior and interior nonstructural systems, is expressed in terms of an *earthquake capacity ratio*, which is the ratio of seismic capacity to seismic demand for critical structural members and connections.

ATC 14 sets up standard definitions for the 15 building types and provides a list of questions for each type of building that is designed to uncover significant physical weaknesses. These *Evaluation Statements* are written so a *true* response implies that the building is adequate in the area of concern and does not pose a *life-safety* hazard. A *false* statement indicates an area of concern that might be a *life-safety* hazard, and therefore needs detailed study. Following each statement are appropriate recommendations for detailed analysis with corresponding acceptance criteria to be used. By following this process, weak links in a building's structural system are identified and related to *life-safety* hazard, based upon the probability that one or more of the following will occur.

- The entire building collapses
- Portions of the building collapse
- Components of the building fail and fall
- Exit and entry routes are blocked, preventing evacuation and rescue of the occupants.

ATC 22 Evaluation Methodology

ATC 22, *A Handbook for Seismic Evaluation of Existing Buildings*, is a second-generation document. It is built upon ATC 14 by refining the procedures, expanding the commentary, and incorporating *strength design* concepts of the 1988 edition of the NEHRP *Recommended Provisions for Development of Seismic Regulations for New Buildings* (Ref. 10). The document format was modified from ATC 14 into a handbook for easier use by evaluating engineers. The 1994 edition of the *NEHRP Provisions* was published in 1995 (Ref. 11).

NEHRP Evaluation Handbook

The *NEHRP Handbook for Seismic Evaluation of Existing Buildings* (1992), hereafter referred to as the *NEHRP Handbook* (Ref. 9), is a more recent evaluation methodology than ATC 14 and ATC 22 and builds on these two previous methodologies.

The *NEHRP Handbook*, also referred to as FEMA 178, was developed by the Federal Emergency Management Agency (FEMA), using the Building Seismic Safety Council (BSSC) consensus process, to develop an updated revised version of ATC 22, *A Handbook for Seismic Evaluation of Existing Buildings*. It covers:

- Structural and nonstructural elements
- Foundation and geologic site hazards
- Checklists, diagrams, and sketches to aid in the seismic evaluation.

The *NEHRP Handbook* is similar to ATC 22 and follows the same evaluation procedures, using *true* or *false* evaluation statements. The evaluation statements are based on *strength design*. Seismic evaluation criteria are based on the 1988 NEHRP report, *Recommended Provisions for the Development of Seismic Regulations for New Buildings* (Ref. 10). The NEHRP *Handbook* for existing buildings utilizes a base shear of 67% of the *NEHRP Provisions* for new long-period buildings and 85% of the *NEHRP Provisions* for new short-period buildings. It sets forth the evaluation procedures in considerable detail, including:

- Site visit and data collection
- Selection of evaluation statements
- Follow-up field work
- Structural analysis for evaluation statements that have been found to be *false*
- Final evaluation
- Preparation of evaluation report.

The *NEHRP Handbook* presents a general procedure that can be applied to almost any type of building construction. Generally, it involves the use of a set of statements evaluating:

- The basic building system
- Vertical systems resisting lateral forces
- Diaphragms and/or horizontal bracing systems
- Structural connections
- Foundation and potential geologic site hazards
- Nonstructural elements that are falling hazards that involve life-safety concerns.

These evaluation statements are specially tailored to the 15 common building types described in ATC 14. They provide a detailed *checklist* of potential weaknesses for each building type and are more comprehensive than the evaluation statements found in ATC 14 or ATC 22.

Recommendations for Evaluation of Existing DOE Facilities

Summary

For DOE Performance Categories 1 and 2, it is recommended that the evaluation provisions of DOE-STD-1020 be followed using the *NEHRP Handbook* evaluation statements as a *checklist* to ensure that all life-safety issues are addressed in the evaluation. Seismic-loading criteria should be the same as those used by DOE-STD-1020 for new facilities, unless reduced performance goals are permitted. Generally, it is not recommended that one use more rigorous techniques to remove conservatism from the evaluation simply to prove that the SSC is adequate. The decision to require retrofit of marginal structures should be based on the consequences of failure and good risk management principles.

NEHRP evaluation statements also include *Quick Stress Checks* to compare calculated stresses with allowable stresses for critical structural members and connections. For example, in a reinforced concrete shear-wall building, there is a *Quick Stress Check* for the shearing stress in the concrete shear walls. However, instead of using the *NEHRP Handbook* base shear lateral force for calculating the stresses, the base shear lateral force should be obtained from the procedures in DOE-STD-1020. DOE-STD-1020 also provides guidance for comparing the capacity of various critical members and their connections with the seismic demand for these elements to resolve the *false* evaluation statements in the *NEHRP Handbook* . Evaluation options in DOE-STD-1020 can be used to reduce the evaluation seismic forces, but performance goals must still be achieved.

DOE-STD-1020 requires that DOE facilities in Performance Category 3 and higher be evaluated using elastic dynamic analysis based upon site-specific median response spectra. The total dead plus design live load and seismic force demands for each member and its connection is compared to the capacity given by

ultimate strength code-type provisions, including strength reduction factors. Step-by-step evaluation procedures are presented in DOE-STD-1020.

The *NEHRP Handbook* evaluation statements may be used as *checklists* for PC-3 and -4 facility evaluations in a similar manner as was recommended for PC-1 and -2 facilities. The *Quick Stress Check, true* or *false*, evaluation statements should be based on DOE-STD-1020 demand and capacity values. The general evaluation statements per se are very useful and practical.

References

1. *Natural Phenomena Hazards Mitigation*, DOE Order 5480.28, U.S. Department of Energy, Washington, D.C., January 1993.

2. *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities*, DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.

3. *General Design Criteria*, DOE Order 6430.1A, U.S. Department of Energy, Washington, D.C., April 1989.

4. *Uniform Building Code*, International Conference of Building Officials, Whittier, California, 1994.

5. *Safety Analysis and Review System*, DOE Order 5481.1B, U.S. Department of Energy, Washington, D.C., 1986.

6. *Nuclear Safety Analysis Reports*, DOE Order 5480.23, U.S. Department of Energy, Washington, D.C., 1992.

7. *Evaluating the Seismic Resistance of Existing Buildings*, ATC-14, Applied Technology Council, Redwood City, California, 1987.

8. *A Handbook for Seismic Evaluation of Existing Buildings*, ATC-22, Applied Technology Council, Redwood City, California, 1990.

9. *NEHRP Handbook for Seismic Evaluation of Existing Buildings*, FEMA 178, Federal Emergency Management Agency and Building Seismic Safety Council, Washington, D.C., 1992.

10. *NEHRP Recommended Provisions for Development of Seismic Regulations for New Buildings*, 1988 edition, FEMA 222, Federal Emergency Management Agency and Building Seismic Safety Council, Washington, D.C., 1988.

11. *NEHRP Recommended Provisions for Development of Seismic Regulations for New Buildings*, 1994 edition, FEMA 222, Federal Emergency Management Agency and Building Seismic Safety Council, Washington, D.C., 1995.

Chapter

7b

Evaluation Methodologies

Harold M. Engle, Jr.

Introduction

Evaluation of structures for seismic resistance requires investigators to have a clear concept of which structural systems provide assured earthquake resistance. This concept can best be developed by observing the performance of structures in past earthquakes.

Prior to embarking on an earthquake safety program, facility managers should confirm that the structural engineering consultants have a thorough knowledge and understanding of DOE-STD-1020, *Natural Phenomena Hazards Design Criteria for DOE Facilities* and of the performance goal(s) for each building under investigation. If the performance goal for a given building is solely life-safety, extreme conservatism concerning such items as horizontal diaphragm chords, diaphragm collectors, reinforced concrete shear wall demands, and capacity ratios are not warranted. Focusing on deficiencies in damage-control items and allocating funds for their correction can divert already limited funding from structures truly requiring seismic rehabilitation for life-safety.

The awareness and concern of design professionals regarding earthquake hazards have not followed a smooth path. In California, the period between the great San Francisco earthquake of 1906 and 1933 was one of reconstruction and rapid growth, but the large majority of the design profession (architects and engineers) did not incorporate seismic resistance in their projects. The Long Beach earthquake of 1933, which stimulated the subsequent passage of the Field Act governing public school design in California, altered this to some degree. From 1933 to 1971, significant seismic design provisions were incorporated into the *Uniform Building Code* (UBC) used throughout the West. Unfortunately, outside of public school design in California, the efficacy of all aspects of the *Code* lateral force provisions, their enforcement, and the attention and judgment given the provisions by many design professionals were not conducive to achieving consistent results.

Outside California, the majority of the design profession was either ignorant of, or ignored, seismic design. However, during this period, many buildings were constructed that can be classified as earthquake resistant. Earthquake resistance was accomplished either by accident or by designers who had studied past building performance and used judgment in applying code provisions, realizing that code provisions were minimum and sometimes fallible, and that adherence to them did not guarantee earthquake-resistant structures.

The spectacular failure of freeway-overpasses, hospital buildings, and other structural collapses produced by the 1971 San Fernando, California, earthquake made the entire design profession, including code-writing bodies and regulatory agencies, suddenly much more aware of earthquake hazards. This awareness was subsequently reinforced by the 1985 Mexico City, 1989 Loma Prieta, and 1994 Northridge earthquakes. News media coverage of seismic design adequacy in nuclear-power-plant, liquid petroleum gas, natural gas, dam, and other potentially hazardous facilities has greatly intensified public interest and concern. The pendulum has swung from ignorance and apathy in some quarters to extreme concern. Unfortunately, this has resulted in over-conservatism in design parameters used by some designers and regulatory agencies. This over-conservatism does not correlate well with building performances in past earthquakes. For example, the 8-inch concrete shear-wall buildings at the Veterans Administration Hospital withstood the 1971 San Fernando earthquake without structural damage. Seismic design criteria for the new, post-earthquake Olive View Hospital were so stringent that these same undamaged VA buildings could not satisfy them.

The intent of the seismic evaluation process should be to identify actual seismic deficiencies, not to make blanket condemnations.

During the period since the *Seismic Safety Guide* was first published in 1983, a progression of seismic evaluation methodologies has been developed by the *Applied Technology Council* (ATC) and the *Federal Emergency Management Agency* (FEMA). A listing and description of each is contained in Chapter 7a. The recommended method is the *Building Seismic Safety Council's* (BSSC) *National Earthquake Hazard Reduction Program* (NEHRP) *Handbook for Seismic Evaluation of Existing Buildings*, FEMA 178. Qualifiers regarding the input loadings to be used for evaluations are contained in DOE-STD-1020. The NEHRP methodology involves 15 standard building types enumerated in Appendix A of this chapter. The following sections offer brief descriptions of selected building types with comments on their earthquake resistance.

Concrete Construction

Type 8. Concrete Moment Frame: Nonductile concrete moment resisting frame construction has suffered numerous total collapse failures in past earthquakes. The collapse of four buildings in the 1967 Caracas earthquake, the collapse of the Olive View Psychiatric Unit (San Fernando, 1971), and the collapse of the May Company Parking Structure (Whittier, 1987), Figs. 7b-1 and 7b-2, are among a long list of world wide failures of this type of construction. The brittle failures of these nonductile concrete-frame buildings illustrated the need for ductile concrete detailing, including continuity of longitudinal top and bottom steel in horizontal members, adequate confinement of vertical column bars, consideration for seismic effects on *nonstructural* elements, and use of realistic lateral force coefficients. Both ductile and nonductile concrete frames are subject to relatively large displacements, with inelastic energy dissipation occurring after fairly high velocities are reached. In the case of the Olive View Psychiatric Unit, even if the frame had been designed as a ductile frame, it is doubtful that collapse of the structure would have been prevented without the use of higher lateral force coefficients. For this reason, existing buildings with moment-resisting concrete frames, both nonductile and ductile, should be carefully investigated for earthquake safety.

Type 9. Concrete Shear Walls: This type of construction has an excellent performance record in earthquakes when adequate attention has been given to design detail. When the term *shear wall* is used, it should be assigned to walls in which the principal vertical resisting elements, when stressed beyond the elastic limit, provide inelastic energy absorption without collapse hazard. True shear-wall action occurs when wall elements have a height-to-depth (H/D) ratio less than or equal to 4 or have specially designed boundary members. Wall grid reinforcing must be sufficiently anchored to edge trim bars and foundations, so that diagonal cracking occurs before flexure or overturning failure. A series of short, closely spaced columns confined by deep spandrel beams similar to the longitudinal walls of the Hakodate University classroom building (Fig. 7b-3) should not be construed as shear-wall construction.

Present practice in shear-wall design has tended toward the use of high reinforcing steel

Fig. 7b-1. Collapse of the Olive View Psychiatric Unit, San Fernando, California earthquake, 1971.

ratios and heavily reinforced boundary elements. The practice of using large-diameter vertical bars, #8 and larger, in double-curtain wall reinforcement is questionable. These large vertical bars must be confined similar to column and boundary element reinforcement to preclude buckling of the vertical wall reinforcement. In-plane splitting occurred in a 15-story telephone building in Oakland, California (Loma Prieta, 1989); the wall in question contained #8 reinforcing bars at 12 inches on center each way, each face, and was constructed of light-weight concrete.

The failures of the Four Seasons Apartments (Anchorage, Alaska, 1964) and the Olive View Hospital stair towers (San Fernando, 1971) were overturning failures, not failure of shear walls per se. Shear-wall construction in low-rise buildings (approximately 160 feet in height or less) can provide truly earthquake-resistant construction. Shear-wall construction is capable of limiting nonstructural damage and precluding the possibility of ground-vibration-induced collapse. Figs. 7b-4 (Industrial Bank of Japan, Tokyo, 1923), 7b-5 (Stanford Avenue School, Southgate, California, Long Beach, 1933), 7b-6 (Knik Arms Apartment Building, Anchorage, Alaska, 1964), 7b-7 (AT&T, Coalinga, 1983), and 7b-8 (Pacific Telephone, Watsonville, Loma Prieta, 1989, 1.24g peak roof acceleration) are representative of sound concrete shear-wall construction. All of these structures survived the major quakes noted with little or no damage. The Industrial Bank of Japan had numerous interior concrete shear walls, exterior concrete walls, and a complete vertical load-carrying structural steel frame. This building was designed by Dr. Naito, in 1920, to resist earthquake forces (base shear) equal to 10% of the weight of the building.

Probably one of the most important lessons learned from past earthquakes is that shear-wall buildings, particularly older shear-wall buildings designed for lower lateral forces, should have a regular (concentric) shear-wall configuration, both for shear-wall layout and for shear-wall element perforations.

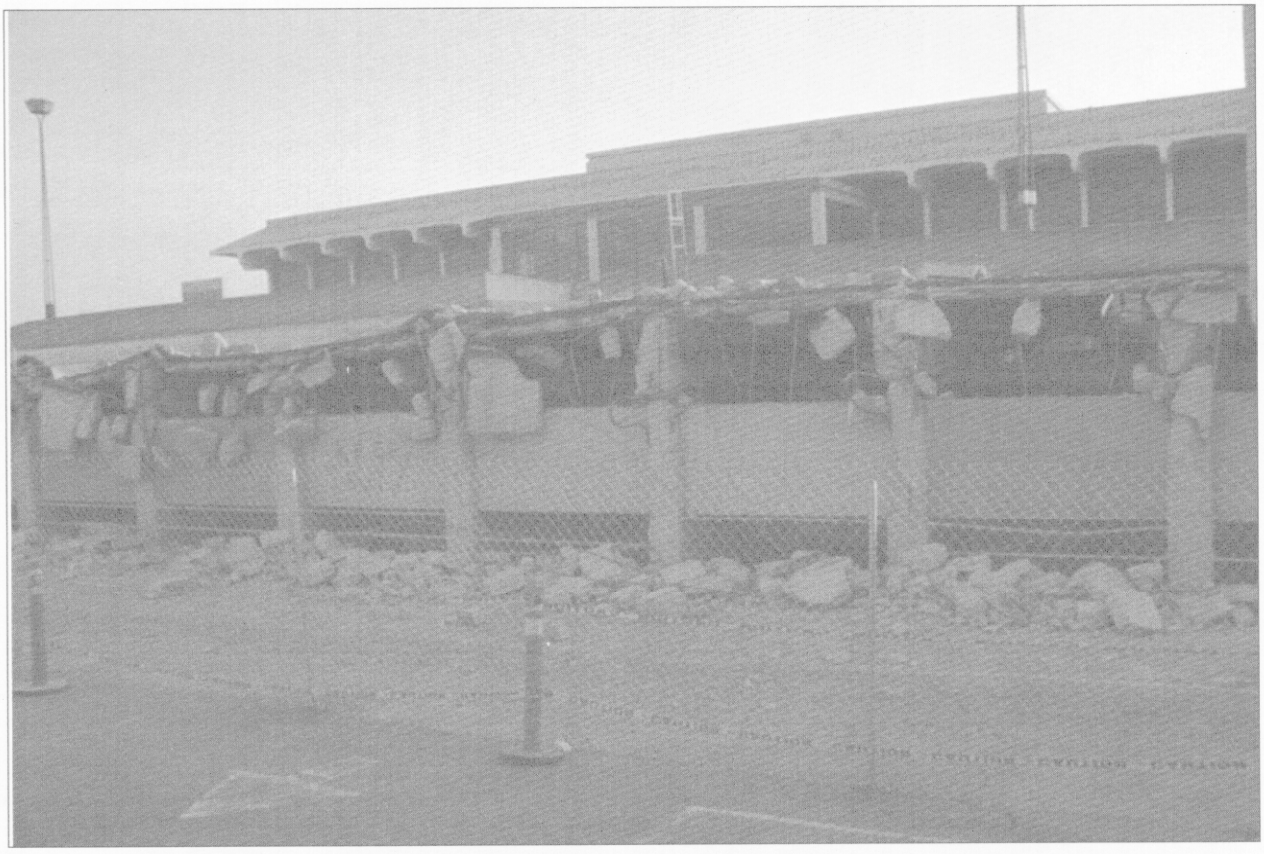


Fig. 7b-2. Collapse of the May Company Parking Structure, Whittier, California earthquake, 1987.

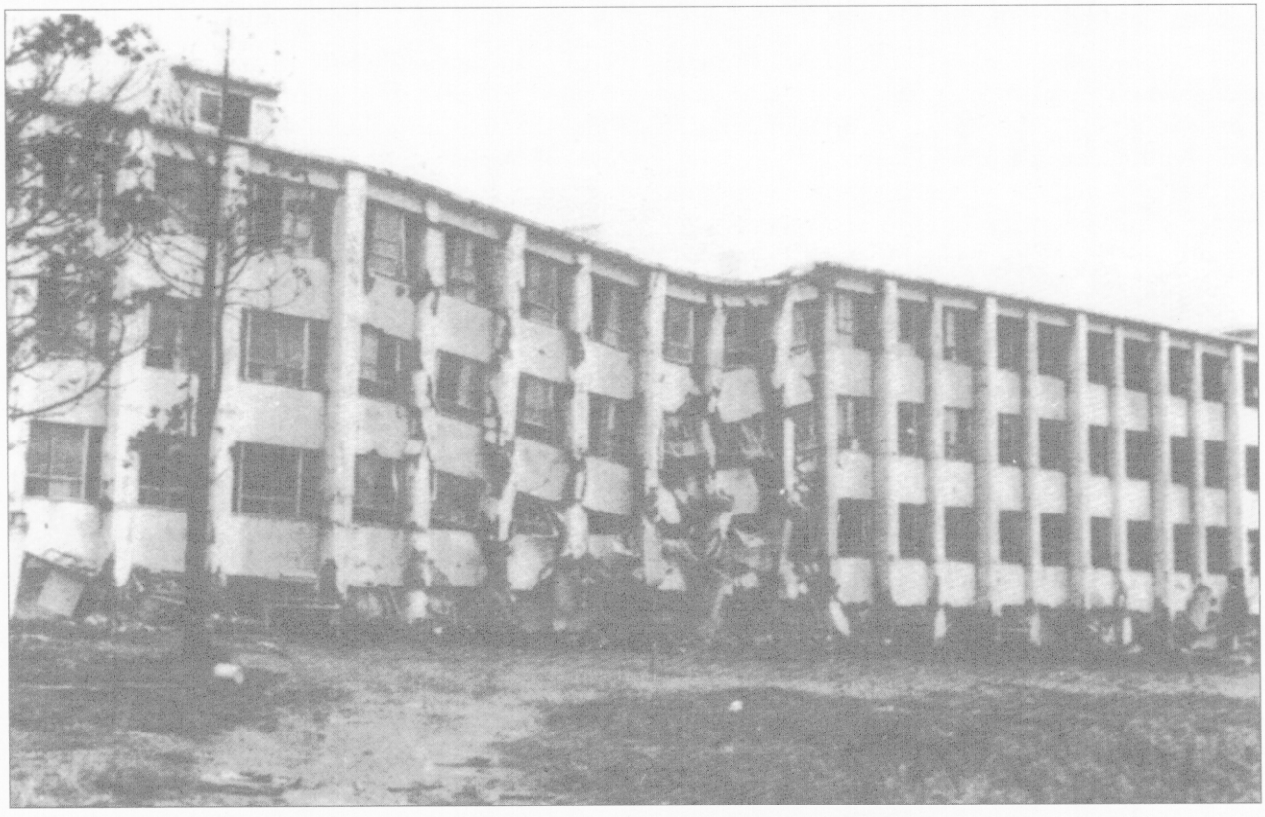


Fig. 7b-3. Hakodate University classroom building, Tokachi-Oki earthquake, Japan, May 1968.

Fig. 7b-4. Industrial Bank of Japan, Tokyo, 1923.

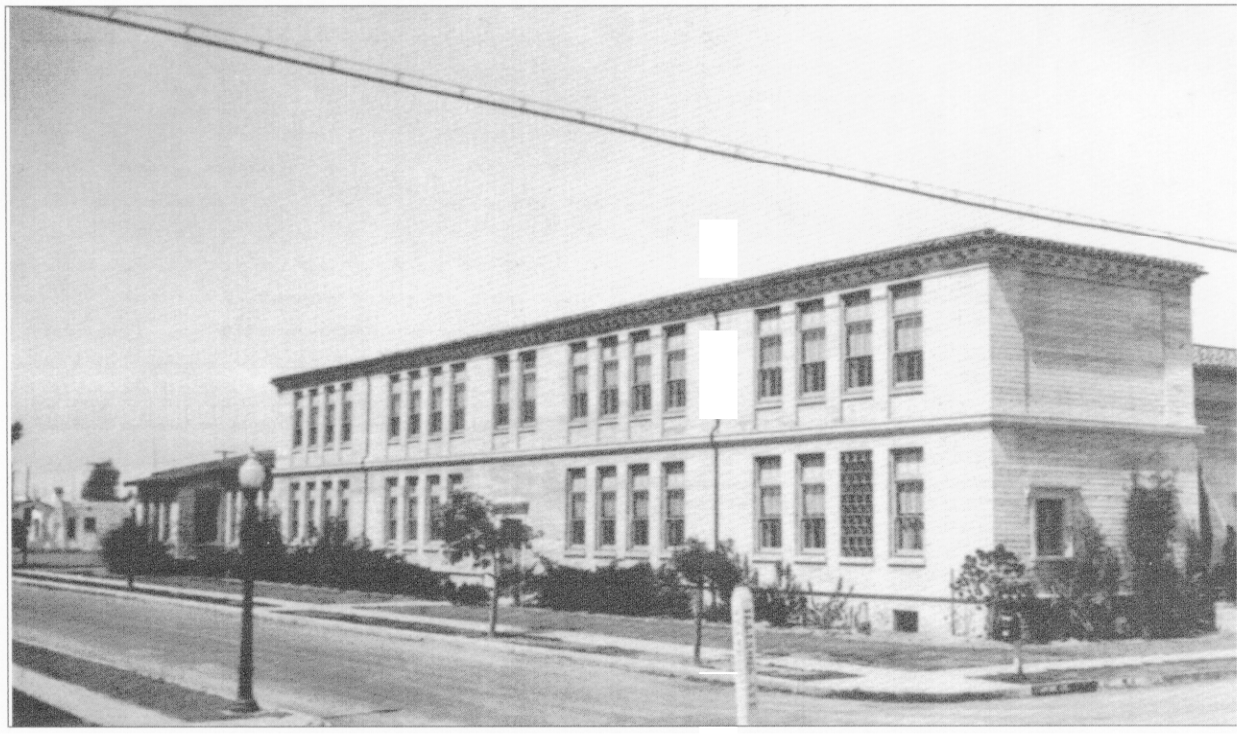


Fig. 7b-5. Stanford Avenue School, Southgate, California, Long Beach earthquake, 1933.

Fig. 7b-6. Knik Arms apartment building, Anchorage, Alaska earthquake, 1964.

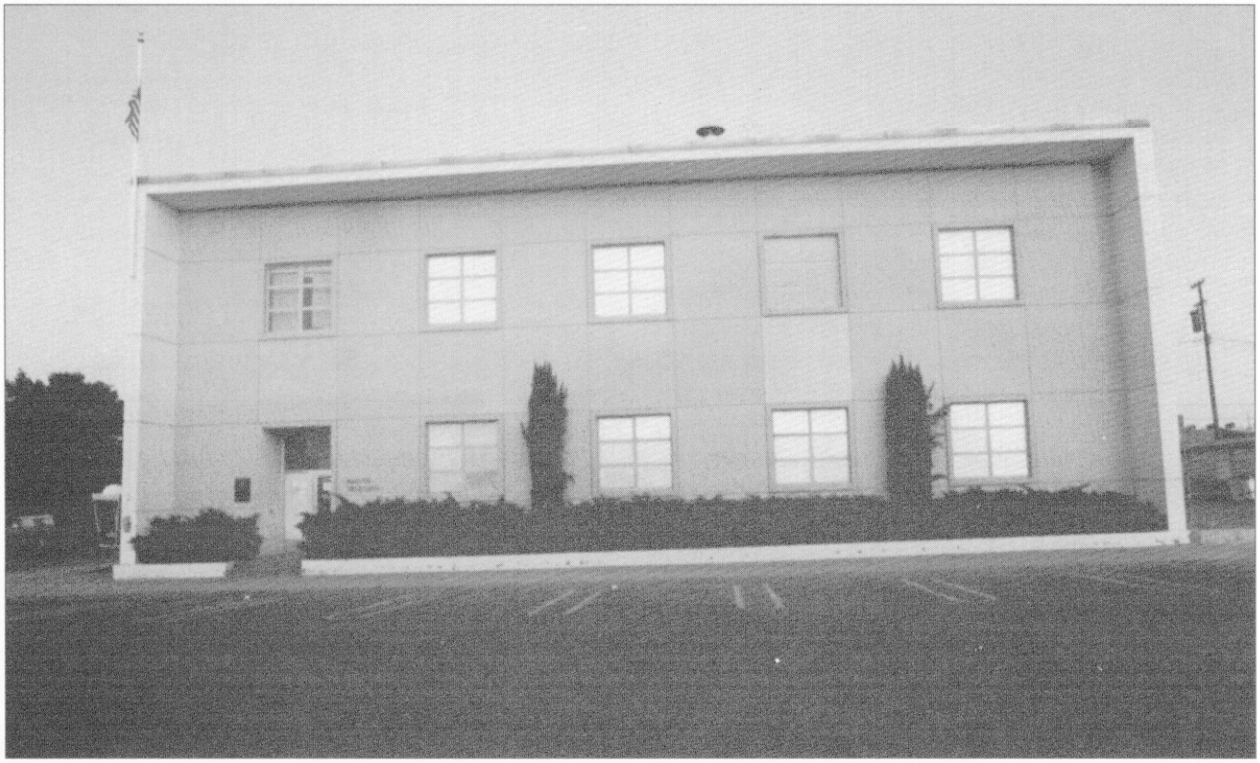


Fig. 7b-7. AT&T building, Coalinga, California earthquake, 1983.

Fig. 7b-8. Pacific Telephone building, Watsonville, California, 1989.
Credit: SEAOC, *Reflections on the Loma Prieta earthquake of October 17, 1989.*

There is a significant difference in the effects of earthquake forces on Type 8 concrete-frame and Type 9 concrete shear-wall buildings. Type 8 nonductile concrete-frame buildings may collapse and kill the occupants. Type 9 concrete shear-wall buildings, in general, provide good examples of repairable earthquake damage that structural engineers like to photograph.

Type 10. Concrete Frame with Infill Masonry Shear Walls: This building type can have extensive variations in performance. Those with well-designed reinforced masonry infill shear walls can perform as well as Type 9 structures. Conversely, nonductile concrete frames with unreinforced masonry infill walls have suffered total collapse in past earthquakes. A common design error in this construction type occurs when no consideration is given to the interaction of the frame and infill wall, with the result that actual forces transmitted to the frames from infill walls greatly exceed original design forces.

Types 11 and 12. Precast Construction: The greatest inherent problem with precast-prestressed construction is the lack of adequate strength in the connection of components. Economic pressures often influence the quality of the connection detail. When the decision is made to use precast-prestressed components, many designers, to justify alleged economy, feel obligated to keep the number of connections to a minimum, so that convenience and speed of erection become the dominant design criteria. A few weld plates with anchors embedded 3 to 4 inches into the edge of a component member cannot equal the strength and toughness of a connection made with conventional reinforcing spaced 12 inches on center and properly embedded in a cast-in-place closure pour. Both precast-prestressed and cast-in-place, post tensioned construction are subject to elastic strain and inelastic (creep) strain caused by prestress forces. When this type of construction is used for floor and roof systems that frame into relatively stiff columns or walls, problems associated with these

strains often develop. Both elastic and creep shortening can cause high moments and shears in columns, and high shear stresses in walls and floor diaphragms. There are numerous cases in which buildings have literally been destroyed by this action. This condition severely limits the capacity of the structure to resist earthquake forces.

Tilt-up precast concrete wall construction is generally combined with flexible, highly-stressed wood diaphragms. For *damage control,* particular attention should be given to the adequacy of the diaphragm chord splice and collector member connections. This caveat pertains to tilt-up construction with welded chord splices, (both pre- and post-1973 UBC). Post 1973 UBC tilt-up construction utilizing proprietary steel sheet metal strap anchors for precast wall anchorage to wood diaphragms, performed poorly in the 1994 Northridge earthquake. Tilt-up buildings with this type of anchorage *require* upgraded wall-to-roof anchorage.

Current construction practices generally use welded splices for horizontal wall panel bars used as diaphragm chords. Numerous weld splice details are used. Some of the details utilized will induce eccentricities at the weld splices. Documented proof of full-time field welding inspection is necessary to ensure the integrity of the welds. Welding inspection records should include preheat used. If documentation of inspection is unavailable, the concrete dry pack should be removed and the welds visually inspected or nondestructive tested (NDT) in the case of full penetration welds.

Steel-Frame Construction

Numerous framing and bracing schemes associated with steel-frame construction have been devised. Five types are listed in the *NEHRP Handbook: Type 3 - Steel Moment Frame; Type 4 - Steel-Braced Frame; Type 5 - Light Steel Frame; Type 6 - Steel Frame with Concrete Shear Walls; and Type 7 - Steel Frame with Infill Masonry Walls.*

Relatively few ductile moment-resisting steel-frame structures (Type 3) had been subjected to major shocks until the recent January 1994 Northridge earthquake. This earthquake caused damage to 100 or more *Steel Moment Resisting Frame* (SMRF) buildings. Damage consisted of cracked welds and cracked columns at the field-welded beam-column joints. Intensive research and testing programs are underway to determine effective retrofit measures and improved beam-column welded joint details. The predominate damage mode disclosed to date appears to be cracking of the beam bottom-flange welds.

Evaluation of Type 3 buildings should include review of all available field weld-test data, visual inspection, and ultrasonic or X-ray testing of a percentage of beam-column joint welds. This requires removal of finishes and fireproofing at selected beam-column joint locations and removal of weld back-up plates.

No earthquake-induced collapses have occurred to Type 3 buildings. The damage to Type 3 buildings in the Northridge shock did not create immediate life-safety hazards; most of the damaged buildings remained functional.

In our opinion, blanket condemnation of all present SMRF (Type 3) structures is not warranted as a result of the damage incurred in the Northridge earthquake. These structures were subjected to severe ground shaking. In some instances peak ground acceleration exceeding twice the present UBC peak ground acceleration for seismic hazard Zone 4. The structures performed as intended by the present code philosophy of preventing life-safety hazards, while suffering significant structural damage.

The number of moment frames used in a given structure has a significant impact on the potential for earthquake damage to the structure. Type 3 buildings in which only a portion of the exterior bays are moment frames with no additional interior moment frames should be given increased scrutiny. The use of relatively few moment frames minimizes redundancy. In this case, an increased percentage of beam-column joints should be inspected for weld defects. Even if no defects are uncovered, this type of framing system should be considered a prime candidate for beam-column connection upgrade measures. It should be noted that buildings with a minimal number of moment frames are usually low-rise structures.

Previous earthquake damage to Type 3 construction occurred in the Cordova Building (Anchorage 1964), in which a column buckled at the southeast exterior wall corner (Figs. 7b-9 and 7b-10). This damage appears to have been caused by lack of consideration for the interaction of frame, rigid curtain-wall, and stair-framing members, and the torsional effects of asymmetrical core walls. In reviewing this type of construction, particular attention should be given to the adequacy of attachment of exterior curtain walls to the steel frame and possible adverse effects of stiff, nonframe elements such as stairways that are integrated with the framing.

In our opinion conformance with all the detail requirements of *Special Moment-Resisting Frame* design criteria contained in the 1991 UBC are not absolutely essential for life-safety considerations. Older low- and mid-rise structures lacking doubler plates required by present column web shear criteria are not a life-safety concern.

Single-story all-metal proprietary buildings (Type 5) with braced and moment-resisting frames normally present minimal earthquake hazard. Wind forces generally govern. However, this type of construction can be susceptible to seismic damage when tension-rod bracing is loose or has been removed.

Conventional buildings with *diagonally braced steel frames (Type 4)* can provide adequate seismic resistance if the bracing members and their connections are conservatively designed for realistic seismic forces. Bracing members must be designed to act as tension and compression members. Inadequate (or lack of) lateral bracing at horizontal and diagonal member joints can result in severe frame damage.

Buildings having well-designed *steel frames with concrete shear-walls (Type 6)* can have all of the drift and damage-control attributes of *concrete shear-wall (Type 9) structures* plus the added redundancy of a complete vertical load-carrying steel frame. The steel frame, in some cases, possesses significant lateral force resistance. Special attention should be given to modern dual-system buildings designed under the UBC with a K factor of 0.8. This design category was originally promulgated in the 1961 UBC. In many cases, this design method was used to achieve absolute minimum construction cost in midrise buildings. Many of these structures (pejoratively referred to as *two-bit frames*) have neither adequate shear walls nor adequate moment-resisting frames.

Type 7 steel-frame structures with unreinforced masonry infill walls, as noted in the *NEHRP Handbook*, are generally older buildings (50-plus years old). This type of structure is, in some instances, a *dual system* structure, with lateral resistance derived from both the masonry infill walls and the steel frame. This was the case in a significant number of buildings in San Francisco at the time of the 1906 earthquake. These buildings included the St. Francis Hotel, the 13-story Merchants Exchange Building, and the 11-story Mills Building. In the case of the Mills Building, the hollow-tile infill walls failed. Building collapse was averted by the moment-resisting column-to-beam connections, which

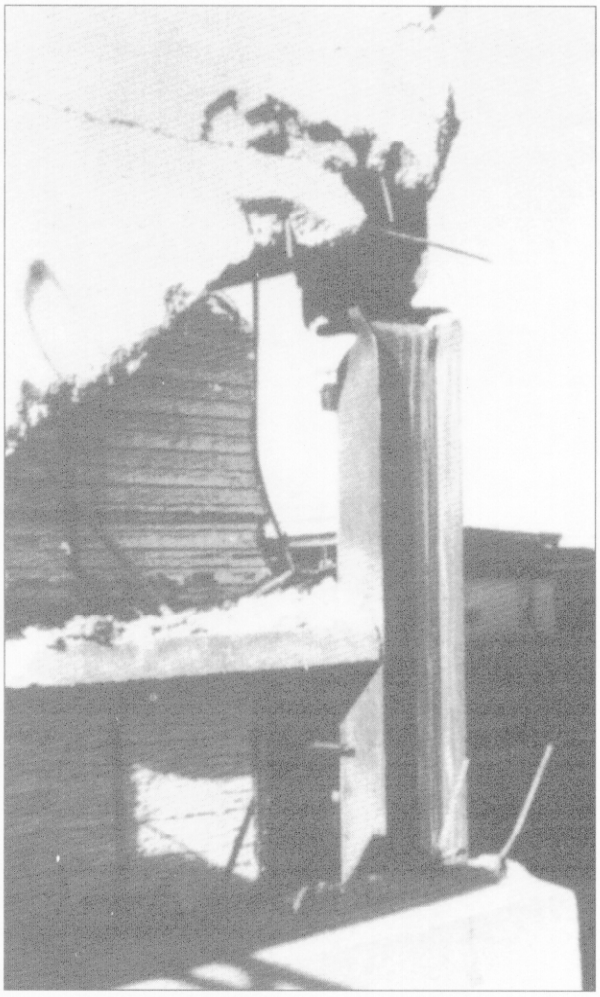


Fig. 7b-9. Cordova Building, Anchorage, Alaska earthquake, 1964.

were designed for wind forces. Later versions of this construction type (1920 and later) pose significantly greater earthquake collapse hazards because the steel frames are almost entirely devoid of moment-resistance. These minimal steel frames, when combined with extremely weak and brittle hollow tile or brick-cavity walls, are true collapse hazards.

Conversely, good-quality solid-brick masonry or concrete block infill walls, symmetrically distributed and properly integrated with the steel frame (even if diagonally cracked) will continue to dissipate seismic energy and significantly contribute to the building's lateral force resistance.

The term *steel-frame structure* is often interpreted as being automatically earthquake resistant. As noted above, steel frames are associated with a variety of bracing systems. Some of these systems provide assured seismic bracing, while others without seismic design provisions, or with serious design defects (Fig. 7b-11: Pino Suarez Complex, Mexico City, 1985), may be collapse hazards. Each building must be considered individually.

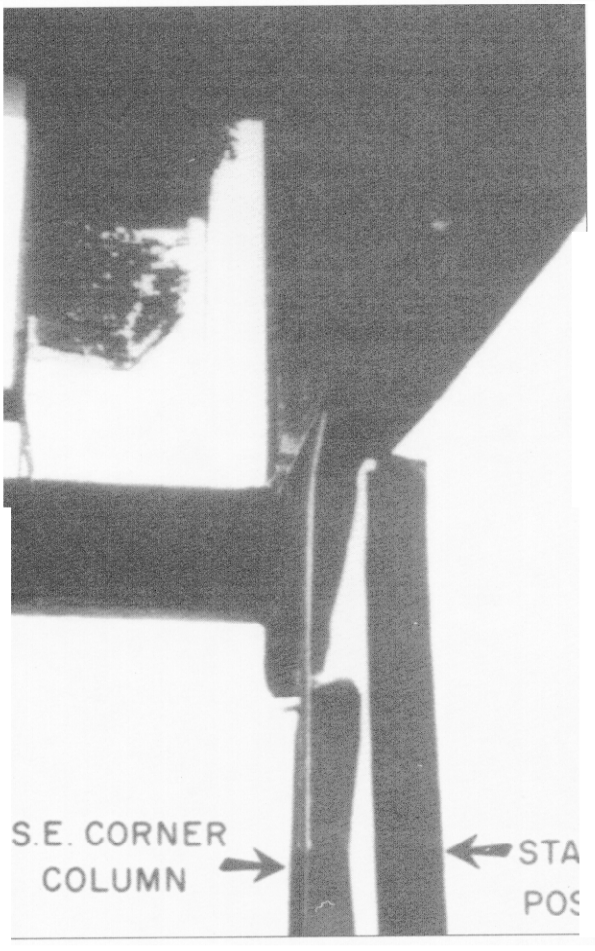


Fig. 7b-10. Construction detail, Cordova Building, Anchorage, Alaska earthquake, 1964.

Masonry Construction

Types 13 and 14, Reinforced Masonry Buildings: The NEHRP Handbook describes two types of reinforced masonry construction: one with flexible diaphragms (Type 13), and one with precast floor and roof framing with or without cast-in-place concrete topping (Type 14). Type 14 buildings should also include buildings with reinforced masonry bearing walls with rigid diaphragms consisting of cast-in-place concrete or steel decking with concrete fill.

Type 14 buildings, *precast concrete plank or tee floors without cast-in-place toppings* should be given special attention. Diaphragm action in this type of construction is, in many cases, dependent on field-welded connections at embedded weld plates. Weld plate anchorage and/or the quality of welds can be deficient to the extent that the diaphragms are truly a loose assemblage of individual precast members. Creep forces induced by pre-stressing can produce diaphragm weld failures and precast member failures.

Reinforced masonry bearing-wall construction, when properly designed, has performed well in past earthquakes. Both grout core and hollow concrete block units can be adequately reinforced using deformed reinforcing bars completely embedded in grout. The use of stacked bond is not recommended for concrete block construction. Fig. 7b-12 shows earthquake damage to a concrete block wall. This picture originally appeared in a report on the San Fernando earthquake, with the erroneous caption, "Damage to well-reinforced concrete block wall." This wall was, in fact, completely devoid of horizontal reinforcing, and the blocks were laid with stacked bond.

Reinforced masonry bearing-wall construction can perform as well as reinforced concrete. Fig. 7b-13 is an example of a well-designed reinforced masonry structure in Whittier, California. This structure did not suffer structural damage as a result of the strong October, 1987, Whittier Narrows earthquake.

Although reinforced masonry can perform as well as reinforced concrete, it is more subject to human error during construction than reinforced concrete. Reinforced masonry, therefore, should have continuous construction inspection for quality assurance.

Type 15: Unreinforced Masonry Bearing (UMB) Wall: All types of unreinforced masonry bearing-wall buildings, whether brick, concrete block, hollow tile, stone, or adobe, have proved to be extremely damage prone with some exceptions. Warehouse and industrial buildings with the following attributes, in general, survived the San Francisco 1906 earthquake with relatively minor damage.

1. Brick masonry laid in moderate-to-good quality mortar without voids or cavities

2. Those with walls containing low height to thickness (h/t) ratios, less than or equal to 12, except for top story walls with h/t ratios less than or equal to nine

3. Those with walls having a low percentage of openings

4. Those with walls anchored to floors and roof with *government* anchors spaced at approximately 5 feet on center

5. Those with site soil conditions not susceptible to an areal or differential subsidence during a seismic event.

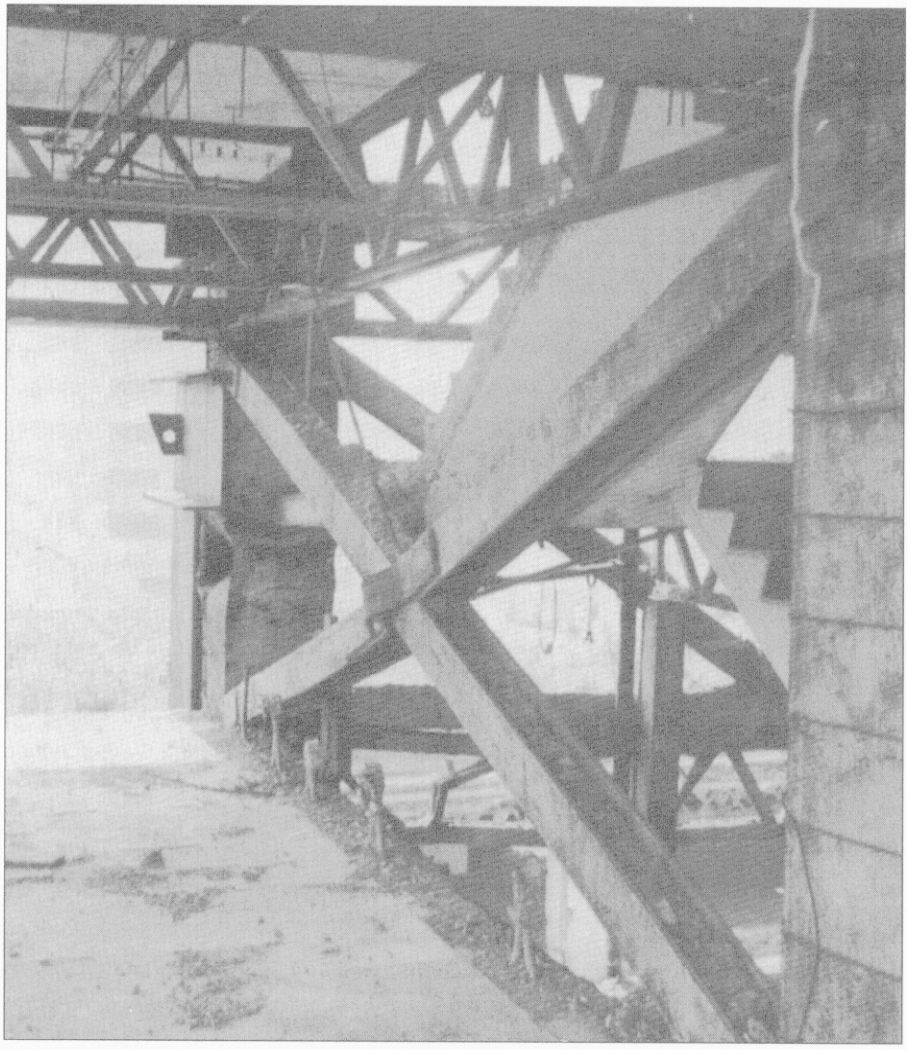


Fig. 7b-11. Pino Suarez Complex, Mexico City earthquake, 1985.
Credit: EERI Spectra, Vol. 5, No. 1, The *Mexico City earthquake.*

Fig. 7b-12. Stack-bond laid, hollow concrete block wall with no horizontal reinforcing; San Fernando, California earthquake, 1971.

Fig. 7b-13. Well-designed reinforced masonry structure with no structural damage, Whittier Narrows, California earthquake, October 1987.

Unreinforced masonry (URM) is inherently a loose assemblage of components. There are only two factors binding the walls together: mortar bond and super-imposed load. Attempts to provide retrofit anchorage in masonry with poor-quality material usually result in effectively anchoring only those masonry units directly connected to the anchor.

Outside the DOE community, the present applicable and consensus code for investigation and rehabilitation of Unreinforced Masonry Bearing-Wall Buildings is the *Uniform Code for Building Conservation* (UCBC) Appendix, Chapter One. This methodology is not recommended for use for DOE facilities.

Two methods for lateral force analysis are contained in UCBC Appendix, Chapter One: the *General-Procedure* method, which parallels the analysis in UBC, Chapter 23, and the *Special-Procedure* method. The *Special-Procedure* method is commonly called the ABK or *Rule of General Application* method. It differs from the *General-Procedure* method in its consideration of the value of cross-walls, vertical distribution of lateral force, and diaphragm capacity.

The *Special-Procedure* method was developed with concern for the societal effects of UMB rehabilitation factored into the design philosophy. The use of interior walls sheathed with archaic materials *(cross-walls)* for damping and shear resistance was an attempt to minimize interior retrofit work and the disruption to tenants in low-income housing. To a great extent, the rationale was based on the past performance of this building type in earthquakes (Fig. 7b-14: Hotel Californian, Santa Barbara, 1925). The failure depicted is quite common for this building type, which has a grid of closely spaced interior walls *(cross-walls)*. The *Special-Procedure* method allows this type of structure to be rehabilitated, in some instances, with a *bolts only* process; i.e., rehabilitation is limited to parapet bracing and anchorage of exterior masonry walls to floor and roof framing.

Both the *General-Procedure* and *Special-Procedure* methods allow unreinforced masonry to resist seismic-induced in-plane shear and out-of-plane flexure. A structure rehabilitated or checked using UCBC Appendix, Chapter One criteria is still an *unreinforced masonry building* even though it meets all UCBC criteria. It remains *vulnerable to earthquake damage*, albeit to a somewhat lesser degree. (See ATC-31, *Evaluation of the Performance of Seismically Retrofitted Buildings*.) Managers should be aware of the poor performance of rehabilitated URM buildings before deciding to use this methodology to evaluate existing URM buildings.

Buildings that have performance goals exceeding those for *Standard Occupancy* facilities should be evaluated using normal UBC criteria for seismic zones 3 and 4 (excluding UCBC Appendix, Chapter One.) These criteria do not allow the use of unreinforced masonry for structural purposes. Essentially the same criteria are used for California public school design and rehabilitation under the California Code of Regulations (CCR), Title 24.

Wood Frame Construction

Types 1 and 2: Wood, Light-Frame and Wood, Commercial-Industrial: As in the case of steel-frame construction, the inclination is to interpret wood-frame construction as earthquake safe. Because of its relatively light weight, the lateral stability of many wood-frame structures is often governed by wind, not *earthquake forces*. However, there are numerous examples of collapse or partial collapse of wood-frame buildings caused by ground vibration not associated with ground subsidence or fault movement through the building site (Fig. 7b-15: Dwelling, Long Beach, 1933; Fig. 7b-16: Dwelling, Santa Cruz Mountains, Loma Prieta, 1989; Fig. 7b-17: Apartment Building, San Francisco, Loma Prieta, 1989). Wood-frame structures are less susceptible to damage that results from design errors than is concrete or masonry construction; however, they are definitely not immune. Normally, the poor performance of wood-frame structures can be attributed to the absence of a rational lateral-force-resisting system and/or to dry-rot deterioration.

Well-designed wood-frame construction can be made life-safe against earthquake ground vibration. This has been demonstrated by the performance of wood-frame schools in California designed in accordance with the

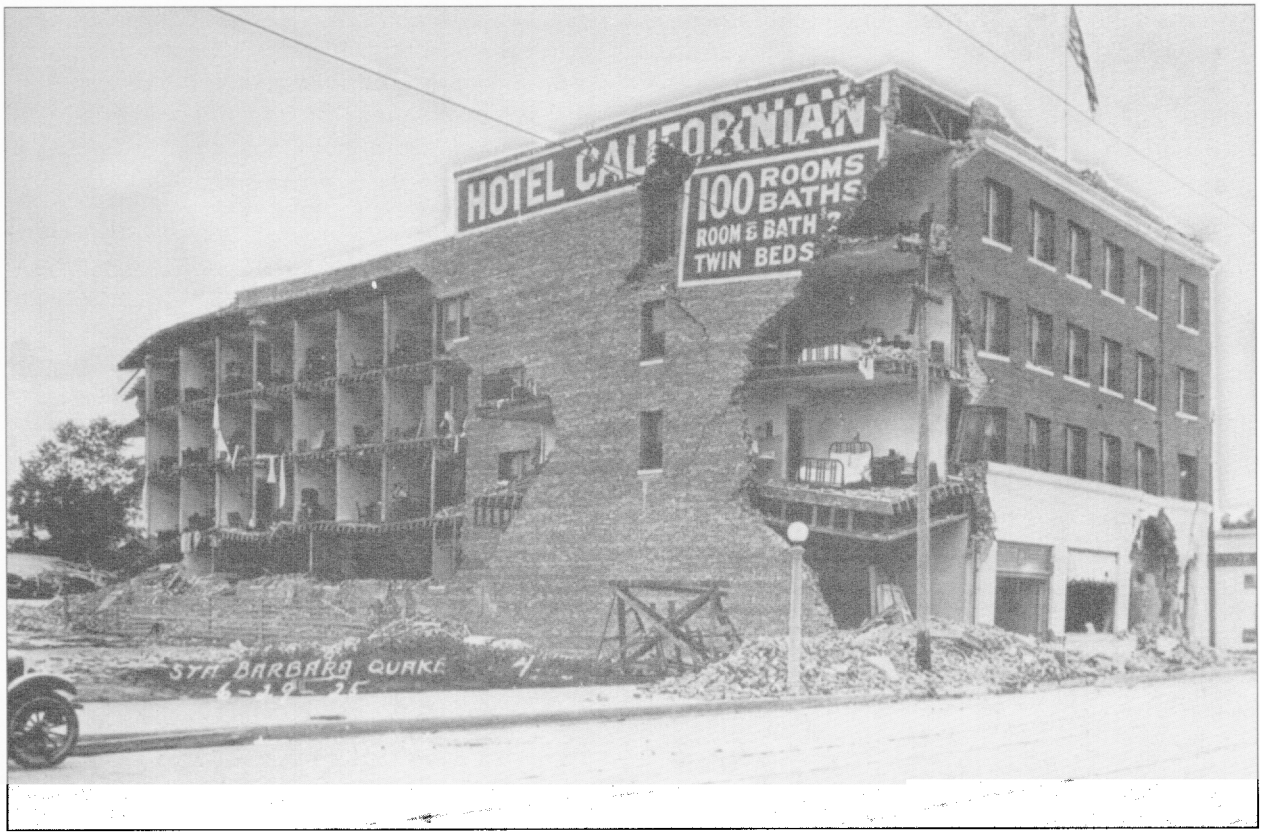


Fig. 7b-14. Failure of the Hotel Californian , Santa Barbara, California earthquake, 1925.

California Code of Regulations, Title 24, which is administered by the *Division of State Architect*. Many other wood-frame structures designed for industrial, commercial, and residential uses also have performed well in past earthquakes. Each wood-frame building should be evaluated individually.

Building Investigation and Evaluation

Optimum information for evaluating existing buildings includes the following:

1. Complete architectural, structural, mechanical, and electrical plans and specifications, including construction change orders and complete plans of all subsequent alteration work

2. Soils reports, including soil boring logs

3. Construction inspection reports and laboratory reports on materials tests and inspections

4. Original structural design calculations.

This information is listed in the order of importance. The complete plan record, including architectural plans, gives reviewers an overall picture that the structural plans alone do not convey. In many instances, so-called nonstructural curtain and infill walls that may not be shown on structural plans have a major adverse impact on structural behavior. Mechanical and electrical plans often provide information about the anchorage of major equipment items, as well as the locations of embedded piping and conduit that may affect structural performance.

The importance of change orders and alteration plans is self-evident. An originally sound design can be seriously impaired by *snap* decisions rendered during the construction process. Alterations to an existing structure by a new group (other than the original design professionals) may impair the original seismic bracing scheme. This point is illustrated in the case of a building at the Lawrence Berkeley Laboratory (LBL). The original structure was circular in plan, with a diagonally-braced, circular structural steel frame. The design was adequate, with no discontinuities in the lateral-

Fig. 7b-15. Wood-frame dwelling, Long Beach, California earthquake, 1933.

Fig. 7b-16. Wood-frame dwelling, Santa Cruz Mountains, Loma Prieta, California earthquake, 1989.

force-resisting system. Additions to the structure involved the removal of some diagonal bracing in the plane of the walls and the addition of structural *tees* welded to the weak axis of eight existing exterior columns to compensate for the removal of the bracing. The new built-up column sections were intended to resist lateral forces by column bending. Close inspection of the plans for modification showed no specific detail for the connection of these vertical *tees* to the truss gusset plates at the top of the columns. Subsequent field inspection revealed that the *T* reinforcing members extended to within 6 inches of the gusset plates and then terminated. The lack of connection between *T* reinforcing and gusset produced a hinge at the point of maximum bending in the column. The *tees* as installed were completely useless in replacing the original diagonal bracing. Fortunately, this condition was discovered during the review of all buildings at LBL in the early 1970s and strengthened by retrofit shortly thereafter. This example illustrates the necessity for carefully reviewing all documented alterations and the need for follow-up field inspections to ensure that the alterations were properly implemented. Often, the care taken in the original design-construction process is not manifest in subsequent alterations.

Soils reports and borings give special insight into the potential for differential settlement, soft-soil amplification, and slope instability on hillside sites. Earthquake shaking can result in differential settlement that, under normal static conditions, would take years to occur. Soft-soil sites can intensify damage, as illustrated in Mexico City (1985) and in San Francisco's Marina District (Loma Prieta, 1989). Investigation of facilities at well-managed institutions usually shows good coordination between soils engineers and design structural engineers. This, however, has not always been the case for buildings investigated at many locations. In many instances, carefully prepared soils reports pointing out soils defects and giving specific foundation design recommendations were completely ignored by structural engineers. This lack of coordination has often resulted in structural damage caused by major differential settlement under static load conditions.

Fig. 7b-17. Apartment building, San Francisco, Loma Prieta, California earthquake, 1989.

Earthquakes can turn such residual conditions into potential life hazards.

Construction inspection reports and laboratory test results provide insight to construction problems and also indicate the care taken to fully implement the intent of the design. Earthquakes act as the ultimate inspector, in that they usually reveal construction defects, sometimes with disastrous results (see Fig. 7b-18: Amfac Hotel roof tower collapse, Loma Prieta, 1989).

Design calculations, if available, aid in the evaluation of the design. However, they are the least important element in the information chain. They should not be used as a substitute for making check calculations as outlined in the *NEHRP Handbook.*

Information Gaps: Many older structures have either no design record or an incomplete record. This lack of information should not preclude an evaluation. The evaluation process is more costly; however, adequate information can be developed without literally tearing the structure apart. At the Lawrence Berkeley Laboratory, the information records were excellent and only a few minor buildings had incomplete records. Many California pre-Field Act (1933) schools and unreinforced masonry bearing-wall buildings in Los Angeles without plan records have been evaluated and rehabilitated. The evaluation and rehabilitation of this type building proved to be economically feasible and effective, as shown by their performance in the San Fernando, Whittier, Loma Prieta, and Northridge earthquakes.

The following lists, with comments, give the general procedures for determining existing structural connection details in seismic-resisting systems.

Wood-Frame Investigation

Types 1 and 2 Wood-Frame Buildings:

1. Remove adequate areas of interior and exterior finish to determine stud size and spacing, top-plate configuration and top-plate splice detail and corner interconnection, sill-plate size and anchorage to foundation, and type of wall and roof sheathing. Gypsum board, gypsum lath and plaster over paper-backed wire lath on line wire, and *straight* (face grain perpendicular to studs) one-inch nominal lumber sheathing, all can be considered as structural diaphragm sheathing, with the following limits. Gypsum board and stucco are brittle materials: once fractured, they provide minimal shear resistance. *Straight* lumber sheathing has limited shear capacity. The shear resistance of *straight* sheathing depends on the resistance of the nail couples. When diaphragms of this type are exposed to the weather, the nails tend to loosen as the wood weathers. Shear walls and diaphragms of the above materials should be considered as structurally viable only within the limits presented in the *1991 Uniform Code for Building Conservation* (UCBC) Appendix, Chapter One. If brittle diaphragm materials show evidence of *extensive cracking* caused by differential settlement or differential wood shrinkage, they should not be considered as structural diaphragms.

Diaphragms sheathed with diagonal one-inch nominal lumber sheathing or plywood can perform as dependable diaphragms for any size wood-frame structure. Diagonal sheathing should be checked for minimum code nailing. Diagonally sheathed and plywood-sheathed exterior walls with stucco exterior should have sufficient stucco removed to determine the existence of *dry rot* in the wood sheathing. Walls without roof eave overhangs are particularly vulnerable to leaks and subsequent *dry rot*. Plywood-sheathed horizontal diaphragms and walls should be checked for plywood grade, thickness, nail size and spacing, over-driven nails, and edge blocking at unsupported joints.

Vertical boundary members in walls should be checked for hold-down anchors to the foundation where the dead load of the wood-frame superstructure is insufficient to counteract overturning forces.

2. Inspect attic and subfloor areas to determine floor and roof member framing and construction details. Particular attention should be paid to the extent, or lack thereof, of interior walls extending through the subfloor crawl space (or, in some cases, the garage space) to interior continuous footings. Probably the most common type of earthquake damage in

Fig. 7b-18. Collapse of the Amfac Hotel roof tower, Burlingame, California, Loma Prieta, California earthquake, 1989.

Type 1 and 2 buildings is caused by the high shear stresses induced in exterior perimeter cripple walls and first-story walls caused by the termination of all interior walls at the adjacent upper-floor level. Dry rot in multiple studs at building corners also is a common occurrence and can be a significant factor in causing wall failures. The existence, condition, size, and spacing of anchor bolts in sill plates is critical.

3. Wood-frame structures with masonry veneers: Special care should be taken to determine the adequacy of the veneer attachment (Fig. 7b-19, Masonry Veneer failure, Santa Barbara earthquake, 1925). Sections of veneer should be removed to determine the type and condition of the anchorage. Wood-frame construction with masonry veneer is subject to dry rot conditions similar to wood frame with stucco exteriors.

Concrete Construction Investigation

1. Remove sufficient finish to determine member sizes and provide access for cutting of cores and chasing.

2. Reinforcing size and spacing can be determined by cutting chases. *Typical* members of a given group can be chased to determine exact sizes and spacing of reinforcing, with other members of the group checked with magnetic detection or X-ray devices to determine if the reinforcing pattern is consistent.

3. Cores for compression tests should be cut from slabs, walls, columns, and beams. Each member category should have sufficient cores cut so that representative average compression strengths can be determined. In many older concrete

buildings (circa 1900 to 1930), the concrete walls have the lowest compressive strengths. In shear-wall type buildings of this vintage, particular care should be taken to determine areas of concrete unsuitable for structural purposes. Concrete average compression strengths should be greater than 1500 psi for the concrete to be considered adequate for structural purposes. Rock pockets and honeycomb can be determined by visual inspection. *Nondestructive evaluation* (NDE) methods include ultrasonic, mechanical-pulse, and Schmidt-Hammer tests.

Steel-Frame Construction Investigation

1. Remove sufficient finish to determine member size and connection detail.

2. Cut coupons from typical members for ultimate tensile and yield-strength tests. (Removal of asbestos fire proofing requires special monitoring and enclosure of inspection areas.)

3. If the frame has bolted joints, bolts should be checked for material type and torque and if they are actually high-strength bolts.

4. Critical welded joints should be checked using nondestructive testing methods. Particular attention should be paid to the inspection of bracing members and bracing connection detail in braced frames.

Masonry Construction Investigation

1. The investigation of reinforced masonry construction is similar to the procedures used for concrete construction, i.e., chasing and core cutting and testing. Grout core masonry should have cores cut to determine the shear strength of the joint between the masonry unit and the grout core. All cores cut from both grout core and filled-cell masonry should be subject to visual examination to ascertain if all joints and/or cells are completely grout-filled. Special care should be taken to determine that reinforcing is installed and that all cells containing reinforcing are grouted. *This*

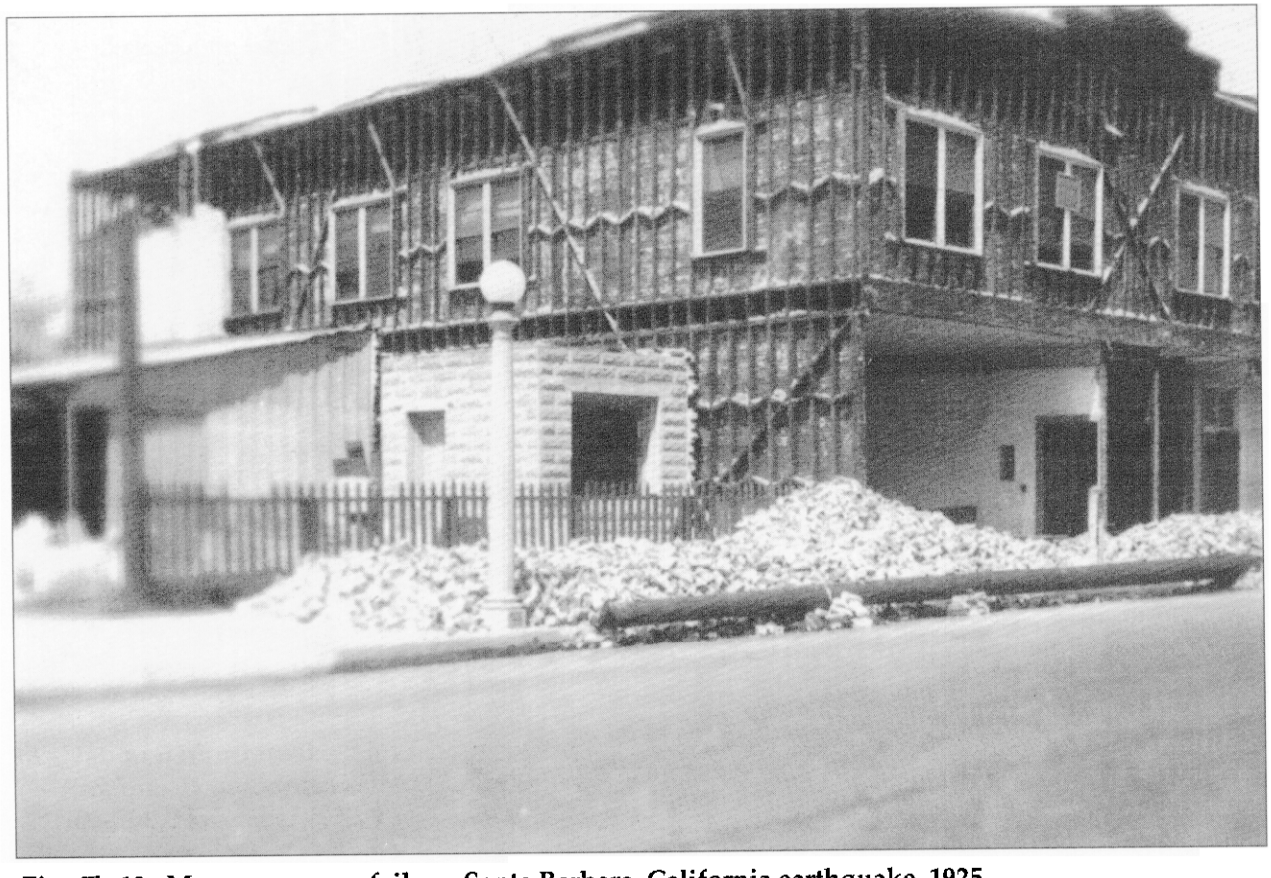


Fig. 7b-19. Masonry veener failure, Santa Barbara, California earthquake, 1925.

cannot be achieved by spot testing and extrapolation. In situ tests can be made to determine shear capacity of mortar.

2. Unreinforced masonry: Required testing for URM walls and existing wall anchors is prescribed in the UCBC Appendix, Chapter One: Material Requirements.

Seismic Design-Check Parameters

DOE-STD-1020 is the governing code for the evaluation of existing DOE facilities. The recommendation for the use of the *BSSC, NEHRP Handbook for Seismic Evaluation of Existing Buildings* should not be construed as a recommendation for the use of two conflicting codes. All the NEHRP check calculations should be made using DOE-STD-1020 design parameters.

Section B.3, Appendix B, of DOE-STD-1020 gives an excellent synopsis of the evaluation process, including alternatives for evaluation parameters based on a structure's projected life. The use of these evaluation alternatives (i.e., increasing the annual probability of exceedance) should be tempered with judgment based on the type of building system being evaluated and the consequences of failure.

Justification for this procedure is completely acceptable when the increased risk is limited to more extensive damage to a ductile system. When the increased risk includes the possibility of sudden collapse in a nonductile system, all parties should be made clearly aware of the consequences of the decision to alter the annual probability of exceedance.

Methodology

The *NEHRP Handbook* provides a comprehensive checklist for the most commonly used building systems. The following suggested amplifications are specifically aimed at information gathering for structures without any plan records and for those with incomplete records.

1. Enhance the NEHRP, Appendix E Data Summary page with the Data Summary format contained in Appendix B of this chapter (7b).
2. Develop plan data based on Section A105 C of the UCBC. (See Appendix B of this chapter.)
3. Note that the base-shear equations contained in the *NEHRP Handbook*, Appendix E, should be changed to conform to DOE-STD-1020.

A thorough field investigation of the structure should *never* be omitted, even if complete design documents (plans and specifications) and inspection records are available. The construction process, even with mandated continuous on-site inspection, is not infallible. Omissions in key details pertaining to the lateral-force-resisting system *can occur even with full-time inspection.*

APPENDIX A

The NEHRP Handbook for the Seismic Evaluation of Existing Buildings, Appendix A, provides general sets of evaluation statements for each of the following:

1. The basic building system
2. Vertical systems resisting lateral forces (moment frames, shear walls, braced frames)
3. Diaphragms (horizontal bracing systems)
4. Structural connections
5. Foundations and geologic site hazards
6. Nonstructural elements.

The NEHRP Handbook, Appendix B, provides sets of evaluation statements for each of the following 15 common building types:

1. Wood, light frame
2. Wood, commercial and industrial
3. Steel moment frame
4. Steel braced frame
5. Steel light frame
6. Steel frame with concrete shear walls
7. Steel frame with infill masonry shear walls
8. Concrete moment frame
9. Concrete shear walls
10. Concrete frame with infill shear walls
11. Precast/tilt-up concrete walls with lightweight flexible diaphragms
12. Precast concrete frames with concrete shear walls
13. Reinforced masonry bearing walls with wood/metal deck diaphragms
14. Reinforced masonry bearing walls with precast concrete diaphragms
15. Unreinforced masonry bearing wall buildings.

APPENDIX B

EARTHQUAKE SAFETY EVALUATION REPORT

File #

Inspected by _____ **Date:**

Plans checked by _____ **Date:**

Reinspected by _____ **Date:**

A. DESCRIPTION:

1. LOCATION ADDRESS: _____

2. OWNER and/or BUILDING NAME: _____

3. OCCUPANCY & CONTENTS: _____

4. HEIGHT: _____ story(ies); _____ feet

5. GROUND FLOOR DIMENSIONS: _____

6.

BLDG. UNIT	Architect	Engineer	Contractor	Year Built

7. SUPERVISION of CONSTRUCTION:

A. DESCRIPTION:

1. OVERHANGING UNIT MASONRY or OTHER LARGE MASS:

Side	Height	Material

2. POUNDING AGAINST ADJOINING STRUCTURE:

Side	Height of Contact	Separation Between Buildings

C. ROOF STRUCTURES

1A. NOT APPLICABLE/NONE _____

1B. CHIMNEYS, ROOF TANKS, SIGNS, EQUIPMENT, PENTHOUSES, ETC:

D. FOUNDATIONS

1. FOUNDATION MATERIAL & SOILS TESTS:

_____

2. FOOTINGS & EXCAVATION DEPTH:

E. BASEMENT

1A. NOT APPLICABLE/NONE _____

1B.

	Thickness	Material
Wall		
Floor		

F. FRAME

G. FLOORS & ROOF

Floor	Thickness	Material	Type & Remark

H. EXTERIOR WALLS

Stories	Thickness	Material	Type & Remark

I. PARAPET

1A. NOT APPLICABLE/NONE _____

1B. HEIGHT: _____
THICKNESS: _____
MATERIAL: _____

2. SECURED BY:

_____
_____

3. CORNICE and/or OTHER PROJECTIONS:

a. Material :
b. Width of Overhang :
c. Secured by :

4. APPROXIMATE % OF WALL OPENINGS:

a. North : _____
b. East : _____
c. South : _____
d. West : _____

5. REMARKS:

_____
_____
_____
_____
_____
_____

J. INTERIOR PARTITIONS

1A. NOT APPLICABLE/NONE _____

1B. QUANTITY, TYPE, MATERIAL, & THICKNESS:

_____
_____
_____

K. ORNAMENTATION

1. EXTERIOR:

2. INTERIOR:

L. MISCELLANEOUS

1. EVIDENCE OF SETTLEMENT:
2. EXTERIOR CRACKS:
3. INTERIOR CRACKS:
4. DEFECTIVE MATERIALS OR WORKMANSHIP:
5. PREVIOUS EARTHQUAKE DAMAGE:
6. APPROX. LOCATION of NEAREST KNOWN MAJOR FAULT(S):
7. UNUSUAL STRUCTURAL CONDITIONS:

M. SUMMARY

Chapter

8

Seismic Retrofit of Buildings

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

The principles that apply to good seismic design for new buildings are equally applicable to the design of projects to improve the earthquake resistance of existing buildings. The difference is that one doesn't have the opportunity to design existing structures from scratch. Analyses, criteria, and design solutions are apt to be less straightforward and retrofit construction more complicated.

When a structure is to be strengthened to improve its seismic safety, the prime objective is to establish a formal lateral-force-resisting system that will perform in a predictable manner. Usually the problem building has other seismic hazards as well as the lack of a predictable lateral-force-resisting system. For example, in older buildings access corridors and room partitions were sometimes constructed of unreinforced hollow-tile blocks, a brittle material not allowed by modern codes. Usually nonstructural elements, such as hung ceilings, light fixtures, mechanical equipment, etc., were not properly anchored.

The combination of problems may add up to a total rehabilitation cost that is overwhelming. A legalistic attitude sometimes prevails that dictates that nothing should be done unless the building can be completely brought up to current code compliance in all respects, not just seismic. This, of course is not good risk management. It is important to separate deficiencies on a priority basis, then take the best first step to improve the situation. Try to set up the most cost-effective and risk-reduction-effective program. Sometimes obvious deficiencies exist that can be remedied so easily and economically that correction should be accomplished immediately. Such would be the case where, for example, it is discovered that rod cross-bracing is not tight, and shaking would cause the bracing to hammer or allow unsafe deflections to take place. Obviously, the bracing can be tightened without delay to abate the hazard. Also, if a brace is missing, it usually can be rather easily replaced. A sophisticated analysis is not required, and it is not necessary to process the correction through an elaborate priority system. The solution is obvious.

The course of action may be much more complicated when serious fundamental deficiencies exist. It may take considerable time to develop an acceptable solution and fund a project for correction. If a deficiency represents an imminent hazard to life safety, action should be taken immediately to reduce the risk, even though the hazard may not be completely eliminated by the action. In some situations, the hazard can be significantly reduced by adding partial bracing or simply increasing the strength of the weakest element. Improving the building's resistance to withstand minor earthquakes reduces the statistical probability of

serious damage or injury, even if the building is not capable of resisting a major earthquake. The exposure of human life can be reduced by decreasing either the number of people using the facility or the daily hours of occupancy. Sometimes the use of the facility can be changed. Risk management, which plays a very important role in the process of building rehabilitation, is covered in Chapter 12.

A common stumbling block in rehabilitation projects is the problem of setting criteria for analysis and design. Sometimes older buildings were constructed of materials that are not acceptable under modern standards and codes. Such materials may be brittle or their earthquake performance unpredictable; allowable stresses may not be published. In spite of this situation, normally it is not feasible to abandon a hazardous building on short notice unless the activities within can be moved to another safe, usable space already available. Usually a building can be strengthened more economically and quickly than it can be replaced if a predictable lateral-force-resisting system can be established for the structure. For facility managers, the key consideration is *predictability*. If an older building does not have a clearly defined and *predictable* lateral-force-resisting system, it cannot be considered safe. If such a system can be established by modifying and strengthening the existing structure or by integrating an entirely new lateral-force-resisting system into the existing structure, then retrofit is generally practical and economical. *Always the key consideration should be predictability.* The performance of the retrofitted building should be predictable and attainable.

Another important consideration is whether retrofit can be accomplished without seriously disrupting ongoing activities within the building. It is very difficult and time consuming to relocate production and research equipment and extremely expensive to lease appropriate space for such activities. On the other hand, many buildings can be strengthened or buttressed from the outside to avoid this disruption. More than half of the 34 buildings rehabilitated at the Lawrence Berkeley Laboratory were strengthened this way. Most of the rest were strengthened with a combination of inside and outside work that minimized distruption. None were completely evacuated during rehabilitation.

Older buildings that have seismic deficiencies usually have other code deficiencies that may require correction if they are affected by seismic-retrofit work. This is true more often when interior strengthening is necessary. Also, interior retrofit usually leads to the modification of electrical and mechanical systems or other nonstructural components. Much of these costly collateral expenses can be avoided if seismic strengthening can be accomplished externally. This approach usually provides more timely life-safety and cost benefits.

Examples of exterior retrofit are shown in Figs. 8-1 through 8-9. Figures 8-1 and 8-2 are before and after retrofit photos of Building 311 at the Lawrence Livermore National Laboratory (LLNL). Building 311 (Fig. 8-1) was originally designed under the requirements of the 1961 Uniform Building Code with a (nonductile) reinforced concrete moment-resisting-frame for 6.7% static base shear. Current codes would require a much higher static base shear and much tougher ductile concrete details. Building 311 was severely damaged in the 5.8 magnitude Livermore, California earthquake of 1980. Three relatively large after shocks with magnitudes 5.1, 4.0 and 4.2 followed within three minutes causing the duration of heavy shaking to last much longer than expected of a moderate earthquake. Nevertheless, the building experienced ground accelerations, later estimated at 25% g (about four times the design base shear), without collapse; this in spite of rather poor detailing and brittle construction. This two-story (37,000 gsf) building was buttressed with a series of external concrete shear walls on three sides for 25% static base shear (Fig. 8-2). Building 311 was evacuated before retrofit because it was unsafe for occupancy. Retrofit cost about 30% of replacement value and required extensive repairs internally and externally. The retrofitted structure was checked using site-specific ground spectra for 50% g.

Figures 8-3 and 8-5 are before and after photos of Building 113 at LLNL. Originally designed for 5.7% g static base shear, this five-story reinforced concrete building (Fig. 8-3) (42,000 gsf) suffered extensive structural cracking in its central-core shear walls which were designed to take 80% of the seismic design load. The remaining 20% was to be resisted by reinforced concrete columns which were

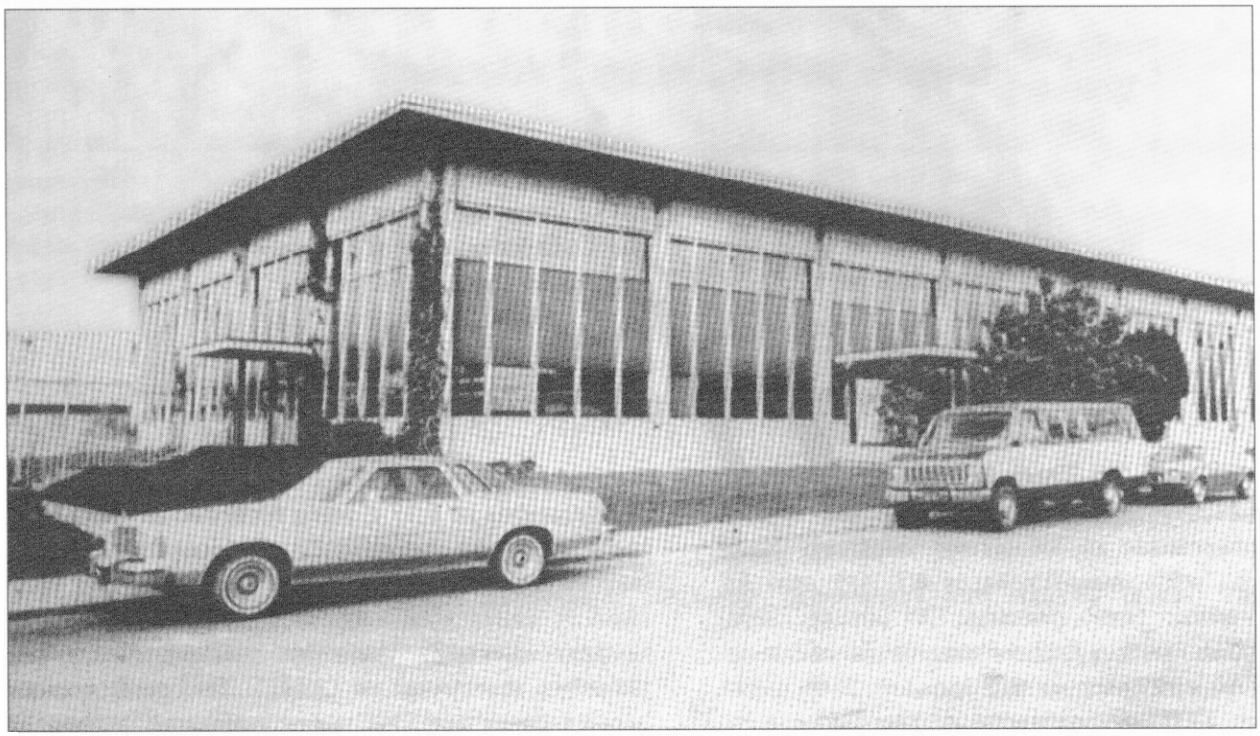


Fig. 8-1. Building 311 at Lawrence Livermore National Laboratory before retrofit.

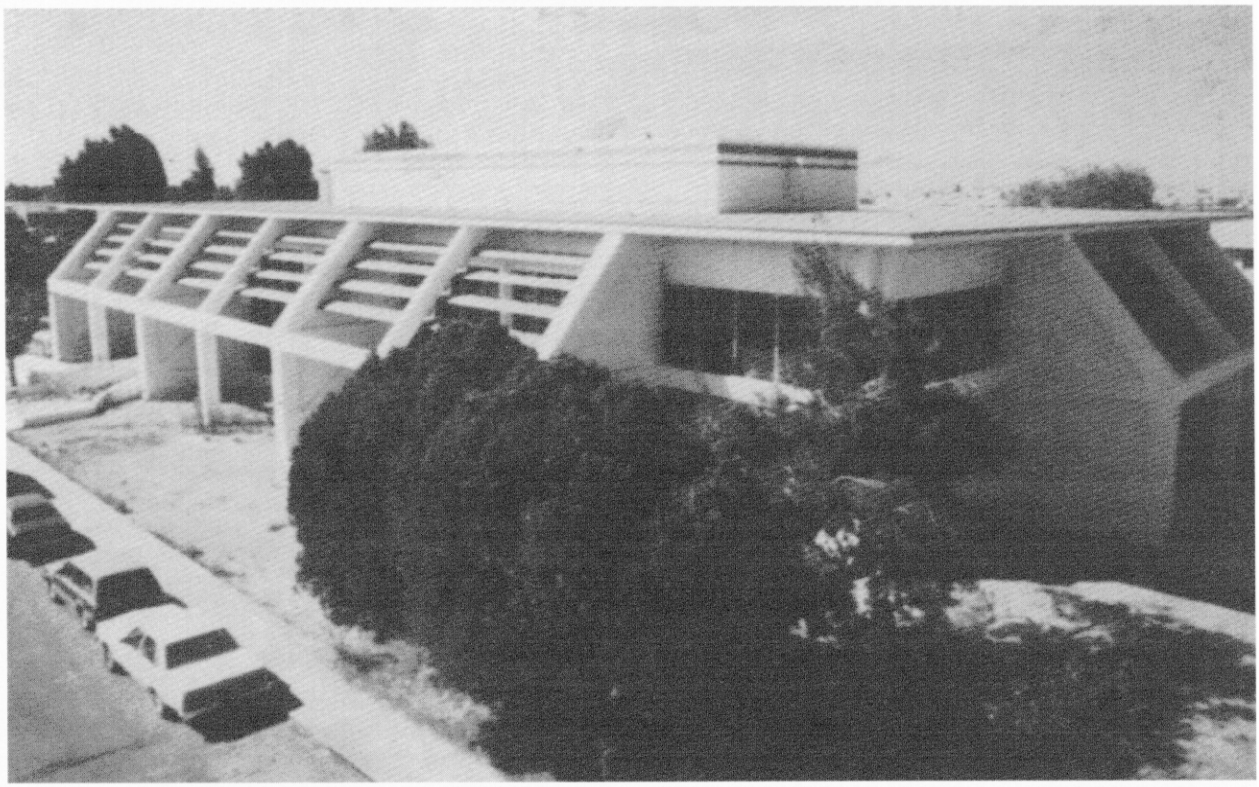


Fig. 8-2. Building 311 after retrofit in 1982.

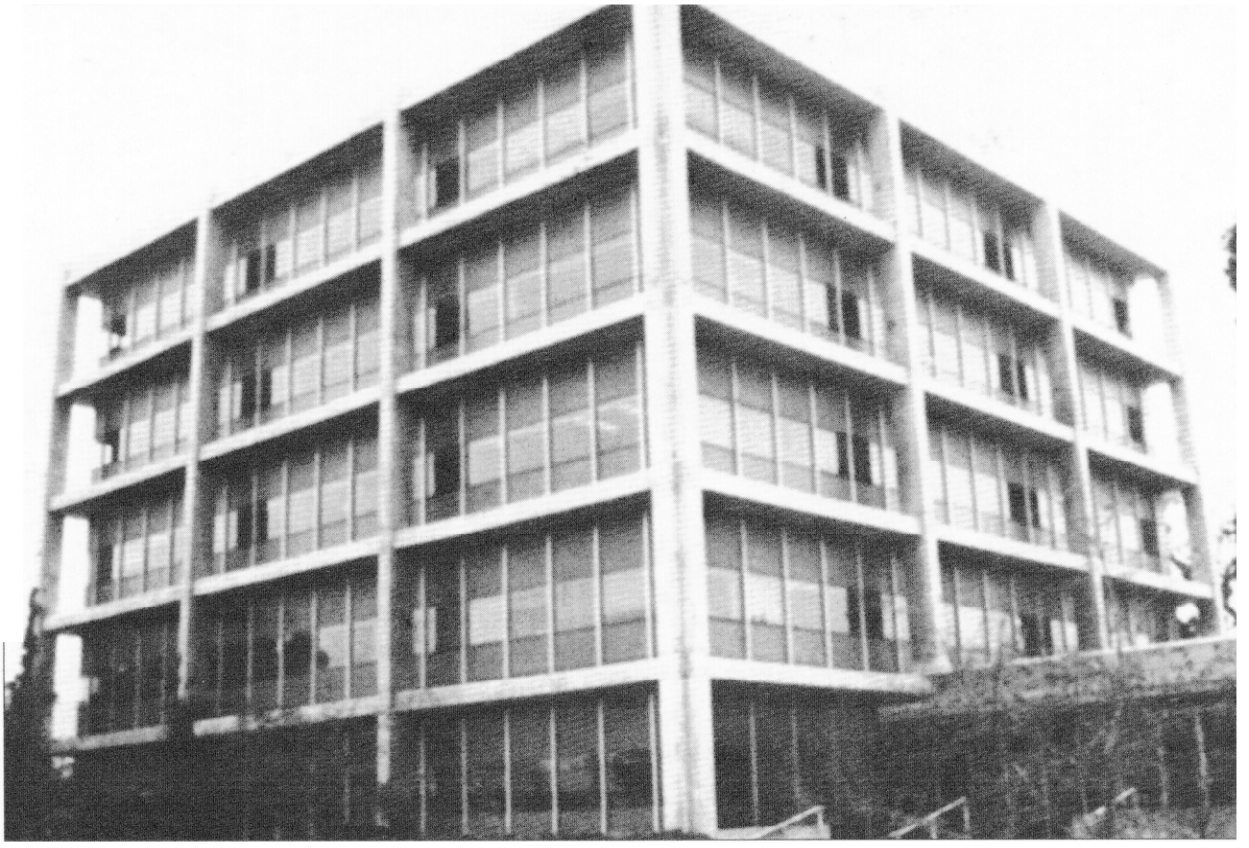


Fig. 8-3. Building 113 at Lawrence Livermore National Laboratory before retrofit.

undamaged in the earthquake. Building 113 experienced ground accelerations approximately four times greater than its design base shear in the 1980 earthquakes. After structural cracks in the central-core shear walls were repaired with high-strength epoxy injection, Building 113 was retrofitted (Fig. 8-4) with structural steel "K"-bracing attached to the outside perimeter and to the existing foundation system. Figure 8-5 shows the completed retrofit which was designed for 50% g peak ground acceleration and cost about 19% of the building's replacement value.

Figure 8-6 shows buildings 50A and 50B at the Lawrence Berkeley Laboratory (LBL) in Berkeley, California which were retrofitted in 1975. These two buildings (constructed in 1960 and 1966) were evaluated in 1973 in the aftermath of the destructive 1971 San Fernando, California earthquake in southern California. Both buildings are reinforced concrete bearing-wall structures, 7-stories at the downhill end and 4-stories at the uphill end. One has an area of 67,000 gsf; the other 64,000 gsf. Each was designed for approximately 8% g static base shear with transverse resistance provided by shear walls and longitudinal resistance provided by moment connections between the 8 feet wide vertical elements at the ends of the window rows and the heavy longitudinal spandrel beams between horizontal rows of windows. Unfortunately, the vertical concrete piers between window rows in the longitudinal bearing walls were nominally reinforced to take vertical loads only. The 1973 seismic evaluations found that the concrete window piers could not resist the longitudinal deflections necessary for the bending resistance to develop in the spandrel beam/end pier connections. It was determined that the brittle window piers would fail in diagonal tension during a strong earthquake causing both buildings to suffer catastrophic collapse.

Both buildings were retrofitted while occupied in 1975 with longitudinal concrete buttresses, one of which can be seen (Fig. 8-6) at the uphill end of the near building (50A). These 3-story buttresses are supported on large, heavily reinforced grade beams with deep seated spirally-reinforced caissons at each end (Fig. 8-7). They were designed for 75% g ground

Fig. 8-4. Building 113 during retrofit in 1981. A steel "K"-brace frame being lifted into place.

acceleration using the Design Response Spectra for Nuclear Power Plants, (Newmark, Blume and Kapur). Total project cost for the 1975 retrofit of both laboratory buildings was approximately 3% of replacement value.

Figures 8-8 and 8-9 show the tubular steel buttresses used to retrofit Building 90 at LBL in 1994. Building 90, a four-story office building (88,300 gsf), was constructed in 1959 with a steel frame moment-resisting structural system designed for approximately 6% g static base shear. Seismic evaluations determined that this overly flexible building would suffer extreme inelastic deflections in a major earthquake rendering it irreparable for future use. Retrofit, which consisted of steel buttresses on three sides, was designed for 20% g static base shear and checked against scaled results of a site-specific time history with (near fault) peak ground accelerations of 70% g. Construction, accomplished while the building was occupied, was completed in 1994 at a structural cost (including engineering) of approximately 14% of replacement value. Additional project costs were incurred to buttress potential landslide

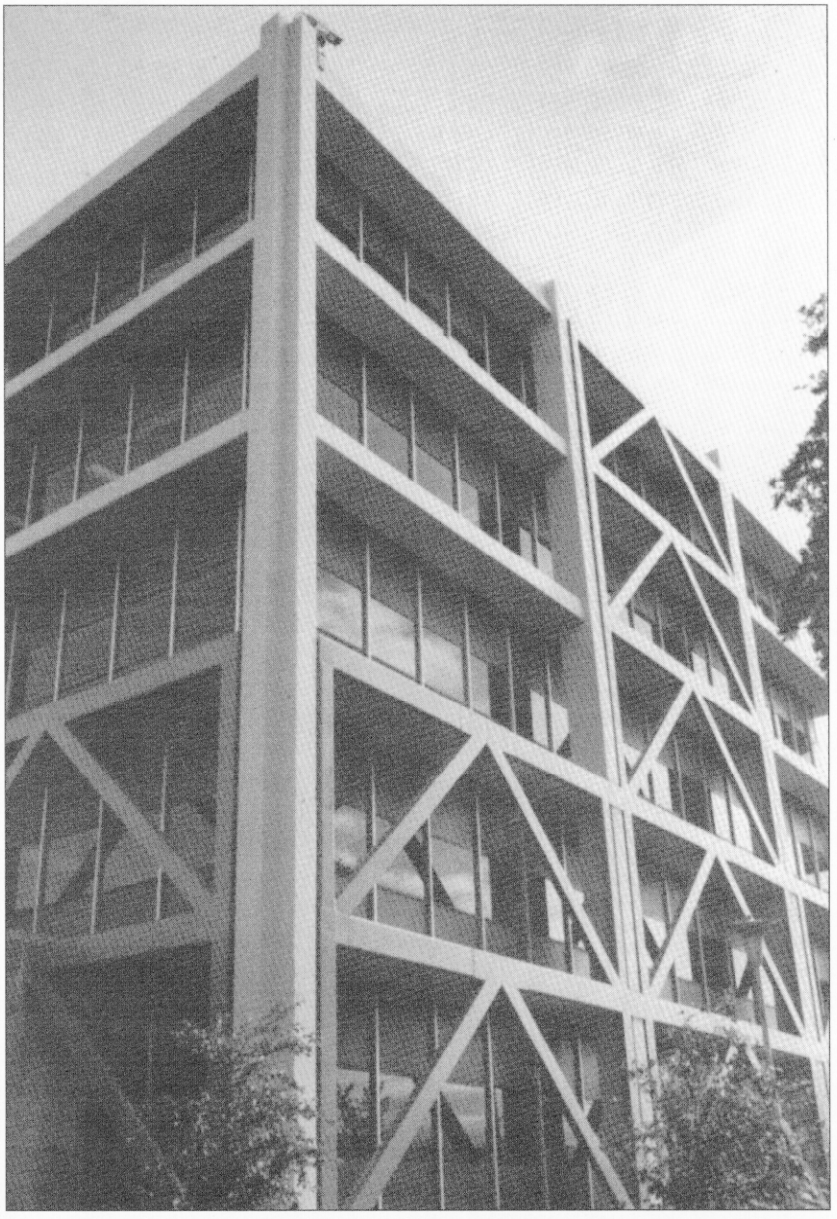


Fig. 8-5. Building 113 after retrofit in 1982.

hazards that would damage Building 90 in a major earthquake.

The U.S. Department of Energy (DOE) has established definitive criteria (Ref. 1) for the evaluation and retrofit of existing structures, systems, and components (SSC), and these criteria are discussed in Chapters 4, 6, and 8a. They are, however, primarily concerned with seismic loadings for design and do not address many of the special problems associated with existing facilities. For example, an existing building may be constructed of nonductile, reinforced concrete that will be driven into inelastic behavior in a strong earthquake. Consequently, a very important goal of the retrofit design is to limit deflection. Thus, deflection may be the controlling factor in mitigating seismic hazard. Seismic rehabilitation of existing buildings is fraught with such special circumstances. Each one is a special case. On the other hand, force levels for which rehabilitation projects must be designed are well defined in DOE criteria and apply specifically to lateral-force-resisting systems.

Designers, particularly those who are inexperienced in earthquake engineering, tend to place too much importance on the specific level of lateral force to be utilized and too little

Fig. 8-6. Buildings 50A and 50B at Lawrence Berkeley Laboratory after retrofit in 1975. Note arrow pointing to logitundinal buttress retrofitted to the uphill end of building 50A.

importance on the details of good design. Lateral force criteria for rehabilitation projects can be established rather easily. For example, if the lateral-force-resisting system for a normal symetrical building is well designed for a static base shear of 0.2g, it should perform in an earthquake as well as if it were designed for 0.8g in a dynamic analysis. In areas where the earthquake hazard is minimal, an analysis using at least 0.1g equivalent static base shear coupled with a ductile design is a reasonable and economical lateral force criteria for retrofit projects. A 0.1g lateral-force factor is large enough to analyze and easily trace through a

Fig. 8-7. Longitudinal retrofit buttress at the uphill end of Building 50A.

lateral-force-resisting system and highlight connections that should be carefully detailed for earthquake resistance. Buildings designed for this loading should resist ground accelerations up to 0.4 g without collapse. (See Chapter 5 Foreword for explanation). For most buildings and locations, it is large enough so that wind does not control the design and the earthquake-versus-wind premium in construction cost is very small. If one then ensures that the lateral force system will act in a ductile way when over stressed, the situation becomes predictable even if the size of the earthquake is not.

The above discussion has been included to put earthquake design loadings in perspective with damage experience. It is not meant to steer designers away from applicable seismic regulations and building codes. Although earthquakes are of an unpredictable nature, the seismic loadings prescribed in building codes provide a reasonable basis for design. When coupled with proper detailing to ensure that the intent of more general seismic provisions is fulfilled, structures will withstand earthquake forces that greatly exceed static design loadings.

In many areas of the United States, damaging earthquakes are rare events, and it has been difficult for engineers to take them seriously in the design of conventional structures. Very few buildings in the midwest and eastern United States have been designed for earthquakes, even in those areas where it is known that damaging earthquakes have occurred in the historic past. A great many buildings are constructed of unreinforced unit masonry (usually brick or concrete block), one of the brittle building systems particularly susceptible to abrupt collapse in damaging

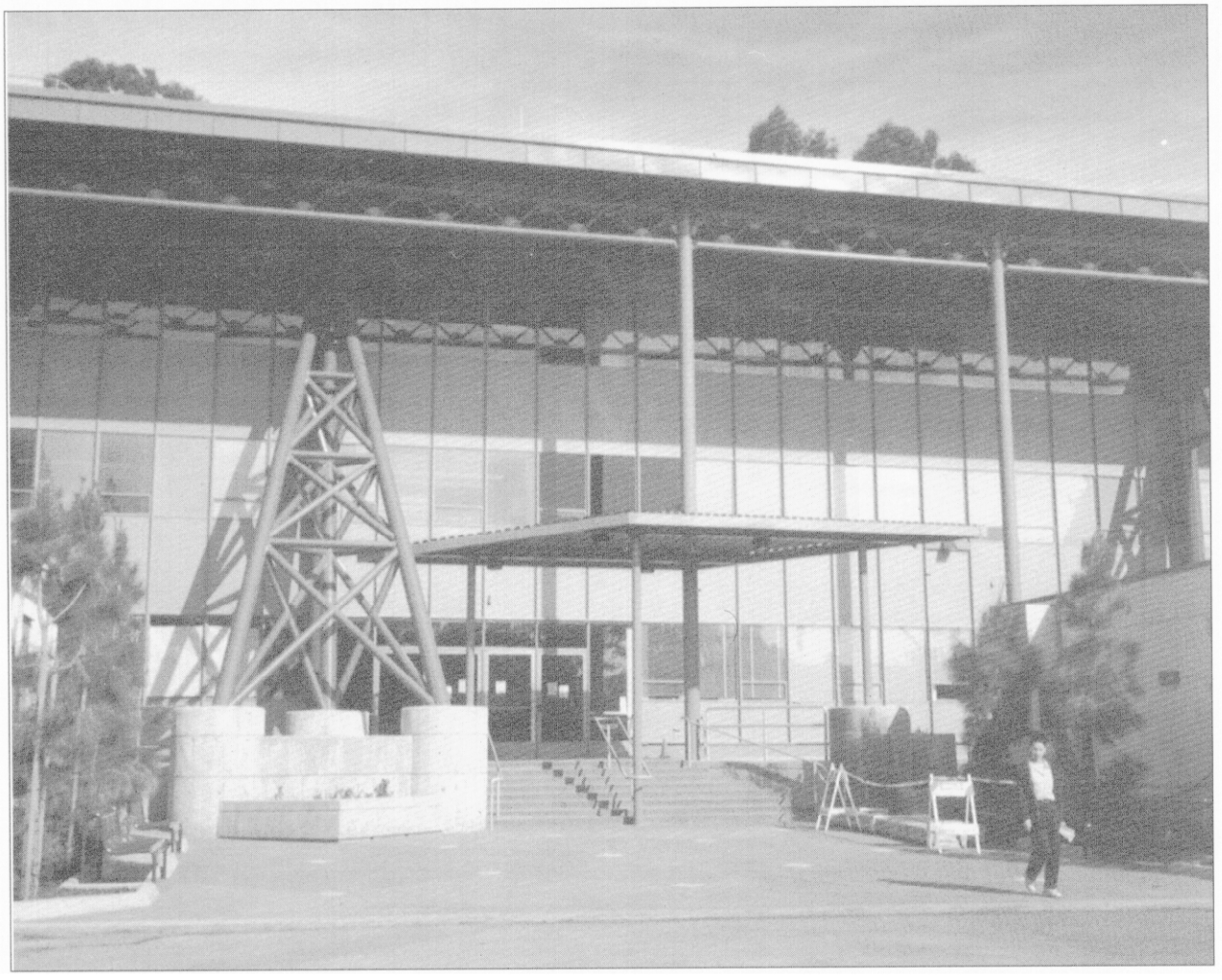


Fig. 8-8. Building 90 entrance area, Lawrence Berkeley Laboratory, after 1994 retrofit.

earthquakes. Significant progress could be made by simply avoiding the use of unreinforced unit masonry in future construction. Unfortunately, all of the hazardous brick buildings that now exist cannot be readily replaced.

About 10 years ago it was estimated that more than 20,000 fatalities could occur in unreinforced masonry (URM) buildings in Los Angeles alone should a major earthquake occur near them. Fortunately, the epicenter of the magnitude 6.6 earthquake that severely damaged the Northridge/San Fernando Valley area of California on January 17, 1994, was about 20 miles from downtown Los Angeles. The duration of heavy shaking was about 9 seconds, and the effects of short-period seismic waves, which tend to damage brittle structures such as unreinforced brick buildings, were significantly attenuated in older Los Angeles.

The City of Los Angeles began an aggressive program to retrofit existing URM buildings (Ref. 2) about 10 years ago. Of 6400 URMs in Los Angeles about 6000 were refitted before the 1994 earthquake under the city's special retrofit ordinance (Refs. 3 and 4). There were proportionately less URM buildings in San Fernando Valley than in Los Angeles because urban development was more recent and many of the URM buildings there had been badly damaged in the 1971 San Fernando earthquake and were then demolished. To make retrofit economically acceptable to owners, seismic criteria were generally geared toward risk reduction rather than full protection of life-safety.

Generally, retrofitted URM buildings performed better than unretrofitted ones, although many retrofitted URM buildings suffered damage. Poorest performance was in those buildings with weak mortar and/or poorly designed retrofits. Overall, many more unstrengthened URM buildings failed than did retrofitted URM buildings. If it were not for retrofits, many more lives would have been lost in the Northridge earthquake. More detailed

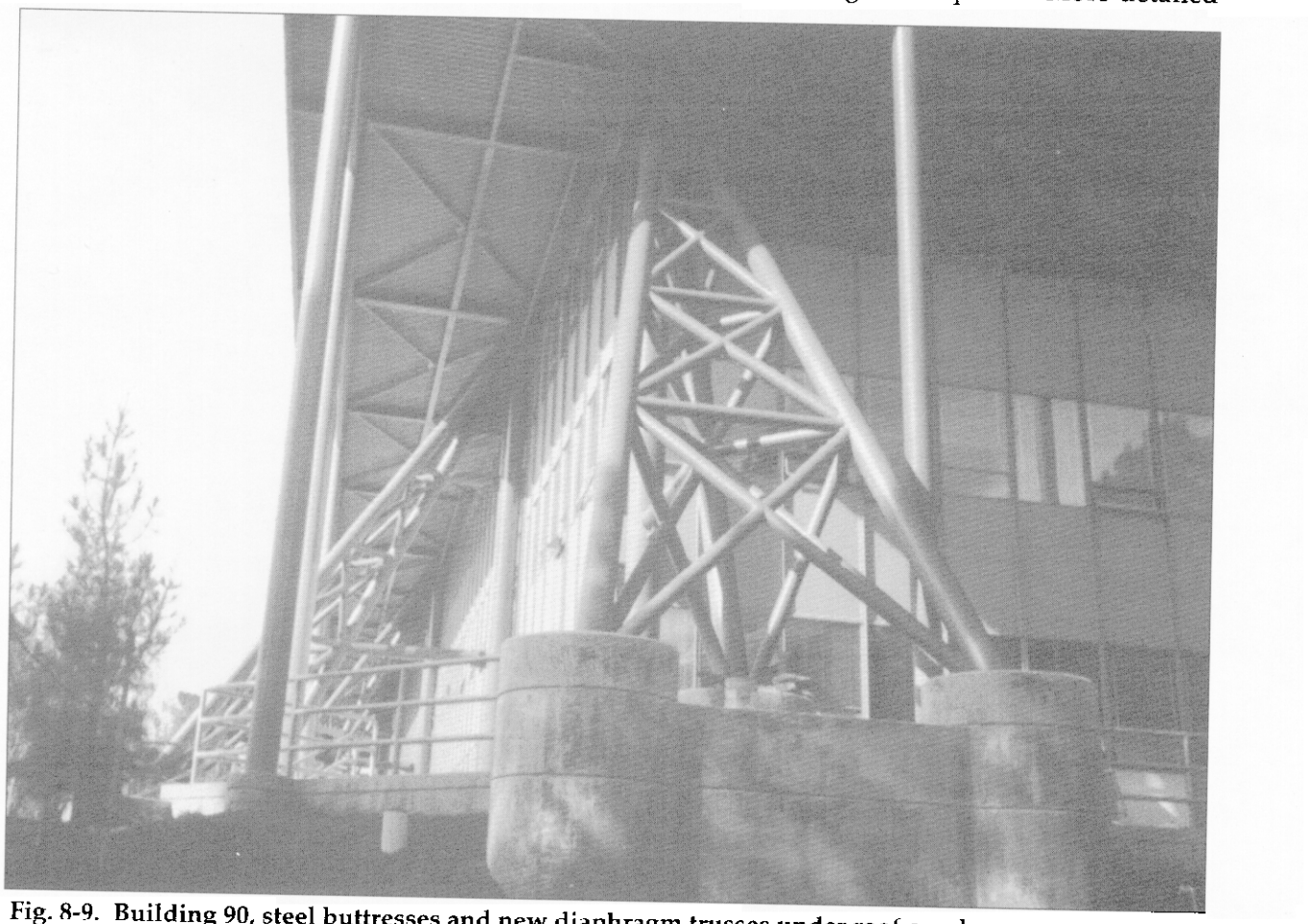


Fig. 8-9. Building 90, steel buttresses and new diaphragm trusses under roof overhang.

comparisons of relative performance will be reported in the future, but it should be well understood that these retrofitted URM buildings are not sufficiently predictable for life-safety purposes. This subject is also discussed in Chapter 7b.

Los Angeles began work in 1974 to develop its ordinance to evaluate existing unreinforced brick buildings in such a way as to codify their future existence and reliability. The work leading up to the ordinance should be valuable to the facility managers who have a large inventory of such hazardous buildings. The report on this work, entitled A Case Study in Hazard Abatement for Older Masonry Buildings, (Ref. 2), was presented in September 1980 by Earl Schwartz, Senior Structural Engineer in the Department of Building and Safety, City of Los Angeles. It provides a methodology for testing, evaluating, and upgrading such buildings, even though they cannot meet today's code for earthquake safety. The report is highly recommended for use in areas of the country where unreinforced masonry buildings will undoubtedly be used until a moderate or major earthquake causes disaster for thousands of occupants of buildings of this construction.

The Northridge event was not a great earthquake , but the intensity of shaking was very damaging even though it was fairly localized and of relatively short duration. On the other hand, the heavy damage to most buildings and parking structures in this earthquake was to be expected because it was primarily related to common deficiencies of design and construction well known to experienced earthquake engineers. One exception was the unexpected damage to welded girder/column connections in steel moment-resisting frames. This subject is discussed in Chapters 3 Foreword, 6a and 7b.

DOE Order 5480.28, Natural Phenomena Hazards Mitigation (Ref. 5) lists the circumstances under which existing DOE facilities are to be reevaluated and how and when reevaluation can be triggered. These requirements are summarized in Chapter 7a. There are also many triggers for retrofit that relate to life-safety, liability, and risk management all of which should be of vital interest to facility managers. A detailed discussion of these issues is included in Chapters 12 Foreword and 12b covering risk management.

In summary, building codes and regulations lay the basis for determination and application of seismic loads, but they rely on criteria generally intended for new buildings, not older buildings of poor construction. Building codes still serve as vital resources, but it is essential that design for retrofit be carried out by experienced earthquake engineers. Effective rehabilitation is very dependent on professional judgment and insight that cannot be adequately provided in codes, regulations, or textbooks. Consequently, the need for third-party review (or peer review) is even more compelling for the design of retrofit projects than for the design of new buildings.

Usually, it is neither feasible nor sensible to correct all of the deficiencies in older hazardous buildings, but it is usually practical to provide a structural solution for the life-safety of occupants.

References

1. Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities, DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.

2. A Case Study in Hazard Abatement for Older Masonry Buildings, E. Schwartz, Department of Building and Safety, City of Los Angeles, California, September 1980.

3. Northridge Earthquake January 17, 1994, Preliminary Reconnaissance Report, Earthquake Engineering Research Institute, Oakland, California, March 1994.

4. The January 17, 1994 Northridge, California Earthquake, An EQE Summary Report, EQE International, San Francisco, California, March 1994.

5. Natural Phenomena Hazards Mitigation, DOE Order 5480.28, U.S. Department of Energy, Washington, D.C., January 1993.

Chapter

8a

Retrofit Guidelines

Frank E. McClure

Introduction

DOE-STD-1020, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (Ref. 1), lists requirements for five performance categories. For Performance Category 1 (PC-1), which covers *General Use* or *Standard Occupancy Facilities*, the primary concern is to prevent major structural damage or collapse that would endanger personnel within or adjacent to the facility. The basic life-safety requirements for PC-1 *structures, systems, and components* (SSCs) are that they will not fail or collapse, partially or totally, during a major earthquake and that they will perform adequately to provide for unobstructed ingress and egress of building occupants. However, damage to the SSCs per se may not be repairable.

Performance Category 2 (PC-2) covers *Esential Facilities* that are of greater importance than PC-1 facilities because of mission-dependent considerations. *Essential Facilities* generally include hospitals, fire and police stations, and other facilities where continuity of function is necessary for emergency operations. The performance goal for these facilities is to maintain the capacity to function and ensure occupant safety. These facilities are allowed relatively minor structural and nonstructural damage that will not jeopardize life-safety or functionality during and after a major earthquake.

Performance Categories 3 (PC-3) and higher cover SSCs that pose a potential hazard to public safety and the environment because dispersible radioactive or toxic materials are present. Performance goals for facilities in these categories are to limit damage so that hazardous materials can be controlled and confined, occupants are protected, and the functioning of the SSCs is not interrupted.

Evaluation criteria and review procedures are discussed in Chapter 7. A Department of Energy (DOE) facility that does not meet the performance goals of the DOE-STD-1020 performance category to which it is assigned is a primary candidate for retrofit.

Recommendations for Seismic Retrofit Design

The goal of the following recommendations is to avoid premature, abrupt, or unexpected failures and to encourage design for ductile behavior of structures, members, and connections and continued functionality of life-support systems during earthquakes. These recommendations are the same for new SSCs as for those to be retrofitted. The design principles for retrofitting lateral-force-resisting systems are more important than the magnitude of the seismic forces to be used for retrofit design. This is because the actual seismic forces are likely to be greater than the design seismic forces; thus, good design and detailing must provide the structure with overload capacity and ductile performance characteristics.

A well-designed and upgraded facility can withstand earthquake forces well in excess of design forces determined from the static base shear derived from the *Uniform Building Code* (UBC) (Ref. 2). Principles for good earthquake resistance for upgraded structures include:

- Good configuration
- Continuous and redundant load paths
- Detailing for ductile behavior
- Tying structural and nonstructural elements together so they function as a unit with deformational compatibility
- Attention to the performance of nonstructural components
- Survival of emergency systems
- Quality assurance for design, materials and construction.

These important considerations are discussed in more detail in DOE-STD-1020. Included are important background information and advice for design professionals.

The primary objective of the seismic retrofit of existing structural systems is to compensate for deficiencies in continuity, stiffness, strength, ductility, toughness, and irregular configurations identified in the various lateral-force-resisting elements of the buildings and their connections. These seismic deficiencies are identified through the application of the seismic evaluation procedures presented in Chapter 7a.

If retorfitted structures are to conform to DOE-STD-1020, strengthening criteria must be the same as design criteria for new structures. Procedures for retrofit should include:

- Review of the detailed seismic evaluation findings
- Verification of existing field conditions, quality of materials, and construction workmanship
- Design of a retrofit system that mitigates the weaknesses found in the evaluation process.

The retrofit design should address all of these considerations to provide the upgraded structure, equipment, and life-support systems with good earthquake resistance. The end result should incorporate a formal lateral-force-resisting system with no weak, brittle, or missing links, which will ensure that all tributary inertial loads are delivered through the structure's foundations into the soil or rock materials below.

Strengthening Guidelines for Seismic Retrofit of Existing Structural Systems

Seismic retrofit design is more difficult than designing a new earthquake-resistant structure. Design engineers must work with existing structural systems and materials, configurations, hidden conditions, and existing architectural, mechanical, and electrical elements. The ability to carry out the seismic retrofit of buildings requires the application of sound engineering principles, judgment, and ingenuity based on experience designing similar retrofit projects and observing their construction.

Structural retrofit of buildings can be accomplished through a variety of approaches, each with its merits and limitations. The following considerations influence or even dictate the choice of retrofit techniques:

- Need for continued use of the building during seismic retrofit
- Architectural layout and appearance of the retrofitted building
- Impact of the retrofit scheme on architectural elements and mechanical and electrical systems, including new requirements
- Cost of the retrofit work
- Other nonstructural considerations.

Most professionals experienced in seismic retrofit agree that if existing bracing elements (and their connections), such as horizontal diaphragms or bracing and vertical bracing

elements (shear walls or frames), are found to be deficient, the addition of supplemental interior or exterior vertical bracing elements will probably be the most cost-effective solution, possibly eliminating the need for strengthening the existing structural elements and their connections.

The following references contain more specific guidelines, commentary and details for strengthening existing buildings, or designing new buildings for earthquakes.

- UCRL-CR-l06554, *Structural Concepts and Details for Seismic Design*, Lawrence Livermore National Laboratory, Livermore, California, September 1991.
- *Seismic Design for Buildings*, Joint Departments of Army, Navy and Air Force, 1992.
- *Seismic Design Guidelines for Upgrading Existing Buildings*, Joint Departments of Army, Navy, and Air Force, 1988.
- *NEHRP Handbook of Seismic Rehabilitation of Existing Buildings*, FEMA 172, 1992.
- *United States Postal Service Seismic Rehabilitation Guidelines*, Applied Technology Council, ATC 26-4, 1992.

ATC 26-4 presents the latest nationally applicable guidelines for the seismic rehabilitation of buildings. It was developed to provide design professionals with feasible design alternatives for upgrading seismically deficient United States Postal Service (USPS) buildings. These strengthening techniques also are directly applicable to DOE Performance Categories 1 and 2 SSCs, and more generally, for all facilities deficient in earthquake resistance. They are categorized according to the 15 model building types in the *NEHRP Evaluation Handbook* (FEMA 178, 1992), that have been used on similar buildings throughout the United States. During development, they were carefully reviewed by structural engineers with extensive seismic retrofit design experience and by contractors with seismic retrofit construction experience.

ATC 26-4 discusses the seismic-resisting elements of each of the 15 model buildings, summarizes their common structural deficiencies, and describes alternative strengthening techniques. Included is a series of drawings illustrating generic and conceptual strengthening designs for each building type.

Although ATC 26-4 is recommended for use by seismic retrofit design professionals, DOE facilities managers and in-house engineering staff will benefit greatly from reading about common seismic deficiencies in the 15 model building types and about the alternative seismic strengthening techniques described therein. ATC 26-4 aids in understanding numerous factors that must be taken into consideration when planning and executing a seismic rehabilitation project. In addition to the 15 building types, it also recommends retrofit conceptual designs for seismically deficient, unreinforced and reinforced brick, concrete block, and hollow clay tile walls. These particular techniques are unique to ATC 26-4 and not presented in other readily available seismic retrofit references.

Seismic Retrofit of Structures Using Seismic Isolation

Seismic isolation, also known as base isolation, is a practical alternative to conventional seismic rehabilitation (fixed-base) of existing buildings, nonstructural elements, and life-support systems. Although this subject is thoroughly described in Chapters 6d and 8c, a brief discussion follows.

Seismic isolation is based on the premise that a structure can be substantially decoupled from damaging horizontal components of earthquake ground motions. Earthquake-induced forces may be reduced by a factor of five from those that a conventionally rehabilitated fixed-base structure would experience. This reduction in earthquake-induced forces may be sufficient to avoid strengthening some structural and nonstructural elements that would have required strengthening if the conventional fixed-base seismic rehabilitation were used.

The use of seismic isolation in the retrofit of existing structures, as well as in the seismic design of new structures, has gained acceptance in recent years. To date, the majority of seismic-isolated structures have been existing structures.

There are five primary considerations that generally motivate owners to choose isolation for retrofit:

- Functionality. The facility must remain open and operational during and after an earthquake or be able to resume operation within a short period of time.
- Protection of contents and nonstructural elements. Important contents and nonstructural elements must be protected against damage from earthquake shaking.
- Investment protection. Long term economic loss from earthquake damage must be mitigated.
- Historical building preservation. Seismic rehabilitation, modification, or demolition of historical building features must be minimized.
- Design Economy. Seismic isolation is the most economical construction alternative because of building size and/or complexity.

The above summary was taken from a paper, *Guidelines for Design Criteria for Base Isolation Retrofit of Existing Buildings*, by Charles A. Kircher, Charles A. Kircher & Associates and Robert E. Bachman, Manager, Civil/Structural Engineering Department, Fluor Daniel Incorporated (1991).

Seismic isolation is not a *cure-all* for all structural and nonstructural deficiencies. Each structure is unique and specially developed seismic-rehabilitation design criteria are needed for retrofit. As well, the same principles of seismic rehabilitation for the fixed-base conventional rehabilitation design must be followed in seismic-isolated rehabilitation designs, even though using lower seismic force levels that result from the seismic isolation.

Chapter 6d provides a detailed explanation of the concepts embodied in seismic isolation and energy-dissipation systems and discusses design principles, code requirements and applications to new construction. Chapter 8c covers application of the technique to the retrofit of existing buildings.

Strengthening Guidelines for Seismic Retrofit of Nonstructural Elements and Life-Support Systems

Retrofit guidelines for nonstructural and life-support systems are based on national standards and guidelines for the design and installation of new nonstructural life-support systems. The following references are recommended:

- UCRL 15815, *Practical Equipment Seismic Upgrade and Strengthening Guidelines*, Lawrence Livermore National Laboratory, Livermore, California, September 1986.
- UCRL 15714, *Suspended Ceiling Systems Survey and Seismic Bracing Recommendations*, Lawrence Livermore National Laboratory, Livermore, California, August 1985.
- The Sheet Metal Industry Fund of Los Angeles (SMACNA), *Guidelines for Seismic Restraints of Mechanical Systems and Plumbing Systems*, Los Angeles, California, 1992.
- The National Fire Protection Association (NFPA), *Standard for the Installation of Sprinkler Systems*, NFPA 13, 1994.
- M-012 Rev. 5, *Mechanical Engineering Department Design Safety Standards Manual*, Lawrence Livermore National Laboratory, Livermore, California, May 15, 1982.
- *Guidelines for Earthquake Hazard Mitigation for Data Processing Facilities*, FIMS in cooperation with VSP Associates, Inc., Sacramento, California, June 1987.
- R. Kennedy et al., *Use of Seismic Experience and Test Data to Show Ruggedness of Equipment in Nuclear Power Plants*, Rev. 4, Senior Seismic Review Advisory Panel (SSRAP). Prepared for the Seismic Qualification Utility Group (SQUG) and in cooperation with the Office of Nuclear

Reactor Regulation, U.S. Nuclear Regulatory Commission, February 1991.

- *Workshop on Walkdown Procedures to Mitigate Natural Phenomena Hazards,* Lawrence Livermore National Laboratory, presented at the Third DOE Natural Phenomena Hazards Mitigation Conference, St. Louis, Missouri, October 18, 1991.

- *Seismic Design and Performance of Equipment and Nonstructural Elements in Buildings and Industrial Structures,* ATC 29 Seminar Notes, Applied Technology Council, Redwood City, California, 1992.

- *Guidelines for the Seismic Rehabilitation of Buildings,* ATC 33, Applied Technology Council, Redwood City, California (in press).

- *Guidelines for Design Criteria for Base Isolation Retrofit of Existing Buildings,* C.A. Kircher and R.E. Bachman, Proceedings of the 60th Annual Convention, Structural Engineers Association of California, Palm Springs, California, October 10-12, 1991.

Other References

1. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities,* DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., April 1994.

2. *Uniform Building Code,* International Conference of Building Officials, Whittier, California, 1994.

Chapter

8b

Retrofit Methodologies

John J. Earle
Daniel Shapiro

Introduction

Earthquake-induced phenomena such as shaking, tilting, sliding, and shifting have long been observed and recorded. Builders and engineers investigated and reviewed each disaster to learn and develop effective design and construction methods to mitigate future destruction. Structural response to earthquakes and consequent damage frequently seemed to defy logic; not easily conforming to discernible patterns. Impressions and observations from many seismic events had to be catalogued to begin to understand building reactions to a vibratory environment and to determine which principles of mechanics were applicable to the total structure, rather than individual parts. Gradually, researchers and engineers pinpointed certain building elements essential for structural integrity during earthquake motion.

This process was difficult. Certain building materials or methods used successfully in nonseismic areas were found extremely hazardous and not adaptable to construction in seismically active areas. For example, unreinforced brick masonry performed well under gravity and wind-load effects, but was not suitable to resist severe shaking. Unreinforced structures of concrete block, precast concrete elements, or similar brittle construction materials have exhibited marked susceptibility to seismic damage. Steel structures, wood-framed buildings, properly reinforced concrete and reinforced masonry showed better resilience and resistance to earthquakes; most were able to survive with less damage.

Older buildings often contain flaws and critical weaknesses, although they may have conformed with the building code enforced at the time they were built. Once their weaknesses are discovered and defined, corrective measures can be taken to provide a new bracing system, restore or strengthen an existing system, install supplemental bracing, alleviate local weaknesses, insert new primary elements such as shear walls, or completely rehabilitate the structure with the incorporation of a new lateral-load-resisting system.

Solutions must be evaluated regarding degrees of risk to life, safety, contents or operations. Other factors to be considered are anticipated useful life of the structure, importance of continued occupancy after a major earthquake, economic expediency of permanent abandonment after severe damage, protection of irreplaceable records and computer storage, or other peformance criteria. Engineering solutions may differ depending on the assessment and resolution of these factors.

Managers should recognize that full compliance with building codes and accepted

earthquake engineering principles will not guarantee resistance to major seismic events with possibility of no damage. Building codes and regulations emphasize the magnitude and application of seismic loads for analysis and include basic design requirements. They do not, however, provide sufficient information to design for damage control. Consequently, it is important that structural engineers experienced in designing for earthquake resistance be employed to provide design services. It is also important that the professional in charge of design fully understands the client's needs regarding performance objectives.

Significant constraints often arise in rehabilitation work. Certain restraints may be imposed because of inability to interrupt experimental programs or production processes or because operational requirements preclude an optimum solution. Compromises may weaken an otherwise effective structural system, especially if certain structural elements have to be offset, penetrated, reduced in size, or otherwise modified. Where such compromises are necessary, they must be compensated for so that equivalent resistance is achieved.

Once a deficiency has been diagnosed, corrective work usually has one of the following goals:

- Temporary or partial abatement to reduce risk and prevent collapse or sudden failure
- Full rehabilitation to a minimum level of resistance to ensure life-safety only
- Rehabilitation of the earthquake lateral-force-resisting system to a specified performance level above that required for life-safety only.

The first goal is an expedient (and temporary) measure to reduce risk until a permanent solution to reach the second or third goal can be achieved. The second goal is intended to ensure life-safety only, but not necessarily protection against significant damage. In principle, structural damage could be so extensive that the building would have to be torn down after the earthquake, but there should be no loss of life. The third goal assumes full protection for life-safety and some degree of property damage prevention.

Temporary or Partial Abatement To Reduce Risk

Once it becomes known that a building is hazardous, it is usually not feasible to evacuate the occupants immediately. In this case, expedient measures should be taken to reduce principal hazards and increase protection of life-safety without regard to survival of the building as a useful structure after an earthquake. One goal of temporary abatement is to brace a building to survive as an integral structure long enough to evacuate the occupants after an earthquake. Another is to buy time until a permanent fix can be achieved or until occupants can be relocated to a safe location.

Sudden local failure of building elements, shifting of the building off its foundation, and partial or complete collapse are obviously major hazards. Any expedient and inexpensive method of bracing that provides temporary protection for life-safety can be used, including any combination of resisting systems, whether they are integrated with or external to the main structure. Local strengthening of floors, walls, stairwells, basement walls, etc., can be used to keep the structure intact until evacuation is feasible or a more effective retrofit is achieved.

It is not necessary for an emergency or temporary lateral-load-resisting system to conform to codes, regulations, or standards to fulfill its objective. However, code formulas are useful in determining appropriate seismic loads. Rough approximations can be used to arrive at lateral force coefficients. Normally, amplification factors can be ignored unless the building is located in an area where soil failure or soft soil can cause major amplification of seismic loads. Advantage may be taken of stress increases allowed by the code or other documents when using tabulated nail and fastener values. Steel bracing may be designed for full allowable stresses, but careful consideration should be given to limiting deflections. For example, deflection limitations would be especially significant for brick walls or brick or masonry veneer where these brittle elements could fall on individuals or block exitways.

Usually the easiest way to improve seismic resistance is to find ways of reinforcing existing structural elements (walls, floors, etc.). This is

normally possible for wood-frame buildings, because they have significant inherent strength (Figs. 8b-1 and 8b-2). Designers can usually create ways of strengthening roof and floor diaphragms by renailing existing sheathing to supporting joists. If there is a linoleum or asphalt tile-finished floor, renailing can be done directly through the floor coverings. Roofing materials may prevent direct access to roof sheathing, but gypsum board or plywood ceiling panels can be nailed to the underside of the roof or ceiling joists to create an effective diaphragm. The most critical point for renailing is at the exterior perimeter of the roof and floors and at the boundaries of interior partitions and corridor walls (Fig. 8b-3). At exterior walls, floor and roof sheathing can be nailed to solid blocking that can be inserted from the inside by removing access strips of wall finishes at wall top plates.

Most existing horizontal (floor and roof) diaphragms can function fairly effectively without easily identified chord elements. Although new chords are usually not a top priority, the designer may decide to provide them at diaphragm boundaries. In wood construction, wall top plates can serve this function; however, they are rarely adequately spliced so they must be made continuous. Renailing these plates can be accomplished by exposing them from the inside. Extra nailing is usually necessary where the top plates have been originally spliced. The actual location of existing splice points must be determined by investigation. Manufactured sheet metal straps can be used to splice existing top plates.

Gypsum wallboard-sheathed walls can be utilized like shear walls. It is best to use walls that are in line (or nearly in line with one another) that are at least four feet in length. Conventional gypsum board nails and nail spacing are usually inadequate for earthquake resistance, so wallboard panels usually require renailing. Such shear walls are most effective if they have gypsum wall board on both sides and both sides are renailed. Critical nailing lines are at the top and bottom plates and end studs. Stronger shear wall panels can be developed by stripping off the gypsum wallboard and applying plywood to the bare studs, or to a lesser extent, by applying plywood over the gypsum wallboard. Sill plates of walls should be renailed to existing blocking or

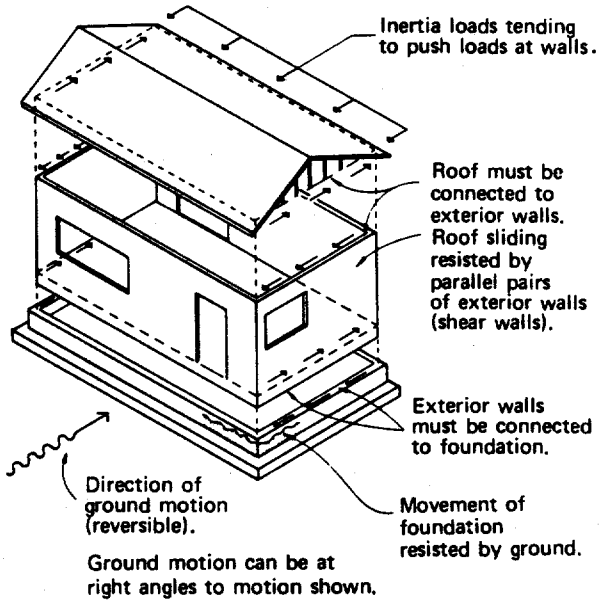


Fig. 8b-1. Seismic loads on typical wood-frame structure.

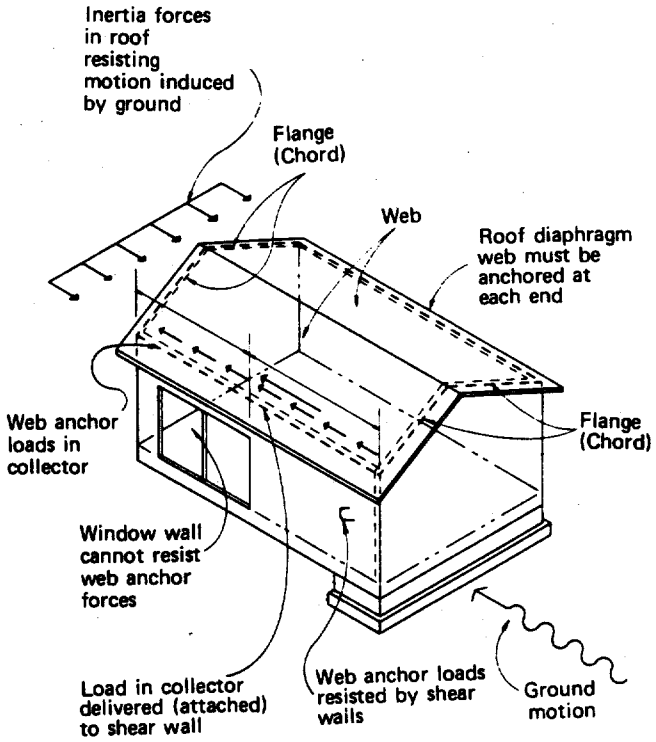


Fig. 8b-2. Elements of seismic resistance.

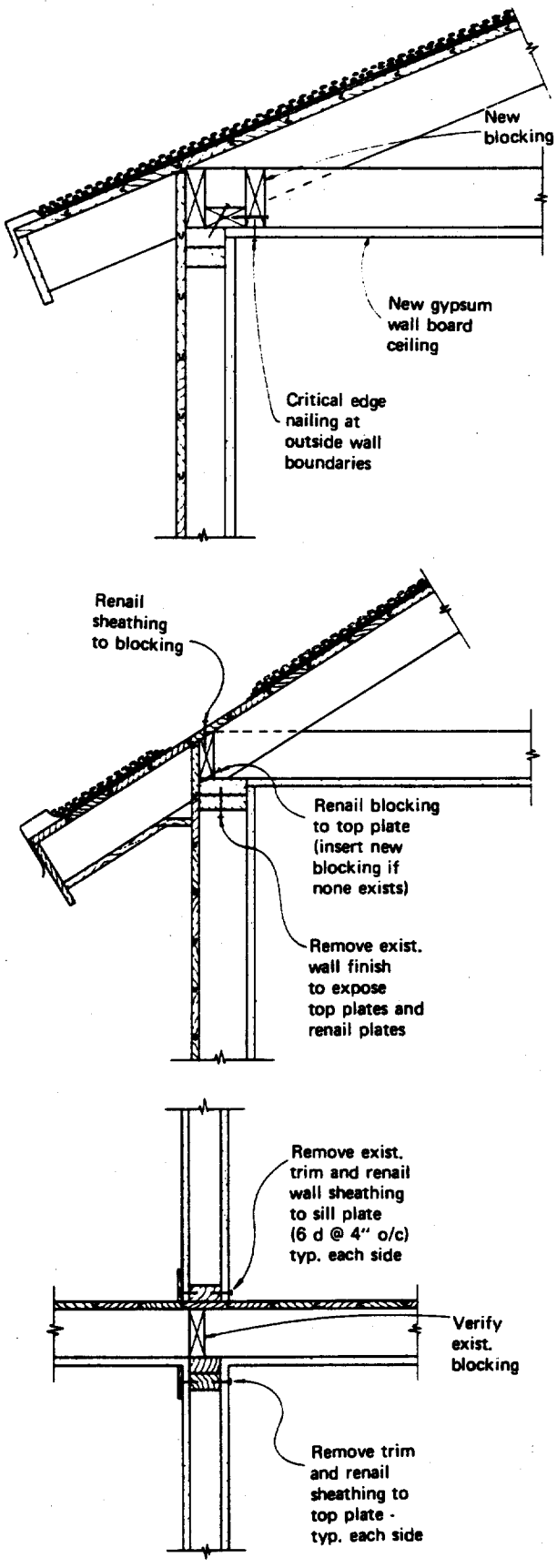


Fig. 8b-3. Critical areas for retrofit renailing in wood-frame buildings.

new blocking inserted in the floor framing to which the sill plates are nailed.

Basement areas or crawl spaces offer the best opportunity to strengthen wood-frame buildings (Fig. 8b-4). Often wall studs and plates are exposed, as is the underside of the first-floor framing. Anchorage of mud sills to foundation walls can be improved by adding drilled-in-place anchor bolts. Blocking can be inserted at plate lines and plywood sheathing applied to the inside face of studs. This plywood should be nailed to mud sills and new or existing blocking. New members to collect lateral loads and bring them to the sheathed walls can be created by applying new framing members to the face of the plywood overlay and nailing through the new collector piece and the plywood to the wall plates or blocking behind. Member lengths as long as possible should be used for these new collectors, and individual lengths must be spliced at butt joints throughout the collector length.

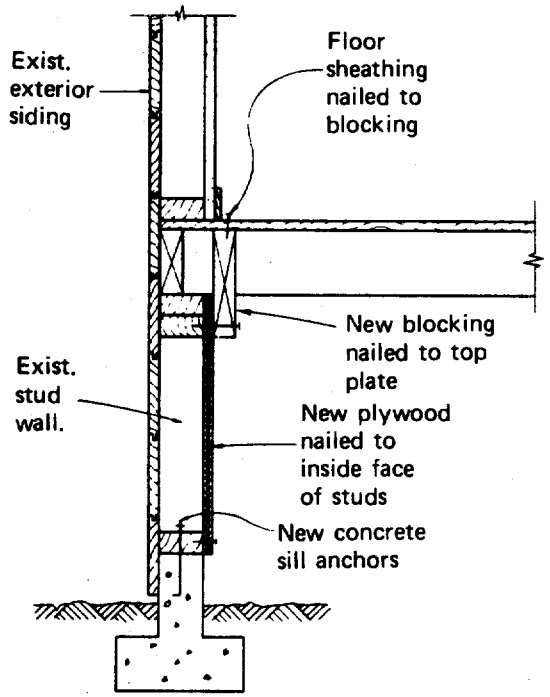


Fig. 8b-4. Wood-frame retrofit in basement or crawl spaces.

Wood structures are flexible and thus can be buttressed by a support system external to the building itself. Cantilevered poles, anchored in the ground and attached to the building, fit this criteria (Fig. 8b-5). Poles should be inserted on each face of the structure, so that opposite poles across the structure can be tied together at the top with a steel rod tensioned with a turnbuckle. The bottom of the poles may be inserted in oversized holes drilled in the ground and the holes filled with concrete.

More rigid and heavier structures, especially brick and reinforced concrete, are not as easily strengthened by temporary measures. Unless new bracing is carefully designed, it may be less stiff than the rigid structure it attempts to brace. If so, the new bracing will probably become effective only after the existing stiffer elements have failed (Fig. 8b-6). This becomes very significant in terms of brittle column failure, which can be sudden if a column buckles. Consequently, bracing or shoring systems must be designed to prevent excessive lateral deflection of concrete columns.

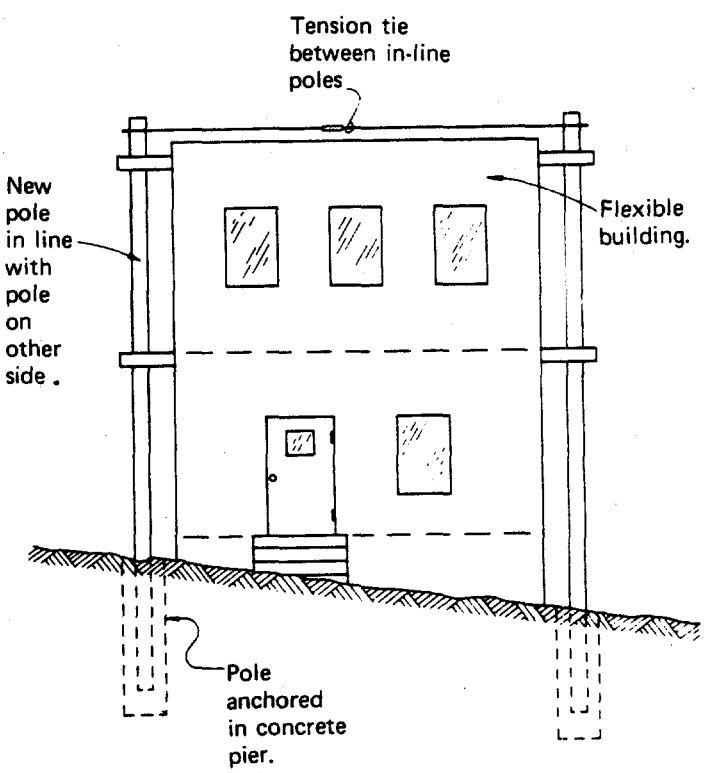


Fig. 8b-5. Wood-frame structure retrofitted with external cantilevered poles.

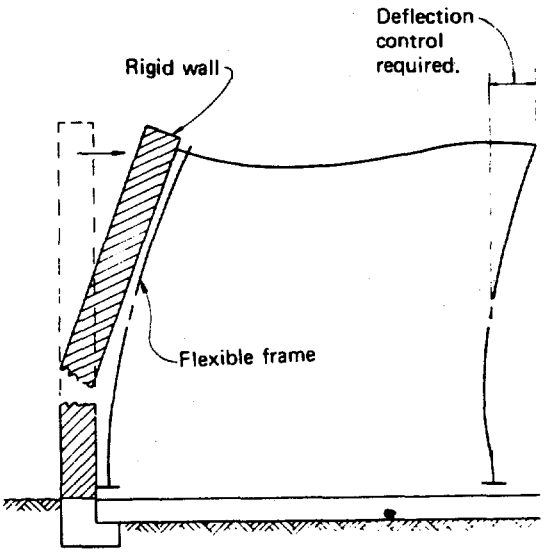


Fig. 8b-6. Failure of brittle wall caused by excessive deflection.

Temporary abatement only serves its purpose when a short, but definite, building life is intended. Buildings that have been minimally braced for temporary resistance should have a schedule for evacuation or permanent rehabilitation. One should not rely on temporary abatement to extend the life of buildings.

Rehabilitation for Life Safety Only

Building code regulations establish minimum earthquake-resistance standards that are intended to protect occupants against injury from localized structural failure and/or building collapse. Structural damage should be limited so occupants can exit the building even if it is otherwise badly damaged. Also, occupants should be protected against falling hazards such as hung ceilings, mechanical piping and ductwork, and electrical fixtures.

When rehabilitating a building to meet seismic life-safety standards, emphasis should be placed on work needed to bring the building into full compliance with the specific seismic provisions of the code. One should expect better building performance and less damage to buildings brought up to this level of structural compliance than for buildings braced for temporary occupancy only.

Economic considerations usually dictate that remedial work be completed with as little interruption and dislocation to ongoing operations as feasible. This should be kept in mind when exploring alternative schemes for strengthening an inadequate lateral-force-resisting system.

Department of Energy (DOE) facilities that pose a hazard to employees, the public, and/or the environment must be brought up to full compliance with the performance goals of DOE Orders and Standards (see Chapters 4 and 8a) or the *Uniform Building Code* if applicable. If a building houses highly valuable equipment, contents, or operations that are sensitive to vibrations and deflections, these considerations may dictate design requirements for seismic retrofit that may be more stringent than usual code standards.

Designers should not proceed without adequate geotechnical data. Geologic hazard maps and other documents that provide information about earthquake hazards in the vicinity should be reviewed. U.S. Geological Survey (USGS) maps can be very useful in this regard. New geotechnical reports are only necessary when existing data are insufficient or DOE regulations or standards require them.

Sufficient geotechnical information, including allowable soil-bearing values, may be available in reports prepared for original construction. However, designers may need verification from the geotechnical engineers who prepared the original report that new loads to be superimposed on the bearing stratum are close enough to recommended values given in the original report to be acceptable. The need for new borings should always be evaluated. Normally, ground acceleration coefficients related to theoretical site periods developed from geophysical tests are only warranted for unusual structures or special performance goals, but soil amplification should be considered in determining force levels.

Most earthquake analyses use static lateral loads based on dynamic considerations. The principal parameter is the structure's natural period. The insertion of additional bracing or shear walls and strengthening or stiffening of diaphragms or bracing trusses (all of which must interact with existing elements) could alter the building's natural period and significantly change its response to earthquakes. Designers must take into account these changed conditions. Codes usually require inclusion of an importance factor (I) when calculating seismic loads. For most buildings I = 1 is acceptable, but for *Essential Facilities* that must be operable during and after an earthquake, the factor is raised to I = 1.25 in the *Uniform Building Code* (UBC) and as high as 1.5 for related nonstructural and support components. These higher I factors are applicable to fire stations, police stations, communications centers, first-aid stations, garages housing emergency vehicles, and buildings housing hazardous materials or unusually valuable equipment. For retrofitting DOE facilities with special performance requirements see Chapters 4 and 8a. Keep in mind that good design principles and techniques are usually more important than the magnitude of force levels.

Concentric diagonal bracing carrying only axial load (tension or compression) is effective in resisting earthquake loads when retrofitted into an existing building. However, the connections of bracing elements sometimes exhibit brittle qualities and fail before the strength of the bracing member is fully developed. Consequently, bracing members are designed for loads greater than the calculated axial force (see UBC 1994), but the allowable stresses used to size the member may be increased 1/3. For retrofit, however, the authors recommend that connections be designed to resist the code required loads in the bracing member without the 1/3 increase in allowable working stresses. Members and connections other than the principal earthquake bracing may be designed on the basis of working stresses with the 1/3 increase where applicable.

Eccentric braced frames are coming more into use in earthquake design. Such systems add ductility not available in concentric braced frames. Energy is absorbed in a link beam, lessening the potential for buckling of the diagonal brace. It is necessary to follow published design procedures and detailing recommendations for the connections between the brace and the link beam.

All acceptable building materials can be used to strengthen or create a lateral-force-resisting system. Each material has its

advantages. Structural steel is relatively light and easily altered at the construction site. It can also be welded during construction, an important consideration when it is necessary to cut and fit members into unusual situations. Wood has some of the same advantages as steel, but must be joined together by a limited variety of connectors. Reinforced concrete is especially useful because its heavy weight can be used in *dead-man* anchors, buttresses, drilled piers, and shear walls. Reinforced concrete block masonry can be useful when adding new shear walls because individual block units are simple to work with and easily transportable. Also, block walls require no forming.

Whenever possible, it is best to restore, strengthen, and/or replace the primary bracing elements already existing in a structure. One begins at the roof level by assessing the capability of the roof diaphragm. Most older wood-frame roofs were built with diagonal sheathing that has significant strength as a diaphragm. However, it is necessary to uncover the sheathing boards and verify the nailing at the perimeter exterior walls. If existing diaphragm strength is poor, such as where straight-laid sheathing exists, a plywood overlay can be placed on top of the existing sheathing and nailed in accordance with code regulations. New nailing of the plywood to the old sheathing and of the sheathing to joists and/or blocking is needed. This is especially important at the perimeter exterior walls. Because old wood hardens through aging, predrilling may be required for proper installation of nails. If the existing sheathing is badly deteriorated (dry rot, termites, etc.), it may be necessary to strip it off and replace it with plywood. Diagonal truss bracing also can be used on the underside of the roof joists when some effective way can be found to transfer the roof loads into the truss and the truss loads into the wall-bracing elements.

Care must be exercised to ensure that the new diaphragm system is properly anchored to chord members at the diaphragm boundaries (usually the exterior walls) (Fig. 8b-7). Existing wall top plates (in wood construction) can be used as chords and collectors, but plate internailing is required, with special attention given to plate splices. The diaphragm loads must be transferred to the plates through blocking or joists. Sheet-metal clip angles are useful in connecting the blocking or joists to the

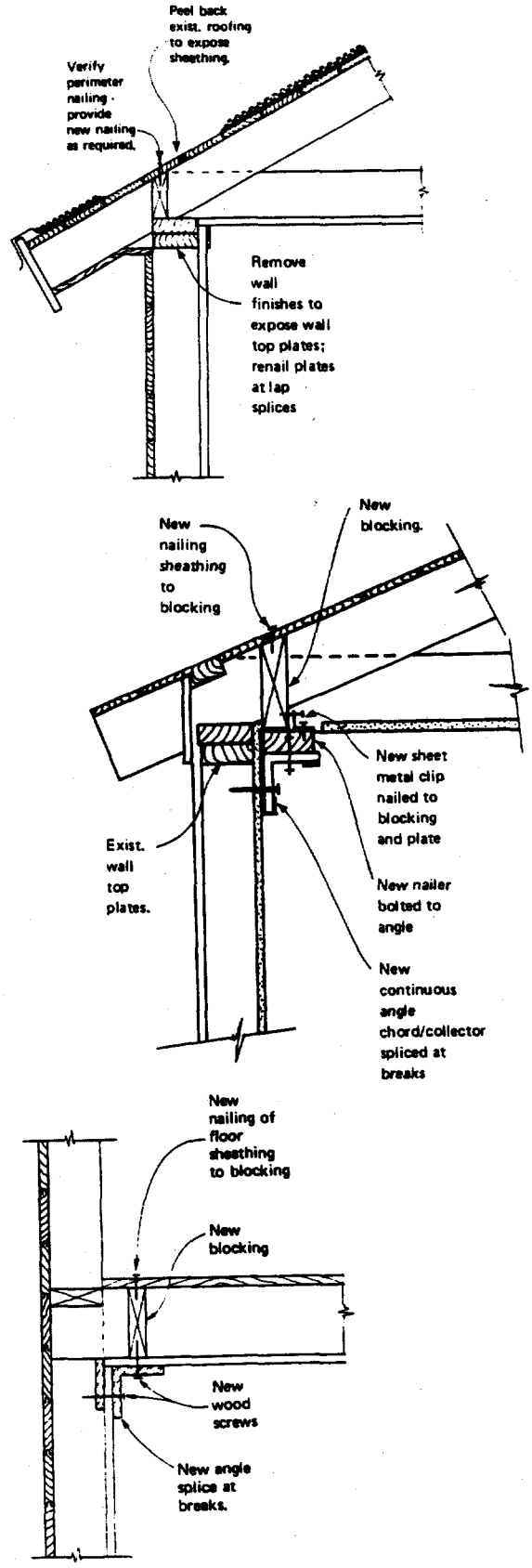


Fig. 8b-7. Retrofitting diaphragm connections in wood-frame structures.

plates. Where top plates are inaccessible or inadequate, steel angles or channels can be inserted at the roof ceiling level and the diaphragm loads delivered to these chord members by nailing or screwing to wood nailers attached to these steel members. Screws are a very effective and positive way of making the shear transfer at diaphragm boundaries.

Steel-framed structures without a proper diaphragm can be corrected by removing the existing decking and replacing it with hat-shaped steel decking (Fig. 8b-8). The steel decking is easily welded to the existing steel framing. If supplemental framing members are needed, they can be inserted in the existing framework before applying the steel deck.

Heavy materials such as concrete precast elements (in roofs or floors) that significantly contribute to inertia loads should be replaced wherever feasible with lighter materials such as steel decking. Where replacement is not possible, interconnection of the precast units, a common weak point, should be studied and, if found deficient, should be improved, or a new lightweight concrete topping can be applied.

In steel-frame buildings, existing struts and purlins may not be properly spliced for continuity. At floor or roof diaphragm boundaries, the members selected as chords should be spliced so that the member becomes continuous for the full length or width of the building. Purlins or struts selected as collectors must be similarly spliced.

Shear walls or diagonal bracing can be used to transfer floor and roof diaphragm loads to the foundation. Diagonal bracing is usually the simplest and least expensive solution (Fig. 8b-9). For exterior walls, the diagonals can go on the outside face of the wall, but the effects of out-of-plane eccentricities must be taken into account. For light loads, knee-braced columns can be used, but generally the diagonals should continue to the foundation where they can be anchored directly to existing footings or to new concrete footings. The use of diagonal bracing may result in large uplift loads, and those members taking the vertical component of the diagonal member must be anchored into a foundation of sufficient weight to hold it down. Drilled concrete-filled friction piers can be used to resist uplift if the foundation has insufficient weight (Fig. 8b-10). Friction values should be obtained from geotechnical engineers. Diagonal members must also be anchored for the horizontal force component that the foundation (or piers) must resist.

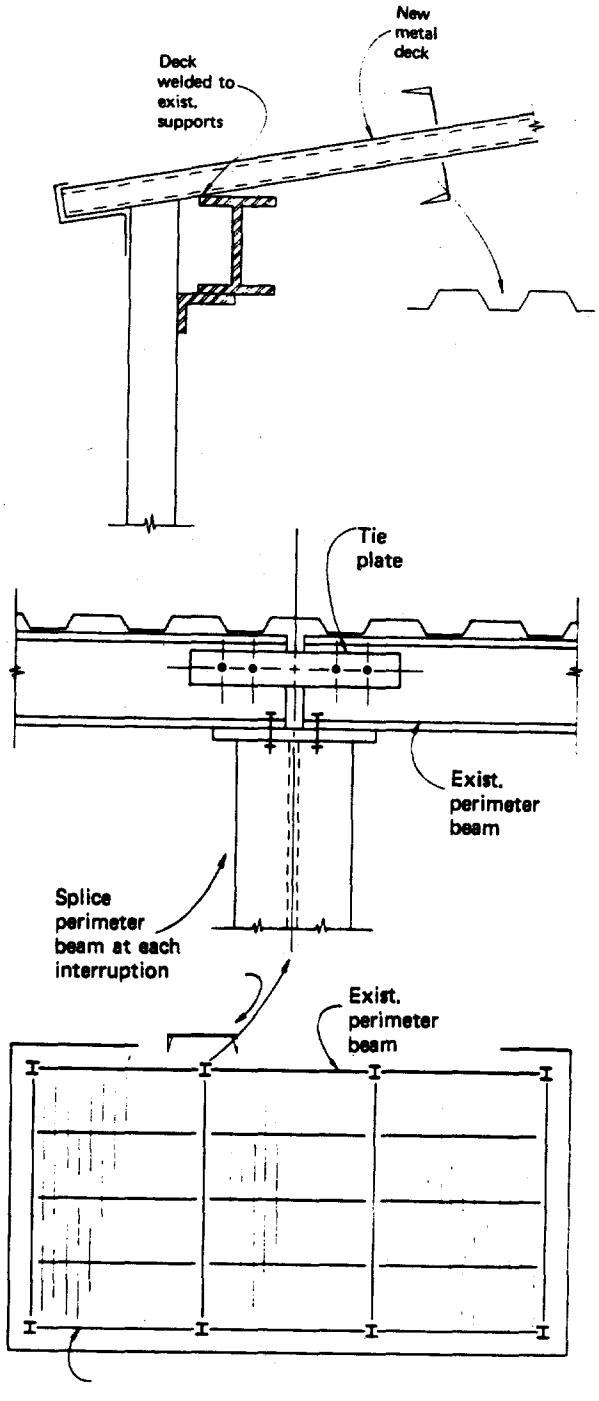


Fig. 8b-8. Replacement of ineffective roof diaphragm with steel decking.

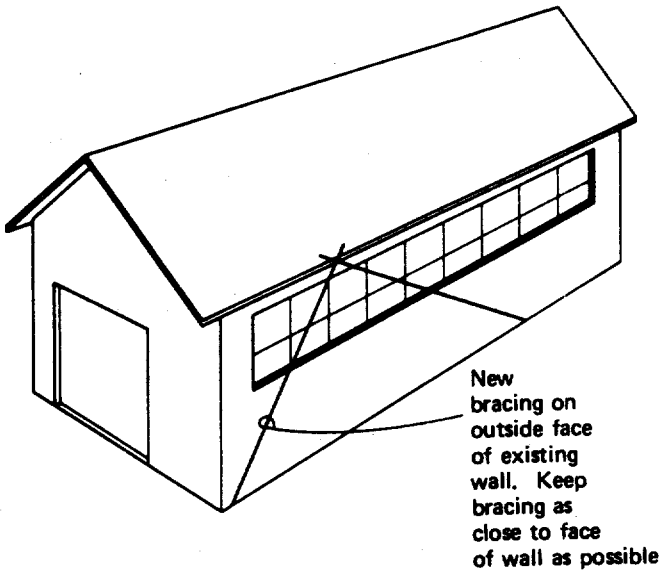


Fig. 8b-9. Retrofit using exterior diagonal bracing.

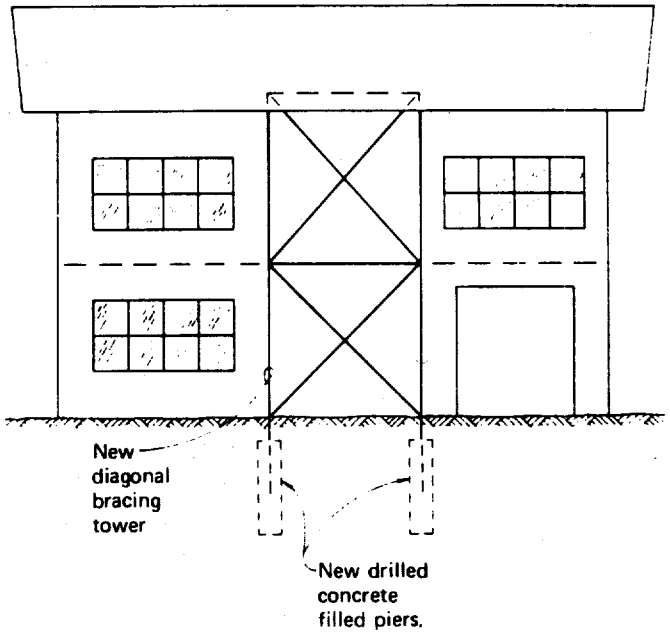


Fig. 8b-10. Use of drilled concrete piers or caissons to resist uplift from retrofitted diagonal bracing.

Older structures may have been designed for wind load and/or seismic loads that are much less than can be realistically anticipated. Because compression members are designed on a conservative basis because of buckling criteria, existing bracing members may be adequate for earthquake bracing using the one-third allowable stress increase. However, rivets or bolts at the connections are usually inadequate. To compensate for this, the individual bracing members can be welded to the gusset connection plates to increase the strength of the connections (Fig. 8b-11).

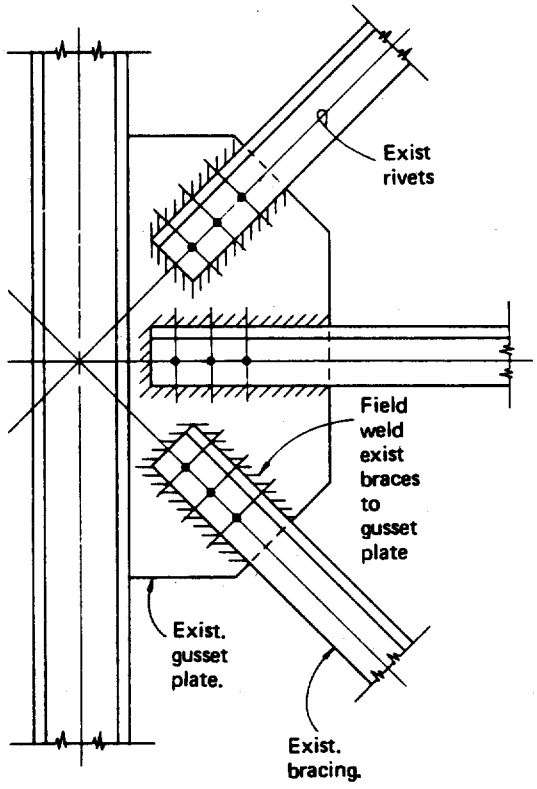


Fig. 8b-11. Use of welding to increase the strength of older bolted or rivited connections.

If walls are available to act as vertical shear elements or panels, they can be tied together through the use of horizontal collectors. Steel angles can be used effectively as collectors for both concrete and wood walls (Fig. 8b-12). At a concrete wall, collector loads can be delivered to the wall by bolting the collector angle to the wall face with drilled-in-place concrete anchors. Allowable anchor values may be selected from the manufacturer's recommended working load values if verified

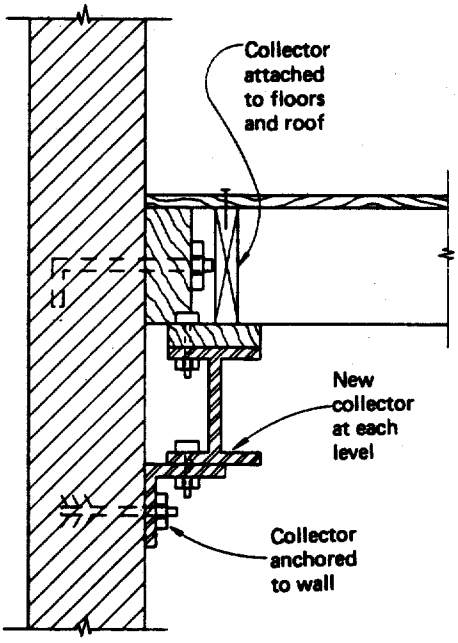


Fig. 8b-12. Retrofitted steel collectors.

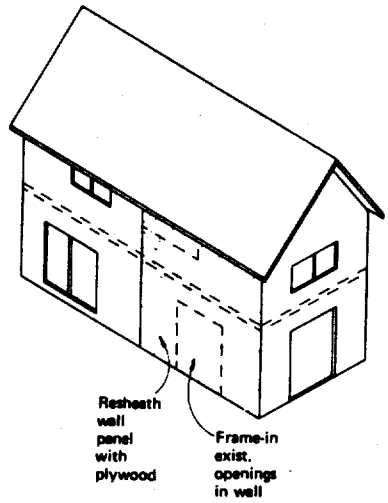


Fig. 8b-13. Resheathing openings in wood-frame walls.

by code certified test results. If diagonally braced bays are spaced intermittently, they also must be tied together by collectors, provided a continuous member capable of carrying the loads is not already in place.

Where existing sheathing materials on wood-frame, shear-wall elements are not able to resist earthquake shear loads, the walls can be resheathed with plywood, or they can be strengthened by closing up window or door openings and sheathing them, or additional wall panel elements can be added through architectural rearrangement (Fig. 8b-13). Uplift can be resisted by using hold-downs anchored to the face of existing footings. If the available footing is not heavy or strong enough to resist overturning effects, hold-downs with rods anchored into new footings of larger size or drilled, concrete-filled piers can be used.

Existing concrete and unreinforced masonry walls can be strengthened by placing new walls adjacent to existing walls and creating composite action by chipping slots or keyways, or inserting steel dowels in the face of old walls (Fig. 8b-14). This usually involves enlarging

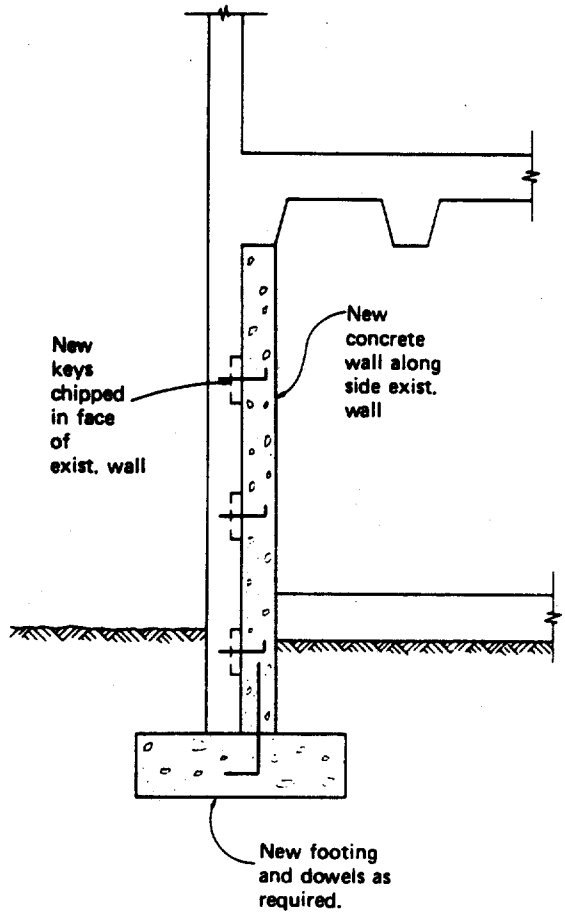


Fig. 8b-14. Strengthening existing concrete or unreinforced brick walls.

the existing footings. In lieu of forming and placing concrete against old walls, the new concrete thickness can be applied by a pneumatic process (air-blown concrete) such as *shotcrete*. The new concrete placed against the old walls must be reinforced for seismic loads perpendicular to and in the plane of the walls. These composite walls must be tied to floor and roof diaphragms. Composite action can also be accomplished through the use of drilled-in-anchors projecting from the old walls into the new. Longer shear walls can sometimes be created by filling in openings between columns or closing up windows.

Reinforced concrete-block masonry is an acceptable alternative when the greater shear strength or weight of reinforced concrete is not necessary. Concrete block is a versatile building material that eliminates forming and can be cut to fit into and around openings. Reinforced concrete-block masonry walls must be tied at the floor and roof lines to resist seismic loads perpendicular to the face of the wall. Block walls also should be dowelled to adjacent columns and beams.

Buildings on sloping sites may be damage prone in a strong earthquake, especially if they are supported by poor foundations or the hillside is not stable (Fig 8b-15). Depending on the position of the buildings relative to the top of the slope, it may be possible to anchor the building or individual wall lines to massive *dead-man* anchors placed in more stable soil formations (Fig. 8b-16). Tension ties can be used to anchor shear-wall lines to the dead-man anchor. These ties can be reinforced concrete beams cast in trenches dug in the ground. Tie rods can be installed in slant-drilled cased holes and the entire assembly filled with concrete. Dead-man anchors must be designed to withstand the load through passive resistance of the soil. In analyzing bending and shear stresses in dead-man anchors, consideration should be given to the distribution of passive soil pressure that may vary along the length of the dead-man anchors.

Bracing of retaining, enclosure, or basement walls can be accomplished either by bracing the exposed faces of the walls or by tying the walls back into the soil behind the walls utilizing the cohesive strength of the soils (Fig. 8b-17). Rock anchors or tie-backs can be used if the drilling crews have sufficient space to set

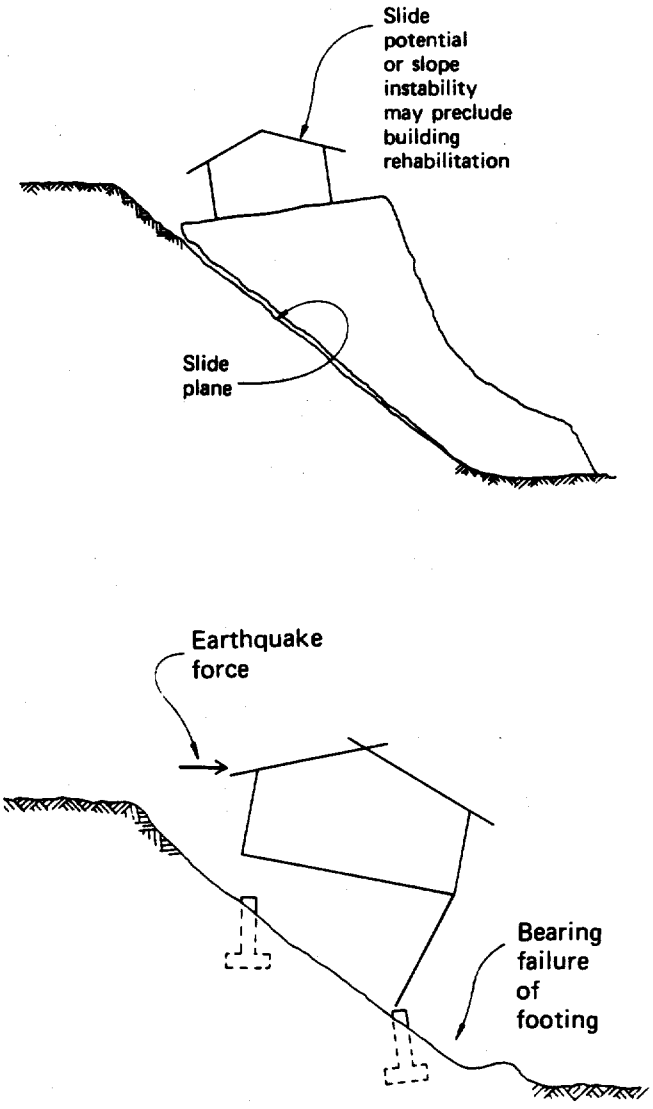


Fig. 8b-15. Potential seismic failures at hillside sites.

up drilling equipment. Anchorage of tie-backs must be located behind the theoretical slope failure plane in the soil. The location of the failure plane and the capacity of the anchor ties must be established from data furnished by geotechnical engineers and confirmed by testing. If tie-backs are to be permanent, measures must be taken to prevent corrosion along their full lengths.

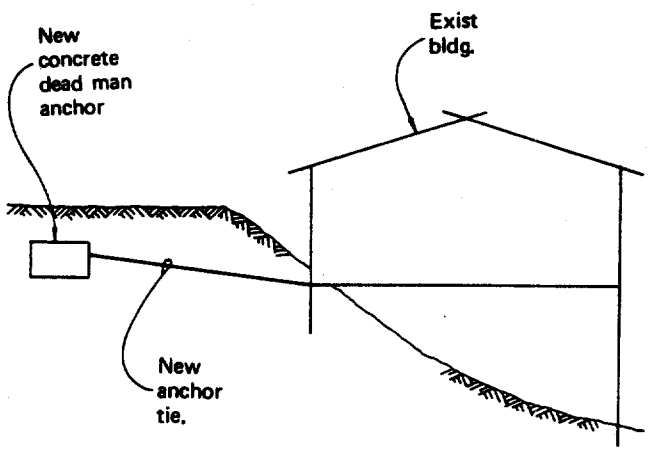


Fig 8b-16. Use of dead man system to anchor slopeside building.

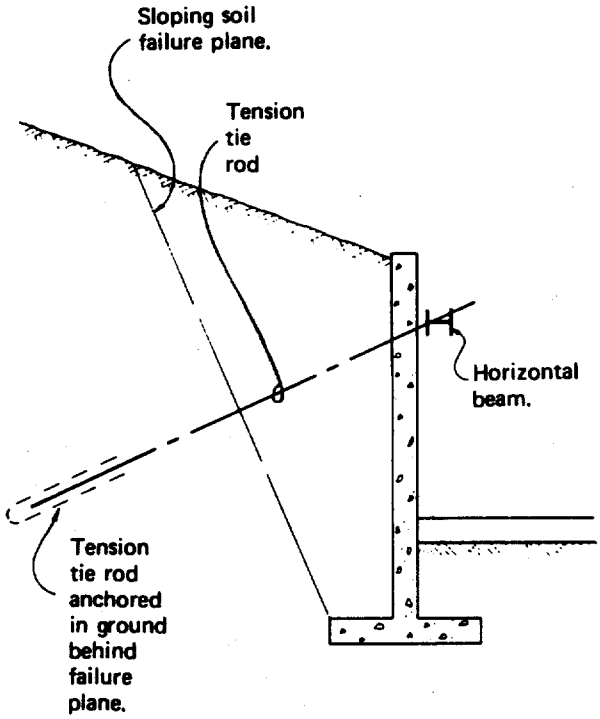


Fig. 8b-17. Use of tie-backs to resist seismic loads.

Full Rehabilitation of Earthquake Lateral Load Systems Above that Required for Life-Safety

In seismic rehabilitation work, the principal goal is generally life-safety. Limiting property damage entails more effort, time, and cost. Rehabilitation work may cause disruption to operations or require evacuation of all or part of a building to do the work. Plans for installing the selected lateral-force system should be carefully reviewed by management, staff, and the operations personnel affected by the disruption, as well as those involved in the plan-check process. The costs of disruption or temporary evacuation should be included in budget estimates. Contingency also should be included for unforeseen costs that are inherent in rehabilitation work, particularly when the work may be affected by operational delays.

Seismic design codes and guidelines for rehabilitation are frequently updated, particularly in the aftermath of damaging earthquakes. Thus, it is necessary to review an existing lateral-load-resisting system in light of the latest developments. Previous assumptions may no longer be valid or in agreement with current practice. A prime example of this situation involves the problems with welded joints in steel frames damaged by the 1994 Northridge, California, earthquake described in the Foreword to Chapter 3.

Similarly, new information about local soil conditions and geologic hazards should be heeded. Known unstable soil conditions should be avoided or corrected.

For highly irregular or unusual situations, designers should not rely on the static force method of analysis. If a building can be realisticaly mathematically modeled and dynamic properties adequately defined, a dynamic analysis should be undertaken.

It is well to emphasize that design for seismic retrofit should incorporate the best concepts for new design. For example, ductility should be incorporated in the rehabilitation in so far as possible. Reinforced concrete moment frames must comply with code requirements for concrete special moment-resisting frames with ductile detailing in high seismic zones.

The compatibility of new and old framing must be carefully considered. Also, deflections of elements that are not a part of the lateral-resisting system (especially columns), but required to resist gravity loads, must be limited to avoid failure. A graphic example of this type of failure was the relatively new parking garage at California State University,

Northridge, during the 1994 Northridge earthquake.

Insertion of new moment or braced frames and/or shear-wall elements may require extensive foundation work (Fig. 8b-18). Shear walls normally have edge or boundary members that carry large axial loads. These direct loads must be carried down to firm foundation-bearing materials. Shoring is sometimes necessary if existing footings have to be enlarged, and underpinning of existing walls and columns may be required where existing footings have to be deepened and/or reconstructed. Sometimes, drilled concrete-filled caissons can be placed adjacent to existing footings and axial wall loads delivered to the caissons through transfer grade beams (Fig. 8b-19). Shear walls may have to be thickened to the extent that they project beyond the original face of the building. In general, it is best to avoid any offset between the center of gravity of the load and the gravity axis of the foundation. In practice, some eccentricity can be tolerated in earthquake design if torsional effects are taken into account. Because foundation work can be the most expensive part of a seismic rehabilitation, geotechnical engineers should be consulted to determine the consequences of exceeding soil-bearing pressures during dynamic loading.

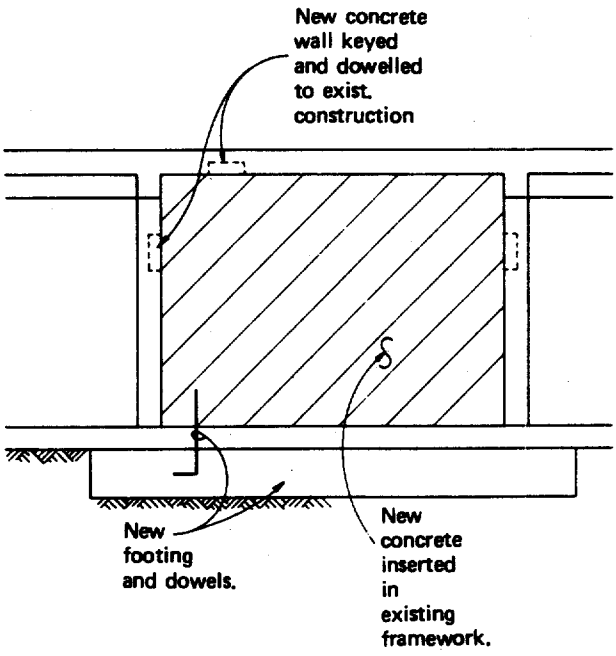


Fig. 8b-18. New foundation for retrofitted shear wall.

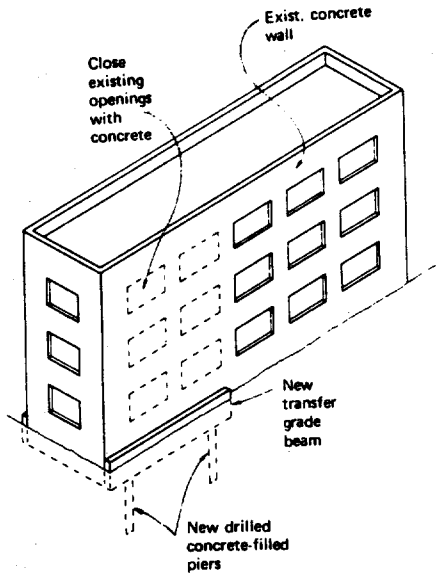


Fig. 8b-19. Use of grade beams and concrete caissons to resist seismic overturning loads from retrofitted shear walls.

Incorporating shear walls is usually the simplest and most direct method of strengthening a building for earthquake loads. Shear walls are most easily located in exterior wall lines, provided that floor and roof diaphragm ratios and diaphragm shear strengths are adequate to span between exterior walls. If diaphragm ratios need correction, or if openings impair the strength and stiffness of the diaphragm, shear walls can be located on interior wall lines (Fig. 8b-20). Corridor walls are most convenient. Because shear walls may create barriers within the building, their locations must be coordinated with other life-safety and functional considerations.

Closely akin to shear walls are buttresses or wing walls. Buttresses can be placed at the exterior ends of the structure, in line with existing shear or exterior walls (Fig. 8b-21). Buttresses must be adequately tied to the existing structure to prevent the building from tearing away from the abutment. To anchor new concrete to the face of existing concrete walls, drilled-in anchors can be inserted into the existing wall face with hooked rods projecting from the anchor. In this way, new concrete can then be placed against the existing wall face encompassing the new anchors.

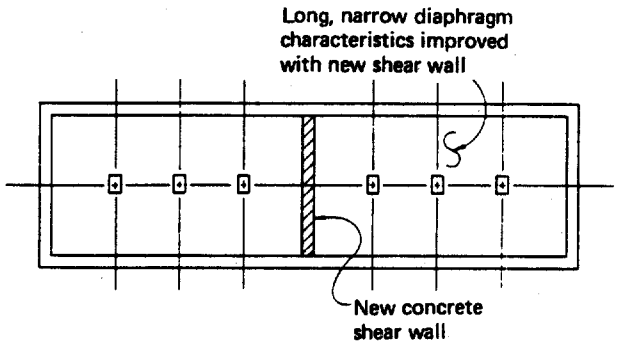


Fig. 8b-20. Use of shear walls to improve the seimic resistance of roof and floor diaphragms.

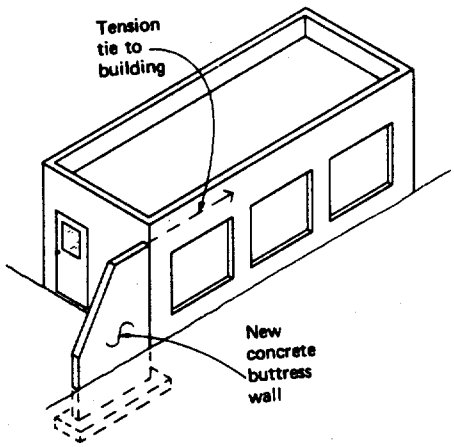


Fig. 8b-21. Buttressing existing structures.

Because buttresses or wing walls are usually located outside the building envelope, they will not have external vertical loading to resist overturning. Careful attention must be paid to foundation design and lateral buckling.

Both diagonally braced and moment-resisting frames can be utilized for strengthing structures (Fig. 8b-22). Longitudinal frame elements are most conveniently kept adjacent to exterior wall lines. Transverse frames can be introduced wherever convenient. Struts and ties must be used to collect and deliver loads to the frame. Floor and roof diaphragms must be secured to collectors. Existing architectural facades may require alteration to accommodate new framing. Generally, moment frames should not be combined with more rigid bracing, such as diagonal bracing or shear walls, nor should any system be employed that will introduce significant torsion.

For wood buildings, plywood shear walls are a preferred solution. A complete system of collectors, ties, and struts should be provided. Roof and floor diaphragms may require reconstruction.

Concrete floors can be strengthened by adding a topping slab, keyed or otherwise bonded to the existing slab. The existing slab has to be analyzed to confirm that it can carry the added weight of the topping slab, especially where heavy live loads are anticipated. Lightweight concrete should be used for this topping slab (even if its shear properties are somewhat less than normal weight concrete) to minimize the added inertia load. Diaphragm shears can be reduced by altering the existing floor area tributary to the diaphragms by inserting new shear walls or bracing lines, thus reducing diaphragm spans. New chords and collectors can be introduced by threading and splicing reinforcing bars for the required lengths.

UCRL-CR-106554, *Structural Concepts and Design Details for Seismic Design,* published by the Lawrence Livermore National Laboratory (LLNL) in September 1991, is an excellent reference for designing rehabilitation projects as well as new buildings.

Some of the newer developments in earthquake-resistant design include energy-dissipating systems such as base (or seismic) isolation and passive devices that absorb energy by slipping, flexing, and distorting structural elements that become sacrificial restraints. These systems require a larger amount of lateral displacement of the supported structure than do conventional fixed-base buildings. Utility lines entering the building have to be altered to accommodate large movements. Basement areas may have to be enlarged.

Rehabilitation by seismic isolation is especially useful because it can often be accomplished while the occupied portion of the building continues in use. It results in less

violent movement of the building, affording better protection for the building's contents. Chapters 6d and 8c provide detailed discussions of seismic isolation and energy-dissipation systems for new and retorfit designs, respectively.

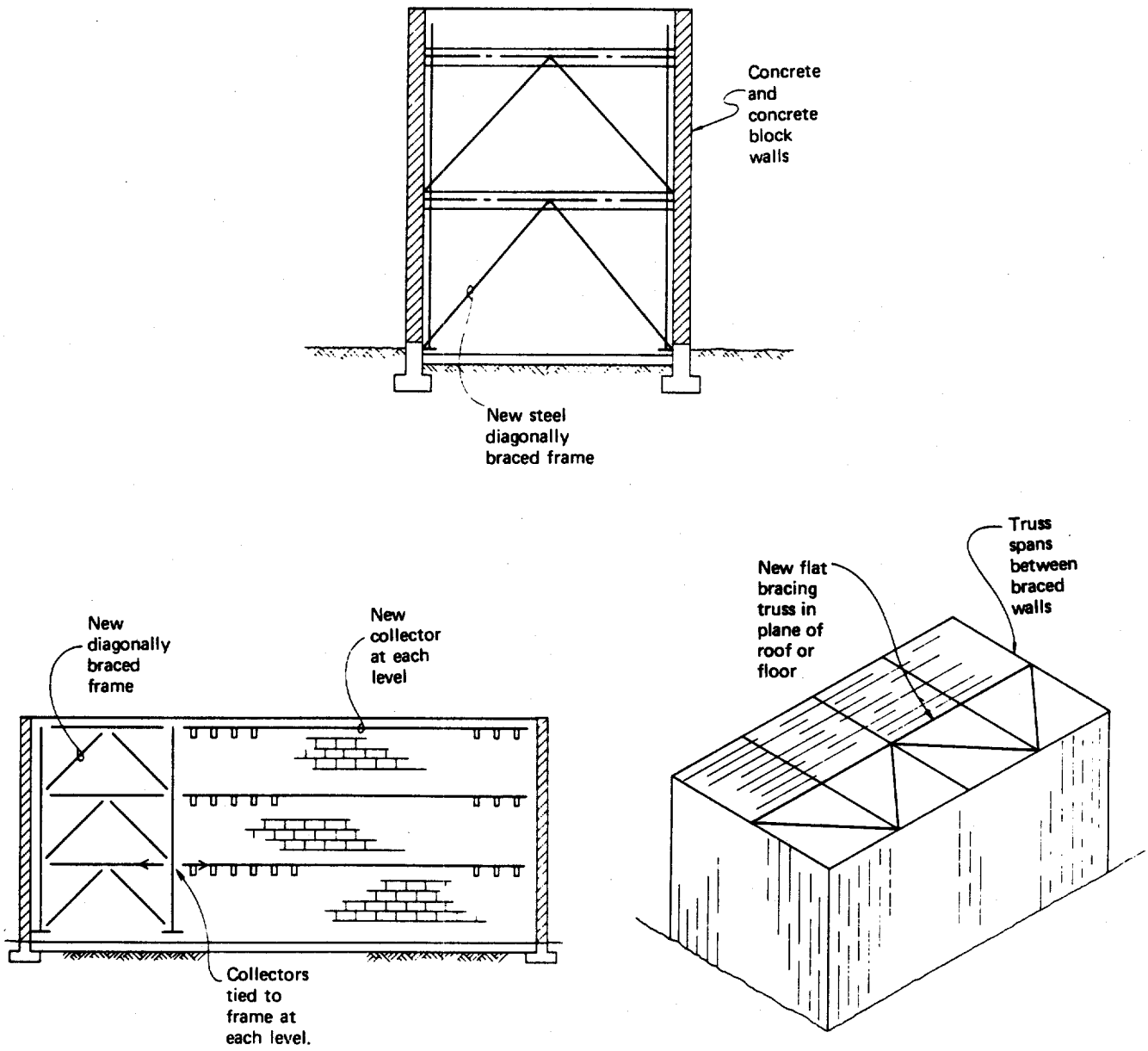


Fig. 8b-22. Use of diagonal bracing to retrofit walls and diaphragms.

Chapter

8c

Seismic Isolation and Passive Energy Dissipation for Building Retrofit

Eric Elsesser

Introduction

The use of seismic isolation or passive energy dissipation for upgrading existing buildings is increasingly an economically viable alternative. Since the first historic building in the United States was seismically upgraded using seismic isolation, a number of other buildings and structures have been upgraded using seismic isolation or passive energy dissipation systems. This chapter discusses issues such as physical limitations, economic considerations, and possible techniques for retrofitting.

Possible Candidates

Existing buildings for which seismic isolation or passive energy dissipation can be effective include those with historic value with archaic materials and brittle finishes, those with precious or motion sensitive contents, and those where interior retrofit construction might not be feasible because of the sensitivity of the contents or processes housed within the structure. Use of isolation retrofit systems, rather than more conventional construction, can often minimize disruption to the existing superstructure.

Seismic Isolation Systems

As discussed in Chapter 6d, seismic isolation means *de-coupling* the building from the ground, severing the rigid structure-to-ground contact with a mechanical device usually located between the structure and its foundation. The de-coupling device is designed to dissipate most of the destructive earthquake energy. Isolation bearings allow large lateral displacements to occur within each isolator, thereby protecting the building superstructure from usual harmful seismic distortions. Seismic isolation devices dramatically reduce seismic impact on buildings and structures. A reduction in seismic forces in turn reduces demands placed on the building structure and assemblies.

De-coupling is achieved by significantly altering the fundamental period of vibration and increasing the damping of the isolator/structural system. The response of the building (structure) can be reduced to 1/4 or 1/5 of that experienced by a fixed-base structure. Isolation increases overall displacement at the structure/isolator interface, but does not increase story displacement (drift).

The isolation systems discussed in Chapter 6d have been effectively used in retrofitting existing buildings.

Major Issues

The major issues to be addressed in determining the feasibility of using seismic isolation for retrofit are similar to those discussed for new building/structures. Necessary behavioral characteristics for the system to be used are the same, namely; self-centering capability, vertical rigidity, and durable materials.

Seismic isolation may be the optimum solution for upgrading buildings where more conventional strengthening methods such as adding steel bracing or concrete shear walls would adversely affect the function of the facility, or where such construction would not be practical because of cost or feasibility of relocating process lines, toxic storage, or sensitive equipment.

Technical limitations that should be considered when judging the feasibility of seismic isolation are that candidate buildings should possess fundamental periods of vibration of 1.5 seconds or less, or be no more than eight to twelve stories in height. Economics tends to dictate that it is only feasible to use seismic isolation for one or two-story buildings if they house special occupancies or critical processes.

Other issues to be evaluated are:

- **Building Resonance**

The basic fixed-base building period must be such as to not induce resonance with the isolation system.

- **Ground Resonance**

The effect of geologic conditions, such as soft soil, may considerably lengthen the period of the building-soil system and make isolation undesirable and unwarranted.

- **Superstructure**

The superstructure system must have sufficient strength and stiffness to be compatible with the isolation system. Existing structures may require supplemental strengthening.

- **Uplift Forces**

The building superstructure supported by the isolators must be proportioned to avoid uplift forces on the isolators. Isolators cannot realistically withstand uplift (tensile) forces.

- **Constructability**

Building framing must allow for installation and future removal and replacement of the isolators. A sufficiently deep crawl space or basement is required to allow isolators to be moved. Individual isolators may weigh from 1,000 to 3,000 lbs or more. Otherwise, the necessary space must be created (constructed) which could require shoring of the building, excavation, and construction of new foundations.

- **Distribution Diaphragm**

All isolators in a system must move together in unison. The existing floor or platform above the isolation interface may need to be strengthened to act as a rigid diaphragm.

- **Isolation Gap**

A 12- to 30-inch-wide gap, or space, is required around the entire isolated building to accommodate horizontal movement of the isolator system. The actual dimension needed depends on the specific isolation system used. Provision for this movement will require specially designed utility interfaces with the building, and specifically detailed entrance/exits, elevators, and stairs.

- **Fire Rating and Durability**

Isolators need a fire rating equal to that required for the structure above. Also, isolators must be constructed of durable materials with a life expectancy of fifty years or more.

- **Testing, Maintenance/Replacement**

Testing requirements for isolators used for rehabilitation are the same as for new buildings. All isolators should be inspected periodically. Those showing evidence of deterioration should be replaced.

Analysis Procedures

The analysis procedures for designing isolator systems for retrofitting are similar to those used for new buildings. Depending on the building, significant effort may be needed to determine the response characteristics of an existing building. In-situ material strengths and moduli, framing and connection details, and evaluation of the physical condition will be required. Code requirements for analysis of new or existing structures with seismic isolation are similar; see Chapter 6d.

Isolation Retrofit Projects

The Salt Lake City and County Building

The first seismic isolation retrofit project in the United States was the City and County Building in Salt Lake City, Utah, originally built in 1894 (Fig. 8c-1). It is a monumental, highly ornamented, unreinforced brick and sandstone structure measuring 130 feet by 270 feet in plan, with five main floors and a clock tower 240 feet in height. The seismic vulnerability of this structure, due in part to its lack of reinforcement, is accentuated by its proximity to the Wasatch fault zone. See Ref. 1.

The structure, which is approximately symmetrical on both sides, is supported by bearing walls of unreinforced brick and sandstone masonry which rest on sandstone plinths and 8 feet-6 inches wide continuous concrete footings (Fig. 8c-2). The interior brick bearing walls have a maximum thickness of 24 inches at the base. The exterior walls, which have an exterior wythe of sandstone masonry, reach a base thickness of 36 inches. Multiple wythes of brick in each bearing wall are bonded together solely by the original sand-lime mortar, which was badly deteriorated in many locations. The central clock tower is supported at the base by four solid piers of sandstone masonry, which are L-shaped in plan and have a maximum dimension of 13 feet.

The idea of using base isolation on the City and County Building was put forth in 1984 as a less destructive alternative to conventional retrofitting techniques.

Fig. 8c-1. City and County Building, Salt Lake City, Utah.

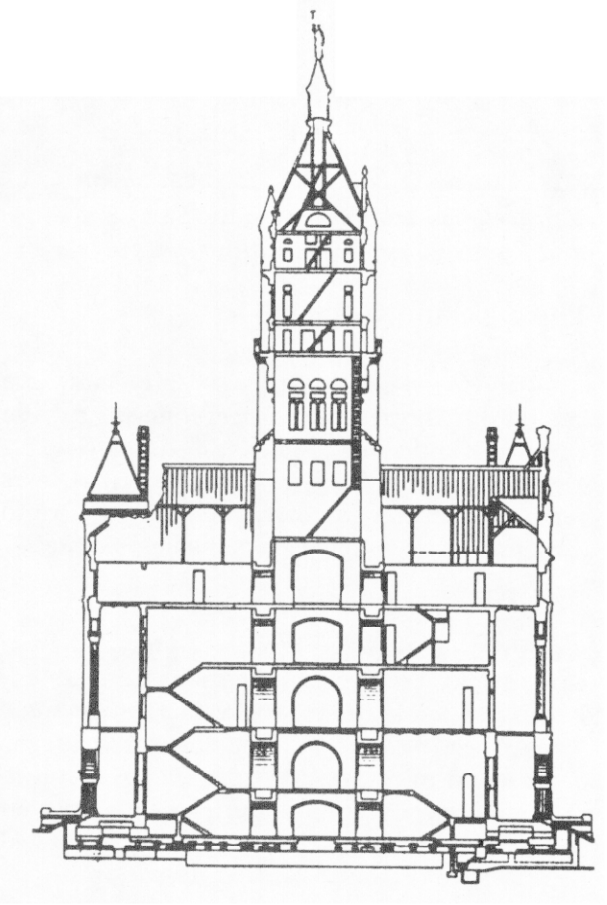


Fig. 8c-2. Cross section of the Salt Lake City and County Building.

A bearing installation scheme (Fig. 8c-3) was implemented, in which each masonry wall was sandwiched between a pair of reinforced concrete side beams, which were notched into each wall to allow direct bearing of the wall, and then tied together through the wall by regularly spaced

concrete cross beams and ducted prestressing rods. Once these beams were cast and post-tensioned to the wall, portions of brick and plinth below the cross beams were then removed, creating a space in which the isolators and bearing plates were installed to bear on existing concrete footings. A similar scheme was developed for the central tower.

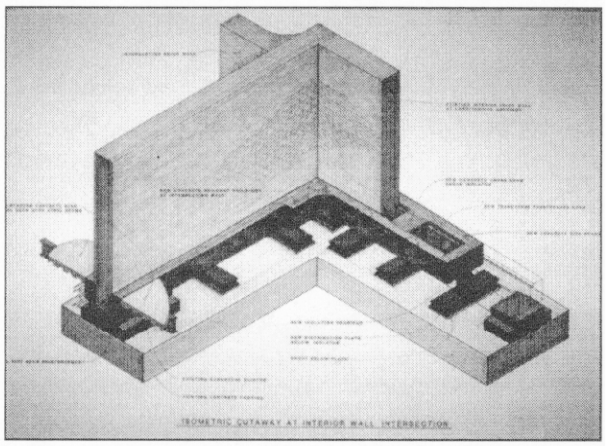


Fig. 8c-3. Isolator layout for the Salt Lake City and County Building retrofit.

The Oakland City Hall

The Oakland (California) City Hall is a landmark, historic 19-story steel-framed structure with brick and stone perimeter cladding (Fig. 8c-4). Completed in 1913, the structure was seriously damaged in the 1989 Loma Prieta earthquake, and required strengthening. See Ref. 2.

An evaluation and review of proposed post-earthquake repair schemes was conducted, which included base isolation and conventional strengthening. Base isolation retrofit was determined to be feasible, cost-effective, and to provide superior seismic performance to other strengthening alternatives. Computer generated figures provide a visual comparison of the response of the building with a conventional fixed-base retrofit to that of the building retrofitted with seismic isolation (Fig. 8c-5).

The Oakland City Hall required tower strengthening and stiffening to satisfy the dynamic response of the isolated building. Large composite steel and concrete transfer trusses installed under the existing steel frame act as diaphragms to spread seismic loads from the

Fig. 8c-4. Oakland City Hall, damaged in the 1989 Loma Prieta, California earthquake.

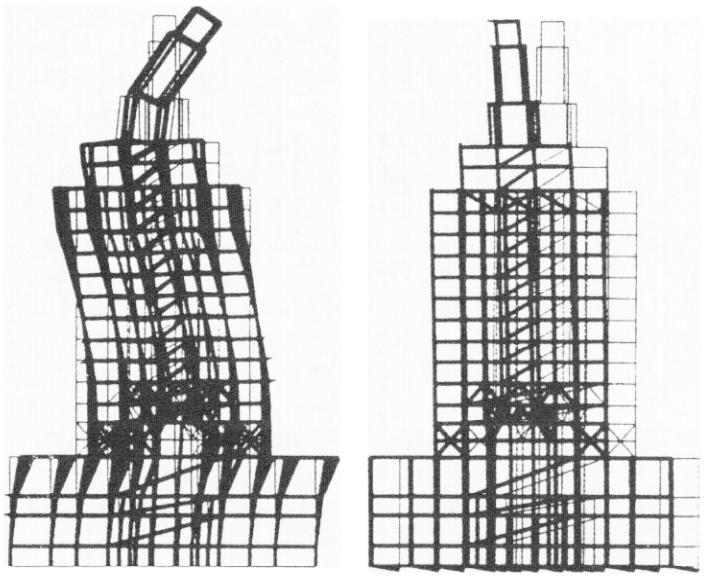


Fig. 8c-5, Oakland City Hall, comparative seismic responses; fixed-base structure versus isolated structure.

tower to 111 elastomeric bearings at the structure/foundation interface. Transfer of loads in existing columns to the new bearings required a phased process of shoring, jacking, and load transfer. The Oakland City Hall is currently the tallest building to be seismically isolated.

San Francisco City Hall

The seismic upgrade design for the San Francisco (California) City Hall (Fig. 8c-6) (built in 1914-15) is completed, and construction began in 1995. Seismic isolation was selected over conventional strengthening because of the unique vertical configuration of the structure and the dynamic response characteristics of the large dome, which is 300 feet in height.

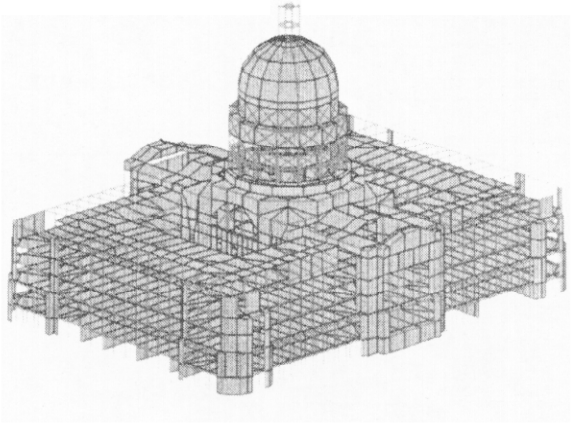


Fig. 8c-6. San Francisco City Hall, California.

The San Francisco City Hall is a *National Historic Landmark* and is architecturally monumental and structurally unique. It is a five-story building with a rectangular plan occupying approximately two city blocks. Plan dimensions are 309 feet by 408 feet. The seismic repair and upgrade, necessitated by 1989 Loma Prieta earthquake damage, was tailored to specific historical, functional, and structural response needs of the building. Both fixed base and base isolation repair options were studied. Analyses indicate that isolation will produce a base shear (for the 10%/50 year earthquake) of about 0.20g compared to base shears for fixed-base options (with shear walls or steel bracing) of about 0.60g to 0.80g. Interstory lateral deformations (drift) with isolation also will be significantly reduced from those with conventional retrofit. Base isolation requires stiffening of the existing structure to control the dome response, but because of the limited seismic response with isolation, major strengthening of brittle historic stone finishes is not required.

More than five-hundred isolators are planned for installation on top of existing footings at the base of the steel columns. Phasing of the installation, in conjunction with temporary bracing, is required to maintain structural integrity during the installation process.

Bearing Details

Details for installation of bearings will vary with each building. An existing building must accommodate the following:

- An isolation platform or distribution diaphragm
- Space for isolation bearings and crawl space access
- Load transfer from existing columns and/or walls to isolation bearings must be physically possible
- Temporary bracing to insure stability during the construction.

Figure 8c-7 shows the variation in configuration and detailing of isolation bearings for four different seismic rehabilitation projects.

ISOLATION BEARING

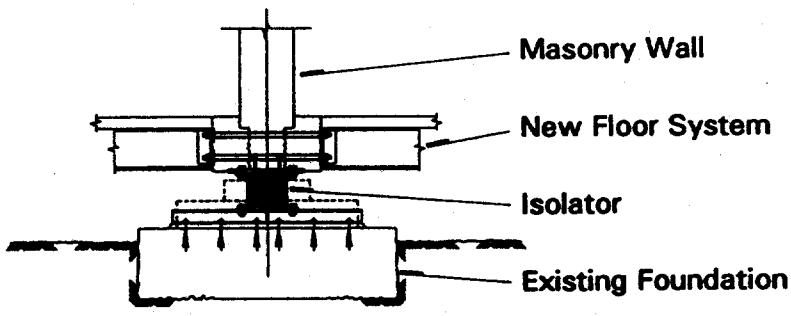
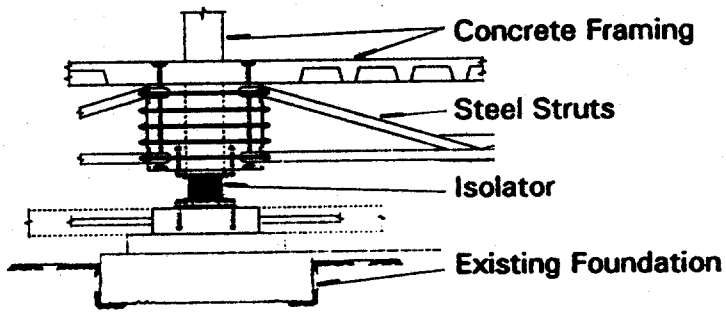
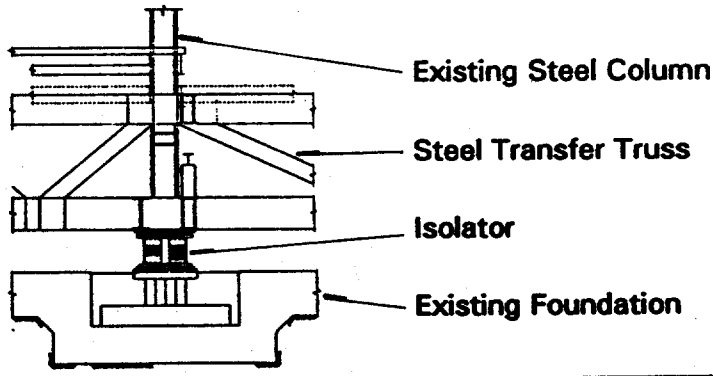
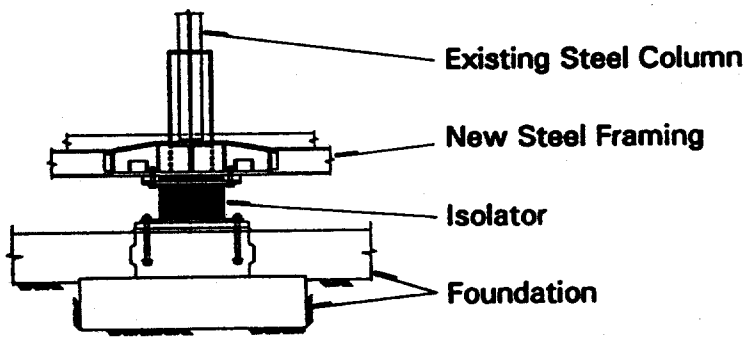
Project	Isolation Bearing
City and County Building **Salt Lake City, Utah** *Structure:* Unreinforced Masonry Bearing Wall **5-Stories plus Tower**	
Hays Hospital **Fort Ord, California** *Structure:* Reinforced Concrete Frame **8-Stories**	
Oakland City Hall **Oakland, California** *Structure:* Steel Frame with Masonry Infill Walls **14-Stories plus Clock Tower**	
San Francisco City Hall **San Francisco, California** *Structure:* Steel Frame with Masonry Infill Walls **5-Stories plus Dome**	

Fig. 8c-7. Configuration details for isolation bearings used in four seismic retrofit projects.

Passive Energy Dissipation Systems

Seismic isolation may not be feasible for buildings with long periods. For these buildings, passive energy dissipation may be a viable alternative. Such systems reduce building response to seismic ground motions by increasing structural damping and dissipating earthquake-induced energy by supplementing the energy-dissipation capability of the building's structural framing system.

The function of energy dissipation devices is to increase the energy dissipation capability of the system without increasing the displacement/deformation of the building structure. In many cases the deformation can be decreased up to 50%.

The effect of damping on the seismic response of structures is covered in detail in Chapter 6d. The use of passive energy dissipation systems for reducing seismic response in existing buildings is receiving increased attention and study. Retrofit of a small two-story bank office building was recently completed in San Francisco, California using the *Added Damping and Stiffness* (ADAS) system .

A feasibility study was recently completed using passive energy dissipation to reduce horizontal displacements (story drift) in a large 17-story federal government building in Los Angeles, California. The results of the study are under review by the responsible agency.

The Santa Clara County General Services Building in San Jose, California, was seismically retrofitted recently using visco-elastic dampers (Ref. 3). The existing building suffered considerable damage during the 1989 Loma Prieta earthquake, mostly to nonstructural elements, such as partitions, furniture and fixtures.

Peer Review

Peer reviews should be conducted for all seismic isolation and passive energy-dissipation retrofit projects. The recommendations given in Chapter 6d for new construction also apply to retrofit projects.

Key issues for peer review are:

- Appropriate and realistic performance goals
- Feasibility of installing isolation bearings
- Appropriate seismic ground motion for retrofit design.

References

1. *Base Isolation of the Existing City and County Building in Salt Lake City*, E. Elsesser, Proceedings, the International Seismic Isolation/Historic Preservation Symposium, Salt Lake City, Utah, May 1988.

2. *Repair of Five Historic Buildings Damaged by the Loma Prieta Earthquake*, E. Elsesser, S. Naaseh, M. Walters, V. Sattary, A. Whittaker, Proceedings, Conference on Seismic Retrofit of Historic Buildings, San Francisco, California, November 1991.

3. *Utilizing Visco-Elastic Dampers in the Seismic Retrofit of a Thirteen Story Steel Framed Building*, P. Crosby, J. Kelly, J.P. Singh, Proceedings, ASCE Structures Congress XII, Atlanta, Georgia, April 1994.

Chapter

9

Seismic Evaluation and Design Considerations for Operations and Contents

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

In recent years, there has been an increasing awareness of seismic hazards posed by nonstructural elements in buildings, such as suspended ceilings, ducts and piping, mechanical and electrical equipment, elevators, file cabinets, book cases, partitions, glazing, architectural ornamentation, and miscellaneous movable objects. Damage to building mechanical, electrical, and plumbing equipment received particular attention following the Alaska (1964), San Fernando (1971), and Managua (1972) earthquakes, and significant codification has since developed, although somewhat slowly. More recent earthquakes in California, Santa Barbara (1978), Livermore (1980), Loma Prieta (1989), and Northridge (1994) have demonstrated the extreme hazards to life-safety and property damage posed by nonstructural elements and contents in buildings. In the Santa Barbara and Livermore earthquakes, structural damage was overshadowed by damage and costs related to nonstructural elements, and by secondary effects that made emergency responses very difficult. In both earthquakes damage was very localized. Although personal injury was minimal, it was obvious that good luck rather than preparation saved many people from serious injury. In each of these earthquakes, public institutions, the Lawrence Livermore National Laboratory and the University of California at Santa Barbara, were situated in the area of heavy shaking, and the lessons learned fell upon responsive management. In each case, the earthquake experience was well documented and the earthquake safety-improvement programs that followed combined to provide excellent reference material for managers in charge of similar facilities. In particular, after the 1980 Livermore, California earthquake the Lawrence Livermore National Laboratory published UCRL-151815, *Practical Equipment Seismic Upgrade and Strengthening Guidelines*, (September 1986) which includes numerous examples of nonstructural damage as well as guidelines for retrofit.

Chapters 9a through 9d cover earthquake safety as it relates to contents, equipment, and operations that are not part of a building's structural system. It's important to know that nonstructural elements in existing buildings generally were not designed or originally installed with earthquakes in mind. For example, items of mechanical and electrical equipment are often installed in buildings without being adequately fastened to the structure or anchored to foundations. Although

this chapter does not treat nonstructural elements of buildings comprehensively, several recommended references covering the subject are listed.

Shops, laboratories, and production facilities contain many heavy objects that will travel, rock, topple, or break during earthquake shaking. Such actions may induce secondary effects that can be even more hazardous, such as breakage of containers holding toxic chemicals or explosive gases. Normal office equipment and furniture also pose significant hazards for injury. Side opening and heavy file drawers supported on low friction bearings usually are ejected during an earthquake if the cabinets do not have positive latches. Wide drawers in flat tracing files similarly become massive, multiple missiles when accelerated out of cabinets. Bookcases and storage shelving that are not properly restrained topple or discharge their contents to clutter and block nearby space.

Many examples of common seismic hazards posed by building contents are given in Chapters 9a and 9b. The approach to mitigate these hazards is relatively simple. One has only to observe a building space and carefully consider what will probably happen to the contents if the building is violently shaken. Consider the working stations of the occupants (Fig. 9-1) in relation to the imagined action of hazardous contents. Will a tall bookcase fall over a desk chair (Fig. 9-2)? Are the cabinet file drawers aimed at the occupant? Are heavy objects stored above the workstation? Will the exit door be blocked by fallen debris? Remember, when violent shaking takes place, an occupant may not be able to maneuver quickly. One can be very helpless in this situation. The *cure* has to take place before the shaking starts to make escape from injury more than a matter of luck. A well-organized building manager system, coupled with a program to educate building occupants

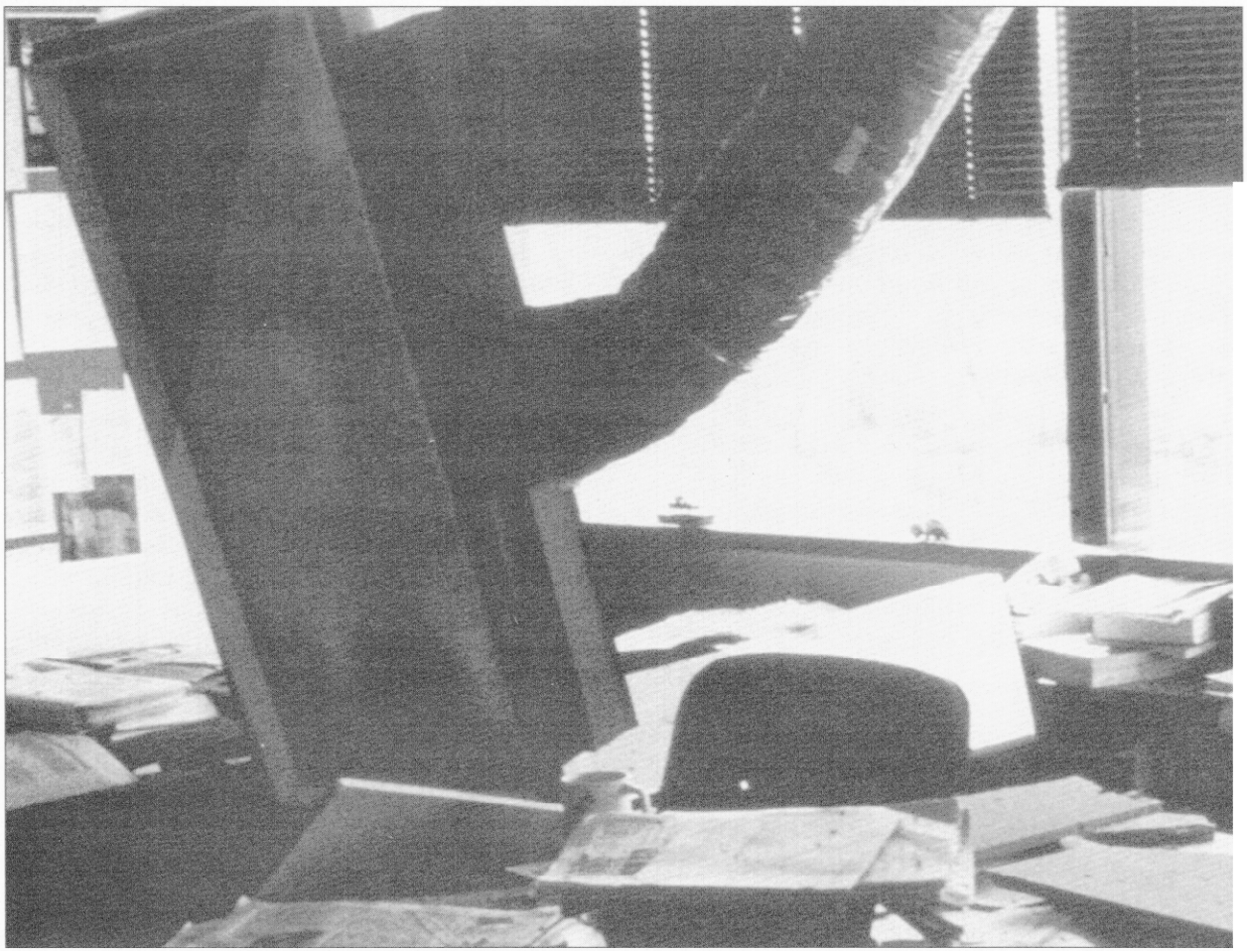


Fig. 9-1. Failure of heating and ventilating system components over workstation, Livermore California earthquake, 1980.

and their supervisors about earthquake safety, can be quite effective if supported by a systematic inspection program. The overall success of this combination is dependent on the following principles:

- Building managers must be given authority to carry out their rather difficult responsibilities. Usually, this can be achieved by appointing the person with the highest supervisory responsibility in the building as *Building Manager*. Most of the time-consuming work associated with the responsibility can then be delegated, but the *Building Manager* must remain accountable to management for the safety of occupants.

- An educational program is most effective if the occupants, supervisors, and building managers have a real understanding of how building contents react to damaging earthquakes. This is best achieved by showing photographs or videos of actual earthquake damage, backed up if possible by open discussion with earthquake engineers who have observed such damage. The reaction of contents to violent shaking, vividly recorded in Kobe, Japan during the 1995 Hyogo-Ken Nanbu earthquake, is very impressive.

- Seismic safety inspections should be scheduled and carried out in conjunction with other routine inspections, such as fire safety and industrial safety, so that the economic burden is minimized. Although inspectors should be trained to visualize the effects of heavy shaking on building contents, one does not have to be an earthquake engineer to get the job done. Of course, common sense must be used. For example, when heavy bookcases are tied together and finally braced or bolted into a wall, the wall itself must provide adequate strength for anchorage. Gypsum wall board does not provide sufficient strength, and the wall framing itself, particularly if it is a partition that terminates at a hung ceiling,

Fig. 9-2. Unanchored bookcase overturned on workstation, Livermore, California earthquake, 1980.

must be adequately braced. Also, modular furniture systems often incorporate and support heavily loaded shelving which can cause serious injury if the system is not seismically restrained. It is of prime importance to appoint a single individual to focus on earthquake preparedness related specifically to building contents and operations.

- Finally, the results of seismic safety inspections must be communicated to someone who can arrange for corrective action. If the *Building Manager* accompanies the inspection party, this first-hand experience usually provides increased motivation for correction. However, there must be an avenue for the *Building Manager* to pay for corrective measures that require labor and materials, such as equipment tie-downs and bracing for storage racks or library shelving (see Fig. 9-3).

The seismic performance of lifelines is covered primarily in Chapter 11a, however, one of the most critical components in electrical power systems, the emergency power generator, also is a nonstructural element. During the 1994 Northridge, California earthquake three acute-care hospitals, Holy Cross, Olive View (now named Sylmar) and Northridge, were functionally disabled primarily by nonstructural damage (Ref. 1). Except as noted, all three of these modern hospitals suffered:

- Sprinkler and other waterline breaks and leaks
- HVAC equipment anchorage failures
- Large oxygen tank base failures
- Toppling of unanchored cabinets and equipment
- Communication failures

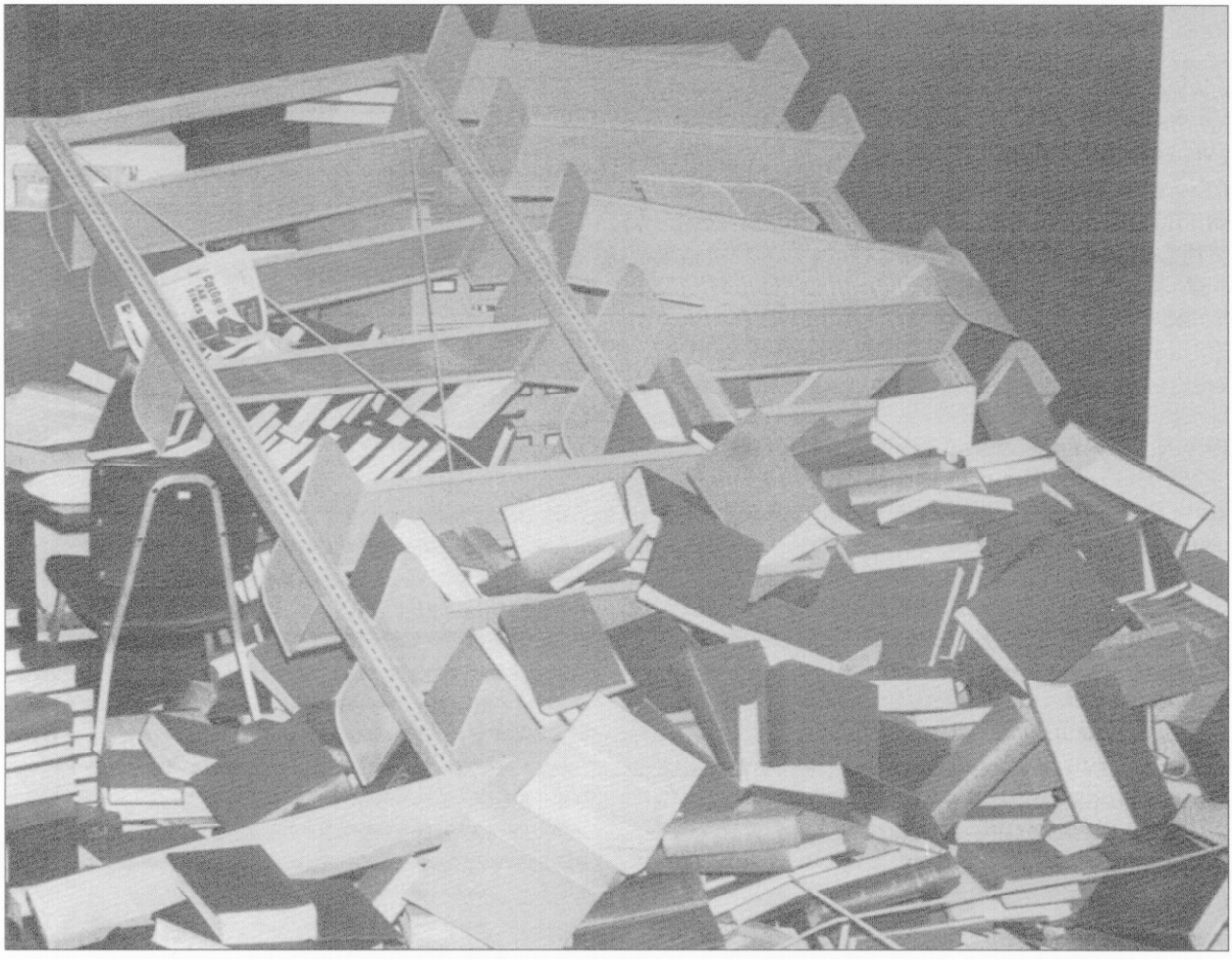


Fig. 9-3. Collapse of library shelving on workstation, Livermore, California earthquake, 1980.

- Elevator damage
- Firefighting system failure
- Medical gas failure (except Northridge)
- Backup power outage (except Northridge and Holy Cross)
- Water service outage
- Gas service outage
- Electrical service outage

Two of the largest and newest facilities in the San Fernando Valley, Olive View and Holy Cross, were effectively shutdown by nonstructural damage for the week following the earthquake. At six medical facilities in the affected region (including Olive View) emergency power generator systems failed to operate. California has very stringent state regulations for hospital design and construction; however, the success of building code requirements will always depend primarily on the application of sound judgment by designers and project managers as well as dedicated maintenance and operations practices by facility managers.

Chapter 9a provides an overview of experience with building contents, equipment and related hazards during earthquakes. Chapter 9b covers methodology for the evaluation of typical nonstructural hazards. Chapter 9c describes how data-processing systems are subject to seismic damage and what to do about it. Chapter 9d covers the subject of seismic experience data for various types of physical plant equipment.

Reference

1. *Turning Loss to Gain*, Seismic Safety Commission, State of California, Report to Governor Pete Wilson, 1995.

Chapter

9a

Overview

James L. Stratta

Inspection of Contents

In the aftermath of an earthquake, the number of injuries, loss of life, and amount of property damage resulting from causes other than structural failures is inversely proportional to the quality of the inspection and the precautions taken to reduce hazards in facilities prior to the earthquake. Explosions, fires, toxic fumes, falling objects, sliding objects, and other serious conditions often occur during seismic activity. An awareness of how such potentially disastrous events (often triggered and/or intensified by sequential collisions) can happen facilitates development of means to minimize injury or damage, and the chances of their occurrence. Some steps that can be taken to prevent or mitigate such problems are described in this chapter. These are summarized below, then discussed in more detail in the sections that follow.

Explosions can occur in several ways. Whenever hazardous compounds such as liquids or gases are to be stored, expert advice should be sought regarding both the physical facilities and operational procedures to be used with respect to seismic safety. The compatibility and proximity of adjacent materials to be stored in a given space, suitability of containers, ambient temperatures and pressures, presence of electrical devices, and location of the storage areas should be considered. The basic objective is to protect occupants and facilities from the effects of impact, spillage, and/or mixing of such materials in the event of an earthquake.

Gases stored in containers or distributed in building piping, including heating fuels or any other material that could be ignited by a spark or elevated temperatures, should be monitored and, where advisable, seismic shutoff valves installed to reduce the hazard. Electrical equipment and conduit should be securely anchored and provided with expansion joints or flexible connections at points most likely to be heavily stressed in an earthquake. Seismic disconnect devices should be considered for electrical service where arcing could lead to fires or explosions.

During the past several years, great progress has been made regarding *earthquake shut-off valves*. These valves may be used for gas lines up to 72 inches in diameter to shut down the flow of liquids or gases, whichever the case may be, for whatever reason, and to turn off electricity. During the 1989 Loma Prieta and the 1994 Northridge, California earthquakes, there were instances of ruptured gas lines. In some cases, these valves prevented gas leakage and possible ensuing fire or explosion.

Dangerous fumes are normally exhausted, neutralized, or absorbed by special ventilation systems. These systems should be made earthquake-resistant or retrofitted so they will

continue to exhaust or contain such fumes in the event of an earthquake. An insidious situation exists, however, where backup emergency power is supplied to an environment in which explosive gases could be present in the aftermath of an earthquake. For example, emergency power may be required to ensure continuity of ventilation of a space in the event of a normal power outage. If, as the result of an earthquake, explosive gases are released in that space and normal power is lost, emergency power may still cause sparking and possibly an explosion. In this situation, an earthquake-resistant, explosion-proof electrical system may be necessary.

Falling objects often create very serious problems during earthquakes. In addition to injuring people, they may create obstructions to egress from buildings. For example, free-standing or inadequately anchored lockers or vending machines in hallways often slide or topple, blocking exit corridors (Fig. 9a-1). Such obstructions may cause panic in an already stressful emergency condition.

Sliding objects, particularly heavy, unanchored equipment or partitions, may travel across the floor, creating one or more of a variety of serious problems, examples of which will be given later. Intense shaking can also cause other equipment-related problems. For example, heavy compressed-gas containers may fall over and slide or roll with considerable velocity. If the tank valve is broken off, the gas can abruptly escape causing the container to react like a missile.

Research equipment housing hazardous dispersible materials, such as radioactive or other highly toxic substances, carcinogens, or viruses, may overturn and rupture, releasing their contents.

Seismic hazards from nonstructural building components and equipment present in research facilities provide ample justification for carrying out systematic building-content inspections for earthquake safety. Practical means for preventing or mitigating such hazards are discussed in the following sections.

Fig. 9a-1. Overturned vending machine in access corridor.

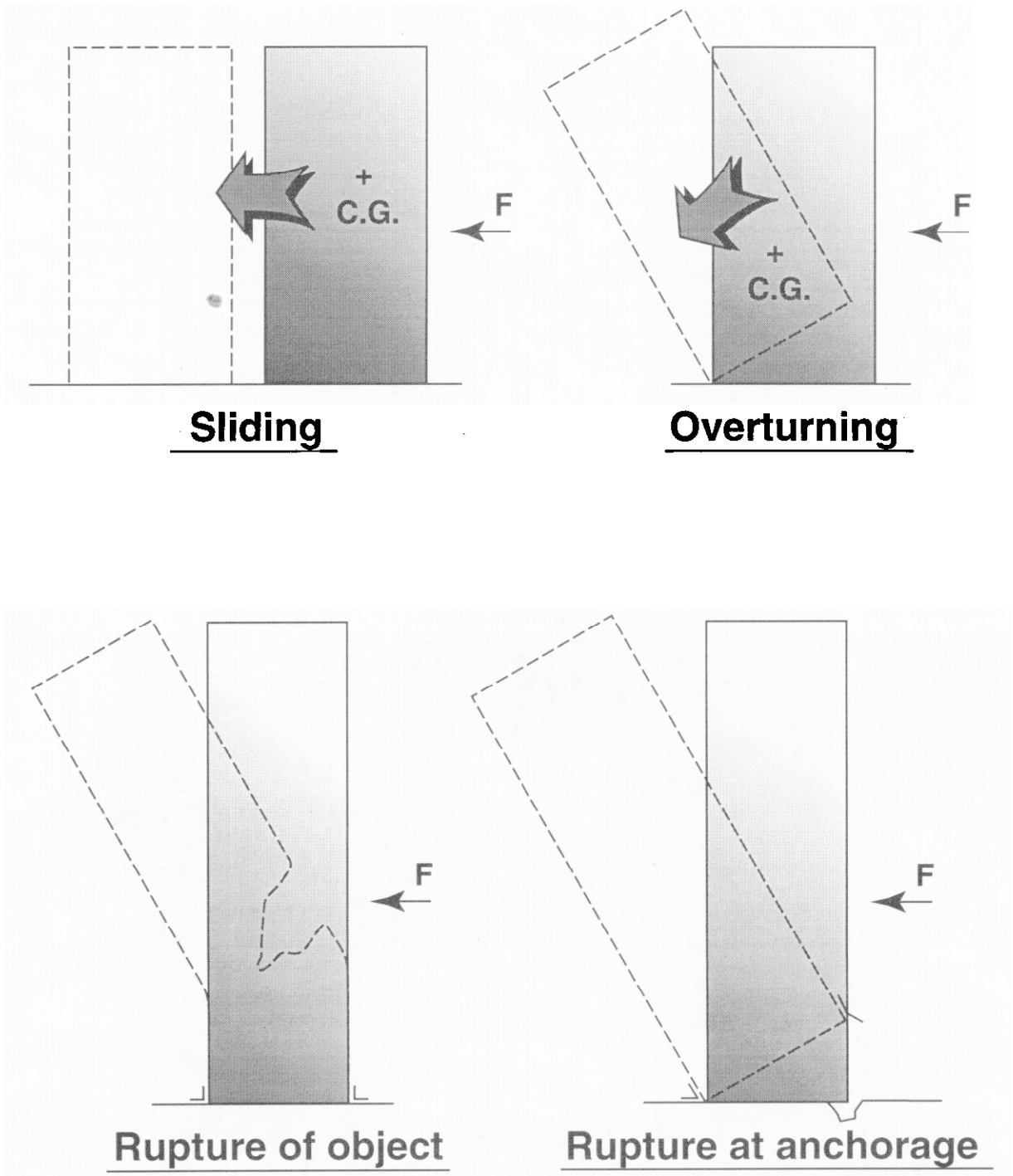


Fig. 9a-2. Four modes of equipment displacement and failure.

Explosions

When storing explosives, particular care must be taken to prevent the four modes of equipment displacement and failure shown in Fig. 9a-2. Expert consultation should be employed. Typically, chemical storage cabinets are not designed or constructed to prevent spillage or breakage of containers during an earthquake. However, an earthquake may cause bottles containing chemicals to slide off shelves or out of cabinets, fall to the tabletop or floor (Fig. 9a-3), and break. If any mixture of these spilled chemicals can cause an explosion, fire, or other potentially dangerous event, precautions to avoid such problems should be taken to mitigate the hazard.

Generally, chemical storage cabinets and door latches must be modified to prevent spillage or breakage during earthquake shaking.

Good seismic safety practices include the use of spillproof, unbreakable containers that are not subject to attack by acids or solvents. Providing a lip or guardrail for each shelf is suitable only where relatively small amounts of low-hazard materials are involved. Unfortunately, a railing may not prevent containers on a partially filled shelf from overturning , even if it restrains them from falling off the shelf during an earthquake. Sturdy storage cabinets with positive spring closures and catches on the doors offer better protection than open shelves. Workability of closures and catches should be verified at each building inspection and/or by users. If visibility of cabinet contents is desired, wire-reinforced glass can be securely fitted in door frames. Full shelves and only compatible materials in each cabinet are recommended. In building areas containing significant amounts of gases that can combine to cause hazardous mixtures, provisions should be made to preclude accidental mixing. For example, seismic shut-off valves may be installed at main gas-supply points, or at appropriate distribution-line locations, as best suits the situation. Explosion-proof electrical fixtures and switches should always be used where sparking could cause a problem.

Fig. 9a-3. Spilled chemicals in *"wet"* laboratory.

Fires

Generally explosions also cause fires. Therefore, the same precautions as those previously outlined for explosions apply to fire prevention.

Storage of flammable chemicals should be very carefully monitored. Cabinets containing hazardous chemicals must be securely anchored to prevent overturning, falling, or sliding. Doors need positive latches to prevent opening caused by shaking. Rooms housing flammables should have properly rated firewalls, ceilings, and doors. Curbs, drains, mechanical equipment, and electrical fixtures also should comply with fire-protection codes. Operating procedures for storage and use of flammable chemicals should comply with good practice for seismic safety.

Electrical fires can result from several causes. Short circuiting is probably the greatest hazard and also the most difficult to control. Because short circuits may occur as a result of structural failure or building collapse, seismic safety evaluation of existing buildings should include inspection of electrical devices and building wiring.

A number of essential conditions must be met to reduce electrical hazards related to operations and building contents during an earthquake. Proper anchoring can prevent overturning of switchgear, transformers, motor control units, and similar equipment. Fig. 9a-4 shows a damaged motor control panel at the Olive View Hospital after the 1971 San Fernando earthquake. As a rule of thumb, this type of equipment should be anchored for a static force equal to a minimum of one-half of the weight of the equipment acting horizontally at the center of gravity.

Energy and water-supply lines of various types also should be protected. Electrical ovens, boilers, and other high-amperage units should be securely anchored to prevent motion tearing of electrical feeder lines that could short and start fires. Outdoor tanks storing volatile flammable gases or liquids, such as propane, must be anchored to prevent loss of contents, explosions, and fires.

Fire-sprinkler systems should be periodically checked to ensure that materials are not stored adjacent to lines or mains where their collapse onto the sprinkler system could cause a rupture in the lines during seismic motion. It is vital that a water supply for fire protection be available in the aftermath of a damaging earthquake. On-site storage with emergency pumping facilities may be necessary for proper risk management.

Toxic Fumes

The two major sources of toxic fumes are those generated as a result of an earthquake and those that were generated prior to the earthquake and are being exhausted. In some laboratories, toxic fumes may be involved as a result of processes used in research or other routine operations. Research processes generally are conducted in hoods that exhaust the gases which then pass through high-efficiency filters or scrubbers prior to emission

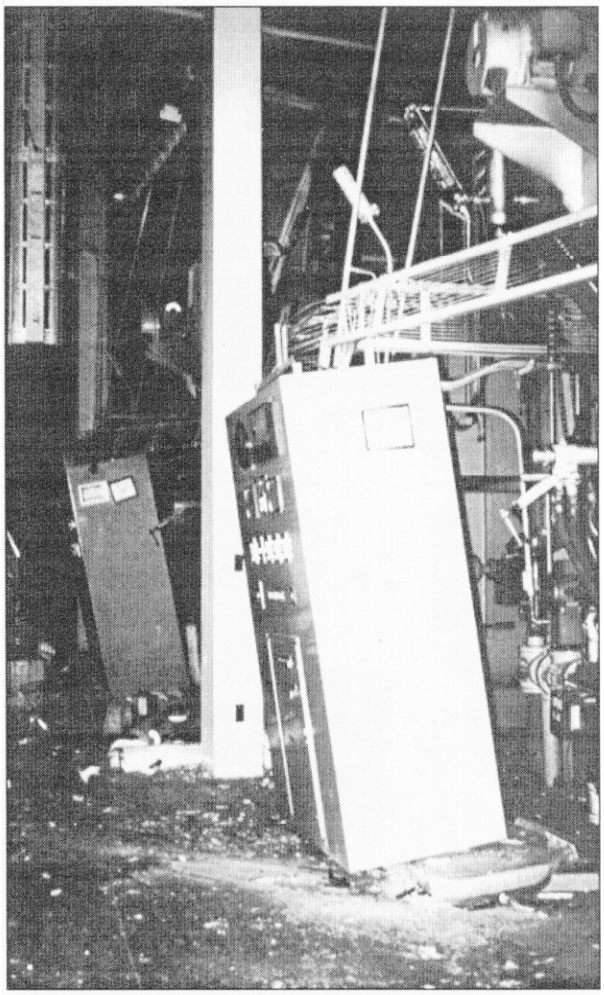


Fig. 9a-4. Failure of damaged motor control panels, Olive View Hosptial, San Fernando, California earthquake 1971.

to the outside atmosphere. For most hazardous gases, it is imperative that filter and scrubber systems will resist earthquakes and that exhaust fans and scrubber systems are connected to an emergency generator so that continued operation is ensured if a power failure occurs. Hoods or benches where these gases are generated also should be carefully anchored to prevent the modes of failure shown in Fig. 9a-2.

Most fumes originating from seismic shaking result from a mixture of chemicals or gases used or stored in a facility. When observing a storage area, a few simple questions will help determine the presence of a seismic safety problem: What will happen if any two of the chemicals stored here come into contact with one another? What will happen if they mix with water? Or, if they come into contact with acids or solvents that are stored nearby—what then? If the answer to any of these questions indicates hazardous consequences, positive means should be made to prevent contact in the aftermath of an earthquake.

Sometimes toxic gases are piped in, but more often equipment is connected to gas cylinders, canisters, or dewars. Whatever the source, hazardous gas must be prevented from creating a hazardous atmosphere as the result of heavy shaking. Where large amounts of gases are maintained or supplied, the use of earthquake shut-off valves should be considered. Plating facilities also should be carefully inspected with the same questions in mind. Tanks are likely to overflow so mixtures of these liquids should be analyzed. Sliding and overturning of tanks should be prevented with well designed anchorage. Highly toxic gases, such as diborane and silane, are extremely dangerous. A small leak from a pressurized container could develop a death cloud more than a mile in diameter. The storage and use of such highly toxic materials are regulated by local, state, and federal regulations. The Uniform Building Code (UBC) and the related Uniform Fire Code (UFC) include special provisions for handling and storing such materials, but detailed seismic design requires the special insight that only an experienced earthquake engineer can bring to the problem.

Falling Objects

Objects may fall from shelves, bins, or tables. A rather innocuous-appearing object is the locker or

Fig. 9a-5. Failure on inadequately braced library stacks.

cabinet, multiples of which may line corridors or other access ways. During seismic motion, tall cabinets or shelves can tip over unless anchored to the walls. If they fall in a corridor, injury to personnel who are in the passageway may be minimal, but the resultant blockage of egress (Fig. 9a-1) from the building may greatly increase the panic felt by those trying to escape the building. Large volumes of heavy books or files stored on shelves along corridors also are extremely hazardous (Fig. 9a-5). Exits and tributary corridors should be inspected regularly for objects that block or congest these areas. Nothing should be allowed to impede prompt exiting of personnel from buildings. Personnel should be instructed not to panic and run from buildings, but during an intense earthquake it is almost impossible to remain calm.

Areas where materials or objects are stored in high places, such as storage rooms, warehouses, shipping and receiving rooms, libraries, etc., should be inspected, keeping in mind the four modes of equipment failure previously illustrated (Fig. 9a-2). Computer rooms where raised floors are neither braced nor anchored may be hazardous if floors could collapse during seismic motion. Tall computers will then almost certainly overturn, possibly injuring personnel and blocking exits. Short circuiting from electrical connections also may occur. Generally, cabling restrains equipment if the computer floor itself survives the shaking. (See Chapter 9c for a detailed discussion of data processing systems.)

Process or research equipment placed on table tops is particularly vulnerable to falling. Usually such equipment is very expensive and difficult to replace. Aside from the loss, dire secondary consequences may result. If the shattered equipment spills hot, toxic, flammable, or dispersible toxic materials, a serious situation is created.

Anything containing highly dangerous material should be carefully designed to prevent spillage or leakage of its contents and be anchored to a stable surface, regardless of its location.

Sliding Objects

The effects of sliding objects are somewhat similar to those of falling objects. However,

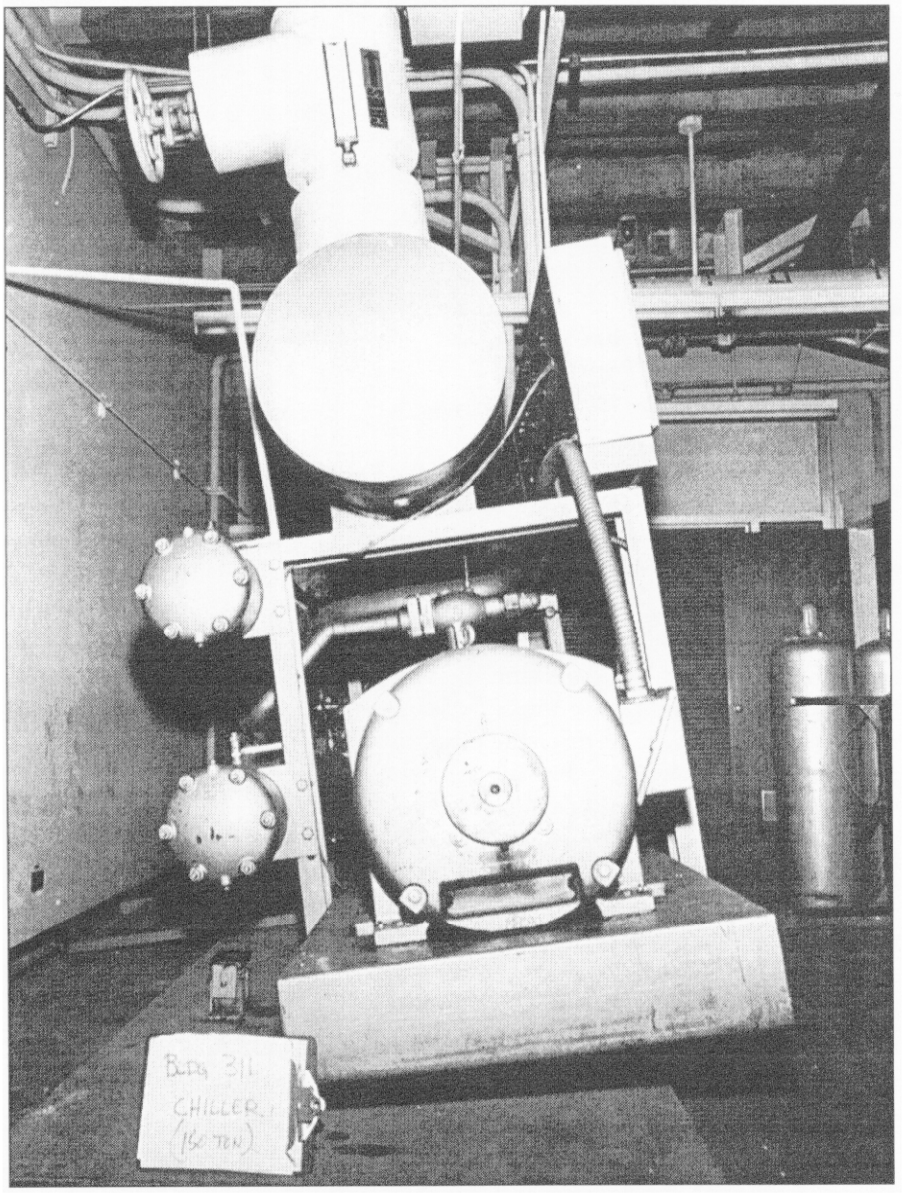


Fig. 9a-6. Failure of anchorage for 150-ton chiller unit.

sliding components of buildings sometimes have unique characteristics and therefore are given special attention in this discussion.

The principal building components to be considered in most cases are those of the mechanical and electrical systems. Such mechanical system components as chillers (Fig. 9a-6), pumps, boilers, cooling towers, and air-handling units require anchoring in a manner that minimizes damage and, for units of critical systems, permits operation after an earthquake. The movement of any of these components, or the breakage of lines leading to and from them, causes these systems to shut down and results in costly property damage. Even in relatively minor earthquakes, property damage to building mechanical systems has been very expensive. The fact that some of these units are mounted on springs to eliminate vibration further complicates the problem. Although expert advice should be sought in such cases, the references at the end of this chapter provide good guidance and methodology for design and retrofit solutions. The motion of these elements and resulting damage result in expensive repairs and may cause personal injury and/or serious structural damage to the building.

In electrical systems, anchorage of substations, switchgear, motor-control centers, panel boards, and emergency generators is very important. Lack of adequate anchorage is the prime cause in failure of such equipment.

In summary, building contents that should be prevented from sliding are those that could make the equipment unusable or create other problems. Heavy X-ray machines should be anchored so that they will be usable in the aftermath of an earthquake. Equipment that is connected to hazardous gases with piping or tubing should be securely anchored to prevent breakage of gas lines. Similarly, unanchored equipment can move or break electrical feeders, resulting in sparking and possible short circuiting.

Overhead cranes present special hazards for buildings and occupants (Fig. 9a-7). Generally, overrunning cranes can be equipped with *keepers* to prevent derailing or falling from the crane runway rail during an intense earthquake.

Generally, anchor bolts for mechanical equipment installations have been too small for adequate anchorage in concrete. It is good practice to use large bolts because most of the cost is in the labor, and anchor bolts in concrete have little ductility.

Miscellaneous Problems

In large, older buildings, certain items should be updated. Ceilings and lighting fixtures over workstations (Fig. 9a-8), and exit corridors should be well braced and anchored to prevent collapse. The exterior facade over exits should be checked to ensure that there are no potentially falling objects, such as masonry or other heavy veneers, parapet walls, or potted plants to rain down on people leaving the building.

Up to this point, the discussion has been concerned primarily with problems relating to harm to personnel. Another very important factor is financial loss. For example, major pieces of electronic equipment may cost hundreds of thousands of dollars. In *earthquake country*, an effective preventive program to reduce the possibility of heavy financial loss is simply good risk management. The cost premium is usually insignificant. The most difficult problems relate to operational procedures and the need for mobility.

Inspections

When inspecting a facility, one should take sufficient time to walk around the interior and exterior of each building, while visualizing a scenario of the aftermath of an intense earthquake. One should imagine how objects would be moved about and the problems they would create. All doors should be opened and all rooms inspected, closets and cabinets included. Spaces above hung ceilings often contain potential falling hazards. Any space may hold potentially dangerous materials or situations. Many conditions that are entirely adequate for efficient daily operation may be inappropriate from a seismic safety point of view. In particular, special attention should be given to spaces and facilities that will be used for emergency recovery.

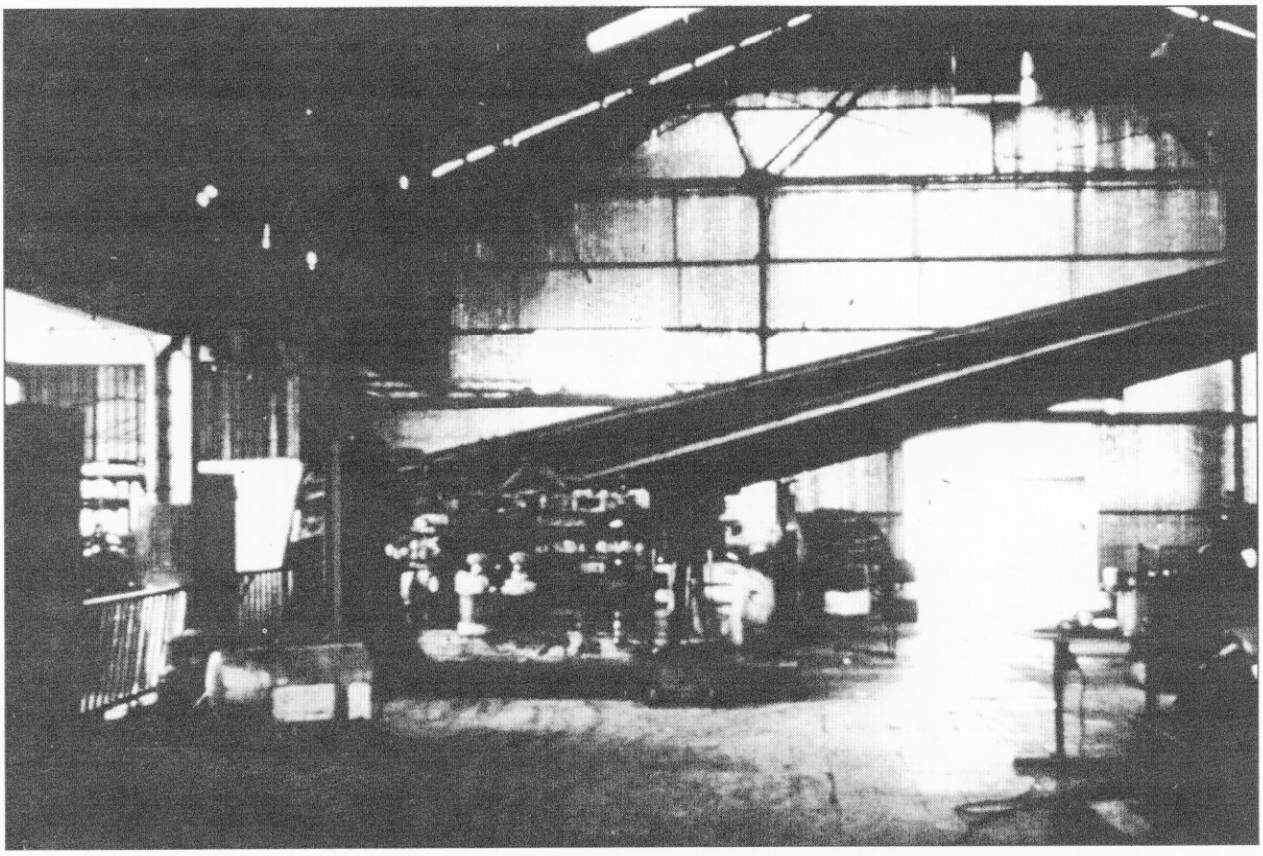
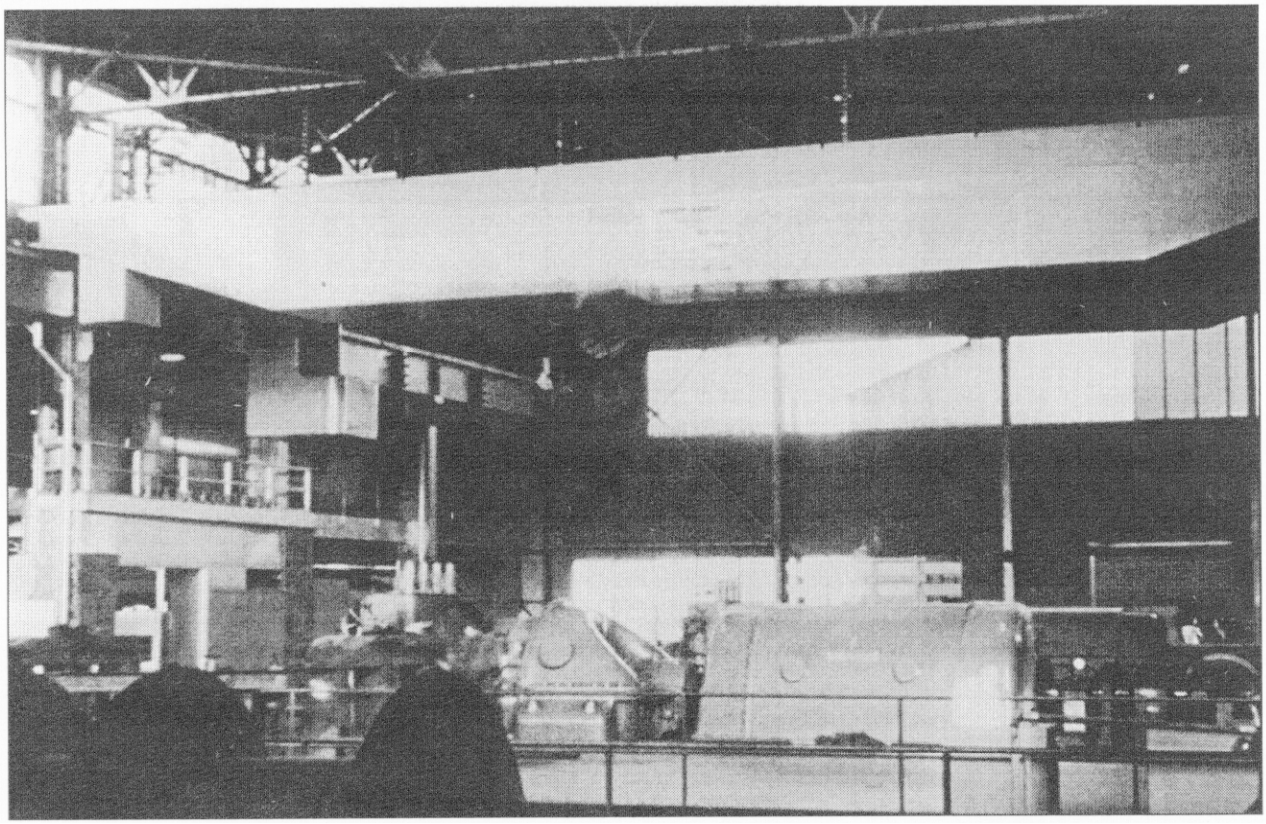


Fig. 9a-7. One end of bridge crane fell from its runway rail during the Managua, Nicaragua earthquake, 1972 (upper photo). In the same earthquake another overhead crane remained operational (lower photo). (Photo by Peter Yanev).



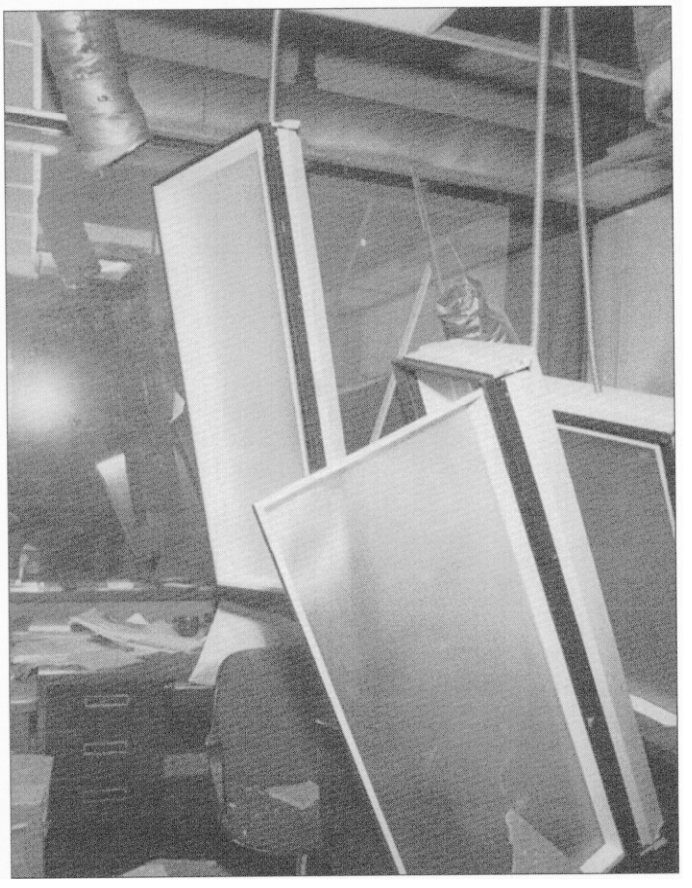


Fig. 9a-8. Failure of hung ceiling, light fixtures, and duct work over workstation, Livermore, California earthquake, 1980.

References

1. *Earthquake Protection of Essential Building Equipment, Design Engineering and Installation*, Gary L. McGavin, John Wiley & Sons, Inc., 1981.

2. *Architectural Design of Building Components for Earthquakes*, MBT Associates for National Science Foundation Research Report, San Francisco, California, 1978.

3. *A Report on Non-Structural Damage to Buildings, Alaska Earthquake, March 27, 1964*, Marx Ayres, Tseng-Yao Sun and Federick Brown, Ayres & Hayakawa, Consulting Engineers, Los Angeles, California, 1967.

4 *Earthquake Resistive Construction for Mechanical Systems, Light Fixtures and Ceilings*, Los Angeles Unified School System, J. Marx Ayres and Tseng-Yao Sun, Ayres & Hayakawa, Consulting Engineers, Los Angeles, California, 1972.

5. *Basis for Seismic-Resistant Design of Mechanical and Electrical Service Systems*, G. M. and T. R. Simonson, Consulting Engineers, San Francisco, California, and Engineering Decision Analysis Company, Inc., Palo Alto, California, 1976.

6. *The Great Alaska Earthquake of 1964*, Engineering Division, National Academy of Science, Washington, D.C., 1973.

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

8. *Study To Establish Seismic Protection Provisions for Furniture, Equipment and Supplies for VA Hospitals*, Research Staff Office of Construction, Veterans Administration, Washington, D.C., 1976.

Chapter

9b

Evaluation of Nonstructural Hazards

Ronald P. Gallagher

Nonstructural Elements

Building contents and the parts of a building that are not part of its structural system are considered to be nonstructural elements. For most buildings, nonstructural elements can be grouped in one of the following three categories:

- Architectural elements
- Mechanical and electrical equipment
- Building contents.

Architectural elements include items such as access floors, ceilings, exterior cladding, plaster and gypsum board walls, partitions, glazing, and ornamentation.

Mechanical and electrical equipment consists of individual items of equipment, as well as systems such as heating and air conditioning, communications, plumbing, power, and elevators. Individual items of equipment include such things as fans, boilers, electrical switchgear, emergency generators, and pumps. In laboratory or process buildings, equipment may include glove boxes, tanks and pressure vessels, high-pressure lines, process equipment, research or production reactors, waste treatment systems, material-handling equipment, hazardous materials storage, and a host of other items not found in ordinary buildings.

Building contents are things placed in a building by occupants. These are items that are moved into a building after it is built; they may include furnishings, bookshelves, storage racks, demountable partitions, laboratory apparatus, data processing equipment, and stored material.

In newer buildings, particularly those located in active seismic regions, many nonstructural items, particularly equipment and architectural elements, are required by building codes to be seismic resistant. In older buildings, however, most nonstructural elements have no seismic design and are much more susceptible to damage.

Performance of Architectural Elements in Earthquakes

One of the major costs of earthquakes is damage to nonstructural elements. Damage to architectural elements can be especially costly, particularly damage to partition walls and ceilings. (See Chapter 9c for a discussion of access floors.) Walls are typically damaged by racking action or by earthquake forces normal to the wall. Ceilings are usually damaged by shaking effects alone. A brief summary of damage to walls and ceilings follows. Additional information on the lessons learned from past earthquakes may be found in Refs. 1 and 2.

Walls are found on both the interior and the exterior of buildings. In newer buildings, exterior nonstructural walls (i.e., cladding) can consist of concrete, metal, stone, or glass panels fastened to the building's structural system. These are normally attached to the building, rather than built integrally with the structural system, so they will not be subject to racking forces. Cladding has failed in a number of instances when connections have broken. This type of damage, particularly when the cladding consists of heavy precast concrete panels, creates a falling hazard and major life-safety risk.

In older buildings, many exterior walls are constructed integrally with the structural system (e.g., steel frame). When constructed of masonry, these walls are often unreinforced and begin to crack and spall brick as seismic damage progresses. This creates a serious life-safety hazard. Usually, strong earthquakes heavily damage unreinforced brick and hollow clay tile partitions, particularly when used as fire walls round stairwells and elevator shafts.

Interior walls are typically constructed of plaster or gypsum board. They often experience cracking, particularly around openings, as a result of racking action. Normally, this type of damage does not pose a life-safety hazard, but is expensive to repair.

Heavy plaster and suspended acoustical ceilings found in older buildings have been known to collapse (Fig. 9b-1). Older, lightweight, suspended acoustical ceilings often have marginally connected runners and cross-runners which support lay-in tiles and are frequently damaged. The older versions, typically constructed before the late 1970s, often have runners and crossrunners that can easily separate, allowing the acoustical tiles to drop out. In these older ceilings, recessed light fixtures and diffusers have frequently fallen out. They are a more serious injury hazard to occupants because of their heavier weight. Newer suspended ceilings have better attachment of runners and crossrunners and diagonal bracing of both runners and recessed light fixtures. Generally,

Fig. 9b-1. Collapse of a ceiling in the control room of a large electrical facility. (Photo: Los Angeles Department of Water and Power)

these improvements have kept recessed light fixtures from falling on occupants, but have not prevented damage. Damage tends to concentrate around the perimeter of the ceiling. This results from pounding of the ceiling against the walls. Consequently, acoustical tiles also fall out.

Performance of Equipment and Building Contents in Earthquakes

Since the great Alaskan earthquake of 1964 and the San Fernando, California earthquake of 1971, where there was considerable damage to a wide variety of nonstructural items, especially equipment, a great deal of information has been obtained about the earthquake performance of these elements. Generally, damage to equipment, and to many building contents, has come from two sources: (1) lack of inherent ruggedness, and (2) lack of sufficient anchorage, restraining, and/or bracing to prevent sliding or overturning. The latter is more frequently encountered and can result in damage or severing of attached piping (Fig. 9b-2), conduits, control wiring, and ductwork.

Items that possess ruggedness will not experience failure if properly anchored (to the floor, wall, table, etc.). Examples of equipment items that have consistently demonstrated ruggedness in earthquakes include: diesel generators, motor control centers, motors, pumps, mild steel piping with welded joints, wall-mounted electrical panels, switchgear, small steel tanks and vessels, and many boilers. While there have been occasional exceptions, equipment of this type has not failed even under extremely strong shaking, provided that it is properly anchored (see Chapter 9d). Other types of equipment, such as delicate laboratory apparatus and porcelain-supported electrical equipment, have often experienced total and sometimes spectacular failures. There are several reasons for the inherent ruggedness of equipment. Diesel generators, for example, must withstand the rigors of start-up and normal operational vibrations. Electrical switchgear must be sturdy enough to survive the shipping and installation process undamaged, and cabinets are built with certain minimum metal gauge requirements.

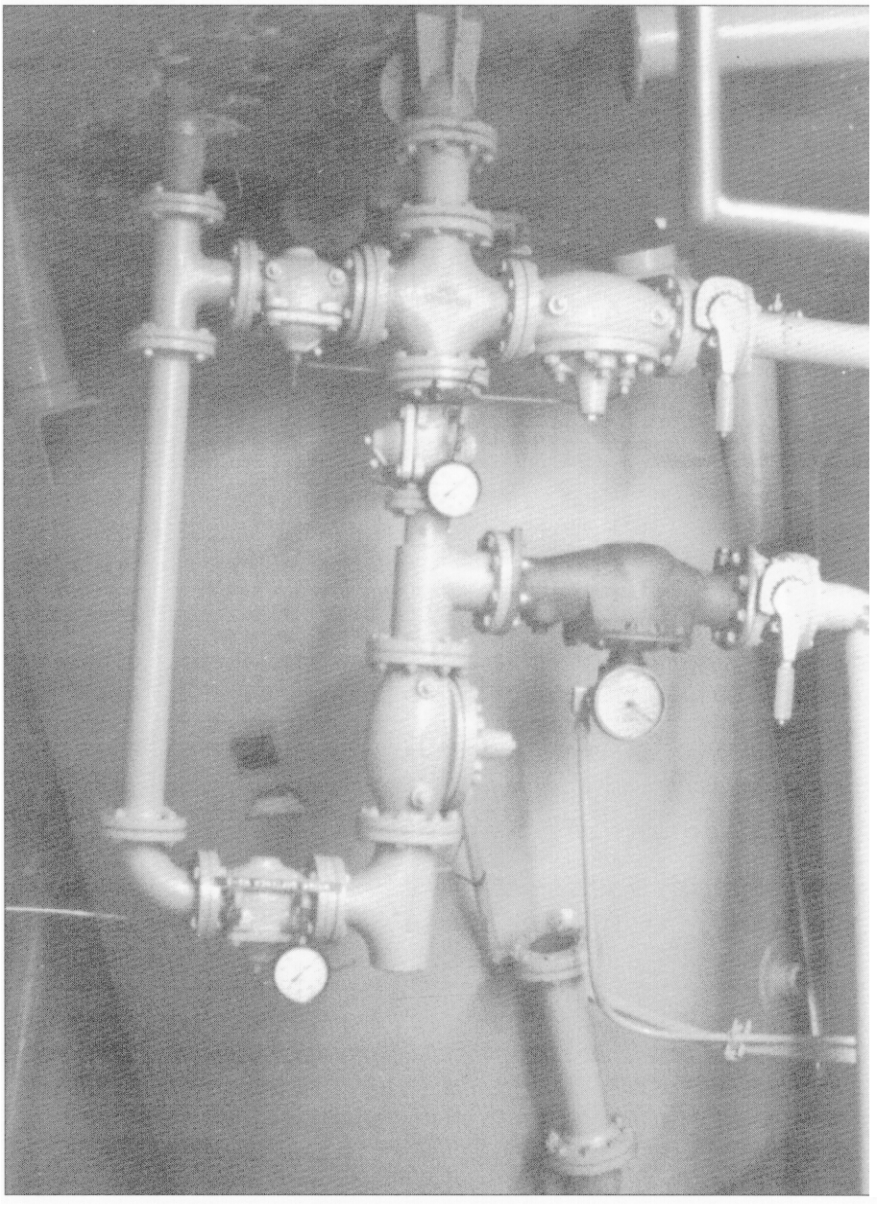


Fig. 9b-2. Broken piping caused by collapse of the legs on a water-softener tank. (Photo: Ronald Gallagher)

The usual mode of equipment failure or damage results from insufficient or inadequate anchorage. Occasionally, equipment is also damaged by being struck by other equipment (i.e., collateral damage) that was either unanchored or has torn loose from its anchorage. For example, large unanchored boilers in the mechanical building of the Olive View Hospital slid several feet during the 1971 San Fernando earthquake, severely damaging some nearby heating, ventilation, and air-conditioning (HVAC) control units (Fig. 9b-3) and one exterior wall of the building. In this same earthquake, large station transformers at the Sylmar Converter Station weighing more than 100 tons broke loose from their anchorage, overcame friction, and moved more than a foot, severely damaging large-diameter electrical cables.

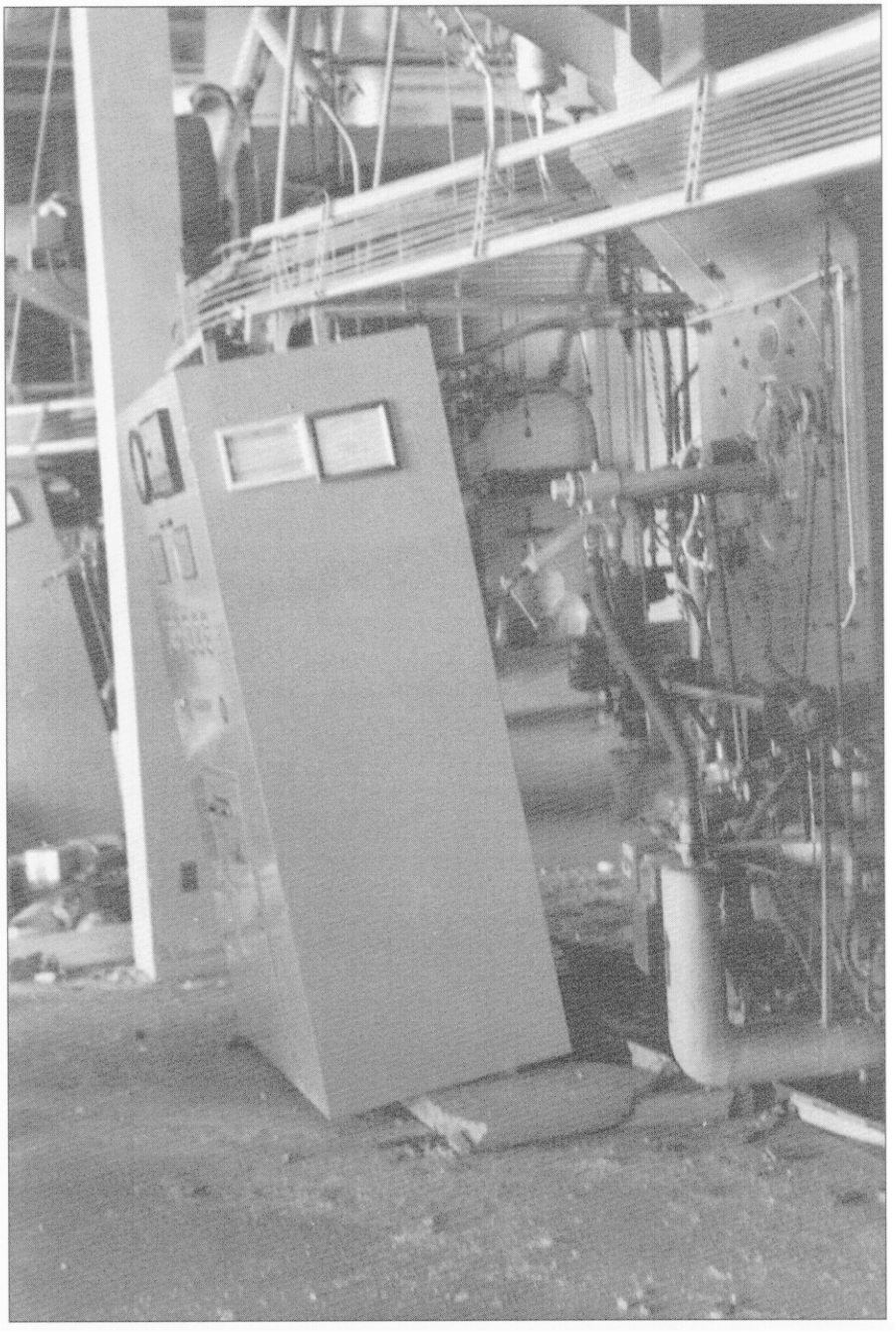


Fig 9b-3. HVAC control units damaged by being impacted by large unanchored boilers. (Photo: Ronald Gallagher)

Building contents such as bookshelves, file cabinets, laboratory apparatus, and personal computers usually suffer damage. During the 1980 Livermore, California earthquake, the Lawrence Livermore National Laboratory (LLNL) experienced heavy damage to building contents (Figs. 9b-4 and 5).

Table 9b-1 provides a representative list of nonstructural hazards and the principal concerns for each item.

Earthquake Effects On Nonstructural Elements

When an earthquake occurs, seismic waves are generated and travel rapidly through the earth in all directions. When these waves pass under a building, the resulting ground motion causes the building to sway back and forth. The motion generated within the building consists of two different but related phenomena:

- Shaking
- Story drift (Interstory displacement).

Shaking can damage nonstructural elements that are not sufficiently

Table 9b-1. A representative list of nonstructural seismic hazards.

Item	Principal Concerns
Architectural Elements	
• Suspended ceilings	Dropped acoustical tiles, perimeter damage, separation of runners and cross runners
• Plaster ceilings	Collapse, local spalling
• Cladding	Falling from building, damaged panels and connections, broken glass
• Ornamentation	Damage leading to a falling hazard
• Plaster and gypsum board walls	Cracking
• Demountable partitions	Collapse (i.e., falling over)
• Raised access floors	Collapse, separation between modules
• Recessed light fixtures, HVAC diffusers	Dropping out of suspended ceilings
• Pendant light fixtures	Stem failure, collapse
• Unreinforced masonry walls and partitions	Collapse, spalling causing debris and falling hazards.
Mechanical and Electrical Equipment	
• Main boilers	Sliding, broken gas/fuel and exhaust lines, broken/bent steam and relief lines
• Chillers	Sliding, overturning, loss of function, leaking refrigerant
• Emergency generators	Failed vibration isolation mounts, sliding; broken fuel, signal, and power lines leading to loss of function, broken exhaust lines
• Fuel tanks	Sliding or overturning, leaks, broken fuel lines
• Battery racks	Damaged rack, dislodged batteries, acid spill
• Fire pumps	Anchorage failure, misalignment between pump and motor, broken piping
• On-site water storage	Tank or vessel rupture, pipe break
• Communications equipment	Sliding, overturning, or toppling leading to loss of function
• Main transformers	Sliding, oil leak, bushing failure, loss of function
• Main electrical panels	Sliding or overturning, broken or damaged conduit or electrical bus
• Elevators (traction)	Counterweights out of guide rails, cables out of sheaves, dislodged equipment
• Other fixed equipment	Sliding or overturning leading to loss of function or damage to adjacent equipment
• Ducts	Collapse, separation, leaking fumes
• Piping	Breaks, leaks.
Building Contents	
• Radioactive materials	Breach of containment, radiation contamination
• Toxics and chemicals	Spill, fumes in ventilation system, fire, hazardous material contamination
• Liquid oxygen tanks	Sliding or overturning, leaks, broken lines
• Bottled gases	Overturning, broken lines, broken heads
• Storage racks	Spilling of contents, overturning
• Lockers, vending machines	Overturning, blocking exitways
• Explosives	Explosion, fire
• Laboratory equipment	Sliding, overturning, loss of function.

anchored or braced and elements that are not rugged. Many items are affected by shaking alone. Story drift is the relative horizontal movement that occurs between floors when the building sways back and forth. It can damage nonstructural elements such as walls by racking action. Elements that are attached to two or more floors, such as walls, cladding, and risers, are simultaneously affected by both phenomena.

A nonstructural item that cannot withstand shaking and/or any relative movement imposed on it will be damaged. This is usually called initial, or primary, damage. It can result in property damage, loss of function, and downtime. A consequence of primary damage may be the creation of *secondary hazards*, additional hazards created by the initial damage.

Secondary Hazards

There are various secondary hazards, some of which can be of far greater consequence than the primary damage. They include such things as:

- Hazardous material releases
- Fire
- Electrical hazards
- Explosion
- Water damage
- Falling hazards
- Collateral or interaction damage.

Most, if not all, of these hazards can be prevented, or the chance of their occurrence greatly minimized, if nonstructural elements are made earthquake resistant. This is particularly true for equipment and building contents.

Hazardous Material Releases

Earthquake damage that causes rupture of containers, tanks, and vessels or breakage of pipes and ducts can release hazardous powders, gases, and liquids into a building (Fig. 9b-6) and possibly into the general site vicinity. After the 1971 San Fernando earthquake, a large electrical facility was closed for several days by mercury

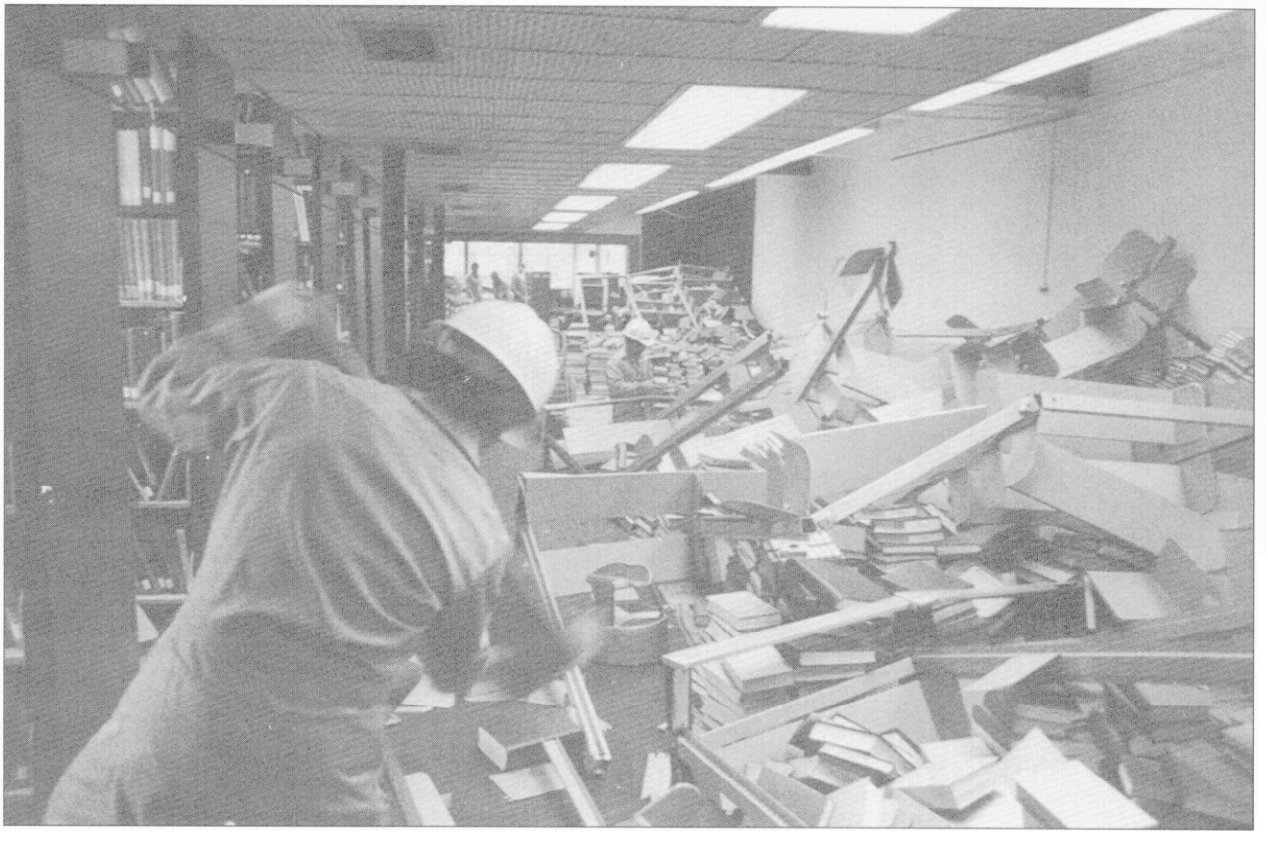


Figure 9b-4. Damage to an LLNL library caused by the 1980 Livermore earthquake. (Photo: LLNL)

contamination. This particular incident was complicated by an electrical fire in the same room. Other hazards include friable asbestos release from damaged insulation on pipes or damaged fireproofing on structural steel.

Fire

Fire is a common occurrence after large earthquakes (see Chapter 11a). Sources of ignition can include gas leaks and electrical short circuits. Major conflagrations occurred after the 1906 San Francisco and the 1923 Tokyo earthquakes (Ref. 3). Many fires were generated in the 1989 Loma Prieta and 1994 Northridge, California earthquakes and the 1995 Hyogo-Ken Nanbu earthquake in Japan. Of perhaps greater concern for DOE sites, however, is the possibility of a fire of local origin. Fighting a fire after an earthquake puts an additional burden on the emergency response organization at a time when its resources will be stretched to the limit.

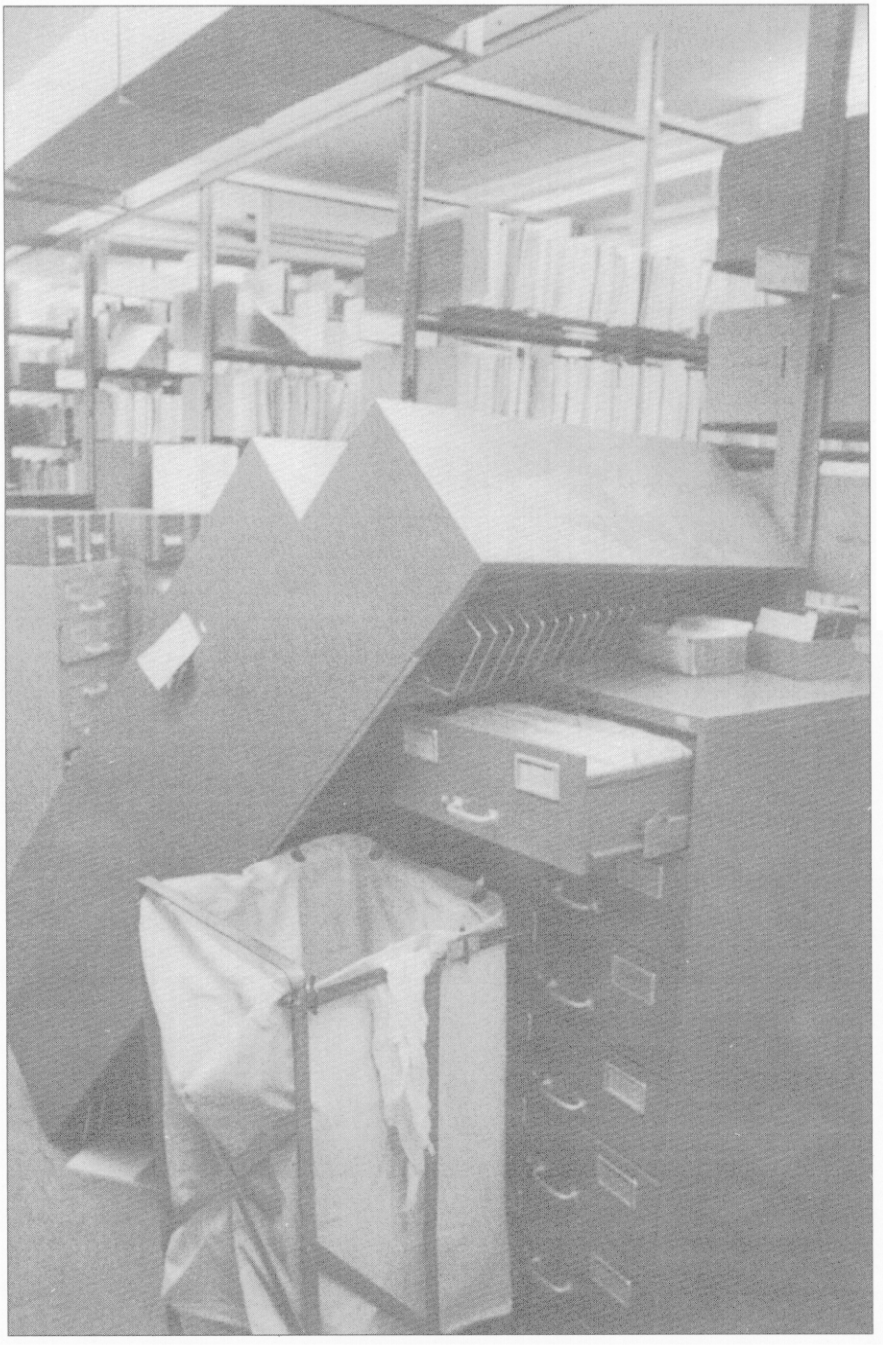


Fig. 9b-5. Overturned file cabinet at an LLNL office after the 1980 Livermore earthquake. (Photo: LLNL)

Electrical Hazards

Electrical hazards may include downed power lines, shorts in wiring, and malfunction of equipment such as relays. Downed lines may be on the ground or on the tops of buildings. It is important not to turn on power to a room or building in which the electrical system or equipment has been damaged. This may lead to sparks and subsequent fire or explosion.

Explosions

Explosions may have various causes. A ruptured gas line can release sufficient quantities of gas into a room where it can be set off by a spark or flame. Explosions may be relatively small, such as those resulting from mixing of laboratory chemicals, or quite large. The latter can result from large quantities of explosive substances released by broken lines, ruptured tanks, or spills in process and storage facilities. It is good practice to *sniff* electrical manholes before activating electrical circuits.

Water Damage

Water damage may result from broken pipes or ruptured tanks. Following the 1989 Loma Prieta earthquake, there were numerous instances of water damage in office buildings. In a data-processing building, a fire sprinkler line burst above a $17 million computer. It was several hours before the water could be shut off because no one on site knew where the shut-off valve was. In a highrise building some 60 miles from the epicenter, sprinkler piping crossing a building seismic joint broke.. The piping had no provisions for relative movement where it crossed building joints. Subsequent water damage was estimated to have caused more than half of the $3.5 million damage to this $70 million building.

Similar problems occurred in the 1994 Northridge earthquake. Several hospitals, with minor structural damage, suffered functional failures because of water damage.

Falling Hazards

Falling hazards are typically parts or pieces of a building that have been damaged or come loose (Fig. 9b-7), and may fall, injuring people. Aftershocks frequently *activate* falling hazards. Falling hazards may include such things as cracked parapets, broken or cracked overhead window glass, damaged ceilings, badly cracked hollow clay tile partitions, and duct work with damaged supports.

Collateral Damage

This is interactive damage to one element caused by the failure of another (Fig. 9b-3). For example, an unanchored item of equipment may slide and/or overturn. In the process, it may damage adjacent anchored equipment. Another example is an unreinforced brick parapet of one building falling on (and possibly through) the roof of an adjacent, lower building.

Finding Nonstructural Hazards in Existing Construction

The ability of a facility to perform its normal functions after a seismic event is dependent on prevention of serious structural damage and control of nonstructural damage. The word *control* is used because it is usually impractical to prevent all nonstructural damage. For example, cosmetic cracking in drywall is generally difficult to prevent and is not a life-safety hazard. However, serious damage to architectural elements, such as fallen ceilings, collapsed access floors, or fallen cladding, cannot be permitted. Also, equipment and systems

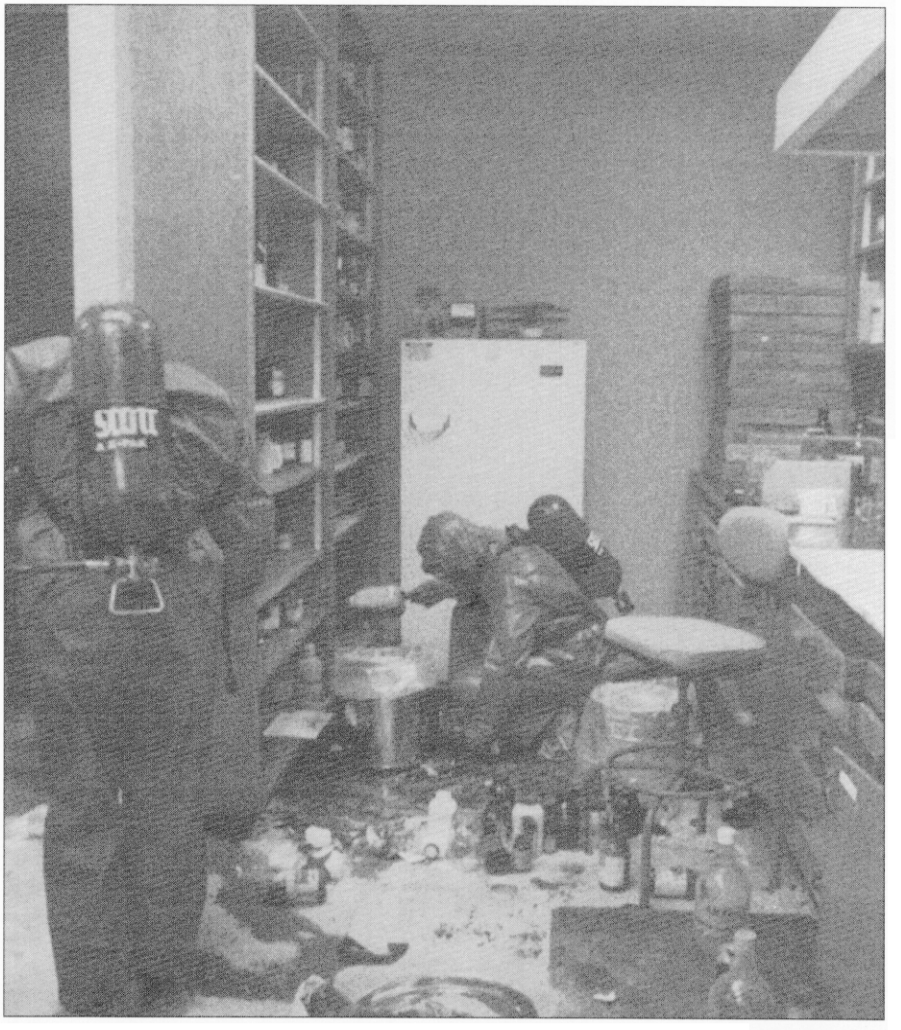


Figure 9b-6. A hazardous materials team cleans up an LLNL laboratory after the 1980 Livermore earthquake. (Photo: LLNL)

in a building can and should be made very damage-resistant, and unnecessary damage to building contents should be avoided.

Because there has been a tremendous amount of costly nonstructural damage, particularly to unanchored equipment and building contents in past earthquakes, it is very important to examine these items, particularly those considered vital to the facility's operations. These are items that if damaged would stop or curtail normal operations, and those that would cause a life-safety hazard.

One effective diagnostic measure is the seismic vulnerability survey inspection. This should be done for most equipment and building contents and for many architectural elements. The inspection is particularly useful when building managers are faced with the task of assessing the seismic vulnerability of large numbers of nonstructural elements within a low budget. It has two main objectives: (1) to identify and list the items considered important, and (2) to rate each item, or system, for its seismic vulnerability (i.e., likelihood of damage) under strong ground shaking.The process begins by developing a list of important equipment, contents, and other items. Seismic engineers tour the facility with staff familiar with each particular area's operations. Major facilities equipment, such as mechanical and electrical equipment located in mechanical rooms or penthouses, can be identified with the assistance of the in-house engineering staff. In operational areas of the building, where laboratory or data-processing elements are found, seismic engineers should be accompanied by operational managers for the area. Important items are thus identified and inspected, and listed with seismic vulnerability ratings. This is usually done by room or area.

The seismic assessment and rating of each item is best accomplished by a two-person team of engineers experienced in seismic design. Each item is visually examined for both overall ruggedness and the presence of anchorage and/or bracing. It is then rated as having a *high*, *moderate*, or *low* vulnerability. This is done on the basis of visual observation and engineering judgment. No calculations are performed or reviewed in this assessment.

Fig. 9b-7. This is a good illustration of a falling hazard. The TV cameraman could easily have been injured or killed by falling brick. (Photo: Ronald Gallagher)

The levels of vulnerability are defined as follows:

Vulnerability	Characteristics
High (H)	Possesses little or no seismic resistance, item may break, slide, or overturn under strong shaking. High probability of damage under strong shaking.
Moderate (M)	Possesses some seismic resistance, but not as much as an item rated low.
Low (L)	Possesses good seismic resistance, should resist moderate shaking without damage. Low probability of damage under strong shaking.

Table 9b-2 provides an example of how this type of survey can be summarized. Items listed are from one room of a large newspaper plant.

The above *walk-through* technique can also be applied to some architectural elements, particularly raised access floors and suspended ceilings. Other architectural elements, such as rigidly attached exterior cladding, unreinforced brick or hollow clay tile partitions, and nonseismic resistant attachment of ornamentation usually require review of drawings. When faced with older construction, experienced structural engineers can identify such things as rigid cladding and unreinforced masonry walls by visual inspection, although drawings may be needed to identify all locations where these occur.

This *walk-through* inspection is very useful for getting an initial handle on the seismic hazards present in a facility and for establishing priorities for subsequent more detailed evaluations and/or remedial strengthening. When faced with items with questionable seismic resistance, it may be necessary to do a detailed structural evaluation. This usually requires making calculations, but it is the only sure way to establish that a particular element has the desired level of seismic resistance. References 4, 5, and 6 indicate practical ways of reducing the risk of nonstructural hazards. Reference 7 provides additional guidance for seismic design of such items as elevators, access floors, and ceilings. Reference 10 is a walkthrough field guide for DOE facilities.

Preventing Nonstructural Hazards in New Construction

Modern building codes such as the *Uniform Building Code* (Ref. 8) and the *NEHRP Provisions* (Ref. 9), have requirements for the seismic design of many nonstructural elements, particularly those that can affect life-safety. In seismic areas, items such as cladding, boilers, electrical switchgear, and emergency generators are routinely anchored and braced during the new building construction process. At those Department of Energy (DOE) facilities dealing with nuclear materials and other hazardous materials, nonstructural items, such as process equipment, ducts, and piping, are often subject to a seismic design requirements far in excess of that required for ordinary construction (see Chapter 4), and because of effective quality assurance programs, nowadays these items are almost always constructed as specified. On the other hand, seismic design of nonstructural elements in ordinary construction is too often overlooked. It is not uncommon to find new or recently completed buildings with most, but not all, of the required nonstructural elements anchored or braced. Often building contents are completely ignored. The problem comes about because several different design disciplines and several different subcontractors typically are involved with nonstructural elements, and seismic design requirements are ignored or forgotten in the design or construction processes.

To avoid these problems, it is recommended that the design team, particularly the mechanical and electrical engineers, be instructed at the outset that seismic design is a project requirement that must be incorporated in the final construction. Postconstruction walk-through inspections, particularly when carried out by seismic/structural engineers, are a simple but effective method to verify seismic design compliance and to identify any omissions.

Table 9b-2. Example seismic vulnerability survey summary.

Mechanical Equipment Room		Vulnerability Rating			Description or Comments
No.	Item	H	M	L	
1.	Motor Control Centers A and B			x	Anchored
2.	Power Panels 1 and 2			x	Anchored
3.	Wall-Mounted Control Panels			x	Anchored
4.	Chillers			x	Anchored
5.	Capacitor Disconnect	x			Unable to verify anchorage
6.	Brine Tank		x		Apparently unanchored
7.	Water Softeners	x			Two of these, both unanchored and may overturn
8.	Air Dryers	x			Two of these, both unanchored
9.	Air Handling Units	x			Unanchored
10.	Surge Tank	x			Unanchored tall slender unit, can overturn
11.	Water Heaters (1,000 and 2,000 gallons)	x			Unanchored
12.	Pneumatic Tube	x			Unanchored
13.	Compressed Air Tanks	x			Unanchored
14.	Air Compressors		x		Three of these. Electrical controls braced. Units on vibration isolation devices which are not anchored to floor but units in small "pits" which will help restrain lateral movement.
15.	Chiller Pumps			x	Anchored
16.	Halon Tanks			x	Well anchored
17.	Return Air Fans		x		Nominally anchored, but anchor bolts look small.
18.	Economizer/Filter Unit			x	Anchored.

References

1. *Reducing Earthquake Hazards: Lessons Learned from Earthquakes*, Publication No. 86-02, Earthquake Engineering Research Institute, Oakland, California, November 1986.

2. *Nonstructural Issues of Seismic Design and Construction*, Publication 84-04, Earthquake Engineering Research Institute, Oakland, California, July 1984.

3. *Earthquakes, Volcanoes, and Tsunamis: An Anatomy of Hazards*, K.V. Steinbrugge, Skandia America Group, New York, New York, 1982.

4. *Seismic Restraint Manual Guidelines for Mechanical Systems*, Sheet Metal and Air Conditioning Contractors National Association, Inc., First Edition, Chantilly, Virginia, 1991.

5. *Reducing the Risks of Nonstructural Earthquake Damage: A Practical Guide*, Bay Area Regional Earthquake Preparedness Project, Oakland, California, October 1985.

6. *Practical Equipment Seismic Upgrade and Strengthening Guidelines*, (UCRL-15815), Lawrence Livermore National Laboratory, Livermore, California, September 1986.

7. *California State Structural Safety Interpretive Manual*, Division of the State of Architect, Sacramento, California, September 1989.

8. *Uniform Building Code*, International Conference of Building Officials, Whittier, California, 1994.

9. *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, prepared for the Federal Emergency Management Agency by the Building Seismic Safety Council, Washington, D.C., 1994.

10. *Walkthrough Screening Evaluation Field Guide, Natural Phenomena Hazards at Department of Energy Facility*, Lawrence Livermore National Laboratory Report UCRL-ID-115714 Rev 2, November 1993.

Chapter

9c

Data-Processing Facilities and Systems

Roland L. Sharpe

Introduction

This chapter discusses issues important to the seismic adequacy of *data-processing* (DP) facilities and support systems including computer processing, telecommunications and other electronic facilities.

DP systems are vulnerable to earthquake damage. Evidence from earthquakes in the United States and foreign countries clearly shows that DP facilities and systems can be vulnerable to many types of seismic damage. These include loss of power, cooling, and other services; overturning of equipment; failure of raised floors; spillage of discs or tapes; collapse of suspended ceilings; water damage; and similar problems. DP systems are of vital importance to many organizations, consequently, a shutdown of more than a few hours cannot be tolerated. DP is used for management information services, database management, process control, research, record keeping, telecommunications, engineering design, payroll, banking, and other critical functions. The costs incurred from interruptions may far outweigh the direct costs of repair.

Ground motions created by earthquakes induce inertial forces and displacement effects in structures and the contents within them. It is therefore important, when designing for earthquake resistance, that all components of buildings, contents, and supporting systems have adequate strength, be tied together, and provisions made for differential motions and displacements. Variables that should be considered when estimating the amount of damage a facility might expect are probable locations of future earthquakes, expected frequency of occurrence, site-soil profile, and building height.

The selection of design methodology and seismic criteria for a DP center is, to a large extent, controlled by the performance criteria needed for the building and its contents. The choice of design approach should be based on the importance and complexity of the proposed facility. When a DP center is considered to be of major importance, a detailed seismic hazard study may be necessary.

It is also prudent to carefully evaluate the potential seismic performance of required support utilities for the DP facility, both off site and on site. This information enables management to make better decisions about which back-up measures are necessary at the facility itself.

The operation of a DP facility is dependent on the proper functioning of a number of systems, including:

- Enclosing and supporting building
- Nonstructural enclosure
- Data-processing equipment
- Support systems
- Supporting utility systems; electric power, water, storm/sanitary sewers, communications.

These issues are discussed below.

Enclosing and Supporting Building

Both the performance of buildings in earthquakes and seismic-resistant design requirements are discussed in other chapters. Thus, only factors pertinent to DP facilities and systems are discussed here. During a seismic event, all components of a building, including walls, columns and floors, equipment systems, and personnel, are subjected to induced inertial forces.

An earthquake is an extremely pervasive natural phenomenon that can simultaneously affect almost all parts of a facility.

The primary philosophy of seismic codes in the United States is to protect the life-safety of occupants and people near buildings. In recent years, it has been recognized that it is also essential that occupants are able to leave a damaged building and that rescuers are able to enter. Further, in the event of a major earthquake, essential facilities such as designated DP centers should be able to function during and immediately after the event. Thus, structures enclosing DP centers should be designed for more stringent performance requirements than ordinary facilities.

Seismically induced building motions generally amplify with building height. Thus, building components and equipment are subjected to increasing inertial and displacement effects the higher their location is in a building. Fig. 9c-1 illustrates variations in design acceleration with height in building versus *Uniform Building Code* (UBC) Seismic Zone. Table 9c-1 gives approximate values for the increase in motion with height in a building. These values are for guidance only; the design of specific installations should be based on results from building dynamic analyses. These factors are important when locating DP facilities or evaluating existing facilities. Special care should be given to making the expected deformations of building framing (flexible, fairly stiff, or stiff) compatible with the nonstructural components and systems, DP equipment, raised floors, and ancillary equipment. Normally, there is an economic tradeoff between building frame flexibility and the special provisions required for nonstructural components, support systems, and their connections.

Nonstructural Enclosure

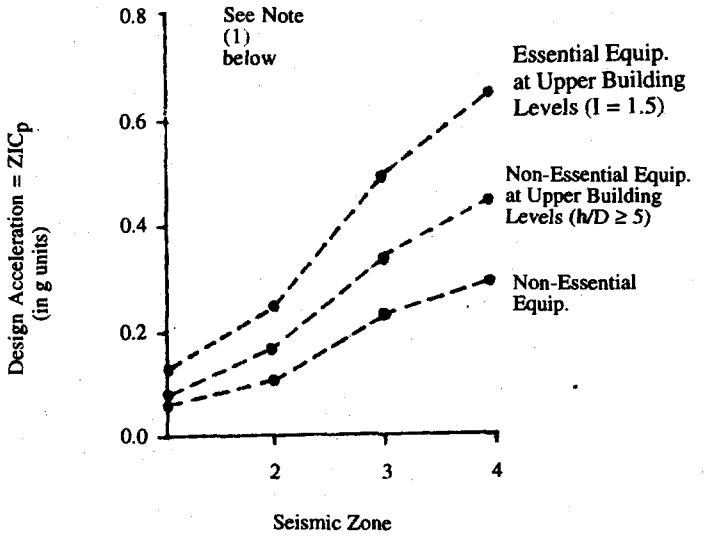
The enclosures for DP centers must be environmentally controlled. Thus, the enclosing structure, partitions, ceilings, and raised floors must be designed to minimize damage to the enclosure during an earthquake. As noted previously, special consideration must be given to ensuring that differential deformations or displacements between the structure and nonstructural elements are compatible.

Partitions

Normally cracking of partitions (gypsum board or plaster) is not of major concern, but the dust from cracking may be a problem for the DP equipment. Thus, an appropriate wall finish or covering should be provided.

Sometimes unreinforced masonry (URM) walls are used for partitions. Where URM partitions exist or are proposed, structural engineers should be consulted to ensure against adverse damage or dusting from cracking or spalling of the URM.

Partitions may be ceiling supported, cantilevered from the floor, or movable. Ceiling-supported partitions should be braced to the building structure above. Cantilever or movable partitions normally lack design for lateral seismic force. If bookshelves or other relatively heavy elements are supported on movable partitions, the seismic resistance of the partitions should be reviewed by a structural engineer and strengthened as required. Partitions should not be supported on



Notes
(1) Curves are based on the assumption that equipment and its support are not "flexible" or "flexibly" mounted.

Fig. 9c-1. Design acceleration versus seismic zone for essential and non-essential equipment types, based on the 1985 UBC; the 1994 UBC produces similar results.

Table 9c-1. Approximate increases of lateral motion acceleration with building height . (VSP Associates, Ref. 1).

Percent of building height	Increase over motion at base
Roof = 100	2.0
90	1.9
80	1.8
70	1.7
60	1.6
50	1.5
40	1.4
30	1.3
20	1.2
10	1.0
Base = 0	1.0

raised access floors because raised floors may not be able to withstand the extra seismic forces. Partitions enclosing a DP center should be supported directly by the building structure.

Ceilings

Ceilings in DP facilities are usually suspended from the structure above to provide space for heating, ventilating, and air conditioning (HVAC) ducts, piping, and conduits. When this space is used as a return air plenum, ceilings are important to proper operation of the HVAC system. The UBC (Ref. 2) specifies requirements for seismic bracing of ceilings. Lighting fixtures also should be secured (per UBC requirements) to protect life-safety and possible damage to DP equipment if fixtures should fall. The Lawrence Livermore National Laboratory (LLNL) published recommendations for bracing suspended ceiling systems in 1985 (Ref. 3).

Access Floors

Raised access floors in the United States provide support for the DP equipment, a plenum for air-conditioned air, and an enclosure for subfloor utilities, wiring, cables, and conduit vital to operation of the DP equipment. Raised floors are an essential element in almost all DP facilities because of the need for under-floor cable access and air-conditioned cooling air for DP equipment cabinets.

Access floors in the United States have been exposed to relatively few earthquakes. There were reports of three floors collapsing in the 1971 San Fernando, California earthquake, some diagonal bracing fractured in the 1980 Livermore-Greenville, California earthquakes, and some damage occurring during the 1989 Loma Prieta and the 1994 Northridge, California earthquakes.

Experience in earthquakes has shown that floor panels can become dislodged and, because of the subsequent loss of support, equipment can tip or overturn. The DP equipment supported on raised floors has a monetary value several hundred times that of the supported floor, so care in the design, specification, and construction of the floor structure is cost-effective. The UBC requires that raised floors be designed for seismic forces in Seismic Zones 2, 3, and 4. The required lateral force for seismic design is:

$$F_P = Z I_P C_P W_P$$

where:

Z = Seismic zone factor

I_P = Importance factor

C_P = 0.75

W_P = floor system dead load plus 25% of floor design live load plus a 10 psf partition allowance.

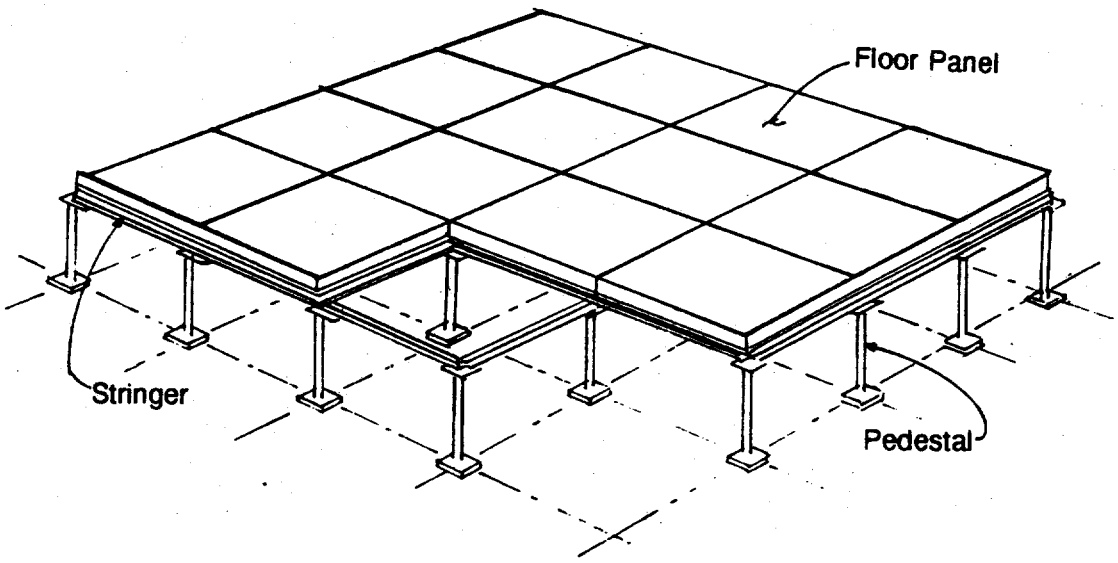
If the DP center is located on higher floors in a building, amplification of seismic motions with building height should be incorporated. Consideration should also be given to limiting lateral deflection of floor support pedestals under seismic loading. A limit of 0.2 inch for an 18-inch-high floor is appropriate for damage control.

Access floors are typically constructed of supporting pedestals, stringers, and removable 2-ft square floor panels. Figure 9c-2 is shows a typical raised-floor system. For best seismic resistance, pedestals should be designed as cantilever columns fastened with anchor bolts to the supporting concrete slab. The stringers should be tightly connected to the pedestal head (top) with small bolts or substantial self-tapping screws. Diagonal bracing can be used, but it generally interferes with underfloor cabling and conduit.

Data-Processing Equipment

The types, shapes, and sizes of data-processing equipment are continually evolving. Computer processing units, disc and tape drives, power-distribution units, coolant-distribution units, data splicers, data storage units, telecommunication cabinets, and voice-mail controllers are some of the types of equipment encountered.

The reported types of damage to DP equipment include overturning, supports sliding into floor openings and equipment tilting or tipping over, power surges causing severe malfunctions, racking of anchored equipment, water-line rupture and flooding access space,



ACCESS FLOOR

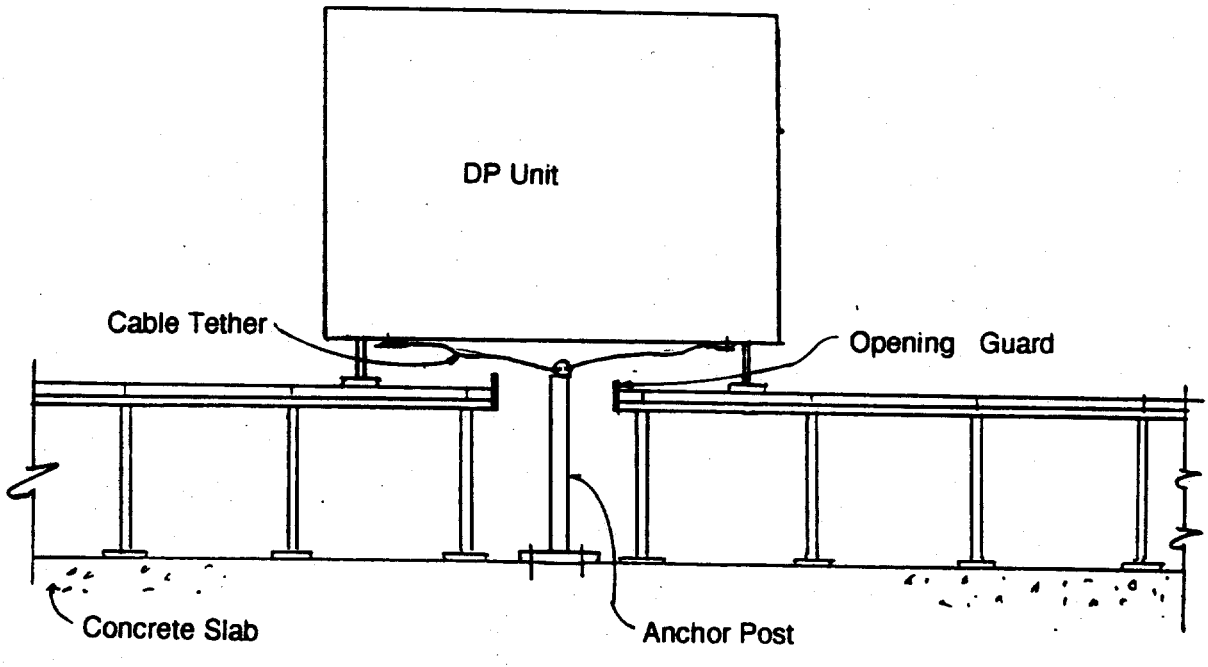


Fig. 9c-2. Raised access floor and tethering concepts.

leveling-pad breaks causing cabinet misalignment and equipment tilt, and desk-top consoles falling off the desks.

Designers disagree whether to use fixed anchorage or flexible restraints for DP equipment. Shake-table testing has shown that anchored equipment cabinets rack/distort, and electronic components within may malfunction. On the other hand, when units that are not susceptible to overturning are allowed to roll or slide, friction between the leveling pads/casters and the floor dissipates energy while the movement partially isolates the unit from the earthquake motions. There is a tendency among knowledgeable designers to allow equipment to move during an earthquake where feasible.

Heavy equipment, such as air-conditioning units and power-distribution units, that is often located in the same room or enclosure with the DP units, should be supported separately from the raised floor. Similarly, units susceptible to overturning, such as those with an aspect ratio (H/B; height to width or depth) greater than about 2.5, or with a high center of mass, may need to be directly braced to the building structure. Bracing can be provided by steel framework extending through the access floor and by bracing the top of the equipment up through the ceiling to the building structure.

Table 9c-2 lists seismic design considerations for DP equipment, including telecommunications and similar systems. Fixed anchorage or bracing may not be the best solution for equipment that is stable against overturning. Fixed anchorage requires that overturning and lateral shear forces be transmitted from the top of the raised floor to the structure below. If the equipment is allowed to slide or roll within limits, much of the seismic energy is dissipated and there is less induced force on internal components of the DP equipment per se. DP equipment can be restrained by under-floor cable tethers and allowed to move a limited amount. Several factors should be considered if a tethering system is used:

- Openings in the access floor should have raised curbs or edges to prevent casters or leveling pads from sliding into the opening.
- DP equipment must be stable against overturning when a reasonably conservative coefficient of friction is assumed between the access floor surface and the equipment base.
- There must be space for DP equipment to move without impacting other equipment or the building structure.
- If DP equipment units or cabinets are adjacent to each other, they may be bolted or connected together or elastomeric pads or bumpers may be provided.
- The location of tether anchors and cable attachments to equipment must take into account the distribution of mass within units/cabinets.
- The design of the tether anchorage to the structure must consider the effect on the access floor of cables pulling taut as DP units move. If cables are anchored to the building floor slab, there is a downward force on the access floor as the cable becomes taut.
- There must be sufficient slack in power or telecommunication cables and coolant lines to allow movement of the DP equipment.

Tethering is especially beneficial if internal components are fragile or cannot withstand severe cyclic inertial forces.

A modified tethering scheme may be a viable option if DP equipment cabinets or units are tall and slender and internal components would be susceptible to malfunction if subjected to large inertial forces. With this concept of restraint, hold-down anchors (usually threaded rods) are installed to resist overturning, and a tethering system is installed to resist the lateral forces after limited movement of the DP units. Care must be exercised to avoid overstressing the anchors in compression.

Support Systems

Ancillary support equipment includes air-conditioning units, uninterruptable power

Table 9c-2.

Seismic design considerations for data-processing equipment and similar systems.
(Modified from VSP Associates, Ref. 1, Table 7.3)

Design Factors	Low Risk	Medium Risk	High Risk
Level of ground motion hazard	Seismic Zone 1	Seismic Zone 2	Seismic Zones 3 & 4
Floor location and building height	Ground floor or basement	Upper floors of mid-rise (3-10 stories)	Upper floors of high-rise (20 or more stories)
Raised floor design	Special	Building code	Nonseismic
Equipment support or attachment	Fixed, anchored, braced, or tethered	Wheels, casters (low friction)	Skids, leveling pads (high friction)
Cabinet geometry	Short and wide (H/B 0.05)	Average (H/B approx. 1.5)	Tall and narrow (H/B >2.5)
Cabinet strength and stiffness	Strong and stiff	Medium	Weak and flexible
Layout	Adequate space (3 feet or more, or very close and tied together)	Moderate space (1.5 to 3.0 feet)	Closely spaced (impact potential) (0.5 to 1.5 feet)

systems (UPS), emergency power generators, precision power units, halon tanks/distribution, voltage controllers, air compressors, heat exchangers, refrigerant units, and cooling towers. Data-processing units operate in a controlled environment, and failure of any of the support systems for other than a very short time can cause the DP units to overheat or malfunction. When evaluating or designing a DP center, every element contributing to its operation must be properly designed for seismic loads.

The matrix in Table 9c-3 shows the relationships between nonstructural/support systems and DP systems. A frequent occurrence during earthquakes is loss of electrical power and, depending on the type of system, loss of telephone communications. During the 1989 Loma Prieta earthquake, there was loss of power to large geographical areas for 48 hours or more. Many phone systems were out of commission because they were dependent on the public utility power system, which was down. Phone systems with emergency power systems continued to function. Water-distribution systems were shut down in some areas because of broken mains. Piping can rupture, UPS battery racks can overturn, and cooling towers can malfunction unless they are adequately anchored, braced, or restrained.

An example of the necessity for reliable back-up systems occurred at one of the world's largest credit-card companies headquartered in San Mateo, California. The DP center lost public utility power, but the UPS system functioned for 15 to 20 seconds until the emergency-power generator came on line. The center continued processing world wide transactions for many hours until public utility power was restored. The emergency power

Table 9c-3. Nonstructural building element relationships, types, possible uses, operational importance.
(From VSP Associates, Ref. 1)

USE CATEGORIES							VULNERABILITY	ELEMENT/SYSTEM	ELEMENT TYPE
ENVIRONMENTAL HVAC	ENVIRONMENTAL POWER & LIGHTING	ENVIRONMENTAL SECURITY	ENVIRONMENTAL ENCLOSURE	DP COMMUNICATION SUPPORT	DP AIR CONDITIONING	DP POWER			
▫	▫			■	■	■	M	SUBSTATION	ELECTRICAL
▫	▫			■	■	■	M	SWITCHGEAR	
						■	H	MOTOR GENERATOR	
						■	M-H	UPS EQUIPMENT	
▫	▫			■	■	■	M	TRANSFORMERS	
						■	H	POWER DISTRIBUTION UNITS[1]	
▫	▫			▫	■	■	H	ENGINE GENERATOR[2]	
▫	▫			■	■	■	L	DISTRIBUTION PANELS	
▫					■		H	MOTOR CONTROL CENTERS	
▫	▫			■	■	■	L	BUS DUCT, CONDUIT, ETC.	
▫					■		M	BOILERS[2]	HEATING
▫					■		H	HOT WATER HEATERS, TANKS	
▫					■		L	PUMPS	
▫					■		H	TANKS	
▫					■		M	MISC. CONTROL CABINETS	
▫					-		L-M	EMERGENCY FUEL STORAGE[2]	
▫					■		L-M	ASSOCIATED PIPING	
▫					■		H	CHILLERS	COOLING
▫					■		H	CONDENSERS	
▫					■		M	HEAT EXCHANGERS	
▫					■		H	COOLING TOWER	
▫					■		L	PUMPS	
▫					■		M	CONTROL CABINETS	
▫					■		L-M	ASSOCIATED PIPING	
▫					■		H	CONTROL AIR COMPRESSORS[2]	
▫							H	EXHAUST FANS	AIR
▫							M	H/V UNITS	
▫					[1]	■	M	HVAC UNITS	
▫						■	H	HVAC UNITS[1]	
▫			▫				H	GRILLS, DIFFUSERS (OVERHEAD)	
▫						▫	L	ASSOCIATED DUCTWORK	
▫						▫	M	FILTERS	
▫						▫	M	HUMIDIFIERS	
				■			M	SWITCHING EQUIPMENT	COMMUNICATION
				■			M	MISC CABINETRY	
				■			L	ANTENNA	
				■			H	BATTERIES	
		■	■				H	MASONRY PARTITIONS ON STRUCT	ENCLOSURE
		▫	▫				L	NONMASON PARTITIONS ON STRUCT	
		-	▫				H	NONMASON PARTITIONS[1]	
▫	■		■				H	PANELIZED CEILINGS	
▫	▫		▫				L	SOLID CEILINGS	
			▫				M	STORAGE RACKS, SHELVING	
			▫				H	STORAGE RACKS, SHELVING[1]	
			■				H	ELEVATOR SHAFT	
			■				H	ELEVATOR EQUIP ROOM	
							M	ENTRY CONTROL SYSTEM	SECURITY
		▫					H	VIDEO SURVEILLANCE SYSTEM	
		▫					L	FIRE PUMPS	
		▫					L	SPRINKLER PIPING	
		▫					M	LOCAL EXTINQUISHERS	
		▫					M	HALON SYSTEM	
	■		■				—	FIXTURES IN PANELIZED CEIL	LIGHT/ POWER
	■		■				—	HUNG FIXTURES — SOLID CEIL	
	▫		▫				L	OTHER FIXTURES — SOLID CEIL	

■ GREATER OPERATIONAL IMPORTANCE

▫ LESSER OPERATIONAL IMPORTANCE

1 ON COMPUTER ACCESS FLOOR

2 INCLUDES RELATED EQUIPMENT

3 DAMAGE LIKELY TO OCCUR IF INSTALLED WITHOUT SEISMIC CONSIDERATIONS
L = LOW
M = MODERATE
H = HIGH

system powered all of the necessary support systems, including cooling, heat exchangers, switchgear, and motor-control centers.

Consideration should be given to providing back-up systems for power and water supplies when DP outage must be limited to a few hours. Back-up should also be considered for longer outage times, unless a very high reliability rating can be given to the public utility supply system.

The support power systems include storage batteries for the UPS, the main power supply, and the back-up generator system for critical support systems. All systems should be designed with adequate seismic resistance.

The need for a back-up water supply system is dependent on the type of cooling system utilized, cooling tower, or chiller. A supply of potable water for personnel should be provided so that the DP center can continue to function.

The procedures for assessing the seismic vulnerability of an existing DP facility or one to be constructed in an existing building are similar to those for new facilities.

Table 9c-4 provides a list of special precautions and recommendations to be considered during design of DP support systems. The Sheet Metal and Air Conditioning Contractors National Association (SMACCNA) manual, *Seismic Restraints for Mechanical Equipment* (Ref. 4), lists seismic bracing and anchorage requirements for ductwork and general piping systems.

Supporting Utility Systems

The continued operation of a DP center or facility is dependent on continued functioning of the following systems:

- Electrical power
- Water supply
- Sanitary and storm sewers
- Natural gas
- Communications.

In addition, site and building access are essential.

Electrical power is necessary for operation of almost all systems in a DP facility, including processing equipment, lighting, HVAC, cooling towers, chillers, water pumps, fire detection and suppression systems, communications, building elevators and escalators, and building security systems.

A water supply is essential for the cooling system, sprinklers, drinking water, and sanitary facilities. Sanitary sewers are needed for waste disposal. Storm sewers are needed for water runoff to prevent or minimize flooding. Most heating systems use natural gas; others utilize propane gas, coal, or fuel oil. The heating system is required to maintain environmental temperature control and provide hot water.

The communications system, whether hardwired or microwave, is dependent on electrical power.

The performance of public utility systems during earthquakes has been mixed, some excellent and some poor. Electrical power systems have had substation damage and malfunction and loss of distribution lines/pole transformers. Water-supply systems have suffered line breaks, shut-down of pumps because of electrical power loss, and storage-tank damage. Sewer systems have suffered pipe breaks. Natural gas distribution systems have had line breaks. Communications have been disrupted following power loss and equipment failure. Personnel access to areas and sites has been impeded by highway overpass collapse and bridge failure. Fire following earthquakes has disrupted virtually all lifelines.

It is important that the reliability of off-site utility systems be evaluated. Redundancy and alternative routing, local distribution, time required for repair and/or restoration of service, and the priority of the DP center for service resumption should be assessed. If the reliability of public utility systems cannot be reasonably ascertained, then (depending on DP outage limitations) consideration should be given to providing back-up systems. Where only a limited outage can be tolerated, back-up

Table 9c-4. Special precautions and recommendations for equipment.

1. **Design equipment supports for toughness and ductility.** Support materials (such as mild steel) and support arrangements should be designed to provide tough performance. Brackets, anchors, etc., should be ductile so that they will bend and not break, and thus continue to carry load and dissipate energy.

2. **Use proper connections.** It is good seismic design practice to balance member strength with connections as strong as the member itself or to provide connections that bend or give rather than break.

3. **Consider the interaction of piping and structure.** Pipes attached to relatively massive, rigid appurtenances and structures can be broken or damaged by differential displacements.

4. **Avoid resonance problems; mount equipment rigidly.** Equipment flexibly mounted in buildings or structures can have their response to earthquake motion considerably amplified. When the period of vibration of an equipment item located on an upper building level is approximately the same as one of the predominant periods of vibration of the building, amplification of ten times the building response or even greater can result. Possible resonance problems can be avoided by stiffening the system or bracing equipment rigidly.

5. **Let equipment move or slide.** Vibrating equipment such as air compressors, fans, pumps, and motors normally are mounted on spring-type isolators to prevent transmission of the vibrations into the building structure. For this type of equipment, restraints can be provided similar to those shown in Fig. 9c-3.

   Other equipment that could be sensitive to inertial forces and deformation may be allowed to slide providing:

   a. The height-to-base width ratio is low enough so that the equipment will not overturn when it is subjected to earthquake forces.

   b. The horizontal movement is limited by means of a restraint or tether.

   c. Energy-absorptive bumpers are provided to minimize damage if the equipment should impact the structure or adjacent equipment.

6. **Consider the effect of interstory building displacements.** Equipment secured between two floors of a building may be forced to accept large interstory displacements under earthquake motion. Designers should take care to ensure that the supporting system can accommodate realistic interstory displacements, which may be on the order of two to four times greater than those calculated from code seismic forces.

7. **For seismic design of equipment, give responsibility to experienced engineers.** Seismic design of equipment supports should be accomplished by engineers familiar with structural analysis techniques and experienced in structural design in seismic areas.

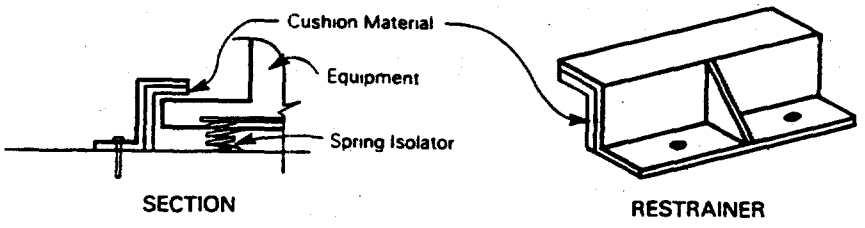


Fig. 9c-3. Equipment restraint.

systems are recommended for facilities located in UBC Seismic Zones 3 and 4.

Access to the site after an earthquake is important to enable personnel to reach work, for delivery of needed materials and replacement units, and to allow ambulances, repair crews, and others to care for injured people, perform repairs, clean-up, and remove debris. Consideration should be given in planning a new facility or evaluating an existing facility for potential access routes and their dependability.

References

1. *Data Processing Facilities - Guidelines for Earthquake Hazard Mitigation,* FIMS, Inc., VSP Associates, Sacramento, California, 1987.

2. *Uniform Building Code,* International Conference of Building Officials, Whittier, California, 1994.

3. *Suspended Ceiling Systems Survey and Seismic Bracing Recommendations,* UCRL 15714, Lawrence Livermore National Laboratory, Livermore, California, 1985.

4. *Seismic Restraints for Mechanical Systems,* Sheet Metal and Air Conditioning Contractors National Association, Los Angeles, California, 1992.

Chapter

9d

Seismic Experience Data and Equipment Damage

Sam W. Swan
Peter I. Yanev

Background

In recent years, the nuclear power industry has sponsored postearthquake investigations for the purpose of collecting performance information about equipment representative of nuclear plant safety systems. Postearthquake investigations have focused on power plants, substations, and large industrial and commercial facilities. These investigations have been sponsored primarily by the *Seismic Qualification Utilities Group* (SQUG) and the *Electric Power Research Institute* (EPRI). Over the years, these investigations have included some twenty strong-motion earthquakes in California and elsewhere around the Pacific rim. About one-hundred different sites have been studied, from which earthquake performance details have been collected for about several thousand items of equipment. In recent years, the two utility groups have sponsored a project to compile the information collected into a computerized database system.

Although the focus of investigations is equipment relevant to power plants, information has been collected on most types of equipment commonly found in most large industrial and commercial facilities. These include instrumentation and control systems, power supply systems, mechanical equipment, and their various interconnections—piping, tubing, conduit, ducts, cable trays, etc.

The resulting equipment inventories encompass a wide range of sizes, configurations, ages, applications, operating conditions, and locations at ground level or above ground in various building structures. The motion experienced at various sites ranges from about 0.20g to 0.80g, with strong-motion duration ranging from a few seconds to almost a minute. The effects of different soil conditions and the filtering or amplification effects of buildings create a variety of frequency content in the base motion experienced by the equipment.

Database inventories usually consist of a large majority of undamaged equipment, that is, equipment that either operated through the earthquake or remained operable once power was restored to the site. A small portion of the inventory of equipment, about 10%, exhibited noticeable effects from the earthquake. Shifting of unanchored equipment along the floor or concrete foundation is the most common example. Another example of nondamaging effects is the actuation of automatic shutoff switches, such as electrical relays, either by

earthquake vibration or by earthquake-induced fluctuations in the electric power supply.

A smaller portion of the equipment inventory experienced some form of damage and was rendered inoperable following the earthquake. That portion of the inventory that consists of damaged equipment offers the most valuable information. The percentage of equipment of a particular type found to be inoperable following an earthquake provides an indication of the tendency of the equipment type to suffer seismic damage. Illustrations of common sources of seismic damage point the way to means for mitigation.

As examples, the sections that follow describe the earthquake performance of three general categories of equipment common to modern industrial and commercial facilities. These equipment categories include:

- Electric power supply equipment
- Uninterruptible power supply equipment
- Rotating mechanical equipment.

Earthquake performance summaries for these three categories provide a basis for earthquake mitigation reviews of equipment installations in conventional facilities. Based on actual earthquake experience, standard industrial-grade equipment has an excellent seismic performance record. There are common sources of relatively rare earthquake damage that can usually be identified in a *walkdown* of facility equipment installations. Other than identifying and correcting these common weaknesses, standard industrial-grade equipment of the types listed above should have no further need for seismic retrofit. Based on past experience, this equipment should be able to withstand the seismic loads corresponding to ground motion acceleration up to about 0.30g.

Electric Power-Supply Equipment

The category of equipment defined as electric power supplies consists of sheet-metal-enclosed switching assemblies and transformers carrying AC power from 120 volts up to 15 kilovolts. It includes ventilated and enclosed transformers, low- and medium-voltage switchgear, motor-control centers (MCC), and distribution switchboards or panelboards. This equipment may be housed separately, but it is often bolted together as a single structural assembly or unit substation. Examples of typical alternating current (AC) power supply equipment investigated following a recent strong-motion earthquake are illustrated in Fig. 9d-1.

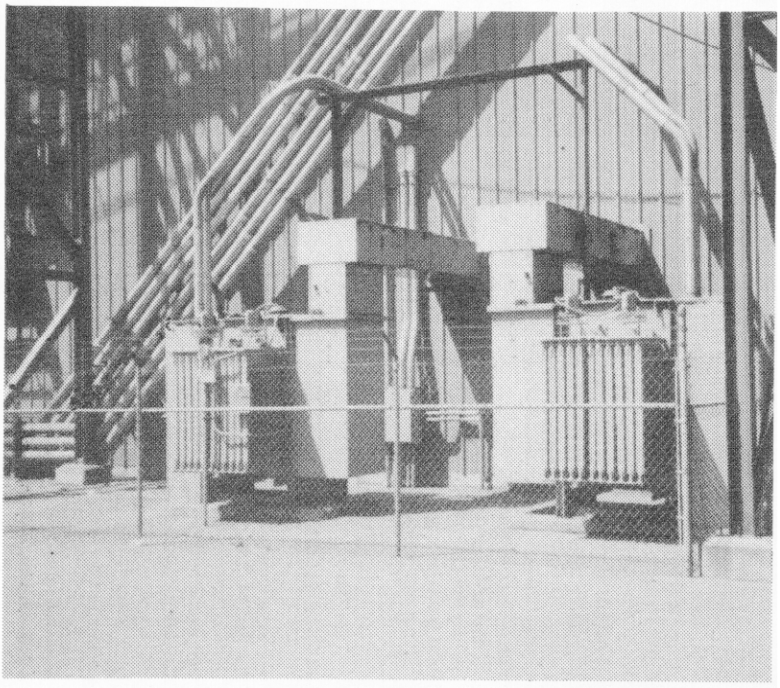
Database Inventory

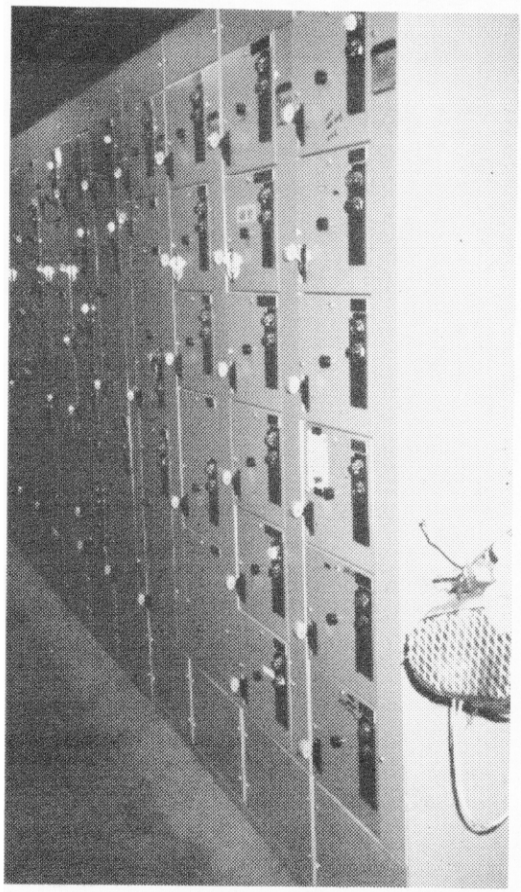
Earthquake investigators have compiled an inventory of around four-hundred examples of motor-control centers, switchgear assemblies, and distribution switchboards, and about two-hundred examples of transformers of various sizes. Fig. 9d-2 illustrates this database inventory in a bar chart.

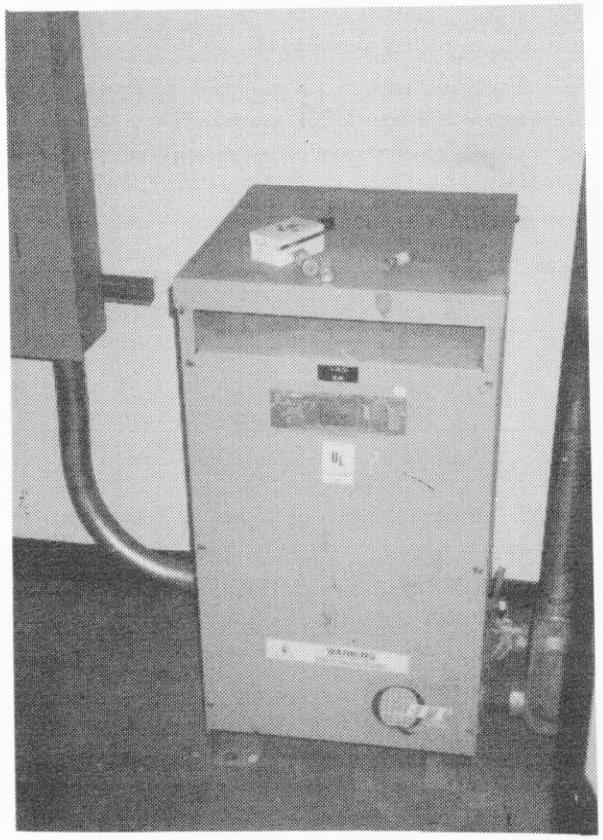
It shows the number of examples of power supply equipment found at various sites investigated. Each bar represents a particular site, arranged according to the peak ground acceleration experienced based on the nearest strong-motion recordings. The postearthquake condition of equipment included in each vertical bar is denoted by the texture of shading in elements that make up the bar. About six-hundred examples of power supply equipment have been compiled from some forty sites, representing sixteen earthquakes. These sites were subjected to peak ground accelerations ranging from about 0.20 to 0.60g.

Typical Earthquake Damage

As illustrated in Fig. 9d-2, there are many instances of nondamaging earthquake effects. These typically consist of shifting of unanchored or underanchored power supply equipment or tripping of circuit breakers when relays are actuated. The bar chart also illustrates thirty-three instances of power supply equipment that were inoperable following the earthquake, or for which postearthquake operability could not be verified. These include equipment that was disassembled following the earthquake for inspection or repair prior to repowering. In these cases, because operability immediately after the earthquake could not be confirmed, the equipment was assumed to have been







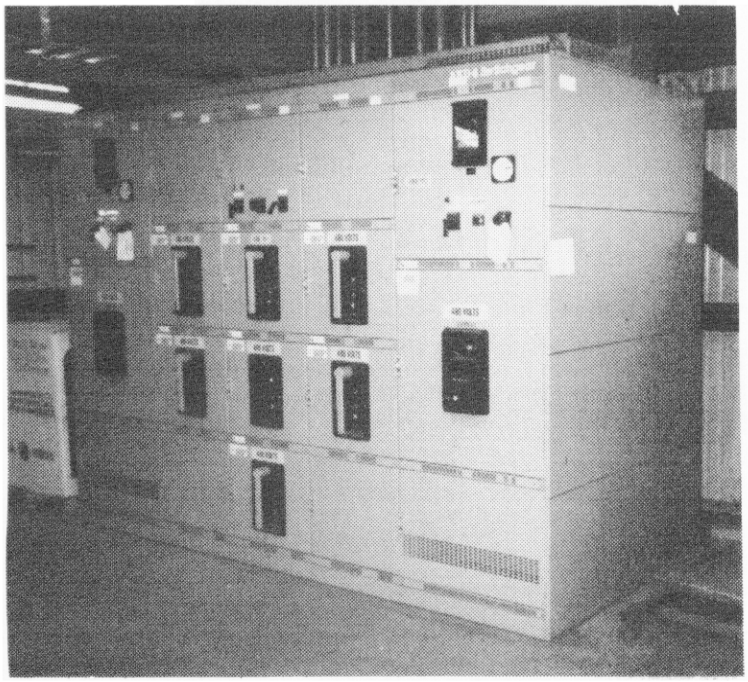


Fig. 9d-1. Examples of recent vintage-power supply equipment at a power plant in Scotia, California, which experienced about 0.50g peak ground acceleration in the Cape Mendicino earthquake of April 1992. Power enters the plant at 12 kilovolts and is stepped down through transformers (upper left) to 2400 and 480 volts. Motor-control centers (upper right) and low-voltage switchgear (lower right) distribute 480-volt power to individual circuits. Lighting and small motors are served by stepping power down to 240 or 120 volts through a distribution transformer (lower left).

Fig. 9d- 2. The database includes about 600 examples of power-supply equipment, representing about 40 sites and 16 different earthquakes. Each bar represents the inventory of equipment at a site. Undamaged equipment are denoted by solid squares, equipment showning noticeable effects, but still operable, are denoted by light texture. Heavy texture denotes equipment damaged and inoperable following the earthquake.

inoperable prior to disassembly. These instances of damage involve eight sites. A general summary can be made of the more significant instances of damage.

About half of the instances of damaged power supply equipment occurred at an industrial site located in the epicentral area of the 1985 magnitude 8.1 earthquake in Mexico. Twelve motor control centers and one low-voltage switchgear assembly were damaged in the second-floor switchgear room of a two-story concrete-frame building (Fig. 9d-3). Anchorage failed in a portion of the electrical cabinets, resulting in overturning. Some cabinets that remained standing experienced yielding in their structural framing, with subsequent cracking in the rigid plastic insulation of internal components. Although the majority of components within the equipment survived the earthquake intact, all cabinets were removed from the building while extensive repairs were made to the concrete frame. Some months later, the motor control centers and switchgear were reinstalled and repowered. To ensure conservatism, all of the equipment in the building was assumed to have been damaged.

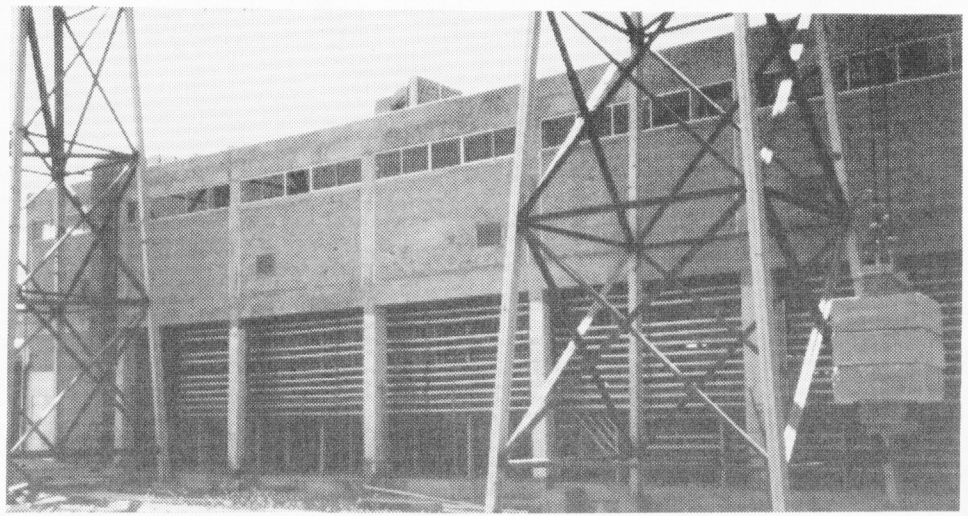
Another example of anchorage failure resulting in loss of function occurred at an oil-pumping plant near the epicenter of the 1983 magnitude 6.7 earthquake near Coalinga, California. A 4160-volt switchgear assembly sheared its anchor bolts, shifted violently across the concrete slab, and impacted a cable-penetration flange protruding from the floor (Fig. 9d-4). The impact loads on the cabinet distorted the steel framing. Circuit breakers were removed to allow the cabinet framing to be straightened. Upon replacing the breakers, the switchgear assembly was repowered and restored to operation. It is unknown if the switchgear would have been operable without first removing the circuit breakers and straightening the cabinet framing. For conservatism, the switchgear assembly was counted as a damaged item.

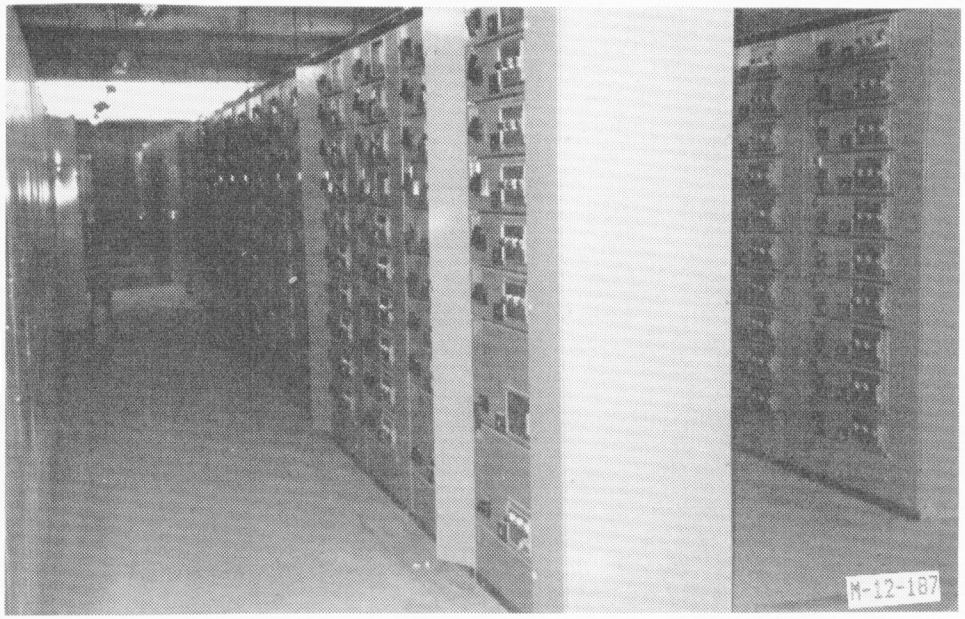
At the same site a unit substation transformer was damaged following anchorage failure and sliding. As the transformer shifted on its concrete pad, it fractured a short section of conduit attached to its base. The conduit fracture apparently frayed insulation on the cable inside, resulting in a short circuit or ground fault within the transformer.

At two sites, problems with power supply equipment were caused by water spray from fractured overhead piping. In the 1989 Loma Prieta, California earthquake, water spray from damaged piping resulted in loss of function in two unit substations serving an electronics manufacturing plant near the town of Watsonville. Swaying of a ceiling-mounted fire sprinkler line caused a sprinkler head to snap off upon impact with a ceiling beam. The resulting water spray flooded two unit substations located beneath the fractured sprinkler. Each unit substation included two transformers attached to a switchgear assembly (six items of equipment total). Because power was lost to the site at the time of the earthquake, the unit substations were not seriously damaged. Several days were required, however, to disassemble, dry out, and reassemble the cabinets.

Similar problems with water spray occurred in the 1971 San Fernando, California earthquake north of Los Angeles. Motor control centers serving heating, ventilating, and air conditioning (HVAC) equipment on the upper floors of the Sylmar direct current-to-alternating current (DC-to-AC) conversion station were sprayed by fractured water lines attached to air handlers that fell from their isolation mounts. During the months while the station's switchyard was being rebuilt, electricians had ample time to disassemble and inspect the motor control centers (MCCs) prior to reasssembly and repowering. It is unknown whether the MCCs would have been able to supply power in spite of the water spray, had power been available immediately after the earthquake.

Additional instances of inoperable power supply equipment include one case of jammed 480-volt circuit breakers, and one case of 2400-volt circuit breakers losing electrical connection to bus bars, apparently as a result of shifting within the switchgear assembly. The remaining instances of damage are electrical burnouts that appear to be caused by defects or installation errors in the original equipment.





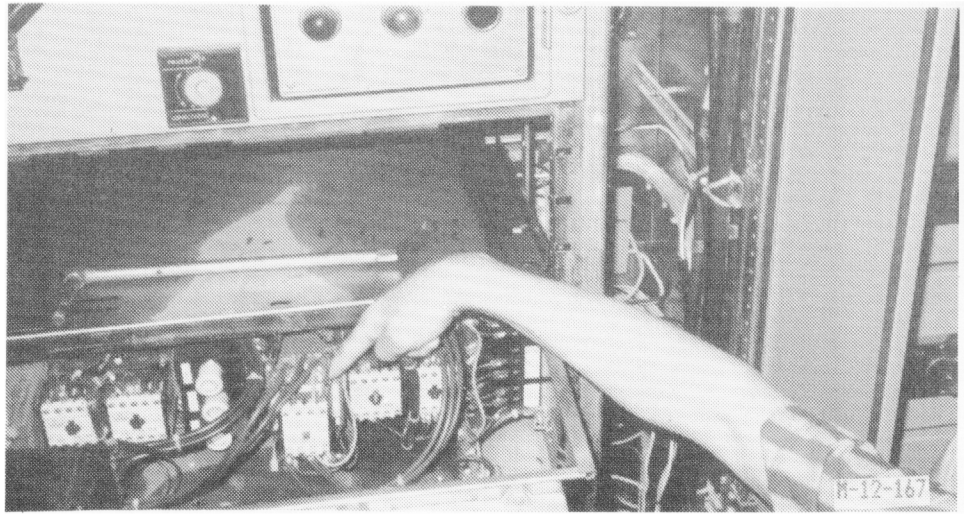
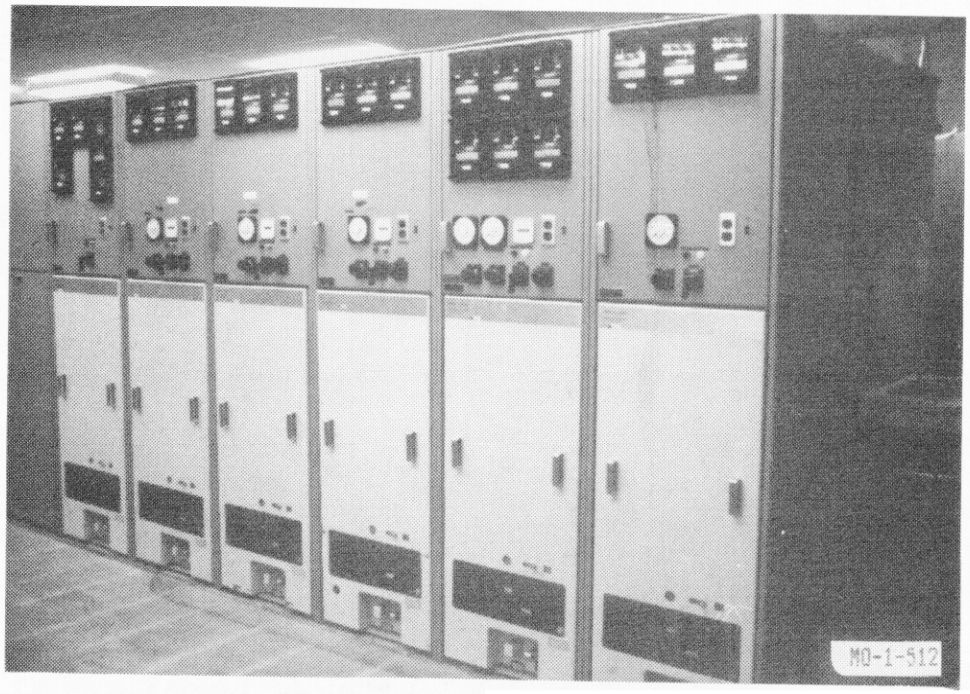
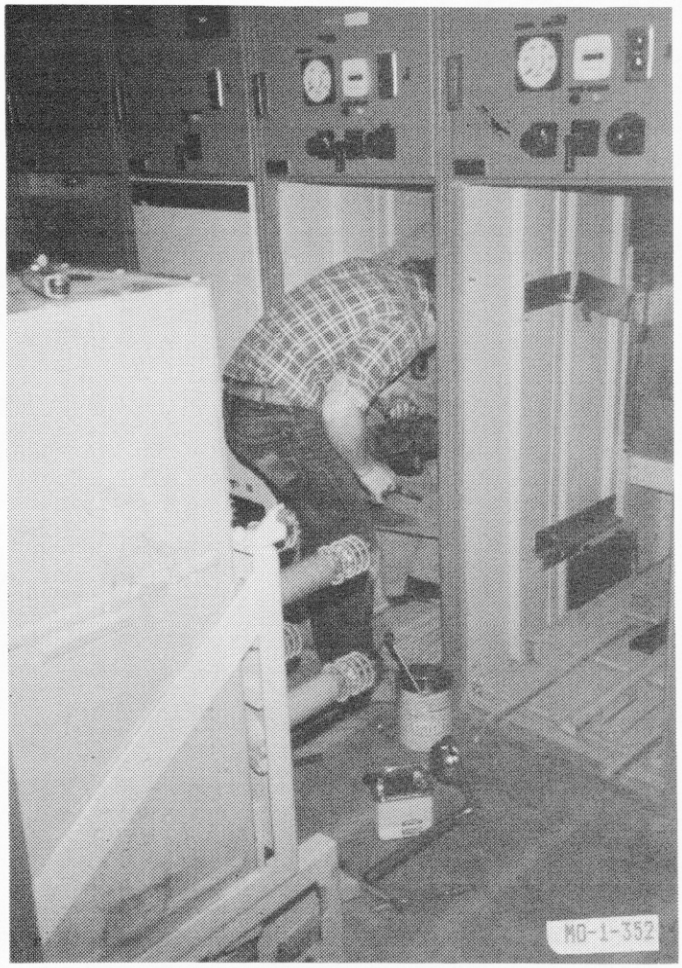


Fig. 9d-3. About half of the instances of damage to power supply equipment derive from one site in the epicentral area of the 1985 earthquake in Mexico. The upper photo shows the two story concrete frame building with masonry in-fill walls. The building houses motor control centers and low voltage switchgear on the second floor (right photo). A combination of anchorage failure and yielding in cabinet framing lead to cracking in brittle insulating material in cabinet components (lower left photo). The building itself was extensively damage and required several months for repair.





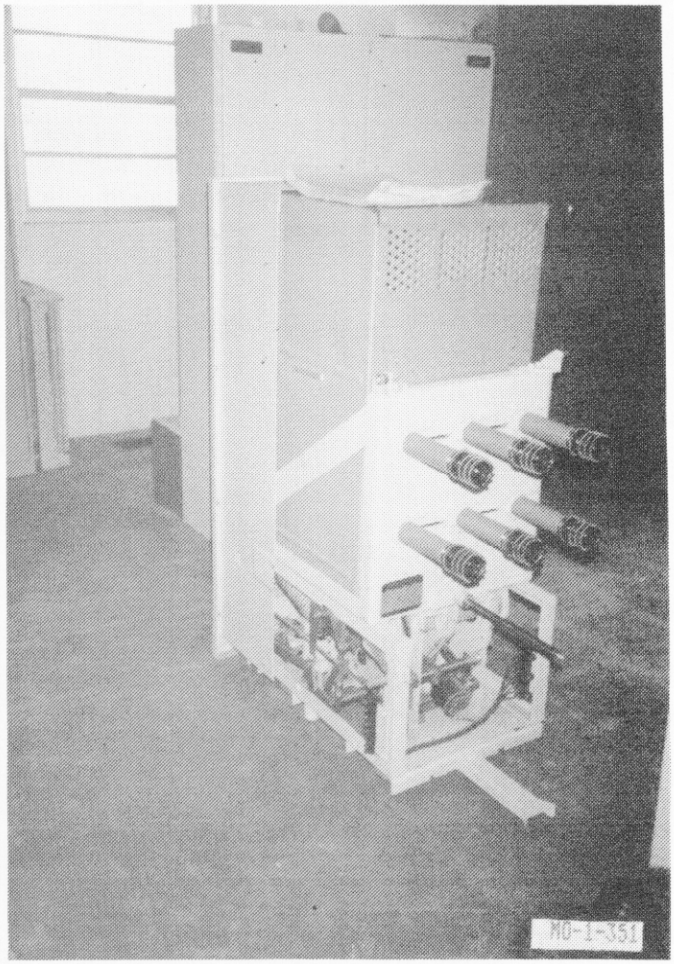


Fig. 9d-4. At an oil pumping plant near the epicenter of the 1983 Coalinga earthquake a 4160 volt switchgear assembly (upper photo) sheared its anchor bolts and slid across the concrete floor of the control building. The base of the assembly impacted flanges protruding from the floor, bending the cabinet framing. Technicians removed the circuit breakers (lower left) and restraightened the framing (lower photo) prior to reassembling and repowering the switchgear.

The more serious instances of damage included in the database indicate that the primary hazards to electrical power supply equipment are anchorage failure, yielding in sheet metal cabinets, and water spray from damaged overhead piping.

Uninterruptible Power Supply Equipment

Uninterruptible power supplies (UPS) are back-up systems that ensure a steady source of AC power to critical circuits in the event of loss of normal AC supply from the off-site grid. The primary components of a UPS system are a rectifier (battery charger), a battery rack, a DC-to-AC inverter, and an automatic transfer switch. The rectifier converts incoming off-site AC power to DC power. The DC output from the rectifier maintains charge in the battery rack. The DC supply is also fed through the inverter and converted back to AC to serve critical circuits such as data-processing or control systems. When the normal off-site AC supply is interrupted, as in an earthquake, an automatic transfer switch senses the loss of power and changes the source of DC supply, feeding the inverter from the rectifier to the battery rack.

A UPS system serving a data-processing (DP) center, which was investigated following an earthquake in 1992, is shown in Fig. 9d-5.

Database Inventory

Earthquake investigators have compiled an inventory of about one-hundred examples of rectifiers and inverters, and about one-hundred-twenty examples of battery racks, representing some forty sites and sixteen earthquakes.

The inventory of rectifiers, inverters, and battery racks is illustrated by the bar chart in Fig. 9d-6.

Typical Earthquake Damage

The bar charts in Fig. 9d-6 include two instances of damage to rectifiers/inverters within the database and nine instances of damage to battery racks. Most rectifiers within the database lost AC power as a result of loss of off-site power, but reenergized when power was restored. This damage is briefly summarized below.

Two DP centers experienced damage to inverters serving UPS systems during the 1987 magnitude 5.9 earthquake near Whittier, California. In both cases, current surges, either during or after the intial shock, burned out internal elements of the inverters. In one instance, surges were arrested by normal fuse protection; in the other instance, capacitors were damaged by the current surge. Current surges appeared to result from an internal ground fault in one inverter, caused by a stray scrap of metal; in the other instance, surge appeared to originate from the off-site power grid.

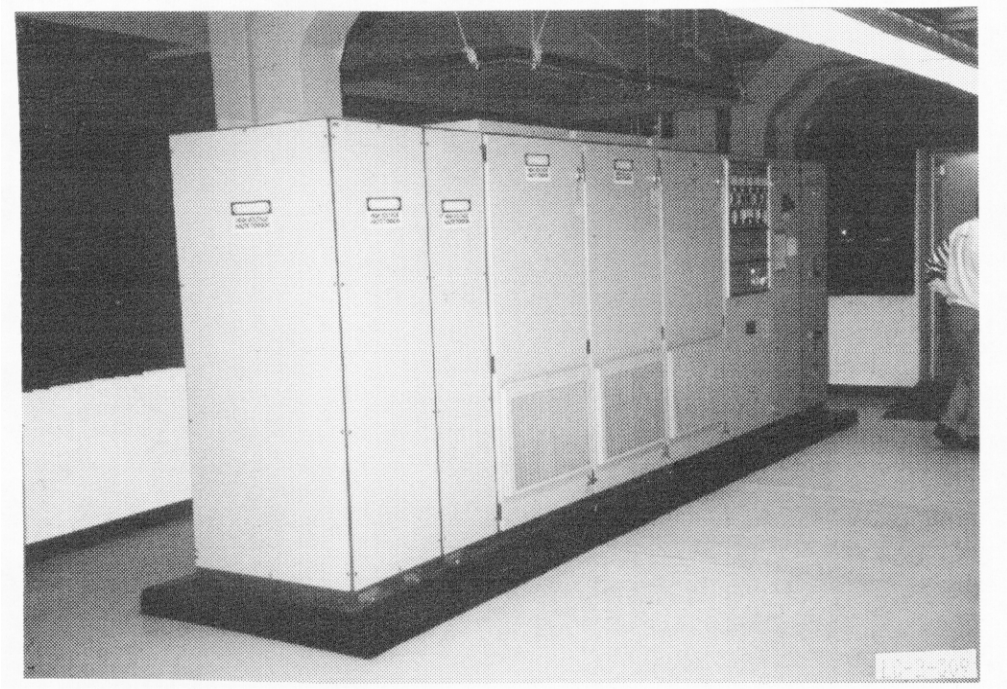
There are nine instances of damage to battery racks within the database. At two sites, batteries toppled from their rack because a wrap-around restraining enclosure was lacking (Fig. 9d-7).

Damage to batteries occurred at telephone switching stations in the epicentral area of the 1990 magnitude 7.5 earthquake in the Philippines. In one instance, a two-tier rack was pulled from its floor anchorage and overturned. In the other instance, several batteries were found to have dislodged internal plates following the earthquake.

A similar instance of internal damage to batteries occurred at a substation in the 1987 magnitude 6.3 earthquake in New Zealand. Several batteries mounted on a two-tier rack suffered dislodged internal plates apparently caused by pounding between adjacent batteries (Fig. 9d-7).

Four battery racks were damaged in one location because of the impact from fallen masonry during the 1985 earthquake in Mexico.

The very limited number of examples of damage to UPS equipment implies that earthquake-induced electrical fluctuations are a credible hazard to inverters. Damage to battery racks is caused by structural weaknesses. Battery racks need to be well anchored. Batteries need to be closely enclosed by wrap-around restraints, with padding between cells to soften interbattery pounding.



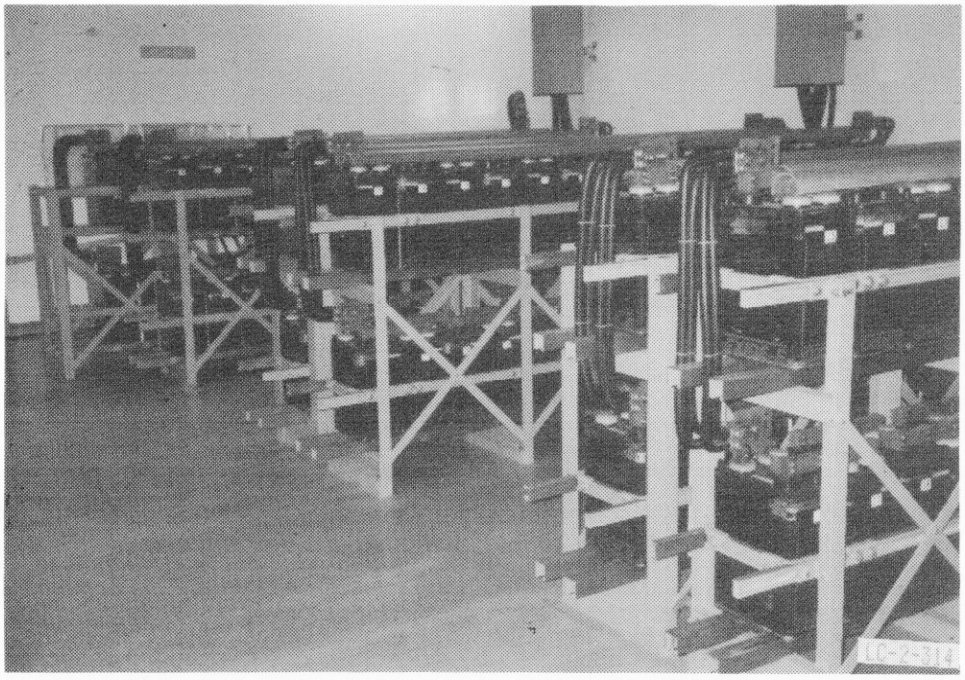


Fig. 9d-5. A recent vintage uninterruptible power supply system at a data-processing center near Ferndale, California, experienced about 0.45g acceleration in the Cape Mendicino earthquake of April 1992. The upper photo shows the cabinet-enclosed rectifier and inverter assembly. The lower photo shows the two-tier battery racks, including a wrap-around *Unistrut* enclosure to prevent toppling of the battery jars.

exture

quares

arth

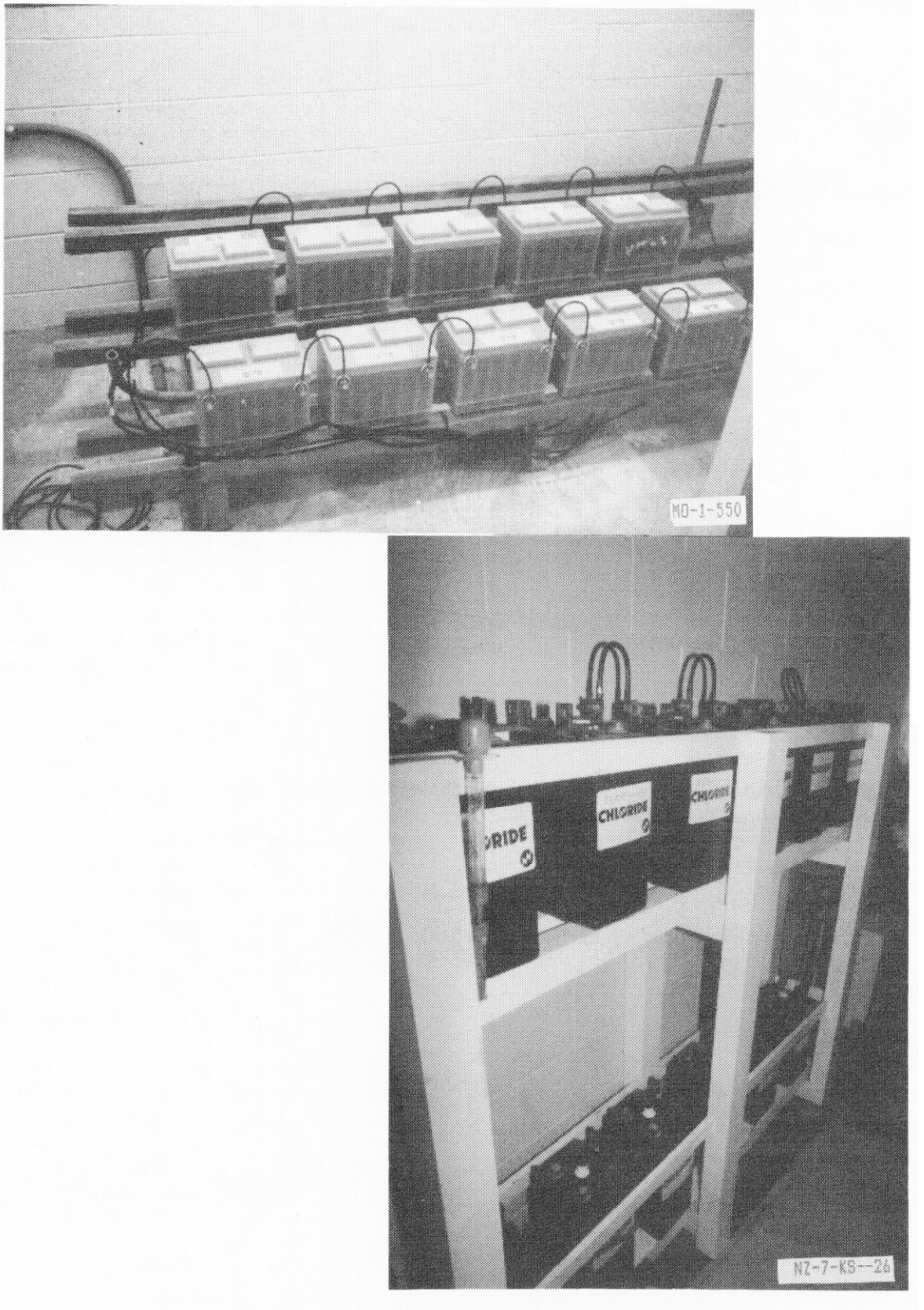


Fig. 9d-7. Batteries that are not provided with restraining enclosures are subject to toppling. The upper photo shows replacement batteries for the set that fell off the rack at an oil pumping plant in the 1983 Coalinga, California earthquake. Batteries not provided with padding between jars are susceptible to pounding damage. The lower photo shows batteries spaced at intervals along a two-tier wooden rack at a substation in New Zealand. Interbattery pounding during the 1987 earthquake dislodged the internal plates in several batteries.

Falling debris is a particular hazard to batteries because of their relatively fragile cell jars and exposed bus bar connections.

Rotating Mechanical Equipment

The rotating machinery equipment category is limited to several of the more common types of motor-driven mechanical equipment found in commercial and industrial facilities. This category includes the more common types of horizontal and vertical pumps; the basic components of central heating, ventilation, and air conditioning (HVAC) systems, i.e., fans and chillers; and gas-turbine or piston-engine-powered emergency generators. This rather diverse mixture of equipment has certain structural similarities and appears to exhibit common modes of seismic failure.

Each of these types of rotating machinery include an electric-drive motor or engine and a rotating impeller, fan, or generator. These two central components are served by attached piping, conduit, and ducting, which provide an often complicated structural link to other equipment and to the enclosing building. Each type of rotating machinery addressed here is often supported on vibration isolation mounts. Examples of typical rotating machinery found at the sites investigated are illustrated in Fig. 9d-8.

Database Inventory

Fig. 9d-9 is a bar chart illustrating the database inventory for rotating mechanical equipment. The bar chart includes database inventories of about 500 examples of pumps, about 200 examples of rotating HVAC equipment, and about 100 examples of emergency generators. These examples cover a wide range of sizes, configurations, and applications in rotating machinery, ranging from fractional horsepower equipment to units of several thousand horsepower.

Typical Earthquake Damage

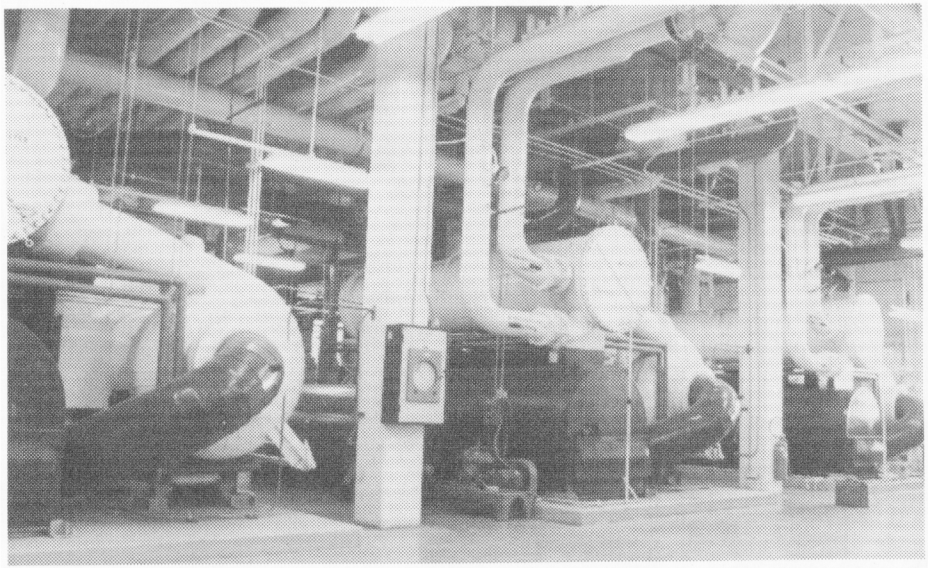
A number of instances of nondamaging earthquake effects are illustrated in the bar chart (Fig. 9d-9). Mostly,these often consist of dismounts from spring vibration isolators, where the equipment nevertheless remained operable (albeit noisy) in its dismounted condition. Other examples of nonfatal effects include offsets and separations in attached ducting, where sufficient air flow was retained for the equipment to continue operating.

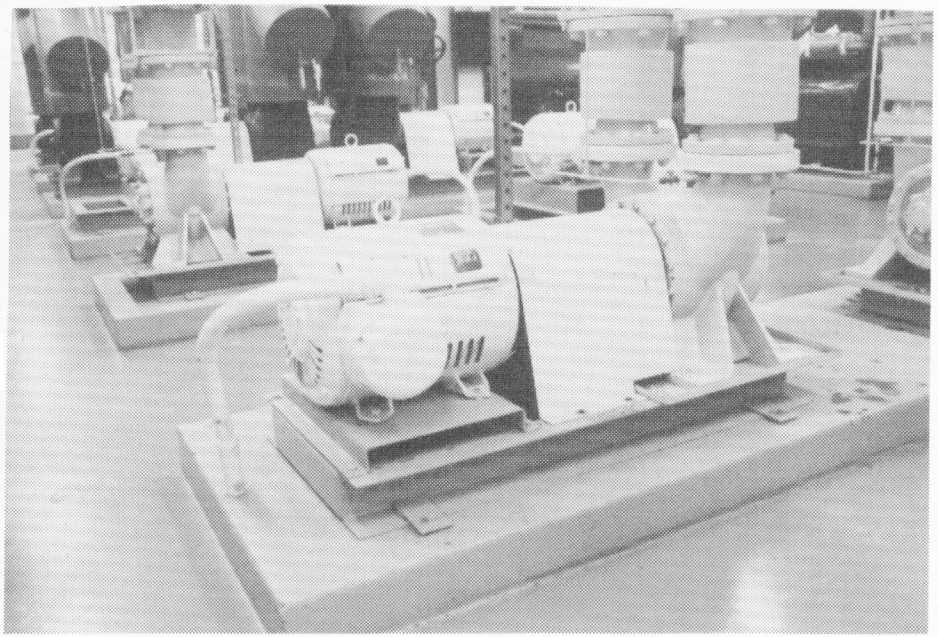
The bar chart includes thirty-four instances of damage where rotating machinery was rendered inoperable following the earthquakes. Brief summaries of the damage are presented below.

Dismounts from spring-vibration isolators are probably the most prevalent and easily prevented source of damage to rotating machinery. At two DP centers near the epicenter of the 1987 Whittier, California earthquake, HVAC chillers were rendered inoperable by spring-isolator dismounts of sufficient displacement to damage attached water piping. At a neighboring DP center, a dismount of an HVAC fan misaligned the motor and impeller sufficiently that the impeller eventually contacted the sheet-metal enclosure following restart.

Two instances of damage to large forced-draft fans providing air flow to boilers occurred at epicentral industrial sites in the 1985 Mexico earthquake (Fig. 9d-10). In both cases, displacement of the large steel plate ducting attached to the fans induced yielding in both the fan enclosures and the impeller-motor drive shafts. The result was excessive misalignment between fan and motor and, in one case, contact of the impeller with the fan enclosure wall.

A similar source of damage to horizontal pumps occurred at an industrial site in Mexico. Severe settlement beneath concrete pads in a water treatment plant imposed vertical offsets in piping attachments to pumps. The anchor-point displacements on the piping created excessive loads at their attachments to the pump impellers, fracturing anchor bolts and motor-impeller drive-shaft couplings. All fourteen pumps in the water treatment plant were affected by the imposed pipe displacements. This single instance of settlement-induced damage at one site accounts for almost half of the instances of damage to rotating machinery illustrated in the bar chart.





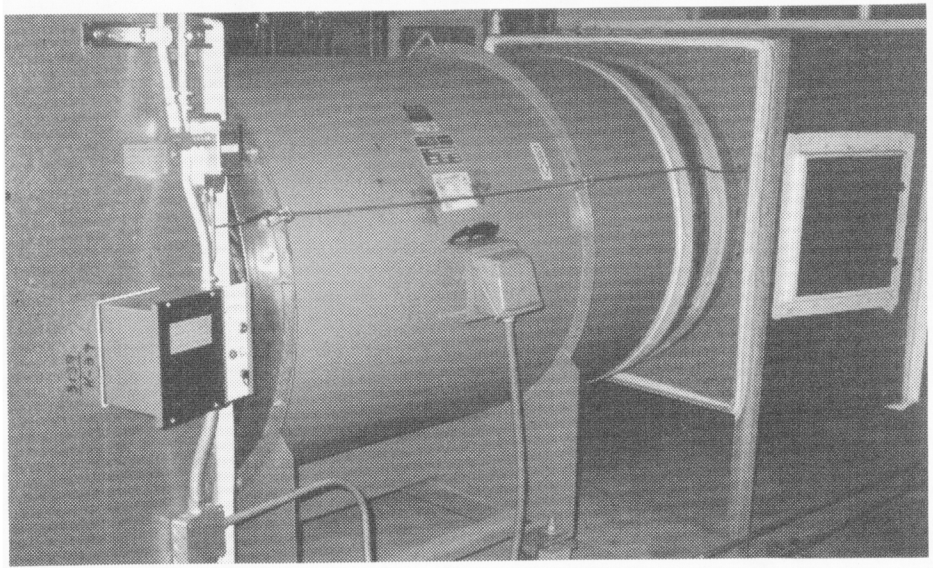


Fig. 9d-8. Much of the rotating mechanical equipment in most commercial facilities is included in HVAC systems. Shown above are centralized water chillers (upper left), chilled water pumps (right), and one of several duct-mounted forced draft fans (lower left) for a large data-processing center near San Jose, California. Strong-motion instruments on the site recorded 0.40g acceleration in the 1984 Morgan Hill earthquake, and 0.25g in the 1989 Loma Prieta earthquake.

Fig. 9d-9. Categories of rotating mechanical equipment within the database include about 500 examples of horizontal and vertical pumps, about 200 examples of chillers and fans, and about 100 examples of emergency power generators.

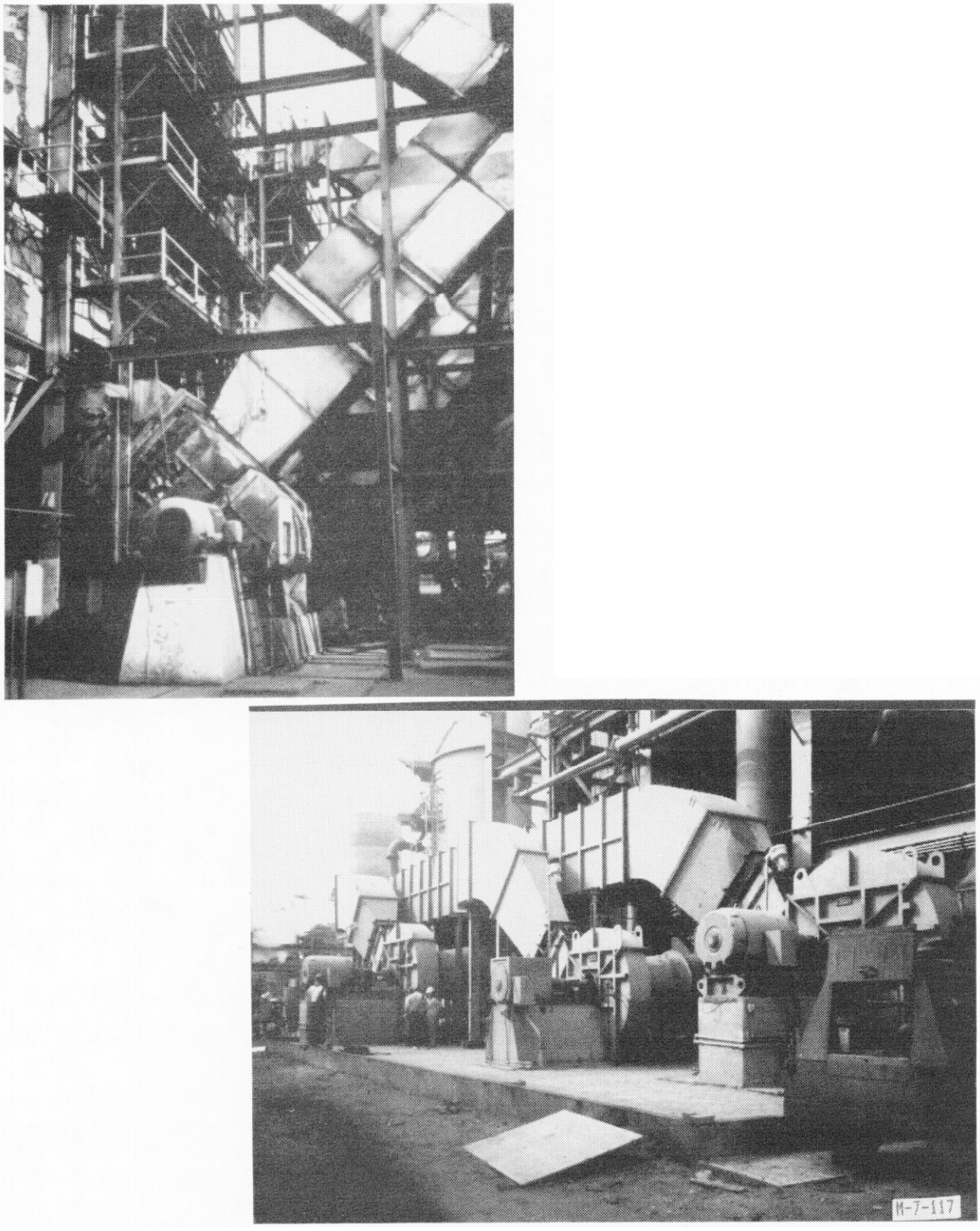


Fig. 9d-10. Neighboring industrial sites in the epicentral region of the 1985 earthquake in Mexico experienced problems with large forced draft fans supplying gas-fired boilers. Severe misalignments occurred in a forced draft boiler fan when shifting of the boiler imposed anchor point displacements on the steel plate ducting attached to the fan (upper left). Similar misalignments occurred in fans at the neighboring site (lower right), where the heavy ducting attempted to follow the response motion of the boiler building.

Anchor-point displacement in piping attachments also caused damage to the HVAC system of a DP center shaken by the 1987 Whittier, California earthquake. In this case, a large chiller dismounted from its spring isolators, shifting several inches. The attached water lines shifted with the chiller in the process pulling them loose from ceiling hangers. This resulted in a combination of pipe displacement, seismic reaction loads, and increased dead load from the piping imposed on the nearby chilled water pump. The excessive pipe loads fractured the pump's impeller casing.

Fractured piping attachments account for most of the instances of engine power generators rendered inoperable following an earthquake. The banana production areas of northern Panama are supplied power from a large diesel generator plant at Changuinola. The 1991 magnitude 7.5 earthquake created severe liquefaction and settlement throughout the Caribbean coastal region. Uneven settlement beneath the diesel power plant imposed vertical offsets between abutting concrete floor slabs (Fig. 9d-11). Piping for fuel and cooling water systems serving the diesel power generators was fractured where it spanned joints between concrete slabs offset by the differential settlement. As a result, seven out of eight diesels within the Changuinola power plant were rendered inoperable because of fractured piping attachments.

One diesel generator in the Changuinola plant was further damaged by an offset of the pillow-block bearing supporting the end of its drive shaft. The pillow block was mounted on the concrete floor slab of the plant, while the diesel generator was structurally isolated on its own concrete pedestal.

Other instances of failure in engine- and gas-turbine power generators were traced to blocked fuel lines and to one instance of apparent overheating during operation. In two instances, generators failed to start because control system components were damaged, apparently as a result of earthquake-induced electrical current surges.

Although there are several instances of miscellaneous causes of damage to rotating machinery, it appears that the most common sources of damage are dismounts from spring isolators and loads imposed from anchor-point displacments on attached piping or ducting.

Conclusions

The performance of the three sample categories indicates that out of about 1,500 examples of equipment subjected to strong motion earthquakes, at most, 5% of the inventory suffered effects that prevented postearthquake operation. This 5% is a conservative estimate. It includes equipment that was operable following disassembly and reassembly, for which the immediate postearthquake operability is suspect.

Instances of damage within the three equipment categories are dominated by common-mode failures at certain sites. About half of the instances of damage to rotating machinery, for example, result from settlement-induced damage to a battery of horizontal pumps in a water treatment plant during the 1985 Mexico earthquake. Similarly, almost half of the instances of damage to power supply equipment was caused by anchorage failure and cabinet yielding in MCCs in a single concrete-frame building at the same industrial site in the 1985 Mexico earthquake. A large portion of the instances counted as failures in UPS equipment resulted from masonry walls collapsing on batteries, again in the epicentral industrial zone in Mexico. Not surprisingly, the magnitude 8.1 Mexico earthquake contributes the largest number of failures of equipment in all categories.

The most common sources of earthquake damage for the three sample categories of equipment are summarized as follows.

Anchor-point displacements imposed on equipment attachments (piping, ducting, conduit) appear to be the most common source of seismic damage. Anchor-point displacements account for about one-third of the instances of equipment damage described previously. The majority of these anchor-point displacements are settlement-induced effects on piping attachments to ground-level equipment.

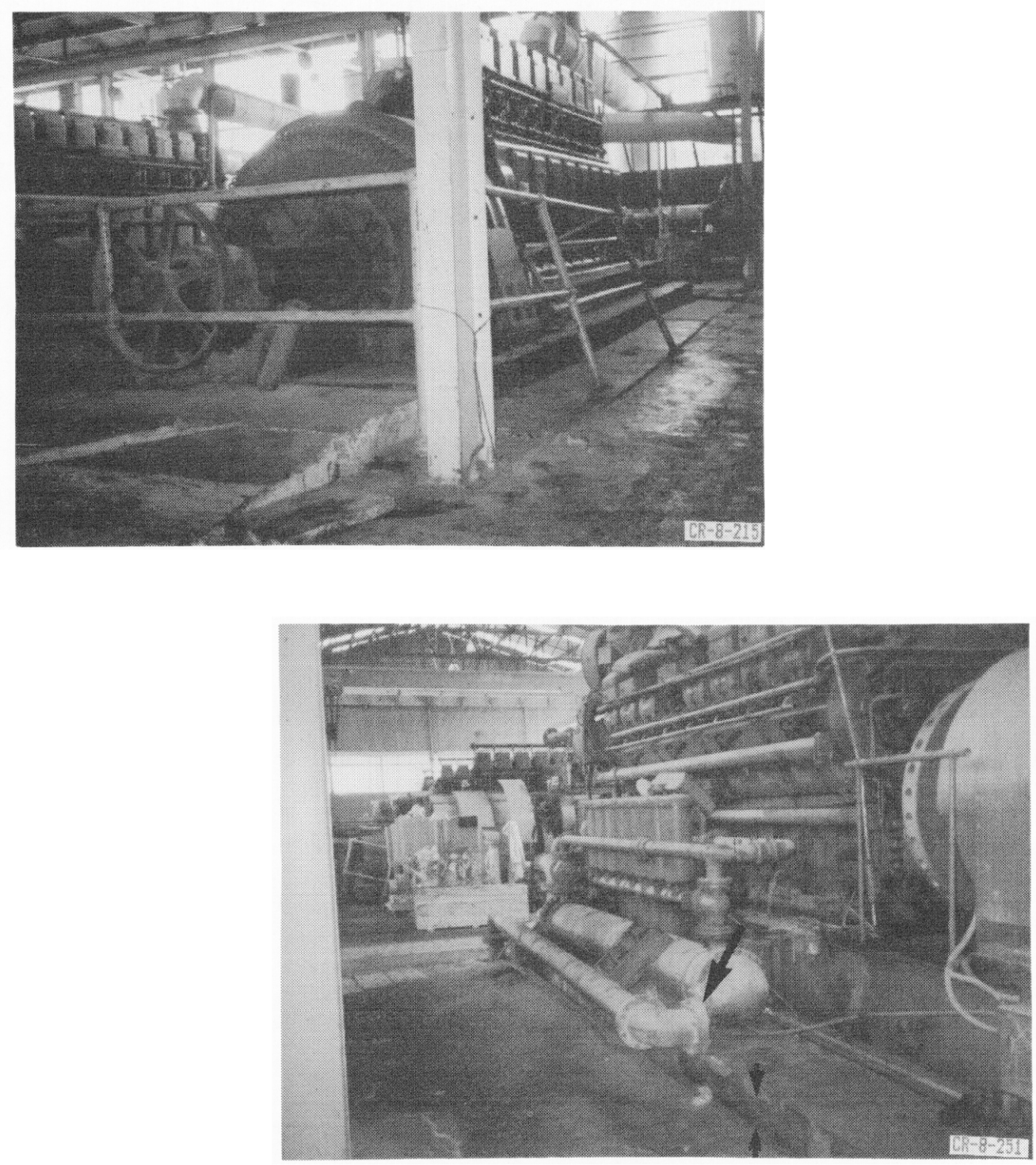


Fig. 9d-11. Most of the diesel generators in the Changuinola, Panama power plant were rendered inoperable due to uneven settlement of concrete slabs (upper photo). Piping attachments that spanned between pedestals supporting the diesels and the surrounding mats fractured due to abrupt vertical offsets (arrows, lower photo).

Damage caused by dismounts from spring vibration isolators accounts for several instances of damage to rotating mechanical equipment. It is apparent that isolation mounts not designed to restrain earthquake displacements are a serious deficiency. However, far more instances are seen in which the equipment dismounted but did not suffer enough damage to be inoperable.

Failure of floor anchorage and subsequent equipment overturning accounted for five instances of failure in power supply equipment and for one battery rack failure. The majority of equipment that stretches or fractures its floor anchorage has been observed to shift without overturning, and thereby remains intact and operable. Unfortunately, shifting without overturning cannot be ensured.

Damage caused by the failure of adjacent fixtures, commonly called seismic-interaction or collateral damage, accounts for more than a dozen instances of equipment failure. These include the four battery racks damaged by falling masonry walls in Mexico and two unit substations and eight motor control centers located beneath fractured, pressurized water piping. Damage from water spray, like damage from ground settlement, is another example of a common-mode failure, causing damage to multiple equipment items at one location.

Finally, there are several instances of damage to equipment internals caused by apparent electrical current surges. This type of seismic damage is the most difficult to trace, and perhaps the most difficult to mitigate. Even if normal surge-protection equipment such as fuses or circuit breakers actuates to protect internals from damage, this will preclude operation of the equipment until the circuit disconnection (e.g., a blown fuse or tripped circuit breaker) is located and corrected.

The more common causes of earthquake damage to equipment described in this chapter form the starting point for implementing practical earthquake damage-mitigation measures. A few suggestions are presented below, both for evaluation of existing equipment installations and for procurement and installation of new equipment.

A conservative failure rate of about 5% indicates that standard industrial-grade equipment does well even in strong-motion earthquakes. Except for equipment that serves a very critical function, no specific provisions for seismic loads should be required other than mitigation of common sources of seismic damage described above. For the rare equipment installation that is absolutely critical, specific seismic qualification by shake-table testing might be justified. It must be remembered, however, that shake-table testing does not simulate all of the sources of seismic damage (e.g., seismic interaction, anchor-point displacement). Attention to the additional suggestions below is also recommended.

Provide Flexibility in Equipment Attachments

Anchor-point displacement can be described as damage to equipment imposed by relative movement of structures or adjacent equipment to which there are relatively rigid attachments. Attachments to equipment typically include piping, conduit, electrical bus ducts, ventilation ducts, or structural bracing. If the equipment has attachments to the structure or to other equipment that could respond to the earthquake with motion in different directions or amplitudes, then the equipment may be forced to act as an inadvertent structural brace.

Provision of *flexibility* in equipment attachments is essential to accommodate differential displacements. Differential displacement between adjacent equipment, or between the equipment and the enclosing structure, seldom exceeds a few inches. Experienced engineers can make judgments (or simple calculations) to determine whether existing equipment attachments can absorb such anchor-point displacements without failure. If there seems to be insufficient flexibility, provision of flexible couplings (e.g., bellows) can be used to solve the problem.

RestrainVibration-Isolated Equipment

Vibration isolators are a common means of mounting rotating machinery to prevent induced operational vibrations into buildings. Vibration isolators may form part of the equipment floor anchorage, or they may support only the rotating component of a larger

equipment assembly and therefore be contained within the equipment enclosure. Vibration isolators also are used to damp out noise from nonrotating equipment such as power transformers. A vibration-isolation system should always be checked for its ability to restrain earthquake-induced dismounts.

Some vibration mounts are housed within a casing designed to restrict lateral displacement and to restrain the inertial loads of the equipment in an earthquake. A quick inspection of the isolator design, perhaps combined with simple calculations, should determine whether a vibration isolator has sufficient strength to prevent an equipment dismount. Assuming a horizontal acceleration of 1.0g through the center of mass of the spring-mounted equipment allows reasonable estimates of the amount of lateral-load-restraining force required of the vibration isolators.

If vibration isolators are not specifically designed to handle seismic loads, retrofit is recommended. Typically this means providing bumpers exterior to the vibration-isolated equipment. Bumpers do not actually contact the equipment. A small gap is provided to allow limited freedom of movement in normal operation and restraint of lateral loads during an earthquake.

Provide Adequate Anchorage

Floor anchorage of equipment is the most obvious provision to prevent overturning or shifting of equipment and subsequent failure of attachments. Where specific seismic criteria do not exist, providing anchorage to withstand 0.50g lateral load through the equipment center of mass is normally sufficient.

Certain installations, such as DP equipment, seldom include floor anchorage, given the need to keep equipment mobile. If floor anchorage of equipment severely impedes normal operations, it may be neglected if the equipment is not critical to operations, or if a low center of mass makes overturning improbable and sufficient slack is provided in attachments to accommodate shifting. See Chapter 9c for a detailed discussion of data processing systems.

Prevent Interaction with Adjacent Fixtures

Checking for hazards caused by the failure of adjacent fixtures is one of the more difficult aspects of practical seismic mitigation. Most often, the focus should be on adjacent or overhead water lines, particularly fire-sprinkler piping suspended from ceilings. Electrical equipment in particular should be checked for nearby weak points in water piping systems as potential sources of water spray.

Impact hazards for sprinkler heads or stress concentration points in branch-line connections are the most common locations where fire-sprinkler piping will fail in earthquakes. Sprinkler piping is typically supported on flexible rod hangers. Allowance should be made for several inches of sway between sprinkler heads and adjacent impact points such as ceiling beams. Where long spans of rod-hung piping are tied together with a short connecting line, a potential failure point exists. The connecting line must accommodate the differential seismic response of the two long spans. Replacing the short rigid connecting line with a flexible connection will allow for differential displacement without failure.

Sprinkler systems have been known to inadvertently actuate during earthquakes, possibly because of dislodged dust in suspended ceilings triggering smoke detectors controlling deluge valves. It is difficult to mitigate the hazard presented by earthquake actuation of a fire system without interfering with fire-safety requirements. Ensuring easy access to a manual shutoff valve is probably the best solution.

Provide Electrical Surge Protection

Outages or fluctuations in the electric power supply to equipment are likely in earthquakes. Hazards from equipment power supplies are perhaps most difficult to mitigate, because they depend on details of circuit protection within the facility at large.

The loss of the normal supply of off-site power is very likely in a strong-motion earthquake. If back-up power is available to the equipment, such as a UPS, there still may be a loss caused by actuation of circuit

protection such as fuses or circuit breakers. There are instances in earthquakes of apparent surges in the off-site power grid (prior to outage) that tripped circuit-protection devices.

Earthquake-induced fluctuations in electrical current are caused by a variety of sources, such as sway of power lines external to the facility. *Single phasing*, or the loss of one circuit out of three phases, is also not uncommon.

Electrical engineers serving the facility can best determine whether the power supply to equipment is relatively *earthquake proof*. Protective relays can be provided in the centralized switchgear to detect a variety of fluctuations in the electrical supply. By concentrating the most sophisticated and sensitive circuit-protection devices in a few locations, such as central switchgear, it should be easy to locate and reset a tripped relay or circuit breaker. This negates having to locate tripped circuit protection, such as blown fuses, further out in the electrical system where they are more difficult to detect.

Summary

With the recommendations outlined above, facility engineers should be able to perform effective *walkdowns* of primary equipment installations and detect the major sources of earthquake vulnerability. Because of the excellent performance record of standard industrial grade equipment, specific design for seismic loads should not be a requirement for conventional operations other than provision of adequate anchorage and flexibility of attachments. Based on past earthquake experience, the probability of failure should be on the order of 1% for equipment installations that have been reviewed for common sources of seismic damage. This observation should be considered applicable for peak ground accelerations up to about 0.30g.

Of special concern are emergency power generators which serve *essential* facilities such as hospitals, fire and police stations, emergency operating centers, and other functions where continued operation is critical for the safety of occupants and the public. During the 1989 Loma Prieta and 1994 Northridge, California earthquakes there were numerous instances where emergency power generators failed at *essential* facilities. It is imperative that UPS systems for such facilities are carefully reviewed for potential seismic hazards and, if needed, retrofitted to function during and immediately after damaging earthquakes. The guidance incorporated in this chapter should provide a practical basis for implementing a seismic safety review of emergency UPS systems. For more detailed information about the review and retrofit of equipment installations in general, see the publication *Workshop on Walkdown Procedures to Mitigate Natural Phenomena Hazards (NPH)* from the third annual U.S. Department of Energy NPH Mitigation Conference, St. Louis, Missouri, October 1991.

Chapter

10

Seismic Design of Concrete Shielding Block Assemblies

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

Massive concrete shielding blocks pose a very special problem in earthquake safety. In 1975, full-sized shielding blocks were subjected to realistic earthquake motions for the first time utilizing the University of California's 90-ton shaking table at the Earthquake Engineering Research Center in Richmond, California. Until then, only theoretical studies were available for reference, and these primarily concerned rocking action and overturning. In instances in which earthquakes were actually taken into consideration, design was usually predicated on the aspect ratio (height-to-width) of blocks. The idea was to prevent toppling. Little consideration was given to sliding; thus, the proximity of heavy shielding blocks to building columns, experimental equipment, or habitable shelters was usually ignored. Experiments on the shaking table clearly demonstrated the sliding hazard. After static friction between the block and the concrete surface of the shaking table was overcome, the table moved almost freely beneath the block. Imagine what would happen if a huge stack of massive concrete shielding blocks impacted a vital building column, located in close proximity, with an acceleration of 0.7g. Nominal anchor bolts in the column base plate would provide little shear resistance against this enormous force.

The simplest and most economical way of avoiding this hazard is to locate shielding blocks far enough away from building columns and occupied areas so that contact is not possible during an earthquake. If a building column must penetrate the shielding stack, the problem is much more difficult. One possibility is to fill the space between the shielding and the column with nonrigid, frangible shielding materials that minimize the impact on the column in the event of differential motion. This expedient is not recommended as a permanent solution, but it could be used to temporarily reduce the risk in an existing installation.

A permanent solution involves anchoring the shielding blocks against movement and designing the stack of blocks to incorporate a predictable lateral-force-resisting system. The important thing to recognize in this approach is that almost all of the inertial energy induced by earthquake shaking must be resisted directly by the lateral-force-resisting-system. Buildings generally have enumerable structural members and joints that, along with nonstructural architectural appendages, flex and deform both elastically and inelastically to absorb large amounts of dynamic energy and thus reduce the effective forces on the lateral-force-resisting system. Unlike a building, little energy is absorbed in deforming very rigid blocks, and

little molecular damping occurs within individual blocks. The redundancy available in most building framing systems is not available in the individual block. Consequently, for shielding blocks, the lateral-force-restraining system should be designed for a static lateral force coefficient close to the effective peak ground acceleration. If the stack is quite high with respect to its depth, consideration should be given to amplification of the induced base motion with height.

It is important that designers analyze shielding restraints as a complete lateral-force-resisting system, including the interaction among the ground, the foundation, and the shielding. A set of calculations covering the complete system, including a narrative description of how the system is expected to function, should be made. The best way to ensure this happens is to require a design review by independent experienced earthquake engineers. There is small likelihood that the persons responsible for designing the experimental apparatus per se can give realistic consideration to seismic restraints; therefore, it is vital that responsibility for the analysis and design is delegated to a structural engineer.

Auxiliary support equipment, important experimental apparata, and sensitive research instrumentation may be attached to shielding assemblies. Often, state-of-the-art experimental apparata are designed and fabricated with no consideration for forces induced by earthquakes. When it is necessary to protect the internals, it is usually feasible to design the equipment base to dissipate some or most of the seismic motion at the base so that the sensitive apparata are not subjected to the full ground motion. This can be accomplished by providing seismic isolation or shock absorbers between the shielding and the equipment so that the apparata experience less acceleration than the shielding. It is essential, however, that there is adequate horizontal clearance between the isolated equipment and the shielding to handle the differential movement without pounding.

The chapter that follows deals primarily with the restraint of shielding block assemblies. The methods described have been utilized in the field without the benefit of a full-scale test by the *master inspector*—the real earthquake. The 1989 Loma Prieta, California earthquake produced only 0.12g at the Lawrence Berkeley Laboratory and .26g at the Stanford Linear Accelerator Center in Palo Alto, California. However, emphasis is placed on design rather than analysis with the objective that possible failure be limited to inelastic behavior of the lateral-force-resisting system rather than collapse, even if the intensity of the earthquake greatly exceeds the design motion.

Chapter

10a

Planning and Design Methodologies for Blocks

Donald G. Eagling
John J. Earle
Daniel Shapiro

Introduction

When operating particle accelerators, it is necessary to shield personnel as well as the surrounding environment from ionizing radiation. For this purpose, concrete blocks are often used as modular shielding elements. Concrete blocks are massive and may slide, rock, or topple in earthquakes. They must be restrained to provide a safe seismic environment for people and prevent damage to adjacent structures and experimental equipment. An official code for designing concrete shielding block assemblies does not exist at present.

Little is known from actual earthquakes about how massive items similar to shielding blocks behave, particularly when restrained. However, experiments with shielding blocks on the 90-ton shaking table at the University of California Berkeley's Earthquake Engineering Research Center have been carried out to investigate both rocking and sliding motions. This research has been documented in two references; *Sliding Response of Rigid Bodies to Earthquake Motions* (Ref. 1) *and Rocking and Overturning Response of Rigid Bodies to Earthquake Motion* (Ref. 2).

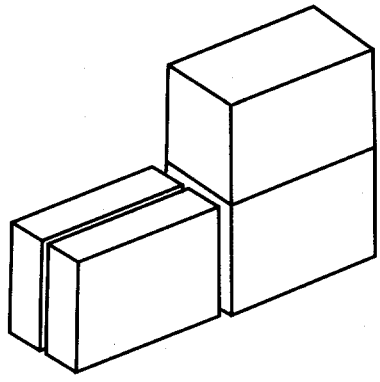
This chapter provides recommendations for the design of shielding blocks for earthquake resistance based on the referenced research and design applications at the Lawrence Berkeley Laboratory. These recommendations are discussed under the following headings:

- Earthquake effects on shielding blocks
- Basic configurations of shielding block assemblies
- Building code considerations
- Seismic design criteria for shielding blocks
- Applications to individual blocks
- Anchorage design
- Prevention of dislocation and overturning
- Design of shielding walls and tunnels for earthquakes
- Design of shielding caves for earthquakes
- Summary of recommended earthquake design for shielding blocks

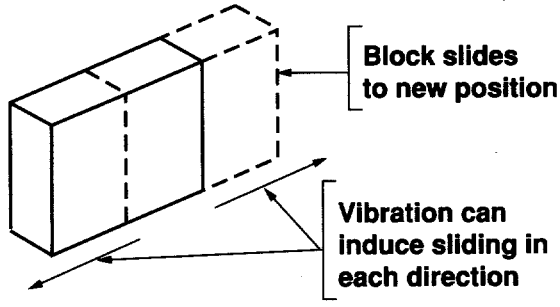
Earthquake Effects on Shielding Blocks

Because of the massive character of blocks, resistance to earthquake shaking involves different considerations than those that apply to typical buildings. Earthquakes may induce responses in blocks or block structures that can cause the following hazardous conditions (Fig. 10a-1):

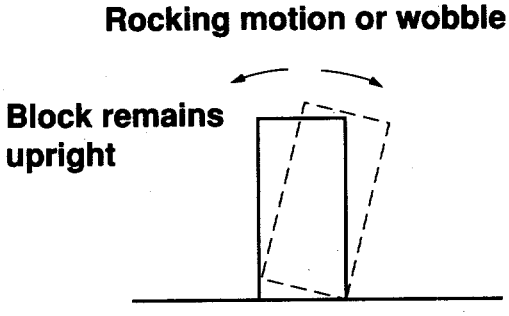
- Sliding or lateral movement
- Rocking, wobbling, or random movement
- Overturning
- Shifting and misalignment
- Collapse of the block structure or roof blocks
- Collision with adjacent internal equipment, building elements, or other obstacles.



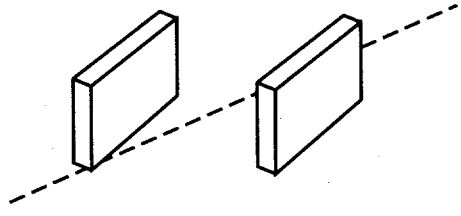
Block Stacks



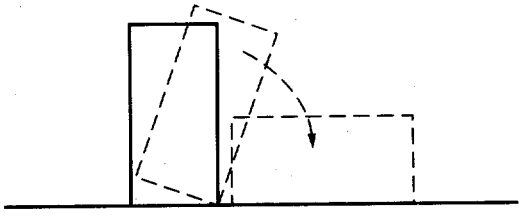
Sliding



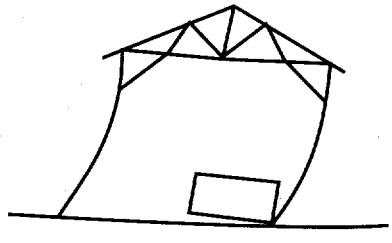
Rocking



Shifting-misalignment



Overturning



Sliding collision

Fig. 10a-1. Concrete block response.

Basic Configurations of Shielding Block Assemblies

Thickness, density, and height requirements for shielding vary. Individual block sizes are usually limited by the capacity of handling equipment available to move them, such as overhead cranes and forklifts. It is necessary to accommodate blocks to varying requirements by stacking them or placing units side by side. Thus, shielded spaces are formed by assembling individual blocks in some required pattern. Blocks may be arranged in many different configurations, depending on the degree of shielding required and the physical area available.

Concrete blocks are durable, heavy, bulky, and very expensive. This makes re-use attractive, but storage costly. Thus, blocks constructed for one project are usually adapted to subsequent layouts not always compatible with their sizes or configurations.

The most common block configurations are (Fig. 10a-2):

- *Individual blocks* in line with or at right angles to one another and without a ceiling cover
- *Tunnels* with two parallel lines of wall blocks with or without roof blocks
- *Caves*, which form enclosures with a roof and four mutually perpendicular walls similar to a box.

Usually, blocks are supported on a reinforced concrete slab on grade or a heavy foundation structure. Such heavy loads should not be placed directly on the ground or on unstable ground or potential landslide areas. Whether loads are supported by a slab or structure, the bearing capacity of the underlying materials and/or structure must be verified as well as the ability of the slab to transmit shear into the ground.

Building Code Considerations

The seismic provisions of model building codes such as the *Uniform Building Code* (UBC) (Ref. 3) have been developed primarily for buildings that have seismic response

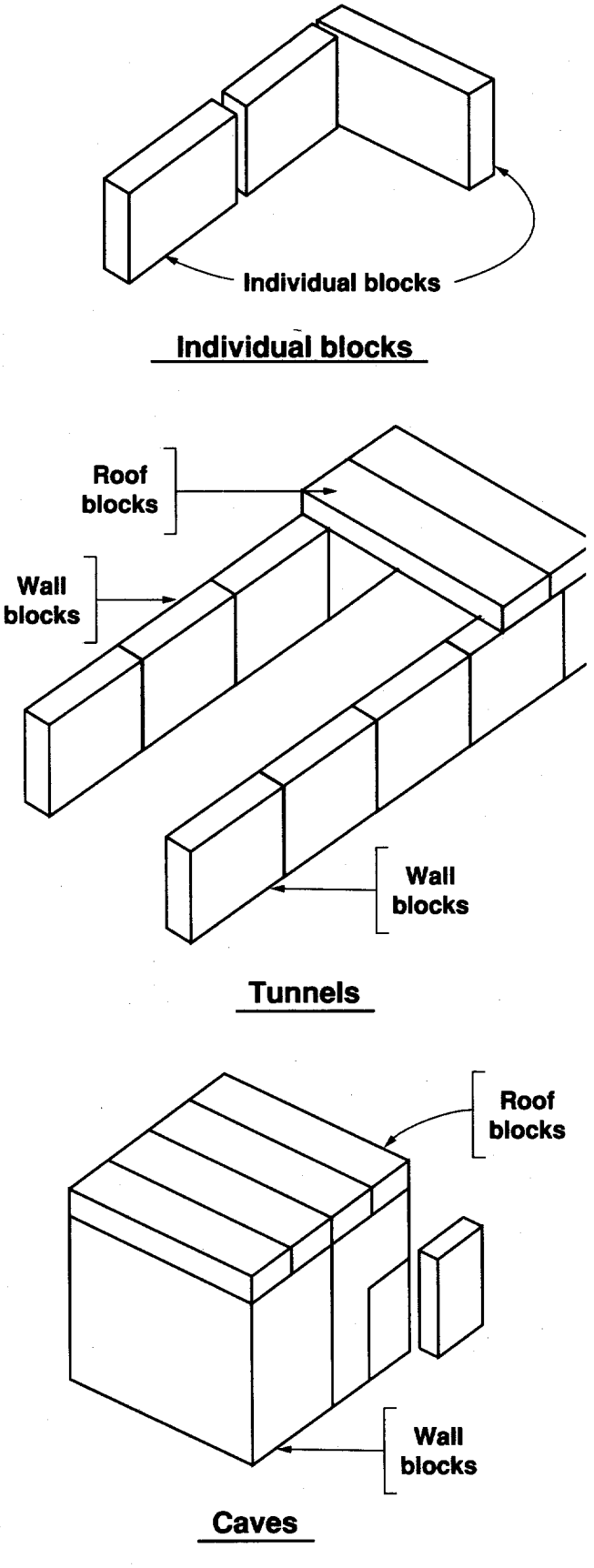


Fig. 10a-2. Block configurations.

characteristics and structural properties not generally applicable to massive shielding block structures. The UBC includes seismic loading requirements for nonbuilding structures, including rigid structures, but these requirements are not intended for such massive and unyielding objects as shielding blocks that, if restrained, are generally fixed to a concrete slab-on-grade or heavy foundation. Blocks are generally of low profile and so stiff that they will respond as a rigid body with constant acceleration, top to bottom. For all practical purposes, a block does not bend or deflect, and very little internal damping can occur. Consequently, almost all of the inertial energy induced in a massive block by lateral ground motion manifests itself as a dynamic base shear force on the connecting system fastening the block to its foundation.

Moderate earthquakes of magnitude 5 and above have been known to produce horizontal and vertical peak ground accelerations over 0.5g near the epicenter and along a causative fault. Effective peak ground accelerations greater than 0.7g have been experienced in major earthquakes. Generally steel-base clip angles that are used to bolt shielding blocks to a concrete slab have too little steel volume to absorb large amounts of energy through ductility (flexual inelastic bending). However, steel is a predictable ductile material in contrast to the brittle concrete into which most bolts are anchored, both in the block and the concrete floor slab. The ultimate failure of steel angle and bolt connectors occurs only after considerable plastic (inelastic) deformation, whereas the concrete that encompasses and anchors the bolts ultimately fails abruptly if it is not reinforced to develop a ductile anchoring medium.

The seismic provisions of the UBC define *static lateral load* as a base shear that is the summation of all horizontal inertial loads acting on a structure at any one time (Fig. 10a-3). The code also prescribes how tributary lateral loads are distributed through the height of a structure.

Shaking-table experiments have shown that various natural rocking periods can be induced into a single unrestrained block (Fig. 10a-4). However, anchoring a single block against displacement causes the block to have a very short fundamental period. Even a *cave* constructed of shielding blocks has a very short

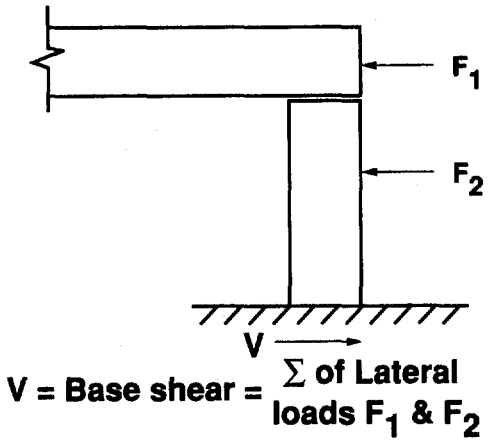


Fig. 10a-3. Static base shear.

natural period if it is braced and as ductile portions of the restraining system experience inelastic behavior, the period of the cave will vary. A dynamic analysis would be very complex and of questionable value. Consequently, it is far simpler and efficient to use a static lateral load (or base shear, V) for analysis related to the weight of the shielding and the effective peak ground acceleration estimated for a site.

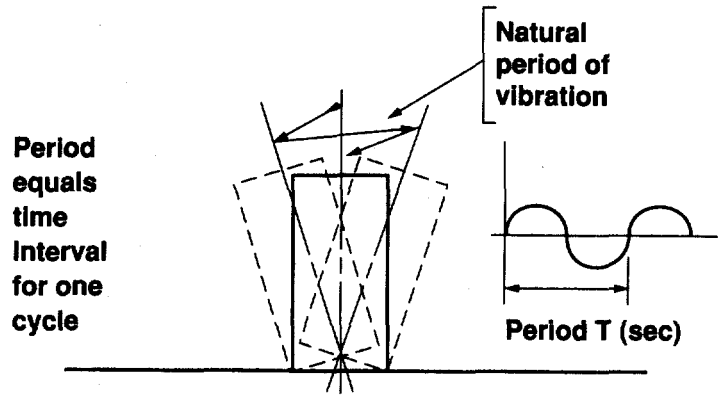
In the text that follows, seismic criteria are those recommended by the authors. They were not extracted from the UBC or any other model code or industry standard.

Seismic Design Criteria for Shielding Blocks

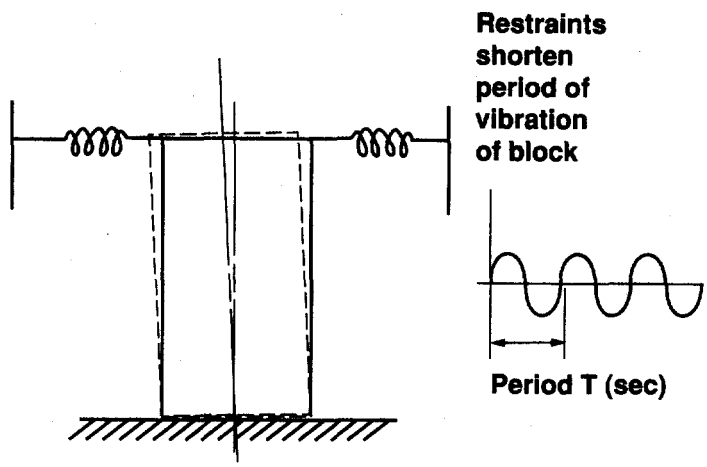
The UBC specifies stresses to be used for design that are interdependent with its other provisions to work properly as an integrated system. It allows working stresses to be increased one-third for earthquakes and wind loads because of their short duration. For mild steel, this means that the maximum allowable stress for earthquakes is close to the elastic yield stress. However, for the purposes of designing lateral-force-resisting systems for shielding blocks, it is more practical to deal directly with yield and ultimate stresses.

These seismic criteria are based on the following considerations:

- Potential risk or frequency of damaging earthquakes and the estimated maximum intensity of horizontal ground shaking at the site



Free vibration



Restrained vibration

Fig. 10a-4. Dynamic vibration.

- The type of failure, ductile or brittle, to be expected from the restraining material being designed
- The consequences of failure (monetary, injury, life-safety, release of toxic materials, loss of emergency services, etc.).

In strong earthquakes, designers must anticipate that heavy shaking may cause stress excursions well beyond the elastic yield stress resulting in plastic (inelastic) deformation (Fig. 10a-5). Consequently, the extreme distortions associated with yield or ultimate strength levels (short of rupture) may make connections unsuitable for future use.

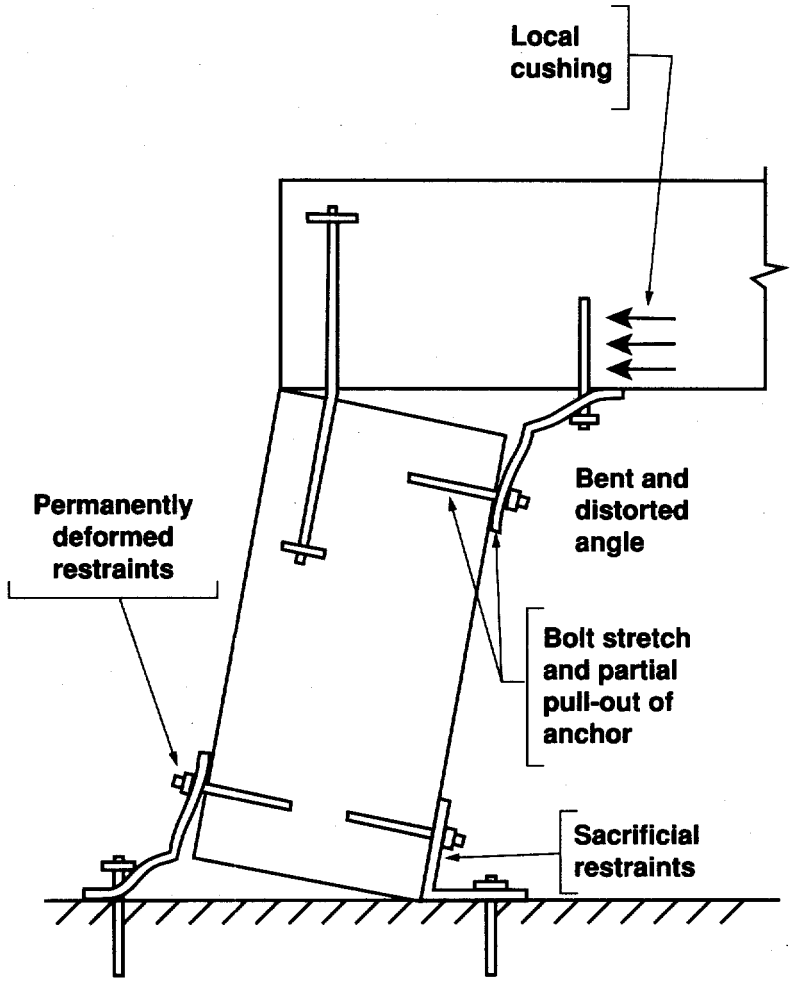


Fig. 10a-5. Inelastic distortions.

Table 10a-1 summarizes the authors' recommendations for static lateral force criteria for the seismic design of shielding block systems. These recommendations are meant to provide a level of safety equivalent to *Standard-Occupancy Structures* and *Essential Facilities* as defined by the UBC which correspond to Performance Categories 1 (PC-1) and 2 (PC-2) facilities as defined in DOE-STD-1020 (Ref. 4). These criteria provide a basis for application in various locations of the United States by utilizing the *Seismic Zone Factor* (Z) from Table 16-1 of the 1994 UBC. A *Seismic Zone Map of the United States* (Fig. 16-2 from the UBC) is reproduced here in Fig. 10a-6 for determination of the appropriate Z factor. Generally, the calculated accelerations ($2.5\ Z\ I\ C_p$) derived using Table 10a-1 recommendations will be greater than the maximum horizontal ground surface accelerations at DOE sites for Performance Category 1 (PC-1) taken from Tables C-5a and C-5b from DOE-STD-1020.

For shielding assemblies that house hazardous operations and/or essential components that must remain operational in the aftermath of a damaging earthquake, the importance factor (I) should be increased to 1.25 in keeping with UBC Table 16-K, *Occupancy Category* for *Essential Facilities*. This increase produces acceleration factors greater than the maximum horizontal ground surface accelerations listed for DOE sites (Tables C-5a and C-5b from DOE-STD-1020) for Performance Category 2 (PC-2). Although *Essential Facilities* should be designed for a base shear (V) with an importance factor (I) of 1.25, nonstructural components of systems and equipment (and their attachments) that must remain operational for life safety or other emergency needs should be designed for a lateral force using I equal to 1.50.

Table 10a-1.

Seismic criteria for static analysis of shielding block assemblies.

$$V = 2.5\, Z\, I\, C_p\, W$$

V = Static Base Shear

I = Importance Factor

Z = Seismic Zone Factor

C_p = Lateral Force Factor

W = Total Seismic Dead Load

Material	Cp	Stress Limit
Steel[1]	0.5	Yield Strength
Steel Diagonal Bracing and connections (Mild Steel)[2]	0.75	Yield Strength
Non-Ductile Concrete		
Bending, Compression and Shear	0.7	0.85 Ultimate Strength
Bearing	0.7	0.60 Ultimate Strength
Anchor Bolts	0.7	0.75 Ultimate Load Value[3]

(1) For structural steel or other materials that exhibit non-linear plastic behavior (similar to steel) beyond yield stress.

(2) Some higher strength steels may not exhibit a usable yield plateau and should be designed more conservatively.

(3) Based on load tests.

Seismic Zone Factors (Z)
(from UBC Table 16-I)

Zone[1]	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

(1) Zone to be determined from UBC Seismic Zone Map of the United States, (see Fig. 10a-6).

Importance Factors (I)

Importance Category[1]	Importance Factor, I
Standard Occupancy Structures	1.00
Essential Facilities	1.25[2]
Hazardous Facilities	1.50[3]

(1) For occupancy within shielding, see UBC Table 16-K, *Occupancy Category*, for definitions.

(2) Nonstructural components of systems and equipment (and their attachments) that must remain operational for life safety or other emergency needs shall have an Importance Factor of 1.50.

(3) For shielding that houses dispersible toxic materials or equipment that must remain operational for life safety systems or containment of toxic materials.

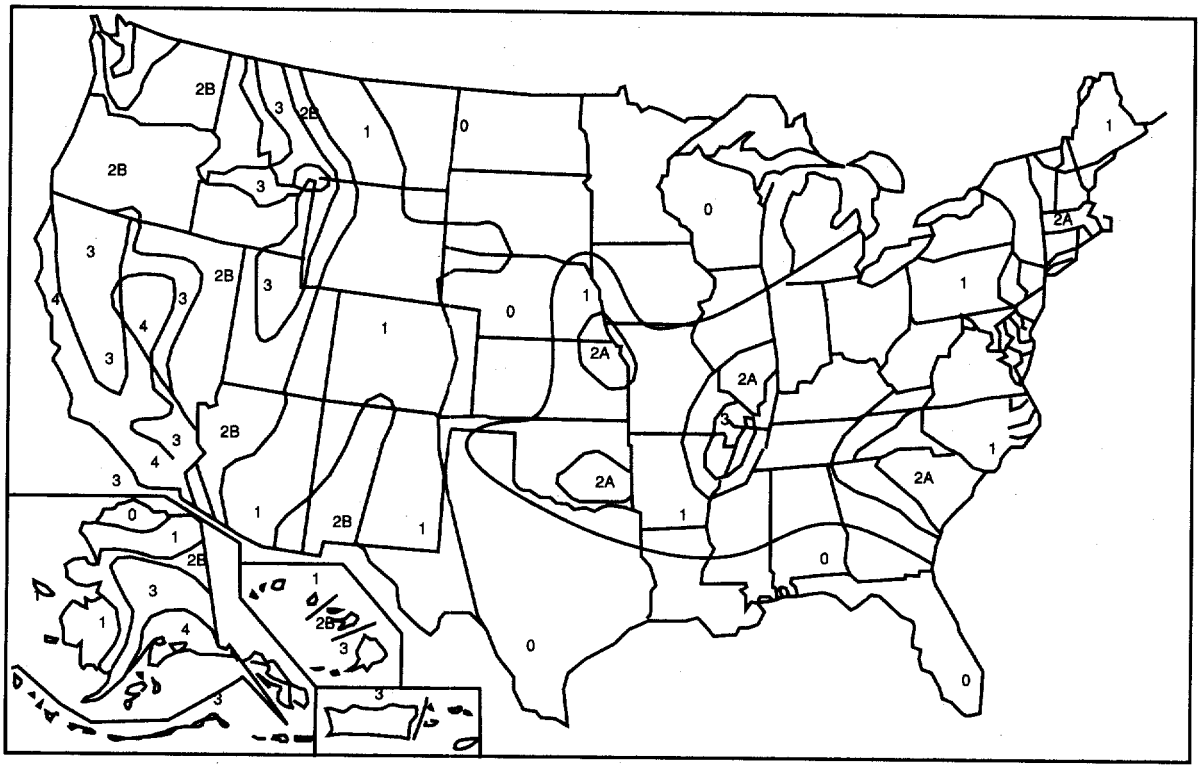


Fig. 10a-6. UBC Seismic Zone Map of the United states (Figure 16-2 from the 1994 edition). For areas outside of the United States, see UBC Appendix Chapter 16.

For design of shielding assemblies that contain highly toxic materials and related equipment that must remain operational, the I factor should be increased to 1.5 instead of 1.25. This produces acceleration factors that are conservative in relation to the maximum horizontal ground surface accelerations listed for DOE sites in Table C-5a and C-5b from DOE-STD-1020 for Performance Category 3 (PC-3). Tables C-5a and C-5b list maximum horizontal ground surface accelerations for Performance Categories PC-1 through PC-4, but recommend site-specific seismic hazard curves for PC-3 and higher. The seismic design criteria in Table 10a-1 have been formulated for PC-1 and 2 categories only. Refer to DOE-STD-1020 (Ref. 4) for specific requirements concerning facilities in Performance Categories PC-3 and higher.

Applications to Individual Blocks

Shaking-table tests show that individual unanchored blocks with a height-to-width ratio less than the static coefficient of friction will usually slide when subjected to horizontal earthquake forces, whereas blocks with a height-to-width ratio greater than the static coefficient of friction will rock. Individual blocks may be made to rock at different frequencies. In earthquakes, both horizontal and vertical accelerations are erratic, inducing chaotic movement. Consequently, blocks should be anchored to resist these phenomena.

When anchoring individual concrete blocks for lateral stability, use of a base shear calculated from Table 10a-1 results in static lateral loads that are conservative in relation to inertial forces that result from effective peak ground accelerations. Because most blocks rest on a slab-on-grade, some energy may be dissipated in mobilizing the heavily loaded slab, resulting in some attenuation of the actual ground acceleration provided the slab is adequately reinforced. Conversely, when blocks rest on structural framing that is integral with a building frame, rather than on a slab-on-grade, there may be amplification of the ground motion. Upper floor accelerations are usually amplified above the ground motion. This amplification can be calculated by dynamic analysis of the base structure and applied to the block as a higher base shear factor.

Of concern is the possibility of resonance between the forcing frequencies or ground vibrations and the fundamental period of the anchored block. An anchored block has a very short fundamental period, so an installation supported on a geologic site having a short natural period (rock) must be designed more conservatively than one on a softer foundation.

An individual block does not benefit from continuity or redundancy inherent in a unitized building system; therefore, it is essential that the base connections that transmit the ground motions to the block do not break or tear away from the floor or the block.

Once the seismic input is determined, other factors should be considered in selection of the restraining systems. If first cost economics dictate, restraint systems may be made expendable for the maximum seismic event so that ductile materials will be stressed well into the plastic or ultimate strength region. However, overturning or nonductile failure must be prevented. Bending, buckling, stretching and distorting of members and anchors are important sources of energy dissipation. At the mild steel yield stress value, with a flat-yield plateau (Fig. 10a-7), considerable reserve strain capacity is available before the steel strains reach the ultimate value and rupture ensues.

Concrete (Fig. 10a-8) and higher strength steels do not exhibit the same type of yield plateau as mild steel; therefore, yield values and reserve capacity are not as easy to define. Because some reserve capacity (short of brittle failure) should be retained for concrete, a factor of 0.85 is recommended for application to the ultimate-stress values for bending, shear, and compressive-stress capacities and 0.6 for bearing-stress capacity. Concrete is a brittle material susceptible to sudden failure or structural deterioration; therefore, it is best to be conservative when relying on its post yield strength properties. It is possible, however, when designing new foundations for shielding-block assemblies to incorporate reinforcing around anchor bolts to impart necessary toughness and ductility.

Anchorage Design

Anchors may be expansion-type anchor bolts drilled into existing concrete work or standard anchor bolts embedded in new concrete (Fig. 10a-9). *Allowable* working loads are usually given as a percentage of ultimate test loads, 25% being the most common. Considering that anchor failure in concrete is abrupt, the ultimate values should be multiplied by a capacity-reduction factor of 0.75 for use in *ultimate-strength* design. When using manufacturers' design values, one should ensure that they have been certified by the ICBO *Evaluation Services, Inc.* (See Chapter 13). Cast-in-place anchor bolts may be designed using ACI-349, Appendix B (Ref. 7).

When anchoring shielding blocks, the basic idea is to provide enough restraint to prevent dislocation and/or toppling of the block. Ductile components of the restraint system

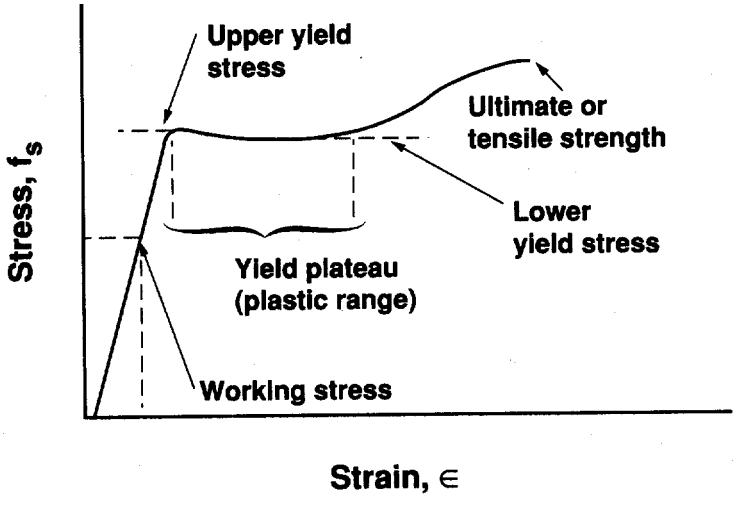


Fig. 10a-7. Structural steel stress-strain curve.

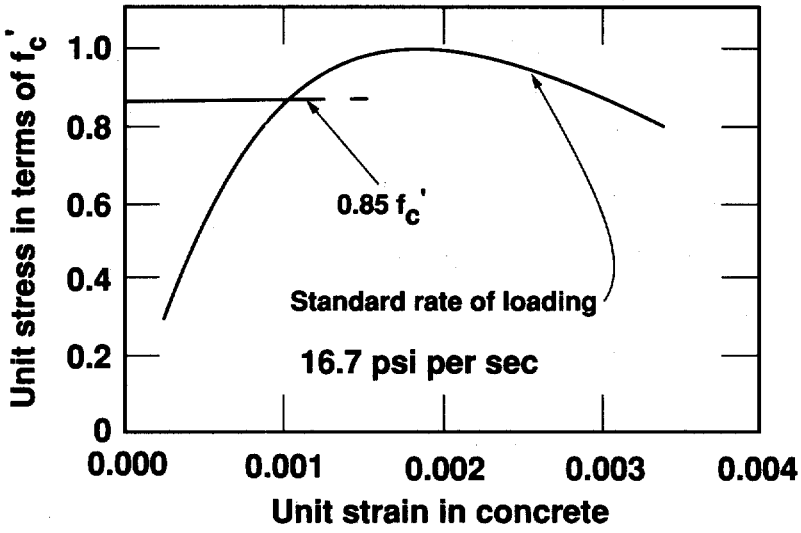


Fig. 10a-8. Typical stress-stain curve for concrete cylinders.

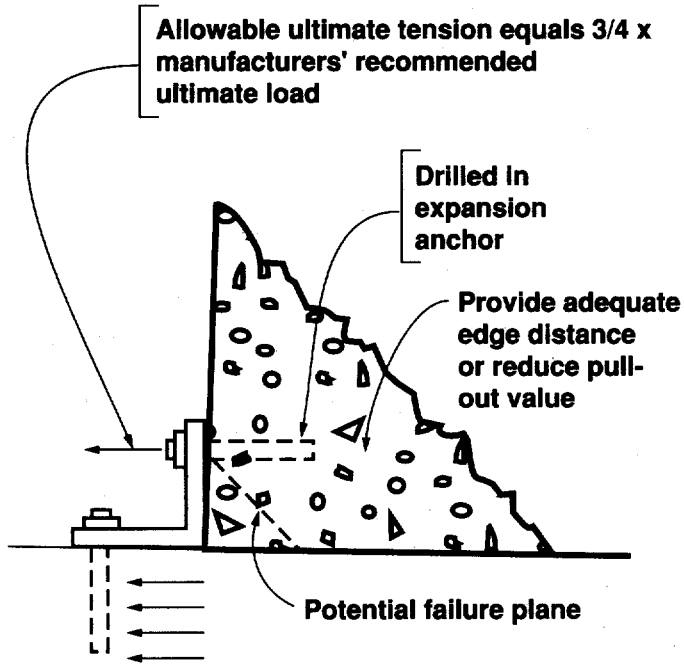


Fig. 10a-9. Anchor bolts in concrete.

should be designed for stresses (or strain conditions) approaching yield. For example, the steel base-clip angle itself should be designed to yield before brittle failure of the concrete holding the anchor bolt can occur. This allows designers to build some energy absorption into the restraining system.

Clip angles used to anchor shielding blocks should be strong enough to provide enough ductility to absorb energy in bending after yielding. When the angle is reinforced with stiffeners, it should not be so heavily stiffened that it rigidly transmits the design shear directly into the anchor bolts without deforming the clip angle.

If the angle thickness is designed to achieve the required ductility in the connection (for C_p = 0.5), its thickness will not be enough to eliminate prying action of the bolts. Therefore, an analysis should be made using the *American Institute of*

Steel Construction (AISC) procedures to compute the additional load in the bolts and drilled-in (proprietary) anchors caused by prying action.

Alternatively, prying action on the bolts can be relieved by using full-depth stiffeners on the base clip angle. Smaller or fewer bolts can then be used however, the connection will have so little ductility that it should be considered as a nonductile connection with C_p=0.7.

It is also very important that the bolts that anchor the angle to the concrete floor slab and to the concrete shielding block have proper embedment in each and adequate edge distance. The distance from an anchor bolt to a free edge of the concrete is an important consideration in anchorage design. If possible, anchor bolts should be located and embedded so that reinforcing steel intercepts potential cracking planes. If sufficient embedment depth and edge distance to develop the strength of the anchor cannot be achieved, then the capacity values for shear and pull-out of the anchor should be reduced appropriately. Shear and tension interaction should be accounted for in the anchor if they can occur simultaneously. If several bolts are parallel to a free edge, the effect of overlapping failure planes on the concrete design strength also must be taken into account. If floor clip angles are used on opposite sides of the block, consider using a through bolt. This eliminates pull out problems, and leaves shear as a consideration. It is still important to have reinforcing bars between the bolt and the edge of the block. For these important considerations, readers are referred to the article, *Guide to the Design of Anchor Bolts and Other Steel Imbedments* (Ref. 5). See also *Anchors in Concrete, Design and Behavior,* (Ref. 6), and *Code Requirements for Nuclear Safety, Related Concrete Structures* (Ref. 7).

Prevention of Dislocation and Overturning

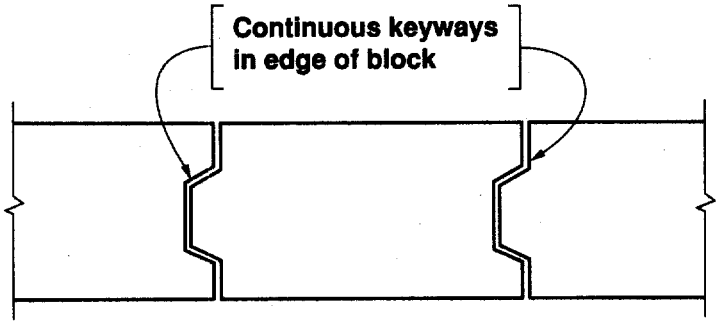
One of the easiest and least expensive ways to minimize block dislocation is to key blocks together at coplanar surfaces and to the floor (Fig. 10a-10). Where blocks in line abut each other, continuous tongue-and-groove keyways along the contact surfaces are effective. Floor keys can be cast in new construction where block locations have been predetermined, but chipping or casting keyways in existing construction is not always feasible. Anchorage is still required in any case. Heavy pins can be placed in holes drilled into existing slabs (Fig. 10a-11) so that they protrude up into holes formed in the bottom of the block when the block is set in position on the floor. Lateral loads are transferred by shear and bending in the pin and bearing of the pin on the concrete, and through shear and bearing at the keyway edges. In lieu of pins at the base, angle clips can be attached to the block and secured to the floor with pins or anchor bolts (Fig. 10a-12). Flat-steel plate inserts with substantial anchors into the concrete can be cast in the face of the base of the block. Base angle clips can then be welded to these plates, or stud anchors can be welded perpendicular to the flat face of the plate and block and used to bolt (clamp) the base angle to the face of the block. The weldment may be the most brittle point in the system. If in the designers' judgement this is so, then a C_p = 0.7 should be applied.

Other anchoring systems can be devised with the function of keeping the concrete floor slab from shearing away from the blocks as the ground induces earthquake motion into the block through the slab.

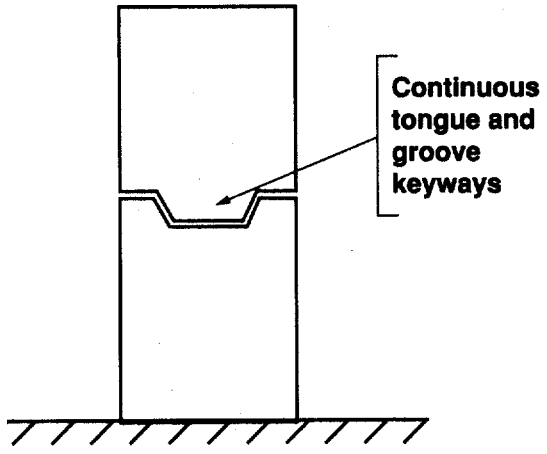
When new shielding can be tied into existing anchored shielding block installations, the ability of the existing shielding to resist movement of the new blocks or assemblies may be utilized. Intersecting blocks or block walls can also be placed to resist displacement. Designers should consider the blocks as a system and provide base anchors, keys, pins, and ties as necessary to maintain elements in place.

Generally, building framing members, especially columns, should not be used to resist lateral loads from massive shielding blocks. Blocks and their restraints should be free and adequately clear of building support members. The use of framing elements that are part of the vertical-load-carrying building frame should always be avoided.

Base anchors (Fig. 10a-13) are effective against sliding and differential displacement, but are not always a positive safeguard against overturning under severe or sustained rocking motion. Overturning restraint (Fig. 10a-14) can be incorporated with base anchors in the form of



Plan



End elevation

Fig. 10a-10. Concrete keyways.

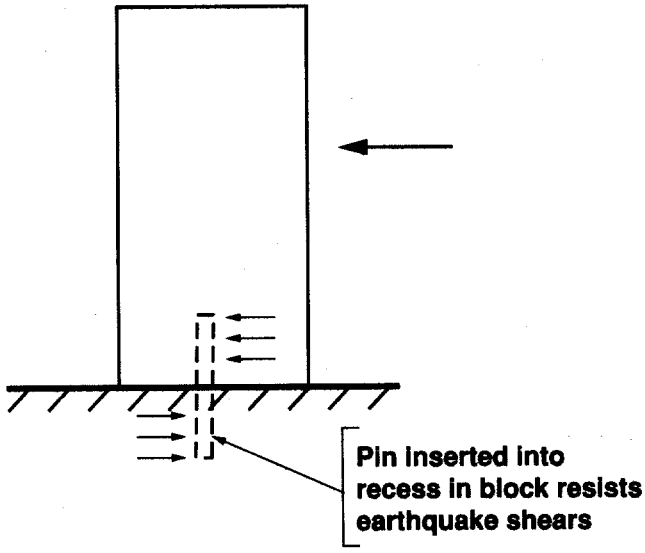


Fig. 10a-11. Heavy shear pins.

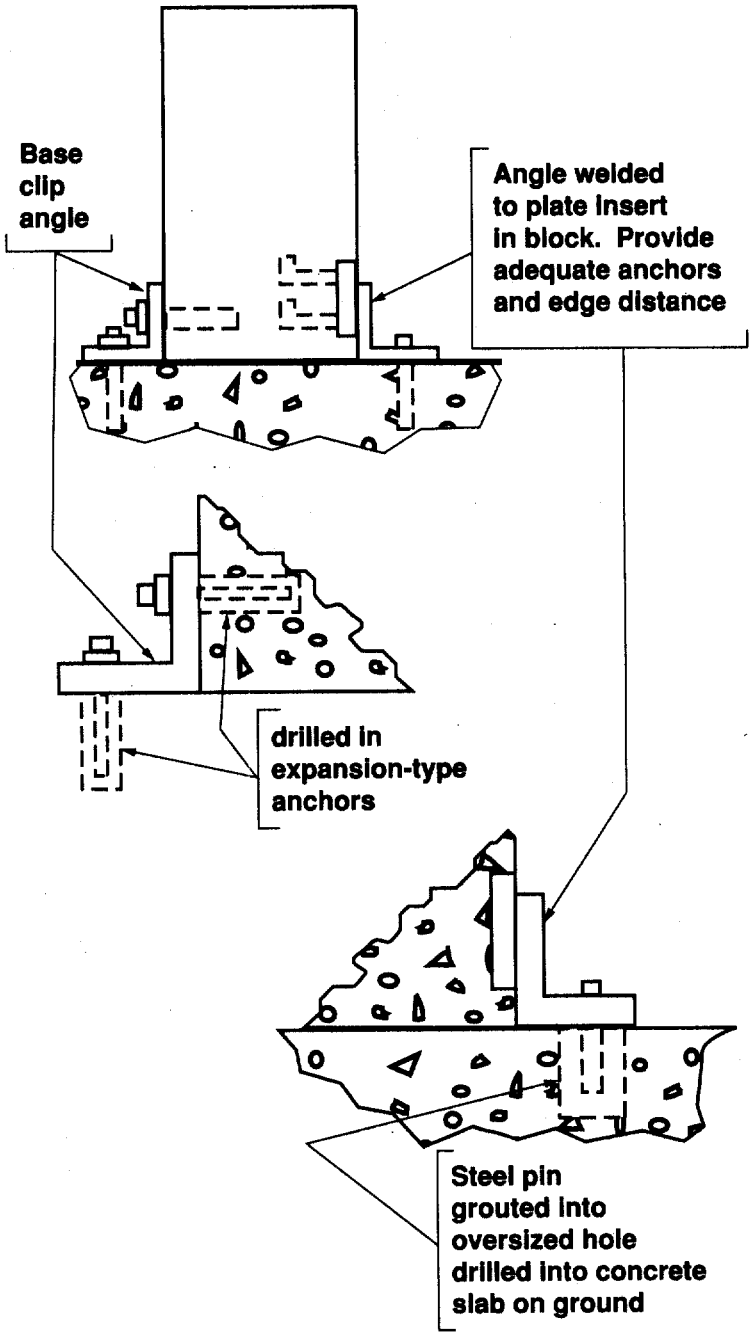


Fig. 10a-12. Securing angle clips with pins or anchor bolts.

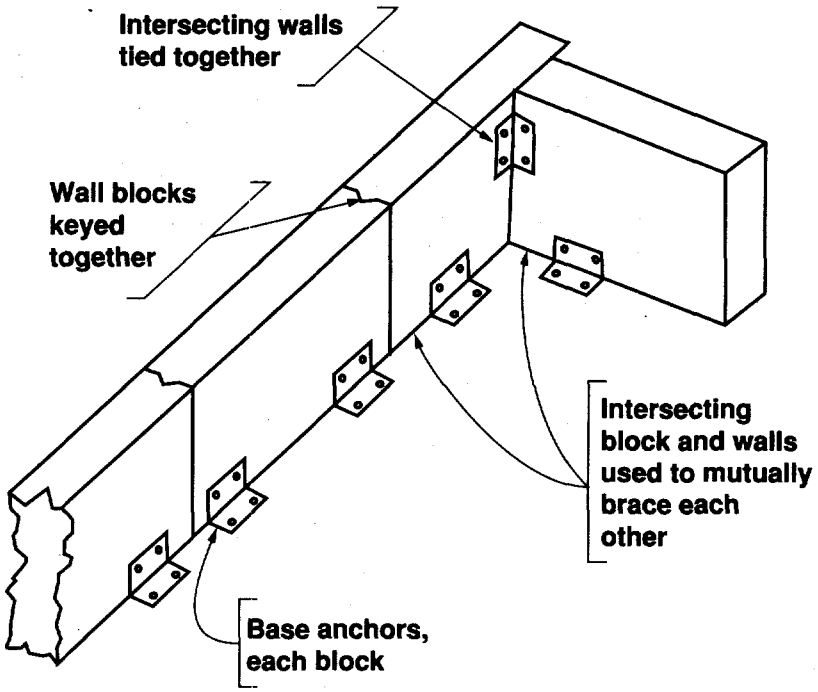


Fig. 10a-13. Base anchors.

tie-down rods or straps attached to block support framing systems, foundations, or other ballast with sufficient resistance or mass to counteract uplift loads.

Diagonal bracing struts perpendicular to the long face of blocks are a good method of maintaining block stability. Bracing struts should rest or react against the top third of the block height. The principal drawback to the use of struts is their encroachment on space adjacent to the blocks. To avoid this problem, cantilevered steel columns can be connected to horizontal beams that engage the faces of the blocks such that the beams span horizontally between columns and deliver the (lateral) block load to the cantilever columns. Cantilever action can be obtained by anchoring the bottom of the column in a caisson or shaft drilled into the ground and filled with concrete (Fig. 10a-15). Formulas are available in the UBC to define the drilled shaft's depth and diameter. Soil-bearing values can usually be interpolated from previously available information or conservatively assumed from visual site inspection, geologic maps, or code tables.

Design of Shielding Walls and Tunnels for Earthquakes

Often, shielding blocks are assembled in a longitudinal tunnel configuration (without transverse walls) with roof blocks supported directly on longitudinal wall blocks. It is imperative that a support and anchorage system for the component blocks be provided to prevent misalignment or shifting of tunnel wall blocks that could cause the collapse of roof blocks.

The premise that the base anchorage should be designed to prevent brittle failure yet provide ductility in the angle is as valid for tunnel configurations as for individual blocks. However, it is possible to take advantage of additional energy-dissipating phenomena at the interface of wall and roof. Connections can be devised that allow for distortion and stress reversal that, along with friction and damping, will relieve the effects of dynamic loading. Non-ductile base connections should be designed for seismic forces based on $C_p = 0.7$, but the ductile structural elements above the base can be designed for seismic forces (see Table 10a-1)

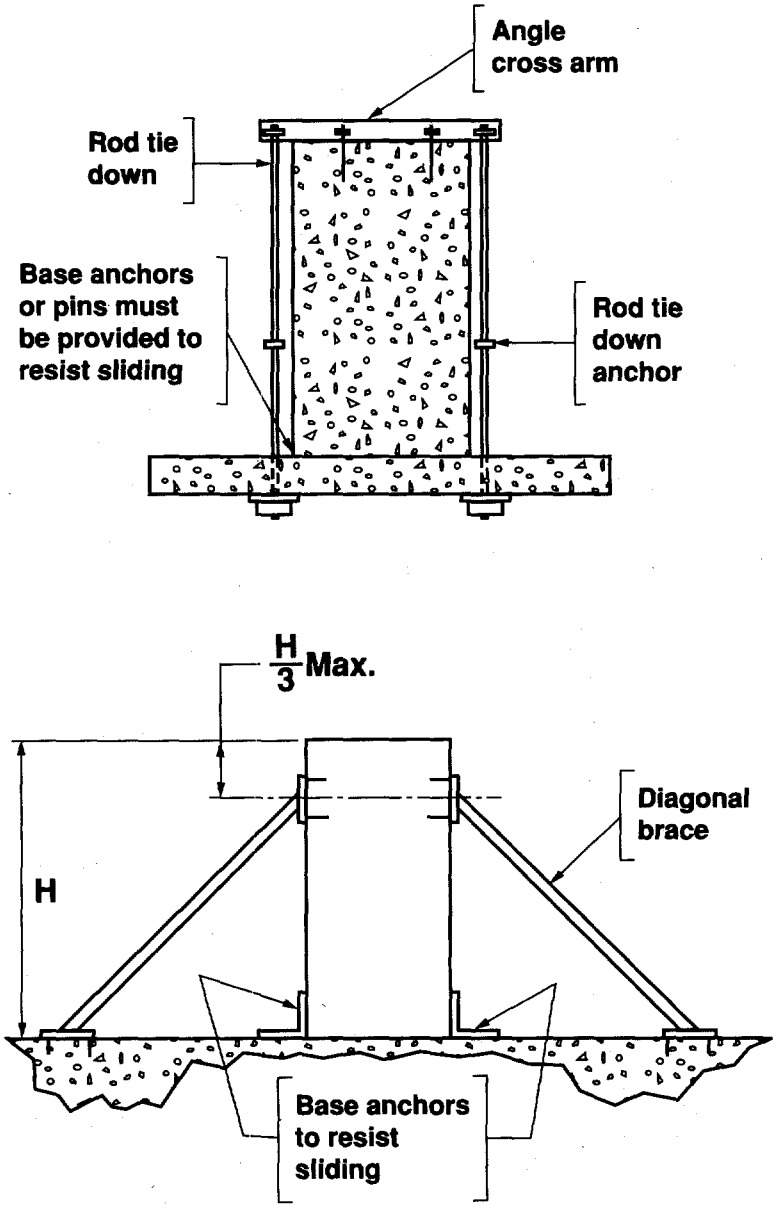


Fig. 10a-14. Overturning restraints.

based on C_p = 0.5 and still meet energy demand through inelastic deformation. It is necessary to design the ductile lateral-force-resisting system and connections to yield and let the strain energy of yielding absorb the load without rupture.

To prevent premature or brittle failure of a connection, one should differentiate between the ductile and more brittle features of the same connection. The ductile or steel portion can be designed for yield stress against seismic forces (see Table 10a-1) based on C_p = 0.5 and the concrete anchorage for the same connection should be capable of withstanding stresses based on 0.85 ultimate stress against seismic forces based on C_p = 0.7. In most instances, the steel

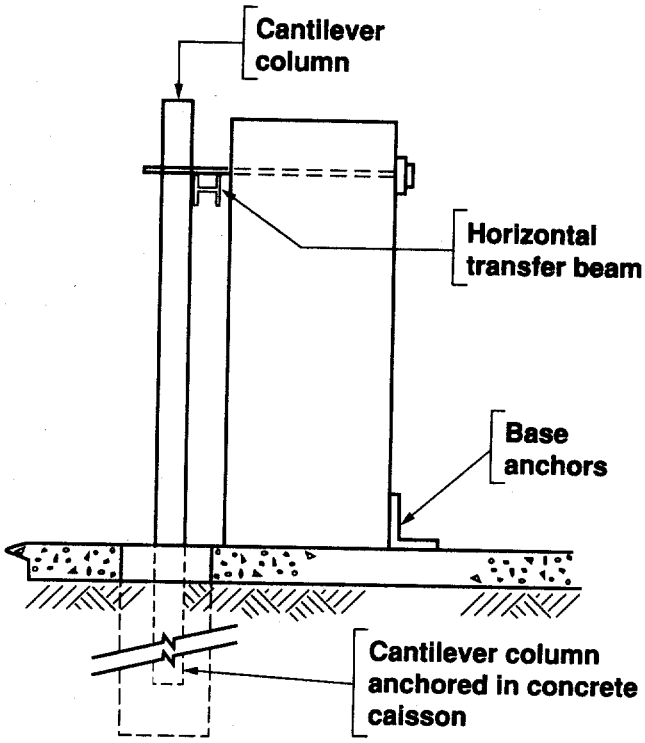


Fig. 10a-15. Cantilever restraints.

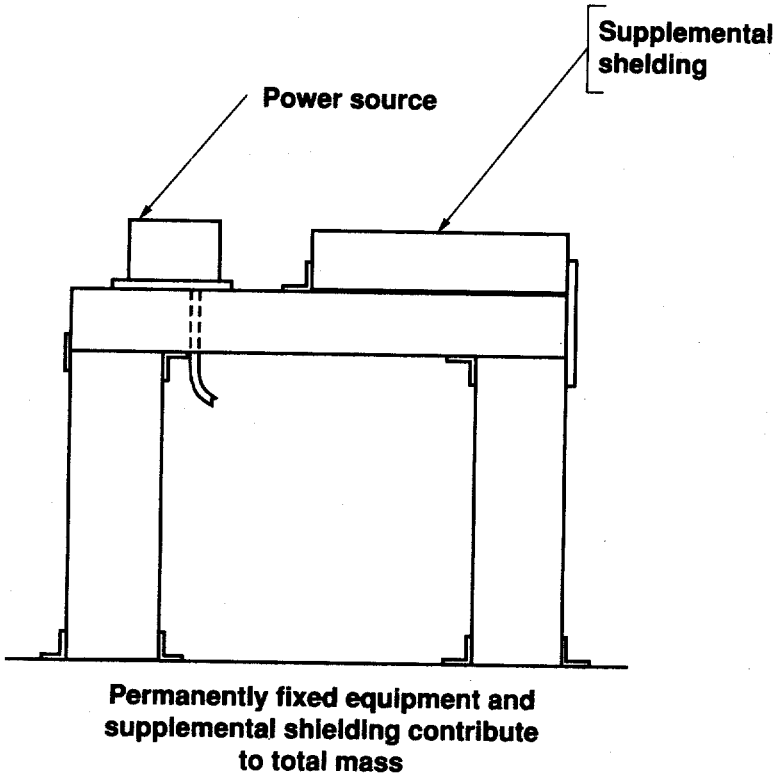


Fig. 10a-16. Tunnel assembly cross section.

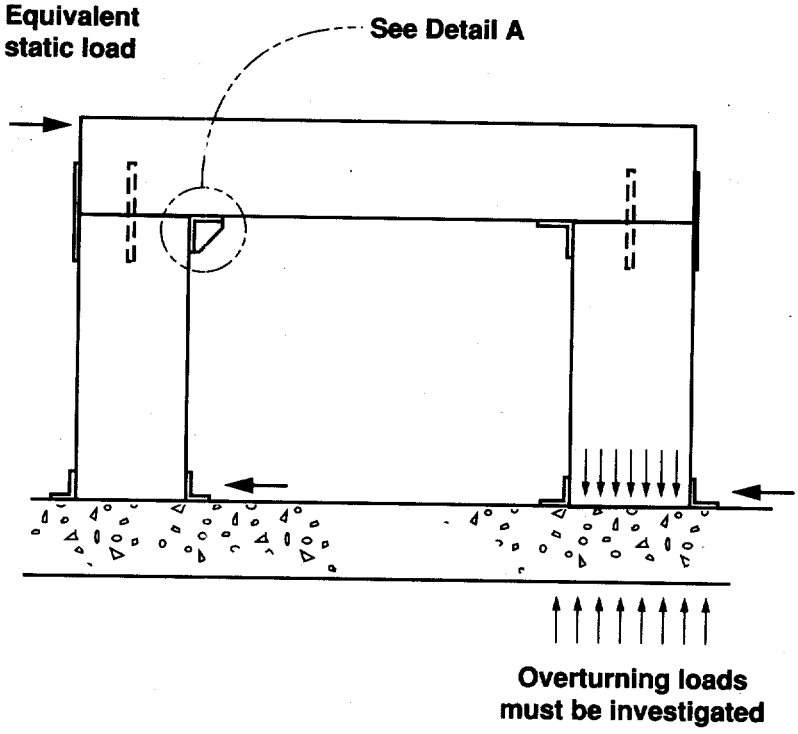
parts (i.e., angles, plates, pins, and bolts) can be designed using the yield strength of the material. However, where a steel member relies on the strength of the concrete for effectiveness, such as anchor bolts, pins, and drilled-in expansion anchors bearing against concrete, the individual part of the connection dependent on the concrete should be sized using $C_p = 0.7$ against maximum bending stresses at 0.85 times the ultimate strength of concrete and bearing stresses at 0.6 times the ultimate strength of the concrete. Thus, concrete anchorages should be tough enough to allow the beneficial dissipation of energy through the inelastic distortion of the ductile materials in the connection.

With block structures having roofs, the question arises as to what gravity loads to use to calculate the seismic design force. Normally, one need only consider the dead weight of the wall and roof blocks. However, heavy experimental equipment, supplemental shielding, and power sources are frequently anchored permanently on the roof (Fig. 10a-16). Unless these accessory weights are effectively isolated, they (or some proportion) must be included in the dead load calculation. This effective dead load (W) used in the equation $V=2.5\ Z\ I\ C_p\ W$, results in the base shear (V), which is the static lateral force to be applied to the tunnel assembly. Materials and equipment placed on the roof, but unsecured will move around during an earthquake, but only a portion of its weight needs to be considered. However, if it is unsecured, it could *walk* off the roof. Consequently, it is best to secure such items.

As with individual blocks, a primary principle is isolation of the tunnel and its components from building supports or adjacent structures. Wall blocks should be keyed to adjacent blocks and roof blocks keyed or mechanically interlocked to prevent relative movement (Fig. 10a-17). Base angle clips and pins can be incorporated to prevent sliding. Frame action can be utilized by incorporating angles and brackets to develop moment capacity at roof block and wall block interfaces. In this situation, the clip angles transfer seismic energy directly into this interlocked *frame* for dissipation rather than relying upon the clip angles alone to dissipate energy. For this reason, clip angles should be sufficiently stiffened to transfer several cycles of stress reversal to the *frames* created by the connections. Moment can be resisted at the base by a couple provided by base clip angles on both sides of wall blocks. Care should be taken in the analysis to account for the fact that the blocks are extremely stiff in relation to the connections. Tiedowns to resist overturning and uplift should be utilized if the base clip angles cannot prevent the blocks from rocking severely.

In tunnel construction, the plane of the ceiling or underside of the roof is a convenient place to put horizontal diagonal bracing. This location is preferred when the roof blocks must be occasionally removed. A horizontal truss system (Fig. 10a-18) can be utilized to receive lateral loads from the blocks. Truss reactions must be transferred to the foundation through frame action of the wall and roof blocks or via extra framing, i.e., diagonal bracing struts and/or frames at the ends of the tunnel or spaced intermittently along the length of the tunnel. To compensate for potential buckling of diagonal bracing, the design force should be increased by a factor of 1.5 for yield stress analysis. To maintain the same concept of resistance for the truss system, one should apply the 1.5 load factor for the yield stress analysis by modifying the earthquake coefficient C_p to 1.5x0.5 or 0.75. This provision has been included in the *Seismic Criteria for Static Analysis of Shielding Blocks Assemblies* summarized in Table 10a-1. First-yield stresses should be utilized for the diagonal bracing with this modified coefficient.

If seismic truss reactions at the ends of the tunnel or reaction points are transferred to the foundation by moment frame action of the wall and roof block assembly, these seismic reactions for the moment frame need not include the 1.5 load-modification factor when designing the moment frame connections and base anchorages. Design of truss member connections to block assemblies (moment frame) should use seismic reactions derived with the 1.5 modifier. When diagonal braces in a vertical plane are employed to hold the horizontal truss in the roof plane, the modified lateral force factor, $C_p = 0.75$, should continue to control sizes and details. Foundation bearing pressures can be evaluated on the basis of reactions calculated without the 1.5 factor, which applies only to the braces and their connections.



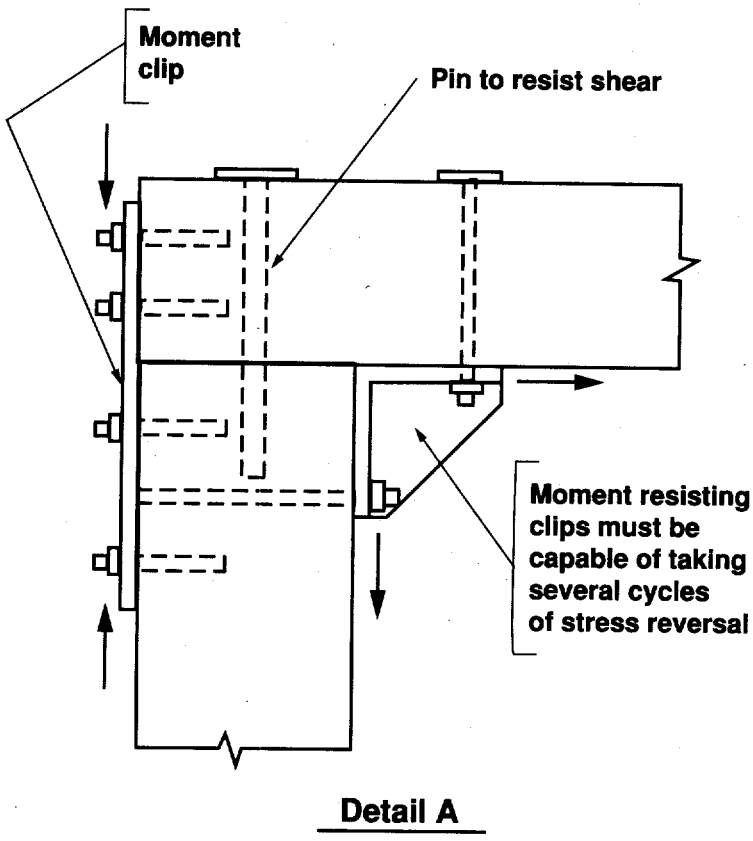


Fig. 10a-17. Frame action details.

Diagonal bracing can also be located in the plane of the top of the roof blocks if the exposure of the framing poses no hazard to personnel and does not interfere with experimental equipment or occasional dismantling. Depending on design loads, connection of the truss diagonals to the blocks (Fig. 10a-18, Section A-A) may be accomplished with anchor bolts, drilled-in-place concrete anchors, welded studs, or pins. Most support techniques used for individual blocks or blocks in-line are equally adaptable to tunnel-like or roofed-over configurations. Diagonal struts can be used to brace each wall block, or a system of horizontal beams and cantilevered columns can be integrated to support the walls. Because the inertial loads to be resisted are large, the problem of controlling deflection is encountered when using cantilevers. A check of deflections is necessary, coupled with an evaluation of potential effects of translations and rotations on the stability of the cell unit. Elastic properties of the ground resistance against the caisson must be considered in determining deflections. The modulus of subgrade reaction is a useful parameter for this purpose.

Design of Shielding Caves for Earthquakes

Because cave facilities resemble buildings more than other shielding configurations (Fig. 10a-19), one might logically reassess the earthquake coefficient for such caves relying on experience with similar massive building structures. Generally, however, the authors recommend the use of higher lateral load coefficients and yield stress capacities that are more in keeping with the primary response of a massive and very rigid structure such as a cave of shielding blocks.

Designers must make sure that there is an uninterrupted path for transferring loads in the bracing members to the ground or bearing medium. Collectors can be used to drag loads from areas tributary to the bracing elements (Fig. 10a-20). Connections and splices must be strong enough to prevent collected loads from breaking the link to the bracing members and isolating part of the cave. This type of failure would leave the cave without lateral support and susceptible to collapse.

Caves by definition usually have four walls mutually perpendicular to each other; therefore, these walls can be used to counteract loads concentrated by collectors. Internal stresses within the blocks are seldom critical, but connections of bracing elements or collectors to the blocks must be strong enough to prevent premature breaking away (Fig. 10a-21). Resisting loads by members in tension and bolts in shear is the preferred approach. Bolts should be high-strength fasteners tightened in accordance with recommended standard procedures. Values for slip-critical bolts should be used whenever high-strength bolts are used in conjunction with welding. Drilled-in-place proprietary anchors may be used; however, cast-in-place anchors or bolts epoxied into drilled holes are tougher and more reliable.

Other building design techniques are useful for caves, the most expedient being the use of the cave roof as a diaphragm. Shear plates can be used to transfer seismic shears from block to block, and structural steel shapes can form flanges (chords) for the diaphragm and load-transfer elements (Fig. 10a-22). Individual wall blocks can be anchored together and to the floor to complete the path to the ground to transfer the dynamically induced forces. Where walls can be unitized, tie-downs at the ends to resist uplift are seldom necessary. Interior cross walls, if permanently located, are useful as additional resisting elements, but they must be anchored to the roof and floor for their proportionate load. Maintaining integrity and alignment are necessary adjuncts to a satisfactory system. Previous emphasis on keys, pins, and mechanical interlocks applies.

Supplemental bracing schemes external to the shielding blocks are sometimes a viable alternative (Fig. 10a-23).

Diagonally braced or moment-resisting frames may be used, but additional caution is necessary when relying on ductile moment-resisting frames. Because blocks are unyielding and resist movement, they prevent the necessary energy-absorbing distortions of the frame from taking place. This could lead to premature failure of the assembly or anchorages because the frame may not be able to absorb the required energy through plastic yielding before failure of the stiffer elements. A careful compatibility

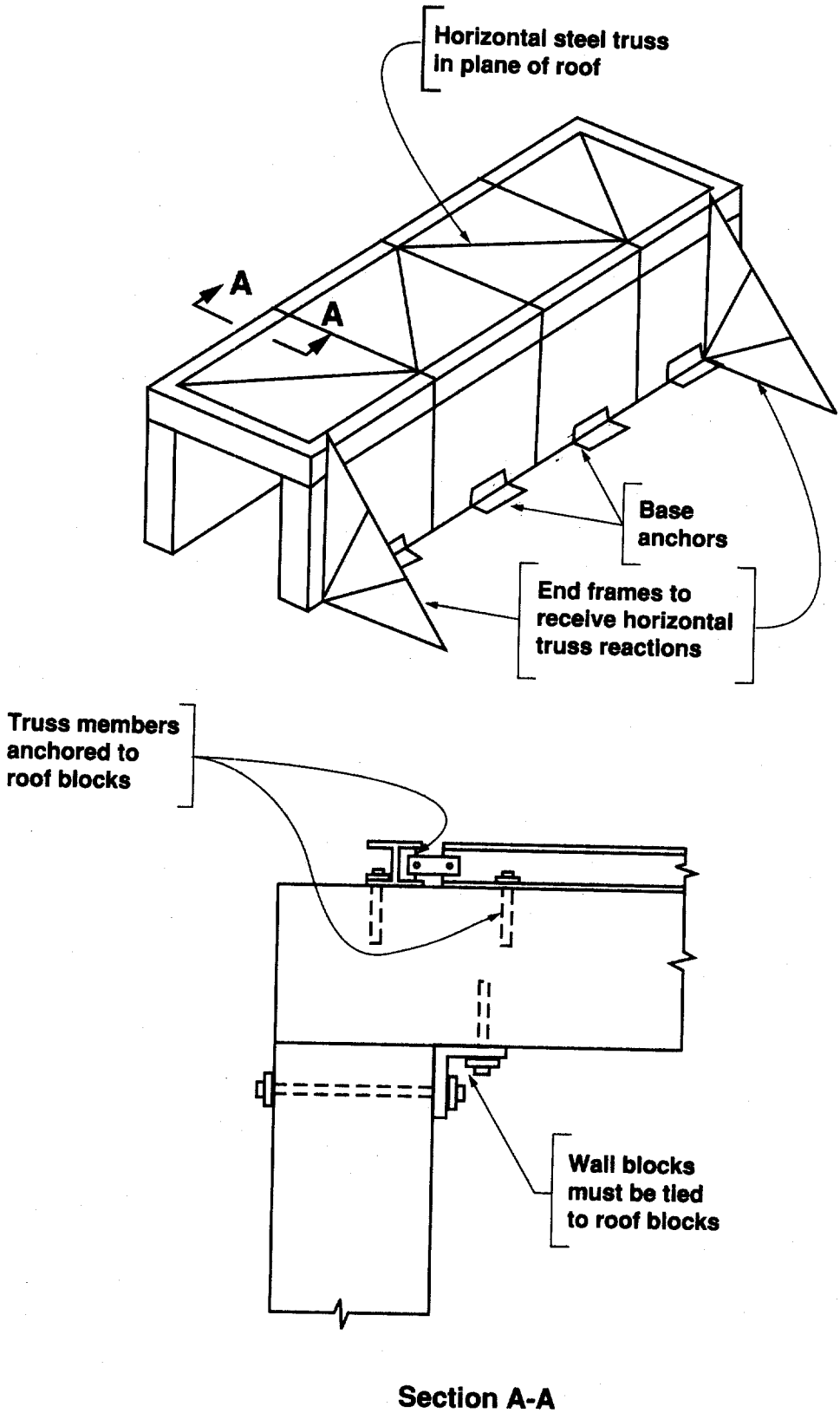


Fig. 10a-18. Horizontal diagonal bracing.

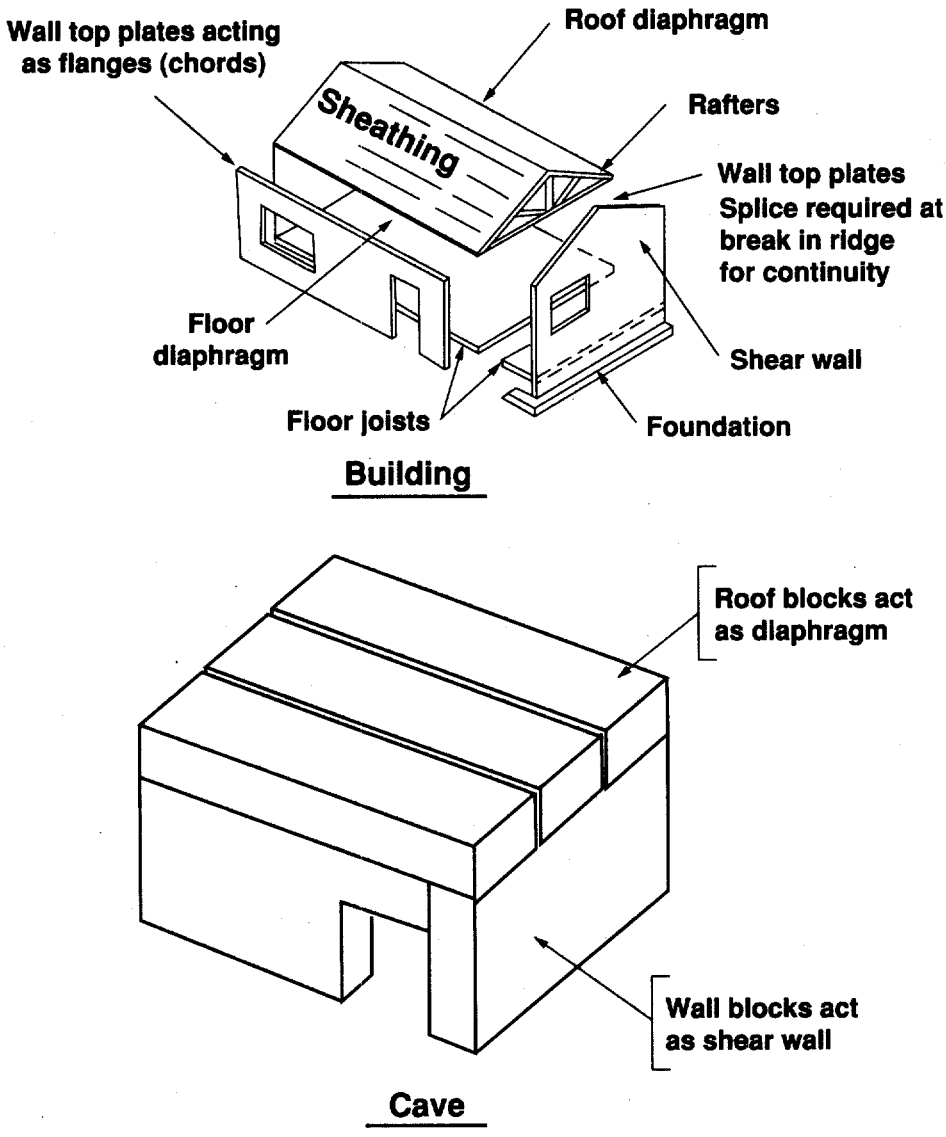


Fig. 10a-19. Cave-building analogy.

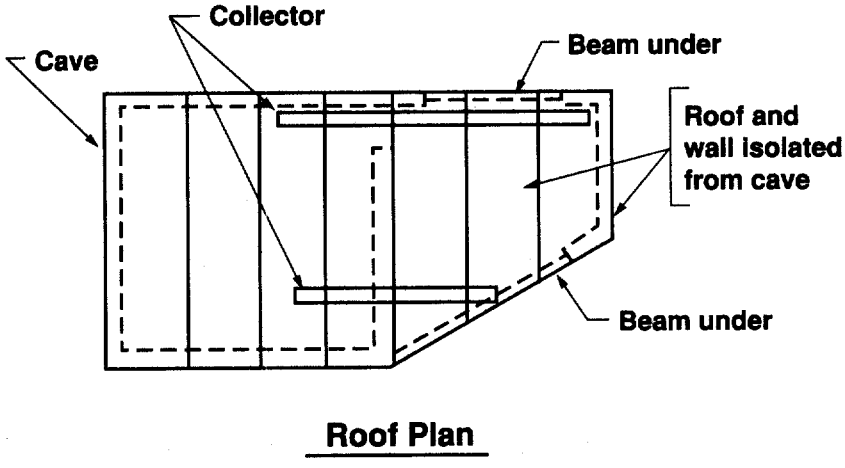


Fig. 10a-20. Load collectors.

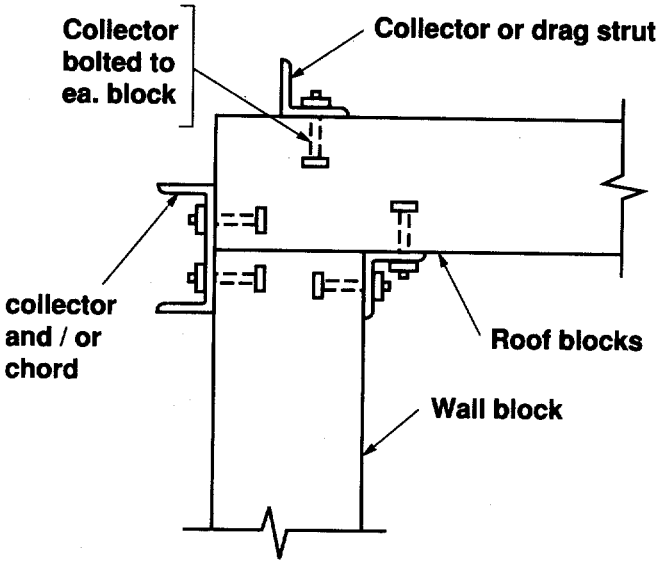


Fig. 10a-21. Cave assembly connections.

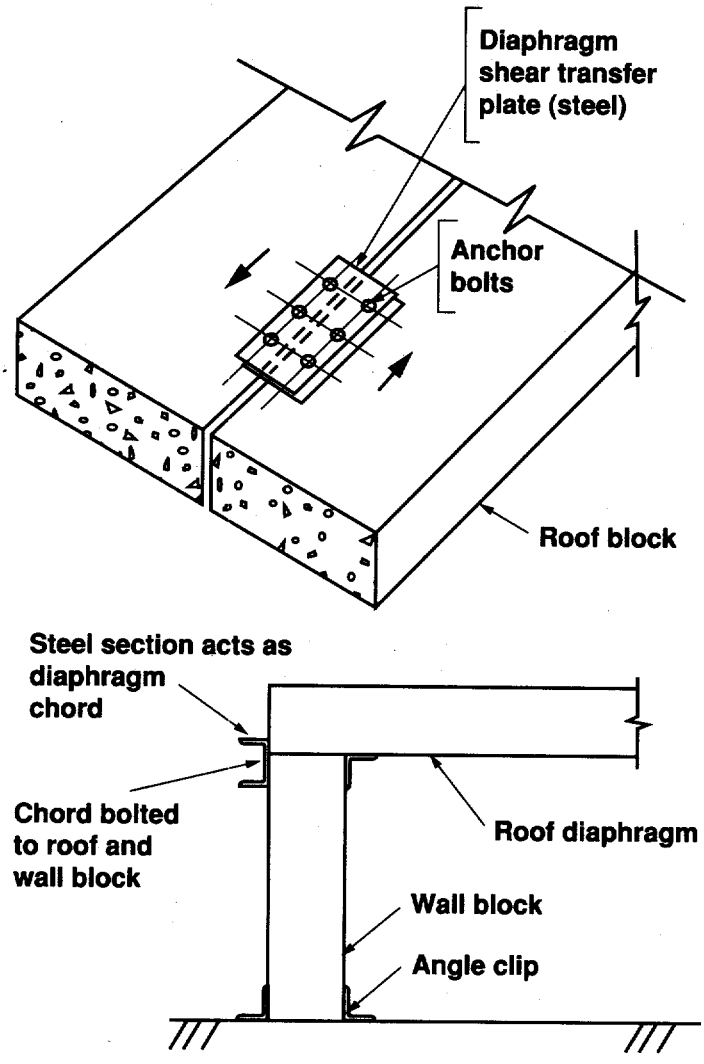


Fig. 10a-22. Shear transfer plates and roof chords.

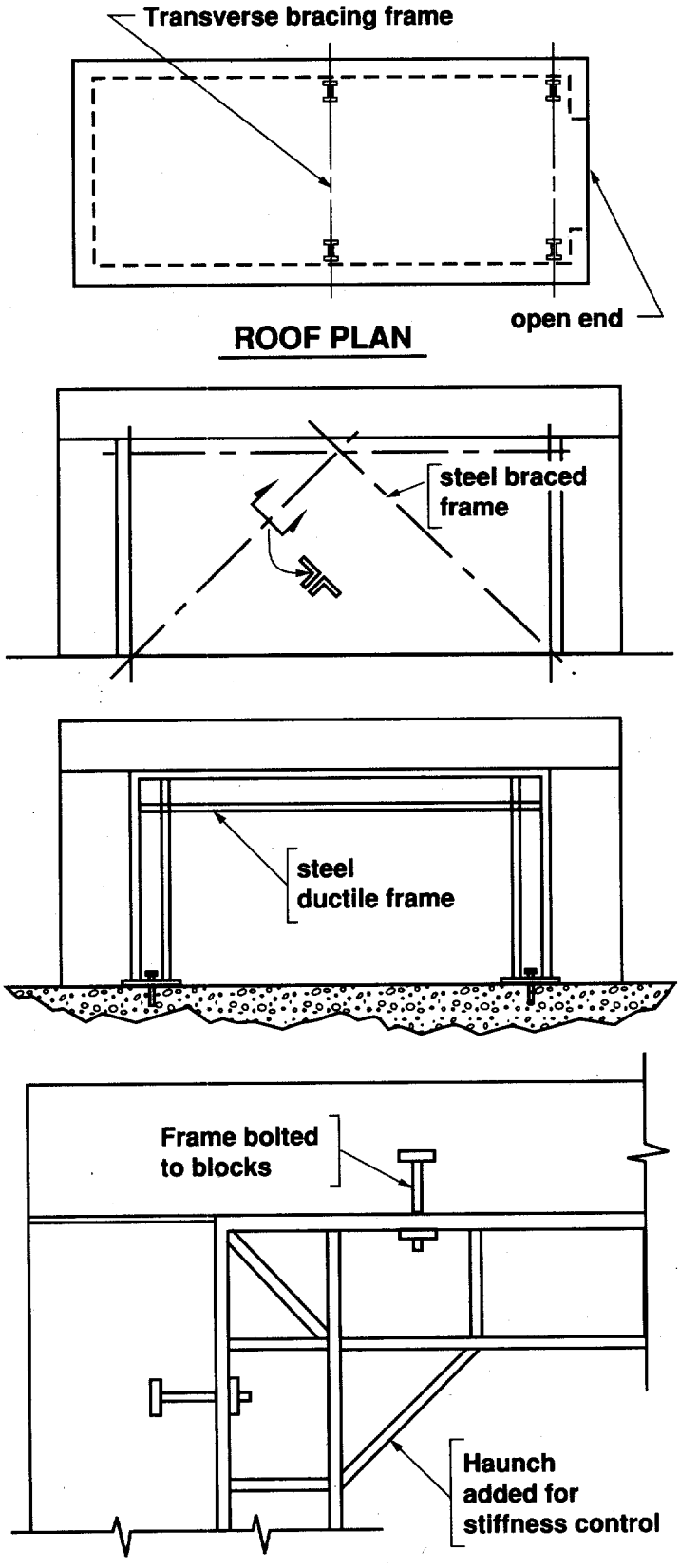


Fig. 10a-23. Supplemental bracing schemes.

analysis is therefore necessary. Usually a stiffer frame is required than would be indicated from a design for moments and shears only.

A concentric diagonally-braced steel frame (Fig. 10a-24), if properly designed and detailed, will also work. It must have sufficient strength to undergo several sudden load reversals during any seismic event; thus, members and connections must be tough enough to resist severe impulse loading. Strength can best be achieved by designing the system to carry as much of the load as possible by members in tension. Toughness can be attained by making the connections of members strong, durable, and capable of developing the full plastic strength of the connected steel members. Regardless of the direction of load (tension or compression) indicated by analysis, connections should be designed to develop the most critical strength of the member used, be it tension or compression, so that severe stress reversals can be accommodated. Thus, the connections will be able to transfer member forces without failing before the member does. It must be assumed that a tension member will be stressed in compression and vice versa; therefore, steel connections should be designed for the actual plastic strength of the member in either mode. Designers should check the building code for special requirements for steel structures resisting forces induced by earthquake motions as related to braced frames, bracing connections, and bracing configurations. Because these columns probably won't carry gravity loads, the strong-column versus weak-beam requirement is not so important.

The eccentric-braced frame is coming more into use in earthquake design. This system adds ductility absent in concentric-braced frames. Energy is absorbed in bending a link beam, lessening the potential for buckling of the diagonal brace. Designers must follow published design procedures and detailing recommendations for the connections of the brace and link beam. The eccentric-braced frame easily adapts to yield and/or ultimate strength analysis.

Some newer developments in earthquake-resistant design include energy-dissipating systems such as base isolation and passive devices that absorb energy by slipping, flexing, and distorting. These systems will result in larger lateral displacement of the isolated structure. Generally, large displacements should be avoided in shielding block installations or accounted for with conservative clearances.

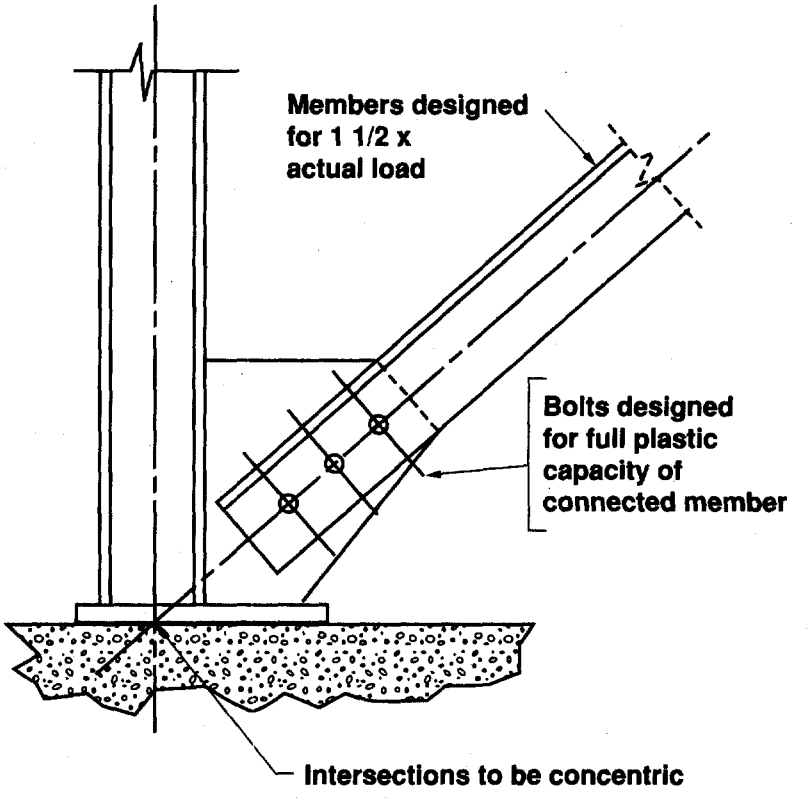


Fig. 10a-24. Concentric brace connection.

Summary of Recommended Earthquake Design Criteria for Shielding Blocks

The earthquake design criteria for bracing concrete shielding blocks should take into consideration site specific seismicity, the type of system used to brace the shielding blocks against earthquake ground motions, and the consequences of failure.

In keeping with Table 10a-1, the formula, $V=2.5\ Z\ I\ C_p\ W$, is used to determine the base shear, V, to be applied for static lateral force analysis and design.

Site seismicity is incorporated by determining the seismic zone in which the site is located from the UBC (Ref. 3) *Seismic Zone Map* (Fig. 10a-6), and selecting the *Seismic Zone Factor*, Z, from Table 10a-1. Alternatively, the effective peak ground acceleration from site-specific hazard curves may be used for Z based upon methodology specified in DOE-STD-1020 (Ref. 4).

The *Importance Factor*, I, is obtained from the appropriate chart in Table 10a-1. This selection should be in keeping with the performance categories described in Reference 4.

The lateral force factor, C_p, varies with the type of bracing system and within the system, by the ductility (or lack of) of each component. For ductile bracing systems, connections, and/or components that are constructed of materials (such as structural steel) that exhibit ductile nonlinear behavior at stresses at or beyond their yield points, a value of $C_p = 0.5$ is recommended, using the actual yield stresses of the ductile materials in the earthquake-resisting system. Diagonal bracing members and their connections should be designed to resist forces resulting from a base shear increased by a load modification factor of 1.5 times 0.50, or a $C_p = 0.75$, to obtain a greater strength and safety factor against buckling. Ductile diagonal bracing members and their connections should be designed utilizing yield stresses.

For nonductile systems and connections constructed of structural materials that do not exhibit reserve strain energy capacity resulting from nonlinear behavior, like non-ductile reinforced concrete, a value of $C_p = 0.7$ is recommended, utilizing ultimate-stress values reduced by an appropriate capacity reduction factor. For normal reinforced concrete, it is recommended that a capacity-reduction factor of 0.85 be applied to the ultimate strength values of concrete for bending, shear, and compressive stresses and 0.6 for bearing stresses.

For the design of anchor bolts or proprietary expansion anchors that depend on concrete for their ultimate-load capacity, use 0.75 times the manufacturer's ultimate-load values (based on load tests), with proper consideration for edge distance and embedment of the anchors. Anchor bolts and shear pins should be designed to be tough enough to allow the beneficial dissipation of energy by the distortion of the steel elements of the connection before failure of the anchorage.

References

1. *Sliding Response of Rigid Bodies to Earthquake Motions*, M. Aslam, W.G. Godden, D.T. Scalise, LBL-3868, Lawrence Berkeley Laboratory, Berkeley, California, 1975.

2. *Rocking and Overturning Response of Rigid Bodies to Earthquake Motion*, M. Aslam, W.G. Godden, D.T. Scalise, LBL 7539, Lawrence Berkeley Laboratory, Berkeley, California, 1978.

3. *Uniform Building Code*, International Conference of Building Officials Whittier, California, 1994.

4. *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities*, DOE-STD-1020-94, U.S. Department of Energy, Washington, D.C., 1994.

5. *Guide to the Design of Anchor Bolts and Other Steel Imbedments*, R.W. Cannon, D.A. Godfrey, and F.L. Moreadith, Concrete International, 1981.

6. *Anchors in Concrete, Design and Behavior*, G.A. Senkin, H.B. Lancelot, III Editor, 1991.

7. *Code Requirements for Nuclear Safety, Related Concrete Structures*, ACI-349, Appendix B.

Chapter

11

Emergency Planning for Earthquake Safety

FOREWORD: The Facilities Manager's Perspective

Donald G. Eagling

Most DOE sites have emergency organizations with well-trained professionals and specialized equipment to handle any type of accident, injury, or hazard on very short notice. Often immediate communication with medical professionals is available within the organization. Limited medical facilities are normally situated on site, available within minutes for treatment of personal injuries. In a major disaster such as a large earthquake, however, the multiplicity of emergencies and injuries to be dealt with simultaneously will overwhelm these special capabilities. In a destructive earthquake, lifelines such as communications systems, energy and transportation arteries, water, and fire-protection systems may be damaged or disrupted. Buildings sustain structural damage. Nonstructural building elements, such as partitions, hung ceilings, light fixtures, heating ducts, and overhead pipes, may fall into building corridors and impede access and egress. Flammable gases, chemicals, and other hazardous materials may leak or spill. Fires may develop.

The aftermath of a major earthquake presents a very different situation than most emergency teams generally face. It calls for a different approach to emergency planning. Self-help is a key element in large-scale emergency response, and preparedness is the *preventive medicine* that reduces the magnitude of the problem.

The most effective stimulus to producing an earthquake preparedness program is a visit by the *master inspector*—the real earthquake. Obviously, this approach can be very costly and is not recommended. A much more practical technique is to develop a model or scenario for the situation that will probably exist in the aftermath of a damaging earthquake.

In the early 1970s, the *National Oceanic and Atmospheric Administration* (NOAA) studied the Los Angeles and San Francisco, California regions to provide earthquake scenarios for emergency planning. The exercise was eye-opening and alarming. As a consequence, many major improvements have taken place that will save the lives of thousands of people when the *big one* takes place. The *scenario* technique, to be effective, must be a practical exercise. The approach involves utilizing the professional judgment of experienced earthquake engineers to produce a likely model of the aftermath of a damaging earthquake.

The damaging 1989 Loma Prieta, California earthquake, an event of magnitude 7.1, was centered about 60 miles south of San Francisco and the Cypress Street Viaduct which collapsed in Oakland. Although the damage was widespread, heavy shaking was attenuated from ground accelerations of approximately 0.65g near

the epicenter to accelerations in the range of 0.10g to 0.25g in the immediate San Francisco Bay Area. Consequently, the Loma Prieta earthquake was not the *big one* for either San Francisco or the Oakland area. On the other hand *the master inspector* proved the value of the 1970 NOAA study and particularly its methodology. This was most evident to those who benefited from its use and those who failed to heed it.

Detailed analysis and time-consuming research is not recommended. The idea is to assume that the entire region is heavily shaken by an earthquake of long duration, and then systematically consider what will probably happen to lifelines: transportation systems such as roads, railroads, bridges, and airports; utility systems such as water, natural gas, and power supplies; communication systems such as radio and telephone facilities; and emergency-recovery facilities such as hospitals, clinics, fire stations, police stations, command centers, and associated equipment.

Locally, the probable condition of on-site buildings and support facilities, roads, emergency equipment, municipal water supplies, water-supply tanks and pumping stations, etc., can be predicted on a judgment basis. The probable condition of the site also can be anticipated. Is fault movement likely? Are there areas of poorly compacted granular soil deposits that may subside during heavy shaking or become subject to liquefaction? Are hillside areas likely to fail in landslides? What is the potential for flooding from water storage facilities? What is the potential for off-site contamination by hazardous materials? Where are personal injuries likely to occur? Will certain areas of the site be isolated from others? How safe are garage facilities that house ambulances and fire engines? Will the water supply be vulnerable to loss when it is needed to fight fire?

The object is to make an educated *estimate* of the multiplicity of conditions and obstacles that emergency response teams may face in the aftermath of an earthquake. The task must be simplified by the heavy use of judgment; otherwise, the development of the scenario can become overwhelming and too time-consuming and costly to be practical. Fortunately, there are a number of experienced *earthquake chasers*—structural engineers, geotechnical engineers, geologists and seismologists—who can supply this type of service effectively and economically, provided that they are properly directed to keep the process simplified.

The scenario technique effectively defines the problems and usually adds new perspective to emergency response planning. The results are often surprising. The need to focus on self-help becomes more realistic. Many new problems become evident. Some have simple solutions. Other hazards can be mitigated, but not eliminated. Priorities are easier to resolve.

Unanticipated events occur in almost every destructive earthquake. Seismic performance of individual buildings and other vulnerable facilities can be stated only in a probabilistic sense. Time of day, weather, and season have significant effects on vulnerability, injuries, and emergency-response capabilities. Detail and accuracy are not so important in the process as is the insight gained for the emergency organizations that will be called upon in the aftermath of the earthquake.

Generally, effective response to widespread damage and injury will require considerable coordination of the usual emergency resources such as environmental health and safety crews, police, firemen, medical personnel, mechanics and craftsmen, equipment operators, communications technicians, plant facilities engineers, and management. The necessity for broad interaction is one of the special problems posed by earthquake emergencies. Once a reasonable scenario is developed, a good approach is to appoint an *emergency preparedness committee* made up of line managers responsible for these various emergency organizations. These people have the special expertise, the resources, and the will to cause preparedness to happen. They will be practical because they will be in the *trenches* when recovery from disaster is required and they must coordinate a response.

When an earthquake strikes, the multiplicity of problems that results is widespread and sudden. The need to know what has happened is of paramount importance, and time is of the essence. Generally, communications systems have serious problems, just when they are needed most. Telephone lines become overloaded and unavailable for emergency use. Relay transmitters for radio pagers often tip over or become disconnected from their power sources. Public address systems often lose house power and become useless if emergency generator or standby battery systems also are damaged and

fail to function. Usually people run out of buildings, so the normal internal public address systems cannot reach them.

Many of these problems can be remedied by modifying existing systems. The local telephone company often has lineload control that can be instituted to free certain predesignated telephones from the overload condition. Generally, by working with the telephone company, arrangements can be made to institute lineload control locally, but it is essential to settle exactly who will make the decision when it is needed. Obviously the telephone center on site and its standby battery racks must be tied down to ensure that it will not be damaged and made inoperative by the earthquake. If underground telephone service lines cross an active fault, precautions can be taken to provide slack and flexibility at the crossing to prevent damage.

Transmitters and antennas may be inadequately tied down or poorly braced against overturning. It is generally a very simple matter to correct this weak link.

Emergency generator circuits should be reviewed to ensure that lifeline communications will stay on-line when public power supplies are lost. The fuel used in emergency generation must not be susceptible to loss. For example, natural gas systems should not be relied upon as backup systems. The generators themselves must be tied down and emergency fuels stored handily nearby.

Public address system speakers can be strategically located outside buildings to reach predetermined gathering spots. *Bull horns* and radios can be made available to building managers and other key personnel who may be an important part of the emergency response communication chain. When other means are not available, the use of *runners* to carry information becomes necessary.

Of course it is essential to harden the usual emergency communication centers available at most sites. The police or security command center and the fire-station command center are obvious examples. When a widespread emergency exists, time is of the essence, so the predesignation of a principal command center with adequate conference room facilities, technical files, maps, and emergency plans is most important. Generally, needs will be greater than resources. Coordination of available resources for recovery is highly dependent on priority control by responsible and knowledgeable managers.

Self-help planning, preparation, and training should be a key element in any emergency response plan for earthquake safety. Although it is essential that the framework for a self-help organization be established by management, ultimate success will depend on the participation of those having local authority and responsibility for well-defined areas of activity and/or locations. It is most important that these individuals are clearly designated and are fully involved in all development work associated with self-help plans; alternates should be designated for each individual *authority*.

Emergency plans must be kept very simple and concise to be effective. People will not read or use long, complex plans. Where possible, reference documents for use during an emergency should be written in the form of checklists. Each designated responsibility or authority should be identified by a generic or functional term rather than a person's name; e.g., building manager. Checklists should be tailored to each role, not generalized to encompass divergent roles. Each checklist should clearly identify responsibilities and locations of necessary tools and supplies. As mentioned above, more than one individual should be designated for each functional role established in the emergency plan. Also, the equipment to be utilized by these individuals should be similarly identified, i.e., a *hard hat* for the building manager should clearly identify that person's title and the building for which he/she is responsible. In action, the *hard hat* identifies functional authority, and unfamiliar faces will not confuse the players.

Communications will be difficult immediately after an earthquake. Just keeping track of information will be a problem. Often the noise level is so great that communication in command centers becomes very difficult. This should be carefully considered in the layout and organization of the functions that must take place. Radios, telephones, speakers, and the individuals who must communicate within the command center need some sound isolation or separation. Information boards and maps need similar consideration. These interactions should be tested in realistic drills to work out the bugs, if possible, before the command center plan is solidified.

Test drills, like the scenario technique for modeling the aftermath of an earthquake, are very effective in bringing a plan for emergency response to a realistic and practical level.

Often professional emergency organizations are reluctant to use volunteers for back-up. Generally, this attitude is a valid one for most individual emergency situations, but in a widespread earthquake extra help is essential. The emergency plan should include designated response teams as support for the professional emergency organizations. Individuals trained in first aid, strong people who can become stretcher bearers, traffic coordinators, runners to assist in communications, *ham* operators, individuals trained in the use of fire-fighting equipment, and people capable of hard physical labor will be needed. Predesignated individuals and locations for reporting should be part of the emergency plan, along with at least minimal training for the jobs.

Inevitably, a major earthquake will be followed by aftershocks that can be a serious hazard for buildings structurally damaged by the main shock. A quick assessment of building safety is always a high-priority task immediately after the earthquake. In addition to predesignating responsibilities for structural review, simplified key plans should be developed for each building to visually identify its lateral-force-resisting system. In this way, if structural engineers cannot make the first quick assessment of quake damage, less qualified individuals can be used to flag damage that seems critical for earthquake resistance.

Generally, the main emergency command center will be separately located from the communication centers for the professional emergency organizations such as the fire department, security or plant protection, safety services, medical clinic, craft shops, facilities engineering, and transportation. This will necessitate a great deal of communication at the main command center to coordinate the overall emergency response.

The number of individuals designated to operate within the emergency command center should be kept to a minimum to reduce confusion and facilitate communication and coordination among the participants who must make command decisions. On the other hand, people in the inner circle have a strong need for staff support. For example, the heavy flow of communication from various and widespread sources will create a need to funnel information into the center without causing a bottleneck. Two or three people may be needed continuously to write down messages delivered by runners and provide carbon copies to those who need them and for file. Other persons must be available to record on display boards such incoming information as the locations of injuries, fires, water and gas leaks, building damage, and other problems that may require action. Display maps with clear acetate covers for grease pencils may be used with a color code to categorize the problem, e.g., red for fire and blue for water leaks.

A communication board or message center will be needed to list those who have been called or contacted. Communicators will be needed to send messages out of the center after decisions are made. Walkie-talkies and ham-radio operators with portable equipment and rechargeable batteries are another very useful resource for communication.

It is extremely important that the functioning of a centralized Emergency Command Center or *Emergency Operations Center* (EOC) does not bog down field operations by forcing all communications to go through the EOC *Commander*. Field supervisors for technical operations responding to hazardous materials spills, building damage, fires, injuries, etc., should have direct radio communication with their technical counterparts (usually department heads) in the EOC. If every message must go through the EOC Commander, the resulting bottleneck stifles the timely flow of emergency communication and interaction between technical leadership in the field and the EOC. Technical department heads who are key members of the EOC staff can provide the EOC Commander with timely, high-quality information only if they are personally knowledgeable of the status of field operations in their areas of expertise. As well, the quality of technical communication between experts should not be unnecessarily diluted by nontechnical relay through an EOC commander whose time should be reserved for management of the overall emergency.

Support personnel will also be needed to handle public information interactions. Invariably after a damaging earthquake, many visitors show up at the scene of damage: reporters, engineers, geologists, seismologists, representatives of public agencies, politicians,

and interested citizens. It is important that this heavy influx does not interfere with the operations of the *Emergency Command Center*. The facility for interaction with visitors should be located somewhere else. Many of the early visitors will be professionals who are capable and willing to assist in support activities. In particular, structural engineers can be extremely useful in assessing building damage, and usually they are well organized professionally to respond to this need. If utilized properly, this assistance is both invaluable and economical in the early stages of response and recovery.

The Earthquake Engineering Research Institute, (EERI), 499 14th Street, Suite 320, Oakland, CA 94612-1934; phone (510) 451-0905, FAX (510) 451-5411, is probably the best resource for pre-planning the use of outside help after an earthquake. The institute is a nonprofit organization that is devoted to finding better ways to protect people and property from earthquake hazards. It is best known for its field investigations of destructive earthquakes. Included in its membership are leading U.S. earthquake investigators from all relevant fields. The EERI has set up volunteer response teams and pre-arranged a methodology for coordination of assistance and investigations in the immediate aftermath of earthquakes.

In 1996, the EERI published a document entitled *Post-Earthquake Investigation Field Guide*. The intent of the publication is to provide plans, procedures, and checklists for field investigations by interdisciplinary professionals to maximize the opportunity for learning in the immediate aftermath of future earthquakes. It covers engineering, geoscience, and social science aspects of earthquakes. The format consists of short commentaries under most specific subjects, followed by checklists. The commentaries summarize lessons learned from past earthquakes, and the checklists provide guidance for investigating new earthquakes. This document is a rich source of information upon which to plan for emergency recovery from earthquakes. In particular, the checklists are recommended for reference in the mitigation of seismic hazards before an earthquake takes place.

Chapter 3b, *Assessment of Damage*, provides a practical discussion of a comprehensive damage assessment program for the sequence of events following an earthquake.

Serious preparation for widespread emergencies should include acquisition and strategic storage of special tools, equipment, fuels, and supplies that may be needed in early recovery operations. For example, breaks in water supply and distribution lines will require emergency repairs or temporary bypasses to get fire protection systems back in service. This can be quickly accomplished if emergency cross-over connections with adapter fittings for *plain-end* water pipe and hose risers to fit standard fire hose are on hand. These emergency cross-over connections can be easily prefabricated using standard rod and socket clamps to fit all sizes of water mains and stored with 2-1/2 inch standard fire hose in 50-foot lengths to provide flexibility to quickly reconnect across breaks of any span up to 600 feet. Similarly, emergency cross-over connections can be prefabricated for natural gas mains.

Tools generally needed in earthquakes (such as shovels, axes, crowbars, *jaws-of-life* cutters, saws, and insulated gloves) can be stored in multiple locations in keeping with the need for *self-help* when widespread damage occurs. Similarly, first-aid and medical supplies can be located in facilities that are safe and suited for use as alternative medical centers.

Natural gas and Liquified Petroleum Gas (LPG) systems pose special explosion hazards after damaging earthquakes. The most effective measure to mitigate the hazard is to install earthquake shut-off valves in the main distribution lines. Placing a single such valve in the main is important, but still leaves too much gas volume and potential for explosion in the distribution system. Similar valves should also be installed at other strategic points.

It is important to look at the potential for loss of water supplies, including those from external sources. Fire protection sprinkler systems are of little value if water service or storage is lost. Where the potential for loss of outside service is significant, the installation of on-site water storage and emergency pumping stations should be seriously considered.

Realistic drills to test earthquake emergency planning are very important. One of the best ways to ensure that such a drill will be effective is to utilize experienced earthquake investigators to review the site plan and develop a damage and injury scenario by which to test the plan. Again, one of the best sources to contact for a

recommendation for a list of such consultants is the *Earthquake Engineering Research Institute*.

Chapter 11a, discusses *Lifelines Considerations and Fire Potential*. A careful reading of Chapter 11a should instill in managers a healthy respect for the potential effects of off-site utility systems upon emergency and recovery operations on-site. These effects should be part of the *scenario* used to develop the site emergency plan.

In Chapter 11b, the *Multihazard Emergency Response Plan* is discussed in generalized terms and principles from the perspective of one who has long experience in public and private sectors in emergency planning and the development and implementation of emergency management systems and training programs. DOE requirements and related guidelines are not discussed in Chapter 11b, but are specifically set forth in DOE Order 5500.3A, *Planning and Preparedness for Operational Emergencies*, 1991. It covers hazard assessments, emergency response organizations, off-site response interfaces, classes of emergencies, notification requirements, consequence assessments, protective actions, medical support, recovery, public information, emergency facilities requirements, training and drills. Also, DOE's *Emergency Management Guide* (1991) specifies a Standard Format and Content for Emergency Plans for DOE facilities.

Chapter

11a

Lifeline Considerations and Fire Potential

John Eidinger

Introduction

The purpose of this chapter is to provide readers with a basic understanding of the earthquake performance of lifelines. Lifelines include water distribution and sewerage, transportation, gas and liquid fuel, electric power, and communications infrastructure. This is too large an array to cover in detail within the limited text of this chapter. Therefore, while all lifelines will be described in general terms, detailed discussion will concentrate on water systems, whose failure in the aftermath of an earthquake can be extremely hazardous. The discussion will cover the high vulnerability of water systems to earthquake damage, the risk of postearthquake fire, the potential for fire conflagrations, and fire protection design philosophy.

One might think that managers of DOE sites would not need to worry about lifelines and lifeline earthquake engineering provided by off-site agencies because these lifelines (for the most part) are located outside the perimeter of the site and owned and operated by others (the lifeline utilities). Facility managers need to know about lifelines for two reasons. First, most DOE facilities have considerable lifeline infrastructure on site, and the seismic performance of these lifelines will affect overall site performance. Second, facility managers must depend, to varying degrees, on the off-site lifelines to support on-site activities. Managers need to consider both on-site and off-site lifeline performance. If off-site lifeline performance is expected to be inadequate, then facility managers may need to provide on-site lifeline redundancies. Further, on-site lifeline infrastructure needs to be suitably designed.

As an example of the postearthquake importance of lifelines, consider the case of a hospital. Most hospitals in *earthquake country* are designed to a high level of earthquake resistance and are equipped with backup power systems. However, is postearthquake functionality of these hospitals really guaranteed? Will off-site communication facilities (like microwave towers) become misaligned or fail in an earthquake because of unanchored batteries rendering the hospital's dispatch system out of service? Will failures in the off-site sewer system contaminate the off-site potable water system, leading to loss of drinking water at the hospital? Will fires break out due to off-site gas main failures forcing evacuation of the hospital? Will the off-site water distribution system remain sufficiently intact to provide delivery of water to nearby fire areas? Will off-site electric-system outages prevent the water department's pumps from working, thereby halting or significantly reducing water flows to fire hydrants? Will failures in the off-site transportation network make it impossible for

fire departments to get their apparatus to the scene of the fire? Everyone of these lifeline vulnerabilities has occurred in past earthquakes. Therefore, it is prudent that facility managers plan for these potential impacts.

Facility managers should know the roles various lifelines have on site facilities in order to plan for these impacts. With this knowledge, they can judge the impact on facility operations if extended outages of various lifelines occur. The possibility of fires following earthquakes should be evaluated, as well as the potential impact on the facility should service from the local water system be unavailable. Finally, managers should consider possible options for mitigating the impacts from such outages. Each of these topics is described in the following text, along with suitable reference material to allow more in-depth study.

If off-site lifeline disruptions will cause impacts at unacceptably high levels, facility managers should consider mitigating the impacts, either by providing an on-site backup lifeline (possibly at considerable cost), or working with the off-site lifeline agency to improve postearthquake service to the managers' facilities (a choice that should become more practical in the future). Alternatively (in some cases), facility managers could simply plan to live with the consequences, especially if the risk-weighted benefit of mitigation is small.

Overview Of Lifeline Performance in Past Earthquakes

The field of lifeline earthquake engineering was probably formalized with the founding of the *Technical Council on Lifeline Earthquake Engineering* (TCLEE) of the *American Society of Civil Engineers* (ASCE). TCLEE was formed in 1974 in the aftermath of the 1971 San Fernando, California earthquake, which caused widespread damage to many lifeline systems.

Since its inception, TCLEE has sponsored three conferences on lifeline earthquake engineering (Ref. 1,2,3) and has issued several monographs (Ref. 4,5,6). Through TCLEE, as well as many other agencies and researchers, a large body of information is now available in the literature.

Probably the most global way of looking at the performance of lifelines in earthquakes is to estimate how long after an earthquake it will take to restore pre-earthquake levels of service. In the following sections, past earthquake performance of water distribution systems is reviewed in some detail; the performance of other types of lifelines is briefly described; and simple guidelines are suggested for estimating potential lifeline outages at the site of a particular facility.

Earthquake Performance of Water Systems

The following paragraphs, parts of which were adapted from Refs. 7 and 8, summarize the types of damage and service outages that have occurred in some past (and projected for some future) earthquakes to water-distribution systems.

The 1906 San Francisco, California earthquake caused extensive damage to the city's municipal water distribution system. Because of broken pipes, water was unavailable in the built-up area. Over the course of three days, small fires that were not extinguished immediately after the main shock grew into conflagrations. The result was that more than 400 city blocks were completely destroyed by fire.

As a consequence, San Francisco constructed an auxiliary water system to supplement the municipal water system that had failed. This system was constructed to be earthquake resistant (as well as could be expected for the early 1900s) so that it could be relied upon to provide water in the aftermath of future earthquakes. However, experience in the 1989 Loma Prieta, California earthquake proved otherwise. In that event, both the municipal and the auxiliary water systems were sufficiently damaged so that areas of San Francisco were again without water. The city was fortunate that there was no wind the evening of the earthquake to spread fires that were ignited. A third water system, made up of portable aboveground water hoses and fire boats for pumping water from the San Francisco Bay, was instrumental in putting out the fires that did ignite.

The 1906 earthquake also prompted the City of Oakland (on the eastern side of the San Francisco Bay) to build an auxiliary water-supply system to serve its downtown areas. Currently, that auxiliary system is no longer operational because it was taken out of service at the time the underground Bay Area Rapid Transit subway system was built through Oakland. Of interest, two cities (Vancouver, Canada and Berkeley, California) have recently begun design of new dedicated fire fighting high-pressure water systems for postearthquake and conflagration fire purposes. Using modern seismic design techniques, these new systems are designed to reliably provide fire flows after the occurrence of large earthquakes.

The East Bay Municipal Utility District's (EBMUD) potable and raw water distribution systems that now serve Oakland and 16 other East Bay communities also were damaged in the 1989 Loma Prieta earthquake. About 130 major pipeline breaks occurred, along with a similar number of service breaks. EBMUD was able to restore water service to essentially all customers within a few days, although this level of damage taxed its maintenance crews to the limit. Most pipeline breaks occurred in the bay mud along shoreline regions of eastern San Francisco Bay. Some notable exceptions included the failure of a 60-inch-diameter concrete-reinforced welded-steel pipe in an area well inland from the shoreline area and away from areas of permanent ground deformations. In that area, ground accelerations were about 0.10g or less. Postearthquake investigations of this pipe showed that poor weld quality was a contributing factor that caused it to break as a result of wave propagation. Similarly, a number of 25-year-old, small-diameter, welded-steel distribution pipes (6-inch and 8-inch diameter) broke in shoreline areas that experienced lateral ground spreading. Postearthquake inspections found that some of these smaller-diameter pipes suffered from corrosion and some failed because of poor field weld quality, which was in turn a function of the type of mix used in the cement mortar lining. The lessons learned from these failures suggest that replacement of segmented lead-jointed cast-iron pipe with continuous welded-steel pipe will not guarantee excellent earthquake performance.

A system model (Ref. 9) of EBMUD's water-distribution system has projected its performance in future earthquakes. The model suggests that a Hayward fault magnitude 7 event, which would result in surface faulting through the middle of the EBMUD system, could cause service outages in parts of the system as long as several months. Three other possible scenario earthquakes, a Hayward fault magnitude 6, Calaveras fault magnitude 6.75, and a Concord fault magnitude 6.5, could similarly cause local service outages of several weeks. EBMUD is now embarking on an upgrading program to improve its postearthquake level of recovery and service.

During the 1989 Loma Prieta earthquake, there were substantial service interruptions to other water systems in the epicentral area. For example, the higher pressure zones of the Santa Cruz water system were quickly drained because of extensive pipeline damage in the soft soil areas along the San Lorenzo River. This resulted in not being able to provide water service to two local hospitals. A concurrent electric power outage prevented the water utility from pumping raw water to its treatment plant serving the area. It was extremely fortunate that there was no wind that evening to spread fire. Water supply to some parts of the city was not restored for up to one week.

Five water tanks collapsed in the San Lorenzo Water District immediately north of Santa Cruz. A one million-gallon tank drained in Scotts Valley, just east of Santa Cruz, when it rocked on its foundation and snapped the connecting piping. Service from the Redwood Estates water system, located in the Santa Cruz mountains near the epicenter, was not restored until five months following the earthquake.

Water treatment plants also were damaged in the Loma Prieta earthquake. Process equipment and baffles were broken up by sloshing water in treatment plants located in the San Jose and Santa Clara Valleys, putting them out of service for up to one month. Because the earthquake occurred in October, after the peak summer water demand, water suppliers could still keep up with demand.

In Washington State, the 1949 Magnitude-7.1 earthquake broke water lines leaving the city of Olympia, the state capital, without water for one day (Ref. 8).

In 1965, a magnitude 6.5 earthquake broke water lines in Seattle, leaving one waterfront area without water service. A recent model (Ref. 8) for Seattle predicts that the city would be without water for up to 20 days in a future magnitude 8.5 event (located 100 km from Seattle), or 9 days after a future magnitude 7.5 event located near the Seattle–Tacoma International Airport.

In January 1994, the magnitude 6.7 Northridge, California earthquake caused serious disruptions to water service in the San Fernando Valley area of Los Angeles. This earthquake caused about 1,500 breaks in the Los Angeles water system, as well as a smaller number of breaks in neighboring districts' water systems. The lack of significant liquefaction over most of the affected area helped keep the total number of pipe repairs to less than 2,000.

In the Los Angeles system, water service was restored to all customers 12 days after the earthquake. This service schedule was accomplished by using a large number of repair crews from the utility's own repair crew force, as well as mobilizing a similar sized repair crew force through mutual aid from water agencies in unaffected areas.

The Northridge earthquake provided very good empirical observations how different types of buried pipe performed under the same ground deformations. On one street, Balboa Boulevard, there were 6 parallel welded steel pipelines; 2 large water pipelines (over 48" diameter); 3 medium gas and 1 medium oil pipelines (12" - 24" diameter). All pipelines were subjected to soil failures at two locations; one where the soil *spread* putting the pipes into tension, and one where the soil *compacted* putting the pipes into compression. At both locations, permanent ground deformations were about 1 foot. In terms of performance, preliminary investigations found that both water pipelines broke; 1 gas pipeline broke; and the remaining pipes did not break. The weld types for the gas pipeline that failed were of the pre-1930 style *gas* welds, which are known to be relatively vulnerable. The differing performance of the other pipelines is presumed due to differing types of welded joints (lap versus butt welds).

The Northridge earthquake also demonstrated that water storage tanks, particularly those that are unanchored, are subject to a variety of failure mechanisms. Unanchored water tanks with flexible pipe connections did well at sites having peak ground accelerations under 0.15g. Damage did occur at (mostly) unanchored steel tanks at sites with higher accelerations, including damage to attached pipes (4 tanks), significant roof damage (3 tanks), loss (or suspected loss, as tank was empty at time of inspection) of water contents (7 tanks), and damage to anchor bolts at one anchored tank. Erosion of soil near tanks that lost their contents, and downhill inundation of structures were observed.

Performance of Other Lifelines

For lifelines built in the United States, a general ranking of postearthquake vulnerability is as shown in Table 11a-1, in order (roughly) from most vulnerable to least vulnerable:

Water systems (most vulnerable)

Sewer systems

Transportation systems

Gas systems

Electric systems

Communication systems (least vulnerable).

This general order was confirmed in a recent study conducted for six types of lifelines in the Everett, Washington, area (Ref. 10). Given the current infrastructure in that area and the estimated capability to repair such infrastructure after future earthquakes, Table 11a-1 provides the predicted service outages for selected customers. The three earthquakes listed in Table 11a-1 represent three possible scenario events. The Puget Trough event is a nearby shallow earthquake (12-kilometer hypocentral distance), whereas the Benioff Interplate events are indicative of moderately distant deep subduction earthquakes (50 and 75 kilometers, respectively).

The trends predicted in Table 11a-1 for future earthquakes have been true for past earthquakes. For example, the longest lifeline outages resulting from the 1989 Loma Prieta earthquake in the San Francisco Bay Area were water and highway bridge lifelines (several weeks). Electric power outages were on the

Table 11a-1. Lifeline service outages in future earthquakes, Everett, Washington.

Earthquake	Puget Trough Magnitude 6.5	Benioff interplate Magnitude 7.0	Benioff interplate Magnitude 8.25
Lifeline	(Days)	(Days)	(Days)
Water	7	2	6
Sewer	7	2	6
Highway bridges	7	1	7
Natural gas	1	0.6	2
Electric power	0.3	0.04	0.08
Telecommunications	0	0	0

order of days, and telecommunication outages were generally a matter of hours, if at all. Experience from other earthquakes confirms these trends. It should also be noted that Table 11a-1 reflects the time to restore a reasonable level of service, sometimes without the same level of redundancy as available before the earthquake.

With respect to highway bridges, some interpretation is needed relative to what constitutes the time to restore service. The 1989 Loma Prieta earthquake damaged the Bay Bridge, which required 30 days to repair, destroyed the Cypress structure (subsequently torn down, and scheduled to be replaced by 1998) and several elevated viaducts in San Francisco, none of which have been completely repaired and put back in service as of early 1995. In contrast, the 1994 Northridge earthquake damaged several (albeit smaller) overpass bridges, all of which were repaired and put back in service within 1 year. The interpretations of the time to restore transportation service thus depends heavily on the availability of alternative detours (in the short term), and the reconstruction effort required (in the long term).

With respect to telecommunication service, restoration depends upon two factors: damage to the hardware (which often has been modest in past California earthquakes), and consideration of the large increase in demand for such service immediately after the earthquake. Table 11a-1 ignores the latter factor. In practice, even if there is no seismic damage to the telecommunication system, the large increase in demand after the earthquake will overtax the hardware's ability to make connections, and *apparent* service to the user will be poor until such time that service demand drops off to a level that the hardware can handle. Excessive demand for service usually lasts about 3 days after large earthquakes.

Estimating Future Service Outages

For planning purposes, facility managers should consider each lifeline serving the site and estimate the potential length of service outage to be expected. Ideally, this review should be performed in conjunction with engineers from the local lifeline agency. Basically, the following three questions need to be answered:

- Is an immediate interruption of service at the facility likely?
- Is a long-term interruption of service at the facility likely?
- Is there the potential for a widespread long-term interruption of service as a result of the vulnerability of a critical lifeline component?

Table 11a-2, *Likelihood of immediate lifeline outage*, addresses the first question. It presents the probabilities of service outage for earthquakes of various magnitudes and *peak ground acceleration* (PGA) values.

Because damage to lifelines is often caused by soil failures, the duration of the earthquake motion has an important influence on the

Table 11a-2. Likelihood of immediate lifeline outage.

Earthquake Lifeline	Magnitude 6 - 6.5 PGA Design Level 0.1- 0.3g	Magnitude 7 - 7.5 PGA Design Level 0.2 - 0.6g	Magnitude 8 + PGA Design Level 0.3 - 0.7g
Water	Medium	High	Very high
Sewer	Medium	High	Very high
Highway bridges	Medium	High	Very high
Natural gas	Medium	High	Very high
Electric power	Low	Medium	High
Telecommunications	Very low	Low	Medium

amount of damage to be expected. For example, a magnitude 6 event near a lifeline may produce PGAs of 0.5g and yet be less damaging than a more distant magnitude 8 event producing local PGAs of only 0.25g. Therefore, for planning purposes, one should estimate both the local PGA value and the magnitude of earthquake that controls the PGA value. This involves examining the process that was used to generate a site-specific probabilistic PGA. For initial planning purposes, this can be avoided by using the following simplifications:

- For many West Coast sites, probabilistic site-specific PGA design levels are controlled by nearby magnitude 7 to 8 events
- For many eastern U.S. sites not near known active areas, PGA design levels are controlled by nearby magnitude 6± events
- For Eastern U.S. sites moderately near known active areas, PGA design levels are usually controlled by either nearby magnitude 6± events or more distant magnitude 7 to 8 events.

For example, assume a site in western Texas with an estimated PGA level of 0.2g. It is probable that no nearby tectonic provinces are capable of large-magnitude events. Thus, for this site, it is more likely that the PGA will be the result of a nearby magnitude 6± event. Therefore, the probability of lifeline outages will be best described by the left-hand column in Table 11a-2.

As another example, assume that a site in Kentucky, located about 100 km east of Missouri, has a PGA level of 0.3g. For this site, the PGA level may be partially controlled by the occurrence of a nearby magnitude 6± event and partially controlled by the occurrence of a magnitude 8± event on the moderately distant New Madrid fault. Thus, the probability of lifeline outages would be more conservatively described by the right-hand column in Table 11a-2.

Table 11a-3, *Likelihood of long-term lifeline outage*, answers the second question. *Long term* is meant to be an outage greater than about three days.

It is important when using Table 11a-3 to know if buried lifeline services to the facility pass through areas of locally poor soil conditions or only through areas of good soil conditions. If the lifeline service does pass through areas prone to liquefaction, landslides, or surface faulting, it is much more likely that lengthy service outages will occur.

The length of the service outage will depend primarily upon how quickly the lifeline utility can repair the damage. All things being equal, assuming the facility served is not a *priority* customer, the time to restore service will be directly related to the total number of repairs the lifeline agency must perform throughout the system, and inversely related to the number of repair crews that the lifeline agency has at hand. Other factors,

Table 11a-3. Likelihood of long-term lifeline outage.

Magnitude	6 - 6.5	7 - 7.5	8+
PGA	0.1 - 0.3g	0.2 - 0.4g	0.3 - 0.7g
Soil conditions	Poor / Good	Poor / Good	Poor / Good
Lifeline			
Water	Low / very low	Medium / very low	High / low
Sewer	Low / very low	Medium / very low	High / low
Highway bridges	Low / very low	Medium / very low	High / low
Natural gas	Low / very low	Medium / very low	High / low
Electric power	Very low / very low	Low / very low	Medium / low
Telecommunications	Very low / very low	Low / very low	Low / low

such as inventory of spare parts and machinery, are normally not limiting factors after a few days because they are usually available through mutual aid.

In order to answer the third question, it is important to understand the hardware, soil conditions, and operational practices of the lifeline agency. Normally, an individual customer does not have access to this type of information. It is necessary for the lifeline agency to perform a study, possibly rather involved, before this question can be resolved. The next sections describe how to perform such a study.

It should be emphasized that Tables 11a-2 and 11a-3 are based on experience from past earthquakes. Obviously, the likelihoods provided (very low, low, medium, and high) are only first-level estimates of performance in future earthquakes. System studies described later should provide better estimates.

Seismic Design for Lifelines

From the previous descriptions, it is clear that some lifelines have not performed well in past earthquakes. An important reason for this relatively poor performance is that much of the infrastructure of most as-built lifelines in the United States has been built outside the jurisdiction of a governing seismic code or standard.

In the western United States, parts of lifelines have been built to a seismic code, particularly lifeline building structures. However, performance guidelines for these buildings have usually been based on the *Uniform Building Code* (UBC) philosophy; namely, to prevent loss of life, not to prevent damage. Generally, stringent attention has not been applied to ensure that important components of equipment are properly anchored or that backup power supplies are provided. Recognizing these problems, some lifeline utilities are now designing for postearthquake functionality of building structures that are an integral part of their lifelines. For example, the Portland (Oregon) Water Bureau's new Water Control Center will be the first seismically isolated structure in the Pacific Northwest (Ref. 11). The Water Bureau chose to isolate its new facility because it is essential that the control center remain operational after earthquakes.

Although the buildings of a lifeline utility may have been designed to some level of seismic code, it is quite likely that much of its infrastructure, particularly its distribution system, has not. For example, essentially all water, sewer, and gas distribution systems use segmented buried pipes, many of which date back to the nineteenth century. These pipes are extremely vulnerable to failure in earthquakes.

Today, most lifelines (both in the eastern and western United States) continue to use segmented buried pipe construction for new additions. Some utilities are incorporating seismic resistant design into these newer pipes. However, in a recent survey of 9 California water utilities (Ref. 12), none had specific upgrade policies to replace old pipe with new

pipe for earthquake purposes. A typical utility upgrade policy was *if it breaks, we fix it.* Some utilities have policies to replace older small-diameter pipe (4-inch or smaller) with newer 6- or 8-inch-diameter pipe, implemented on an annual basis of (typically) under 1% of the inventory of such pipe. The major reason for this type of replacement policy is, however, for improvement of water flow, or troublesome localized repair issues, rather than improvement for earthquake purposes.

Some west coast utilities are beginning to adopt programs to improve earthquake performance of both their existing and new buried pipelines. One San Francisco Bay Area agency has a program to replace gas pipelines, primarily for maintenance reasons. Since the 1989 Loma Prieta earthquake, the agency has accelerated the program to incorporate earthquake improvements. Another San Francisco Bay Area water utility has long had a policy to use only welded steel pipe in areas prone to liquefaction or surface faulting; however, as yet it has no program to upgrade the older cast-iron segmented pipes in such areas.

Thus, for lifeline utilities that have begun to incorporate earthquake provisions for distribution hardware, some postearthquake outages are likely, although the duration of such outages should be shorter.

Codes and Standards

One of the main reasons that lifeline utilities do not incorporate seismic design into their distribution systems is that there are no nationally or regionally recognized codes mandating such design. Further, there is little available in terms of guidelines to accomplish such design. However, various industry groups are now making some progress in filling this void.

On the national level, the *Federal Emergency Management Agency* (FEMA) and the *National Institute of Standards and Technology* (NIST) have been chartered to jointly develop a plan for establishing earthquake design standards for lifelines (Ref. 13). This plan is envisioned to take several years to carry out.

On a state level, the *California Seismic Safety Commission* has developed initiatives for the earthquake performance of various lifelines (Ref. 14). These initiatives reflect the state of preparedness of the larger California electric utilities, but are not otherwise reflected in the lifeline industry as a whole. There is some interest in merging the California and federal lifeline efforts.

Oregon has established a *Seismic Safety Policy Advisory Commission.* As part of that work, a lifelines position paper has been drafted (Ref. 15).

One of the key industry groups that has focused attention on the matter of lifeline standards is the TCLEE. A comprehensive plan to develop lifeline standards was developed in 1992 covering electric power, gas and liquid fuels, telecommunications, transportation, water, and sewerage lifelines. As of early 1995, significant funding for this plan had not materialized; however, NIST is continuing to work with TCLEE to establish priority research areas for lifelines. In many aspects of lifelines, only partial knowledge is available; thus, a significant part of this plan is to improve the current state of knowledge. For example, publicly available system models are needed to allow evaluation of lifelines. As currently envisioned, the planned development of standards will require many years, likely extending into the next century.

What Is Available Now

Certain lifeline utilities have begun some form of seismic assessment of their existing systems. These assessments generally involve six steps:

- **Inventory.**

  The utility inventories its nonrugged equipment. For a water distribution system, it includes buried pipe, tanks, dams, tunnels, electrical equipment, etc. For an electric-transmission system, it includes 500 kV and 220 kV substations. To a great extent, the availability of budget and/or other resources limits a utility's ability to perform such an assessment. For example, Pacific Gas & Electric has more than 1,000 substations, and a walkdown assessment of each substation is a major undertaking.

- **Hazard Assessment.**

  Estimates of seismic demand are made, either deterministically or probabilistically. Hazard conditions include potential ground shaking, liquefaction potential, landslide potential, and surface-faulting potential. More sophisticated hazard assessments include seismic microzonation efforts and estimates of permanent ground deformations. Because the earthquake experience data for lifelines show that most damage occurs in areas of poorest soil conditions (liquefaction, landslide, and surface-faulting areas), it is important that these local areas be identified as part of the hazard assessment.

- **Vulnerability Assessment.**

  Fragilities, damage algorithms, and experience data can be used to estimate the level of damage to the equipment. Different damage algorithms are used to account for ground shaking and permanent ground deformation effects. Good information is now available for estimating building and equipment response caused by ground shaking, especially from reports prepared by the *Applied Technology Council* (ATC), the *Electric Power Research Institute* (EPRI), and the *National Institute of Building Sciences* (NIBS) (Refs. 16,17,18,19). The ATC-13 (Ref. 16) information for building performance is still considered reasonable for California-quality (i.e., seismically designed) construction, but the ATC-13 information on electrical and mechanical equipment, tanks, and other lifeline components is now considered out of date and has been substantially improved through EPRI, NIBS, and other efforts. The NIBS effort extends ATC-13 work by providing *fragility* information for buildings designed to non-California standards, building contents, and all types of lifeline inventories. In addition, the *National Center for Earthquake Engineering Research* (NCEER) has published considerable information over the past few years that has improved the understanding of buried pipe performance. One such study covers crude oil transmission systems (Ref. 20).

- **Performance Assessment.**

  To make an assessment, the combined effects of hazards and vulnerabilities for infrastructure inventory are combined into a single system model. This model is used to predict the level of postearthquake service (usually as a percentage of pre-earthquake service) for the entire lifeline system. The model usually incorporates the lifeline utility's capability to repair damage by evaluating the number of available repair crews, the type of damage, and the inventory of spare parts. From this system model, estimates of postearthquake outages are then made. Some models used for this purpose described in the literature are : the Memphis Light, Gas, and Water Division's water system (Ref. 21), Southern California Edison's electric-transmission system (Ref. 22), East Bay Municipal Utility District's water distribution system (Ref. 23), and San Francisco's auxiliary water distribution system (Ref. 24).

- **Cost–benefit Assessment.**

  For a cost-benefit assessment, direct losses to a lifeline utility are estimated. Occasionally, indirect losses to customers (Ref. 25) also are estimated, including economic losses and casualties. Other models include economic losses (Ref. 26) on a macro-level. The NIBS report provides procedures to consider all types of direct and indirect economic losses (Ref. 19).

- **Develop Improvement Plan.**

  Once the above steps are performed, a lifeline agency can consider various upgrade alternatives. These improvements are then included in the system model, and postearthquake performance is then re-estimated. Alternative upgrade strategies can be considered, looping on this process, until an *optimal* design is reached. The plan can then be implemented, usually over a multiyear horizon.

More than one technical approach has been applied to solve each of the above steps. The current state-of-the-art in lifeline system analysis is still in a formative stage. Therefore, *standard* approaches are not available for each step. The following paragraphs summarize some (but not all) of the

areas in which today's state-of-the-art approaches are still evolving.

- First, there is the issue of appropriate fragility data/ruggedness data to be used in the vulnerability assessment. Since the early 1980s, a large body of experience data from past earthquakes has become available. The amount of this information is rapidly growing. Through the 1990s, this information will continue to be compiled and disseminated, and assessment will become easier to perform with more confidence in the results.

- There is still the issue of how to do the performance assessment. Currently, there is no universal acceptance of what level of postearthquake performance should be expected or required of a lifeline utility. Individual lifeline agencies are cautious about committing to a *standard*, such as full and normal service within three days after an earthquake. Yet the public is being trained, through earthquake emergency-planning measures, to plan to be without lifeline support for three days. This three-day recovery is possibly achievable for some lifelines for some areas. However, there is not yet a standard method of analysis to determine the cost and benefit of achieving this level of performance system-wide. For cases in which the cost-benefit has been estimated, often there has not been sufficient management attention (or capital resources) to upgrade the lifeline agency's infrastructure to this level of performance. Some currently proposed state- and national-level legislation may impose postearthquake performance standards on utilities, but there is not yet a good understanding of whether a three-day outage is *the* correct performance goal, or whether some other goal is more desirable. If no water is available to fight fires for even one day, small fires can grow into conflagrations such as occurred in San Francisco in 1906, Tokyo in 1923, and Kobe in 1995 with unacceptable widespread loss of property and life.

  Currently, the preferred method for examining lifeline seismic performance is to use *geographic information system* (GIS)-based system models. A GIS system model for lifeline analysis should have the following features:

- *Graphic user interface* (GUI): A graphical menu-driven system that nonexperts and experts alike can use.

- *Geographic data base manipulation* (GDBM): The ability to add, modify, and delete elements and database attributes.

- *Data base query* (DQ): The ability to sort information according to user-supplied queries. For example: show all pumping plants out of service because of loss of off-site electric power.

- *Seismic hazard definition* (SHD): Including peak ground-acceleration maps, site-specific response spectra, liquefaction analyses, landslide analyses, fault-crossing analyses, and/or other hazards, as needed.

- *Vulnerability analysis* (VA): Including fragility curves of components for each of the various seismic hazards.

- *Performance analysis* (PA): Given a state of damage to a lifeline network, what level of pre-earthquake service can the lifeline deliver?

- *Restoration analysis* (RA): Given a level of damage and a given number of maintenance crews and spare parts, determine the time after the earthquake needed to restore various levels of pre-earthquake service.

- *Cost-benefit analysis* (CBA): The ability to rapidly perform a series of *what-if* analyses. For example: what is the cost of installing backup power diesels at all pumping plants versus the expected improvement in postearthquake service.

Some GIS systems are described below. The list is not exhaustive, and new systems with more features are becoming available.

- *Full-function GIS systems.* These systems include software products from *Intergraph and Environmental Systems Research, Inc.* These (and other) systems have enormous capabilities in the GUI, GDBM, and DQ areas. They offer full-featured programming languages to allow users to customize the GIS to add the SHD, VA, PA, RA, and CBA parts. In many instances, existing CAD-based drawings can be directly incorporated into the GIS. The drawback to

these systems is that end users must actually develop the SHD, VA, PA, RA, and CBA parts. One such system, WATERFLOW (Ref. 21), is a university-developed code based on the ARC/INFO software product. It has been used for the Memphis, Tennessee water system.

- *Special-purpose GIS systems.* These systems are stand-alone packages, generally developed by engineering firms directly involved in lifeline earthquake engineering. These systems can import information from a variety of sources and have adequate GUI, GDBM, and DQ features. Their strongest benefits are that they include state-of-the-practice SHD, VA, PA, RA, and CBA components. A potential disadvantage of these systems is that they are often proprietary, and end users may become dependent upon the vendor to provide future new features. Two such systems are LLEQE (*LifeLine EQE*, EQE Inc.), which has been used for the San Francisco Water System (Ref. 24); and SERA (*System Earthquake Risk Analysis*, G&E Engineering Systems, Inc.), which has been used for the East Bay Municipal Utility District water system (Ref. 23), the Southern California Edison's electric system (Ref. 22), and the San Francisco Bay Area public transportation system (Ref. 27). The Gisalle program, developed at Cornell University, New York, which has also been used to study the San Francisco water system (Ref. 24) is a university-developed predecessor to LLEQE.

Fires Following Earthquakes

Earthquakes cause fires. It is worthwhile to study what has been learned from past earthquakes to determine what are the main factors causing these fires.

The basic scenario is as follows. An earthquake occurs. It causes various types of damage to lifelines and to residential, commercial, and industrial facilities. This damage causes immediate fire ignitions.

There are two types of fires that could affect a particular facility manager's site. First, a fire can ignite within the manager's facility. For DOE sites, current fire-suppression systems are probably suitable to extinguish such a fire. Of course, the on-site fire-fighting apparatus and water supply may have concurrent earthquake damage that reduces fire-fighting capability.

Second, a fire (or many fires) may ignite at a moderately distant location from the facility manager's site. These fires may overtax the local off-site fire department's ability to extinguish them, especially with concurrent wide spread damage to various off-site lifelines, including the water-distribution system (limiting water flows to hydrants), the electric distribution system (disrupting pumping plants and communications), the transportation network (resulting in lengthened fire-department response times), and possibly collapsed fire stations as well as collateral fire department diversions for victim extraction, etc. The net result may be that a fire that ignites away from a particular facility site may cause a general conflagration that could threaten that facility.

For example, the 1906 San Francisco earthquake caused 52 original ignitions (Ref. 28). Twenty of these fires were extinguished, but not without considerable effort. The remaining fires were not extinguished primarily because of the lack of fire department resources and water. The remaining fires spread into a general conflagration that eventually destroyed more than 28,000 buildings over a three-day period.

As another example, the 1923 Kanto, Japan earthquake caused 88 original ignitions in the Tokyo area. Damage to the water system, limited fire-fighting resources, and high winds eventually led to the loss of some 447,000 houses and buildings from fire, as well as 143,000 dead or missing.

Earthquake experience suggests that there are five main sources of ignitions:

- *Slapping and arcing of above-ground power lines.* The 1983 Coalinga, California earthquake (Ref. 29) caused 15 separate grass-fire ignitions in open country from arcing of power lines.

- *Gas pipelines.* Many buried gas pipeline systems are especially vulnerable to breakage, particularly in areas of soil liquefaction, landslides, and surface

faulting. Broken gas lines can lead to ignitions.

- *Collapsed buildings.* If a building collapses (or is excessively deformed), there is a chance that it will ignite because of electrical arcing or broken gas lines.

- *Fallen debris.* Postearthquake ignitions can occur in noncollapsed buildings. For example, in Coalinga, a house ignited 3.5 hours after the main shock. The fire started in the kitchen when items fell onto heating elements that were energized when power was restored 3.5 hours after the main shock. Another kitchen fire that started for the same reason was quickly put out by people in the house. Another fire was caused by a can opener, turned on by flying debris which overheated, and set the debris on fire.

- *Cooking fires.* The number of ignitions increases if an earthquake occurs during peak cooking hours (lunch time, dinner time).

Currently, there is insufficient information to make accurate predictions of fire ignitions in future earthquakes. However, three empirical formulations are suggested below. These formulations are probably reasonable for estimating an order of magnitude of postearthquake ignitions, although substantial improvements for area-dependent factors can be made. The first formulation is based on Japanese data, the second and third on United States data.

In the first formulation, the probability of collapse of a single building structure is estimated. This can be done using the ATC-13 damage algorithms, building-specific fragility analyses, or by some other means. For Japanese low-rise wooden buildings, based on data for Sendai City in the June 12, 1978, Miyagiken-oki earthquake (Ref. 30).

$$P[D \mid SA] = 0.020145 * SA^{2.525}$$

where:

P [D | SA] is the probability of collapse per building, given SA.

SA is the 5% damped response spectral acceleration at a period of 0.75 seconds, which is approximately the natural period of Japanese low-rise buildings when subjected to heavy ground shaking. The SA should account for the site-specific soil conditions.

Mizuno (Ref. 31) researched the outbreak of serious fires following earthquakes in urban Japan. A serious fire is one that is not extinguished immediately and that spreads to adjacent buildings. The following formula is based on a regression of 114 data points from 12 different Japanese earthquakes dating from 1923.

$$P[FO \mid D] = 0.00289 \{P[D \mid SA]\}^{0.575}$$

where:

P [FO | D] is the probability of fire occurrence per building.

These formulae were applied to Tokyo assuming a repeat of the 1923 Kanto earthquake. For this case (adapted from Ref. 30), SA is between 0.37g and 0.66g, depending on soil type. The model includes an inventory of 326,000 buildings. This leads to a predicted outbreak of 40 fires, as compared to the 88 fire outbreaks reported in the actual 1923 Kanto earthquake. The larger-than-expected number of fire outbreaks is attributable to the earthquake coinciding with the lunch hour and its attendant cooking fires.

There are many implicit assumptions when using these formulae. They include the earthquake resistance of construction, the fire retardant type of construction, the ability for building occupants to immediately extinguish ignitions, and the capability of the fire department to respond quickly and suppress the initial fire outbreak.

In the second formulation (Ref. 32) which is based on U.S. earthquakes in the twentieth century, fire ignitions are estimated using Table 11a-4.

In Table 11a-4, an SFED is defined as a *single family equivalent dwelling* or 1,500 square feet of floor area. A large office building of 1,500,000 square feet would therefore be 1,000 SFEDs. MMI refers to the Modified Mercalli Intensity for the local area.

Table 11a-4. Fire ignition rate.

MMI	One Ignition Per
VI	Negligible
VII	7,500 SFED
VIII	3,500 SFED
IX	2,500 SFED

In the third formulation (Eidinger, et al.), existing United States earthquake data is reformulated to relate the number of ignitions as a function of building stock exposed to various levels of peak ground acceleration, PGA (Ref. 19). The following equation is used to estimate the number of ignitions per million square feet of floor area, versus PGA:

$$N = -0.025 + (0.592 * PGA) - (0.289 * PGA^2)$$

Once the number of fire ignitions, N, is known, the potential for spread of the fire can be developed using relatively complex fire spread models. One model by Scawthorn (Ref. 32), provides a simulation technique which considers fire breaks, wind speeds, fire retardent construction, number of available fire engine apparatus, and concurrent damage to the transportation and communication systems. Another model by Eidinger and Dong (Ref. 19) expands on this work to examine in detail the effectiveness of water supply at the site of the fire, considering the availability of water supply over time after the earthquake.

Using these models in an urban area like the San Francisco Bay Area, the following trends are observed as shown in Table 11a-5 (Ref. 33). One observes that fire spread potential is substantially dependent upon prevailing wind conditions at the time of the earthquake as well as upon the availability of water at the site of the fire when needed after the earthquake. One also observes that under high wind conditions, the potential for fire spread is very high, and does not depend so much on water supply as it does on rapid discovery and control of the initial fire ignition before it begins to spread. In Table 11a-5 the letter B represents a typical residential structure.

Basic Fire Flow Design For Water Systems

One of the threats to a specific site facility (even if the site itself is not damaged by the earthquake) is that the damage to off-site lifelines cause safety implications on site. The previous section described how fire conflagrations destroyed much of the cities of Kobe, Tokyo and San Francisco. Limited fire flows through water-distribution systems played an important role in helping spread these fires. The discussion that follows describes water distribution system fire-flow design requirements commonly used in the United States.

Water utilities in the United States have the primary role of transporting water for normal consumption purposes, as well as for fire-flow purposes. The sizing of reservoirs, pumping plants, and distribution pipe is based on meeting both normal and fire-flow demands.

In California, for example, fire-flow requirements are set by the *Uniform Fire Code* (UFC) (Ref. 34). Local fire jurisdictions adopt the UFC, sometimes with local amendments. It is important to note that actual required fire flows are usually determined by the local fire chief or fire department. The water-supply agency then builds the water system to provide the required fire flow. In some jurisdictions, the fire department and the water department are both part of the same local government or special district. In another situation, one water department may serve many municipalities. Thus, required fire flows for nominally the same types of building may differ in different areas.

Table 11a-5. Fire spread.

Wind Conditions	Structures Burned, B, per Ignition, Poor Water Supply	Structures Burned, B, per Ignition, Good Water Supply
Calm	3-5	0.5-2
Light	7 - 12	3 - 4
High	40 - 50	35 - 45

There are two principal parts to determining fire flow: the rate of flow in gallons per minute available (from one or more hydrants) to a burning building, and the duration or time this flow must be available.

The adequacy of the UFC fire flows has been proven thousands of times per year in major metropolitan areas. It is rare, when a water system actually delivers the required fire flow, that the fire department cannot prevent a single fire from spreading into a conflagration.

However, a recent experience proved otherwise. In the October 20, 1991, Oakland hills, California firestorm (Ref. 35), ten water reservoirs directly within the fire area (containing more than 5,000,000 gallons at the outset of the fire) were drained during the fire. These reservoirs could not be replenished, as pumping plants were inoperative because of power outages. Even if there had not been power outages, the pumping plants could not have provided significant amounts of water. Storage from larger reservoirs (with a capacity greater than 60,000,000 gallons) was only partially usable because of limitations in the pipe distribution network. These limitations hampered fire-fighting operations. This firestorm caused 25 deaths and destroyed 3,000 dwellings. It should be noted that in the areas that ran out of water, the water distribution system had been built to provide fire flows 50% *higher* that the UFC requirements.

The Oakland hills fire was an *urban intermix* fire, which is one where building structures are intermixed with an area of high fuel load. Residential neighborhoods in the Oakland hills were built in a moderately forested area. This type of fire was not envisioned in developing fire-flow requirements for residential areas for the UFC. The fire flows in the Oakland hills were based on building type rather than actual fuel load. Recent studies have shown that a 720% increase in fire flows *above* that required by the UFC would have been needed to supply adequate water supplies during the firestorm.

The lesson to be learned from the 1991 Oakland hills fire has particular relevance for the University of California Lawrence Berkeley Laboratory (LBL), a DOE facility located just one mile north of the 1991 Oakland hills fire area. The LBL site is situated in a similar *urban intermix* zone. LBL and adjacent University of California-Berkeley have an aggressive long-standing fire-protection plan that includes annual clean-ups to reduce grass and other natural growth on site and in the adjacent hills. LBL also has an on-site professional fire department and two 200,000-gallon water-storage tanks and emergency pumping stations located on site to be used in the event that public supplies are lost. These emergency supplies are designed to be operative in the aftermath of a large earthquake.

Because fire following earthquakes could pose a substantial threat to such a facility, the need for careful evaluation is magnified. Very high fire flows are generally needed in *urban intermix* areas, and the on-site water-distribution system infrastructure may be severely damaged from the earthquake itself. In an attempt to mitigate this hazard some years ago, LBL relocated certain on-site water mains from areas of questionable stability to

stable ground and set up emergency supplies of hose and other special fittings to bypass potential breaks. A large number of on-site emergency generators back up public power supplies.

Inherent Vulnerabilities of Water Systems

Water systems are vulnerable to varied types of earthquake damage. Many of these vulnerabilities are similar to those for buildings (such as underdesigned structures, unanchored equipment, unanchored tanks, etc.) and will not be discussed here. Rather, three types of common lifeline vulnerabilities will be described: buried pipe, reliance on off-site electric power, and the reliability of emergency diesels.

• *Buried Pipe.* As described in previous sections, buried water-distribution pipe is particularly vulnerable to earthquake motions. Past studies based on empirical evidence have suggested that the damage to buried pipe is caused by one of two phenomena: *wave propagation* (WP) and *permanent ground deformation* (PGD).

Wave propagation is estimated from the site *peak ground velocity* (PGV) value. Pipe damage rates are proportional to PGV and vary as a function of the type of pipe material and the type of pipe joint.

The pipe damage algorithms shown below are based on empirical data from several pre-1989 earthquakes and benchmarked within ± 20% of actual damage from the Loma Prieta (1989) event (Ref. 36). These algorithms reflect current knowledge, and will continue to be revised as new empirical information from future earthquakes is added to the database.

For pipes subjected to WP only (no liquefaction, landslides, or surface faulting):

$$n = A * 3.2e\text{-}4 * PGV^{1.98}$$

where:

n = repair rate, per 1000 feet of pipe

PGV = peak ground velocity, inch/sec

A is defined in Table 11a-6.

PGD damage is estimated using a more complex method. First, the likelihood that a particular site will actually undergo PGDs (either from liquefaction, lateral spreads, slumps, landslides, or surface faulting) must be estimated. This information must be developed through a geotechnical evaluation of actual site conditions.

Given that the particular site will have a liquefaction PGD, the pipe break rate can be estimated as follows:

$$n = B * 1.04 * PGD^{0.53}$$

where:

n = repair rate, per 1000 feet of pipe

PGD = permanent ground deformation, inches

B is defined in Table 11a-6.

Given the repair rate n, the probability of some type of pipe failure (i.e., one or more leaks or breaks along the length) is given by:

$$P_f = 1 - e^{-nL}$$

where:

L = length of pipe (1000s of feet)

n = repair rate (per 1000 feet)

• *Reliance on Off-Site Power.* Based on experience from past earthquakes, there is roughly a 50% chance of an immediate loss of off-site power if the PGA level is in the range of 0.30 to 0.35g. This simple rule can be used for planning purposes, but clearly it disregards the spatial location of the vulnerable electric lifeline substations, duration of outages, etc.

If a water system provides service to many pumped pressure zones, there is a question about the availability of continued pumping after an earthquake. For best reliability of postearthquake water service, a pumped pressure zone should rely on in-zone water storage for normal consumption, emergency reserves, and fire-flow service. With somewhat lesser reliability, a pressure zone can

Table 11a-6. Pipe break rates caused by wave propagation.

Pipe and joint material	Wave Propagation, A	Permanent Ground, Deformation, B
Asbestos cement, rubber gasket	0.5	0.8
Asbestos cement, cement	1.0	1.0
Cast iron, rubber gasket	0.5	0.7
Cast iron, cement	0.8	1.0
Concrete cylinder, large diameter, segmented	2.0	1.0
Concrete cylinder, large diameter, welded	1.0	0.8
Ductile iron, segmented	0.2	0.15
PVC, rubber gasket	0.5	0.8
Arc-welded steel (large diameter lap weld)	0.14	0.15

be served by pumping plants with backup diesel generator sets; however, most pumping plants are sized to refill storage reservoirs over long periods of time, and reliance on pumping only may result in insufficient fire flows.

• *Reliability of Emergency Generators.* Past experience with the reliability of emergency diesel generator sets has not been entirely satisfactory. For example, in the recent Santa Barbara fire, one emergency diesel did not run, reportedly because of oxygen starvation from the intensity of the surrounding fire. In the Oakland hills fire of 1991, similar oxygen starvation probably would have occurred at 4 to 6 pumping plants had there been diesel generator sets installed (none were). For example, at one pumping plant location, the heat of the surrounding fire caused relaxation of the hoop steel in the adjacent concrete tank, resulting in tank failure (gradual leakage). Some estimates indicate that the heat of the fire reached 2,000 degrees Fahrenheit at this location.

Under nonemergency maintenance situations, a survey (Ref. 37) found that 42% of water pumping-plant emergency generators failed to start up on demand at least once per year (or about 3% of the time).

Under earthquake conditions, the performance of backup diesels also has been less than satisfactory. For example, various reports after the 1994 Northridge earthquake confirmed that backup diesels did not work as intended after that earthquake. Failure rates as high as 50% at essential facilities (hospitals, communication facilities) in the epicentral area were reported, although at this time firm statistics are not available. Failures have been attributed to poor anchorage (vibration isolation systems, either without snubbers or with brittle snubbers), inadequate maintenance, lack of tests under full load, and bad fuel.

Conclusion

It should be apparent that a complete dissertation on lifeline performance would require several volumes. Hopefully, the overview will acquaint readers with major lifeline vulnerabilities, the risk of fire following earthquake, and in particular, an appreciation for water system infrastructure problems. The following references are provided for a more detailed understanding of lifeline performance.

References

1. *The Current State of Knowledge of Lifeline Earthquake Engineering*, C. Martin Duke, Ed., Technical Council on Lifeline Earthquake Engineering First Specialty Conference, Los Angeles, California, August 1977.

2. *Lifeline Earthquake Engineering: The Current State of Knowledge*, D. J. Smith, Jr., Ed., Technical Council on Lifeline Earthquake Engineering Second Specialty Conference, Oakland, California, August 1981.

3. *Lifeline Earthquake Engineering Proceedings of the Third U. S. Conference,* TCLEE Monograph No. 4, Los Angeles, California, August 1991.

4. *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems,* Gas and Liquid Fuels Lifelines Committee, ASCE Technical Council on Lifeline Earthquake Engineering, ASCE, 1984.

5. *Seismic Loss Estimates for a Hypothetical Water System, A Demonstration Project,* C. E. Taylor, Ed., TCLEE Monograph No. 2, August 1991.

6. *Lifeline Earthquake Engineering in the Central and Eastern U. S.,* D. Ballantyne, Ed., TCLEE Monograph No. 5, September 1992.

7. *Draft Plan for the Development of Design and Construction Standards for Water and Sewer Systems, Seismic Evaluation Program Status Report,* D. Ballantyne, ASCE TCLEE Water and Sewer Committee.

8. *Earthquake Loss Estimation Modeling of the Seattle Water System,* prepared for USGS by Kennedy/Jenks/Chilton, Dames & Moore, D. Ostrom, Weidlinger Associates, May 1991.

9. *Water Distribution System, Seismic Evaluation Program Status Report,* prepared for EBMUD by G&E Engineering Systems, September 30, 1992.

10. *Earthquake Loss Estimation for the City of Everett, Washington Lifelines.,* prepared for USGS by Kennedy/Jenks/Chilton, Dames & Moore, D. Ostrom, Weidlinger Associates, May 1991.

11. *Building Can Quake in Rubber Boots,* The Oregonian, October 3, 1992.

12. *East Bay Hills Firestorm Response Assessment, Phase II Final Report,* Prepared for EBMUD by VSP Associates, G&E Engineering Systems and BFA Associates, January 21, 1993.

13. National Earthquake Hazard Reduction Program Reauthorization Act, Public Law 101-614 , November 1990.

14. *California At Risk, Reducing Earthquake Hazards, 1992–1996,* Report SSC 91-08, Seismic Safety Commission, Sacramento, California, 1992.

15. *Lifeline Position Paper of the Oregon Seismic Safety Policy Advisory Commission (OSSPAC),* Report to the Governor and the Legislative Assembly for 1991–1993, second draft, July 14, 1992.

16. *Earthquake Damage Evaluation Data for California, ATC-13,* Applied Technology Council, 1985.

17. *Generic Seismic Ruggedness of Power Plant Equipment,* Electric Power Research Institute, Report NP-5223, prepared by ANCO Engineers, January 1991.

18. *A Methodology for Assessment of Nuclear Power Plant Seismic Margin,* Electric Power Research Institute, Report NP-6041-SL, Revision 1, prepared by Benjamin Associates, R. P. Kennedy, PL&G, I. M. Idriss, and Southern Company Services, August 1991.

19. *Development of a Standardized Earthquake Loss Estimation Methodology,* Risk Management Solutions, Inc., Prepared for the National Institute of Building Sciences (NIBS), September 1994.

20. *Pilot Study on Vulnerability of Crude Oil Transmission Systems,* T. Ariman, R. Dobry, M. Grigoriu, F. Kozin, T. O'Rourke, and M. Shinozuka, NCEER-90-0008, May 25, 1990.

21. *GIS-Based Interactive and Graphic Computer System to Evaluate Seismic Risks on Water Delivery Networks,* R. Sato, M. Shinozuka, Proceedings, 3rd U. S. Conference on Lifeline Earthquake Engineering , August 22–23, 1991.

22. *Understanding the Earthquake Environment, Corporate Strategies,* D. Ostrom, Proceedings, 3rd U. S. Conference on Lifeline Earthquake Engineering, August 22–23, 1991.

23. *Preparedeness, Performance, and Mitigation for EBMUD Water-Distribution System for Scenario Earthquakes,* J. Eidinger, J. Young, CSUEC National

Earthquake Conference, Memphis, Tennessee, May 3–5, 1993.

24. *Performance of the San Francisco Auxiliary and Portable Water Supply Systems in the 17 October 1989 Loma Prieta Earthquake,* C. Scawthorn, F. Blackburn, 4th U.S. Conference on Earthquake Engineering, Palm Springs, California, 1990.

25. *Benefit-Cost Model for the Seismic Rehabilitation of Buildings,* Volumes 1 and 2, prepared for FEMA by VSP Associates, FEMA Reports 227 and 228, April 1992.

26. *Seismic Vulnerability and Impact of Disruption of Lifelines in the Coterminus United States,* ATC-25, Applied Technology Council, 1991.

27. *Model Hazard Mitigation Plan,* prepared for the Metropolitan Transportation Commission by G&E Engineering Systems, Seismic Risk Workshop, January 12, 1993.

28. *Large Ground Deformation and Their Effects on Lifeline Facilities: 1906 San Francisco Earthquake,* T. O'Rourke, P. A. Beaujon, C. Scawthorn, Case Studies of Liquefaction and Lifeline Performance During Past Earthquakes, Vol. 2, NCEER-92-0002, February 17, 1992.

29. *Coalinga California Earthquake of May 2, 1983,* EERI Reconnaissance Report No.84-03, January 1984.

30. *A Model for Urban Post Earthquake Fire Hazard,* C. Scawthorn, Y. Yamada, H. Iemura, Disasters, The International Journal of Disaster Studies and Practice, Foxcombe Publications, London, 1981.

31. *On the Outbreak of Fires in Earthquakes,* H. Mizuno, dissertation, Kyoto University, Department of Architecture, Kyoto, Japan, 1978.

32. *Fire Following Earthquake,* prepared for the All-Industry Research Advisory Council by Dames & Moore, Oak Brook, Illinois, 1987.

33. *Fire Following Earthquake, Indirect Loss Estimation,* prepared for East Bay Municipal Utility District by G&E Engineering Systems, 1994.

34. *Uniform Fire Code,* ICBO, Whittier, California, 1991.

35. *East Bay Hills Firestorm Response Assessment, Phase I Final Report,* prepared for EBMUD by VSP Associates, G&E Engineering Systems and BFA Associates, July 15, 1992.

36. *Earthquake Loss Estimation Methods, Water Systems,* prepared for National Institute of Building Sciences by G&E Engineering Systems, May 1994.

37. *Reliability of Standby Engine-Driven Generators at Water Purveyors' Pumping Plants,* A. A. Webb Associates, April 27, 1992.

Chapter

11b

The Multihazard Emergency-Response Plan

Terence P. Haney

Introduction

The U.S. Department of Energy (DOE) requires that all sites and facilities (including those operated by its contractors) establish and maintain emergency management programs. These programs must include written emergency plans that follow a standard format and content. Site and facility emergency management programs and plans must be commensurate with an assessment of potential hazards and constitute a specific set of elements. The scope and extent of emergency planning and preparedness programs may vary considerably from location to location, based upon the hazards involved and risk levels associated with a specific facility. DOE requirements and related guidelines are referenced in the foreword to this chapter.

To be effective, emergency planning at the site or facility level has to be more than *compliance planning*. The existence of a plan on paper does not necessarily indicate that the organization is prepared for emergencies. Planning and preparedness are not synonymous terms when it comes to emergencies. Unfortunately, some emergency-planning efforts do not have the full measure of support that they deserve and require.

In this chapter, a range of disaster preparedness, operations response, and recovery planning steps will be briefly examined.

The chapter has two main objectives:

1. For facilities that have already completed extensive planning, it provides a practical framework against which current preparedness and planning actions can be evaluated

2. For facilities that need to do more extensive planning, it serves as a description of actions to be taken.

Phases of Emergency Preparedness

Comprehensive emergency preparedness consists of five overlapping phases, as noted below. Each will be discussed in this chapter.

- Hazards and risk assessment
- Hazard mitigation and preparedness
- Emergency-response planning
- Operations-recovery planning
- Training and exercises.

Hazards and Risk Assessment

A discussion of the principal activities associated with this phase follows:

Hazard Identification

The first step is to identify the range of possible hazards and establish the level of risk to personnel and facilities. This hazards and risk assessment provides planner(s) with information that will be the foundation for all preparedness and planning activities.

Conducting a thorough hazards and risk assessment is important for these reasons:

- Properly done, the assessment will quantify the scale of the problem. This will be extremely helpful in other phases when determining specific needs for personnel and obtaining essential equipment and supplies.

- The assessment identifies potential weaknesses in facilities, communications, support systems, operations, and training. These problems can then be addressed in the preparedness and planning phase.

- The assessment provides the basis for identifying which emergency-response functions may be required. Some functions will be applicable to all emergencies, and some will be specific to a certain kind of an event.

Local Government Plans

The second step is to understand the extent and limits of local government plans. Political subdivisions, i.e., cities and counties, are required to have plans in place for emergencies. The quality of these plans vary widely from location to location.

In site emergency planning, it is important to understand the emergency plan for the local jurisdiction and to know how it is expected to work. Too often, site emergency plans are written without consideration for what the local government may be planning for various kinds of emergencies.

For example, for earthquake planning, the local government *may* have pre-established traffic plans that will change or restrict traffic flow on certain surface streets or highways. They *may* have plans for implementing air-space control that would limit or prohibit the use of helicopters or restrict the use of local airports. They *may* be planning to activate emergency ordinances that would restrict unauthorized personnel movements, establish curfews, etc. Any or all of these possibilities could have a major impact on site emergency response and recovery actions. To plan without this knowledge is to plan in a partial vacuum.

The local government may have priorities for how it plans to respond, depending upon the kind, size, and duration of an event and its own capabilities. This could lead to delayed response time for basic emergency services, given higher priority life-protection problems in other areas.

Awareness of local and regional priorities provides a real incentive for developing on-site teams that can at least temporarily alleviate various situations. This would apply for such emergency activities as fire and *hazardous materials* (HAZMAT) suppression, search and rescue, first-aid and medical, etc.

Essential Functions

The third step is to identify and clearly describe the essential functions that must be performed in the event of an emergency. At this point, it is better to separate functions related to emergency response (immediate and short-term) from those that will be necessary in the recovery effort.

Possible recovery functions that may have to be performed are described later in this chapter.

Hazard Mitigation and Preparedness

Identification of existing hazards is essential in reducing potential losses. Developing and implementing an action plan for mitigating these hazards will reduce the level of risk before the emergency occurs. Consequently, personnel and facility losses will be decreased, response time improved, and a faster recovery will take place following an

emergency. Hazard mitigation measures taken before an emergency can also reduce monetary loss and recovery costs associated with an emergency.

There are several steps to be considered in hazard mitigation. Each of these will be briefly reviewed.

Structural Assessment and Mitigation

Qualified structural engineers should make an assessment of all buildings to determine how well they would withstand the effects of particular hazards. The results of the assessment should show specifically what steps need to be taken to bring existing structures up to an assured level of operating performance under the conditions described in the hazards assessment. A structural mitigation plan should be implemented based on priorities related to the consequences of failure.

Nonstructural Assessment

This part of the assessment looks at nonstructural aspects of building use and is particularly relevant to earthquakes, windstorms, etc. It identifies areas for improvement within facilities, such as bracing of bookshelves, file cabinets, equipment, computer floors, ceilings and fixtures, and other considerations. It should also include antennas, chimneys, cooling towers, air-conditioning units, fuel tanks, etc. Mitigation should proceed based on a priority system related to the consequences of failure.

In addition to the other materials referenced in this *Seismic Safety Manual*, two basic and very useful guidelines for use in this phase are:

- *Disaster Mitigation Guide for Business and Industry*, FEMA 190 (February 1990) which is available from the Federal Emergency Management Agency.

- *Guidelines for Earthquake Hazard Mitigation for Data Processing Facilities*, developed by the Finance, Insurance, and Monetary Services Committee of the California Governor's Earthquake Preparedness Task Force (June 1987). This report is available from VSP Associates in Sacramento, California.

Vital Records Control and Data Storage

Measures may already have been taken to ensure the safeguarding of important information. If not, it is essential (at each management level) to make an assessment of what information is critically important. The simplest way to proceed is to ask each manager to determine what information he/she uses on a daily basis that is essential to operations. Once this is established, the next step is to determine what measures are currently being taken to safeguard these records. Again, safeguarding should relate back to the hazards assessment. This may, in some cases, require off-site storage of vital records and planning for alternative facilities for essential functions.

Employee-Preparedness Measures

Each employee should be provided with written material and given orientation for personal-preparedness measures. These should include preparedness both at the workplace and at home. Excellent materials that describe actions that should be taken for a variety of contingencies are available from FEMA, the California *Office of Emergency Services*, (OES), the American Red Cross, and private organizations.

Emergency Operations Centers

Every facility must have a central location for the emergency-response management team to use in coordinating response activities. These facilities are usually called *Emergency Operations Centers* (EOCs). The hazards assessment is the best reference to use in determining where that location should be and what it must be protected against.

It is very important that an alternative EOC capability be available and made a part of the planning. If the primary EOC is located in a structure, it is subject to whatever damage that structure suffers. A simple broken water pipe in a ceiling or lack of proper access can put an otherwise sound EOC facility completely out of business. *It is vital that both the structure per se and all nonstructural elements in EOCs (including emergency backup systems) are earthquake resistant. This should also apply to nonstructural elements in access hallways, stairways and building entrances.*

Temporary off-site work facilities should be considered for the additional reason that on-site locations may be unusable for a period of time. In the recovery phase, an off-site center often provides better communications, access, and support. The hazards assessment, and consideration of factors previously discussed regarding jurisdictional planning, are valuable to planners considering alternative sites for coordinating disaster recovery.

Emergency-Response Planning

The *Emergency-Response Plan* should be oriented toward the *total facility*. In other words, it should cover emergency operations from the *broadest* view possible. The plan should be written clearly and presented in a logical, concise, and straightforward manner. The plan should be kept in a three-ring notebook with tab dividers for each principal section.

Experience indicates that most people do not spend adequate time in reviewing the facility emergency plan. Moreover, key personnel are often out of town or on vacation when the emergency takes place and others must fill in for them. A pocket-size condensed version of the plan provides a *security blanket* that is well appreciated at the time of an emergency.

Format of the Plan

Following is a brief discussion of the major elements that should be covered in an emergency-response plan. There are a number of ways to format the plan. DOE facilities and contractors performing work for DOE must have emergency plans that fulfill specific policy requirements, emergency management procedures and follow a standard format and content (see Foreword to this Chapter). What follows is simply one way to describe all of the material that should be included within an emergency plan.

Introduction

The introduction to the plan should include the following information:

- Management authorization and implementation directives
- Statement of purpose and objectives
- An overview of the importance of emergency-response operations
- General responsibilities for managers and employees.

Organization

This section of the plan should contain the following elements:

- Names of principal organization units and a diagram of the emergency-response organization. Often this organization is different from the day-to-day organization for a facility. This is especially true in larger facilities that have large numbers of personnel and many organizational units
- Names and descriptions of the functions of various support teams. Teams may include the following:

  Assembly coordinators

  Auxiliary ambulance team

  Building inspection team(s)

  Building managers and floor wardens

  Fire and hazardous materials control team(s)

  Medical-aid team(s)

  Search-and-rescue team(s)

  Traffic control team(s)

- Descriptions of the major responsibilities of each member and level of the organization
- Descriptions of reporting relationships within the organization.

Alerting and Activating Procedures

This section should cover:

- Description of activation guidelines and procedures. Some locations may have two or more levels of activation.

Some also may be geared to local jurisdiction planning.

- List work/home phone numbers for key personnel to be notified/activated for various emergency levels. These can also be kept on laminated cards or in pocket versions of the plan.
- Describe emergency lines of succession for key response positions in the organization. These should go down two or three levels.
- List emergency telephone numbers and alternative means to contact off-site essential services and suppliers.

Communications

This part of the plan brings together information about communications capabilities. It should include descriptions and listings pertaining to:

Radio systems

- Number and location of base stations
- Number and assignments of hand-held radios
- Listing of frequencies available for use and their assignments to radios and any other pre-identified uses
- Mobile radio assignments, numbers, vehicle assignments, and frequencies
- Amateur radio operators and/or *Citizens Band* radio systems, including locations and descriptions of equipment
- Listing of local government radio frequencies (ambulance, fire, police, local government, etc.).

Telephone system

- Description of basic telephone switching system used on site, including number of trunks and instruments
- Map showing entry point(s) for incoming trunks
- Map showing locations and numbers of any *bypass* phones (private lines not dependent on or part of the facility's central switch system). These should be on separate instruments whose ringing power is *not* obtained from the facility telephone system
- Map showing locations and number listings for all coin phones associated with the facility
- Number, type, and assignments for all cellular phones (mobile, transportable, portable).

Paging and public-address systems

- Description of all private or subscription personal paging systems in use. (If it is telephone activated, determine if there is an alternative or bypass-entry activation procedure.)
- Description of internal and external public-address systems
- Description of portable public-address systems.

Emergency Operations Center (EOC)

Proper configuration and stocking of the EOC is vital to emergency operations. Listed below are the major elements of the EOC that should be described in the plan. Other specific items related to displays, equipment, and communications that should be available in the EOC are also listed.

Major elements

- Location, layout, and description of primary EOC
- EOC activation and start-up procedures
- Organization and staffing
- Equipment
- Displays
- Communications
- Alternative EOC and/or recovery-site description.

Displays

In a permanent EOC facility, displays should be in place at designated locations. If there is no permanent EOC, displays should be kept in a storage area within or adjacent to the location designated to be the EOC. They should be hung or set up in predetermined or marked locations within the EOC upon activation by the EOC *Support Group*. The following displays are recommended for use in the EOC:

Status Boards (These are formatted and lined white boards)

- Personnel status board (4ft x 8ft) (Fig. 11b-1)
- Facility status board (4ft x 8ft) (Fig. 11b-2)
- Major incidents in progress (4ft x 6ft)
- Response-team status/assignments (4ft x 4ft)
- Casualties (4ft x 4ft)
- Emergency numbers and special notices (4ft x 4ft).

Maps

- Facility plot plan
- Map of local area
- Map of region (showing major routes). Equipment and supplies (partial listing)
- AC and battery-operated AM radio
- Television receiver
- Emergency generator and portable lighting (can be stored in another location and moved to the EOC)
- Location map guides for local area
- Residential and yellow-pages phone directories
- Emergency listings for city and county medical resources
- Utility diagrams for facility (gas, water, telephone, electric, sewer)
- Pads, pencils, erasable markers, map pins, and symbols
- Identification vests, hats, or badges for key supervisory positions and teams.

Emergency-Response Functions

It is not unusual in emergency-response planning to identify between twenty and thirty separate functions that may be required in response and recovery efforts. Each requires a brief description, identification of the organizational units that have primary and support responsibilities for its function, and a checklist for implementation. The person in charge of each function should be visually identified by the function (rather than by person) for ease of recognition.

The list below identifies some more common emergency response and recovery functions. Functions are listed alphabetically within three classifications.

Protection of life

- Care and shelter
- Communications
- Evacuations
- Facilities inspection
- Fire-control operations
- Hazardous materials control
- Medical first aid
- Personnel
- Search and rescue
- Situation assessment
- Triage
- Warning signs and communications.

PERSONNEL STATUS BOARD														
BUILDING	FLOOR	REPORT TIME	EVAC OR SEARCH TIME	INJURIES IN BUILDING		DEATHS	MISSING PERSONS	LOCATION OF INJURED	ASSIST. REQ.		INJURY IN ASSEMBLY AREAS			
				MAJ.	MIN.				S&R	MEDIC	AREA NO.	MINOR	MAJOR	TOTAL
100	1	0830	0810	1	3	0	2	East End		X	101	5	1	6
100	2													
100	3	0840	0815	0	2		5	At Elevator		X				

Fig. 11b-1. Example: personnel status board.

FACILITY STATUS BOARD														
BUILDING	FLOOR	TIME	DAMAGE OBSERVED							UTILITIES STATUS				ASSISTANCE NEEDED	OK TO OCCUPY
			FIRE	GLASS	FLOOR	CEIL.	WTR.	EQUIP.	OTHER	ELEC.	TEL.	WTR.	GAS		
100	1														
100	2	0840		X		X	X			ON	OFF	OFF	OFF	FACILITIES	NO
100	3														
100	4														

Fig. 11b-2. Example: facility status board.

Stabilization of personnel and facilities

- Debris removal
- Emergency information
- Environment, health and safety
- Facilities inspection
- Fatality operations
- Food services
- Public information
- Salvage operations
- Sanitation
- Security and plant protection
- Traffic control
- Utilities service and liaison

Recovery

- Alternative work facilities/locations
- Construction services
- Engineering Services
- Environment, health and safety
- Financial services
- Legal affairs
- Operations Services
- Public liaison
- Supply and procurement services
- Transportation and fuel services
- Vital data and records restoral.

Functional Checklists

Each of the functions listed above should have a checklist of actions that may be required at the time of an emergency.

The importance of the checklists cannot be overemphasized. *Checklists should always be prepared with the assumption that the individual who already knows what to do may not be available at the time of an emergency.*

A partial checklist is included in Fig. 11b-3. Note that checklists do not explain *how* to do the job. They are management checklists designed to determine *what* needs to be done, and in what approximate order.

Resource Materials

Resource materials to be kept in the plan should include such things as:

- Facility maps and *key* plans
- Lists of contractors and vendors for essential services
- Utilities diagrams
- Structural diagrams for seismic inspection.

Employee Actions for Specific Hazards

Some plans will include a section that describes employee actions to be taken for specific hazards. These are often presented in other formats, such as part of the employees' handbook or as a part of individual building emergency plans, etc. If they are not included as part of an emergency-response plan, then they must be made available to all employees in some other form such as instructional warning signs or diagrams.

Specific hazards to be addressed for employees may vary from facility to facility. A typical list is included below:

- Ice
- Blizzards
- Bomb threats
- Civil disorders
- Earthquakes
- Fires
- Flooding
- Hurricanes
- Tornados
- Hazardous-materials releases.

Operations-Recovery Planning

The operation-recovery plan can be a separate document or be a part of an overall facility

EOC CHECK LIST

EMERGENCY MEDICAL OPERATIONS

DEFINITION: Coordinate emergency triage and first aid services. Oversee efforts of volunteer medical support teams. Provide ongoing medical services as required and possible.

EQUIPMENT REQUIREMENTS: Hard hats, flashlights or lanterns, first aid kits and medical supplies, blankets, stretchers/some patient bearing capability, water, communications (hand-held radios) or messengers/couriers for the EOC, transportation for evacuation, sanitary supplies-plastic bags w/ties, tissue, access to disposal area.

EMERGENCY ACTIONS
(Complete as necessary)

- ❑ Determine extent of injuries and triage all casualties
- ❑ Administer appropriate basic first aid
- ❑ If communications permit, call 911 and request assistance. Follow other medical emergency procedures; contact Medical Emergency team, Ext. 465 and instruct people at top gate and lower gate to direct emergency vehicles
- ❑ If offsite fire and/or paramedic support is provided, coordinate patient handling with those units
- ❑ If support is not provided have EOC attempt to contact local medical facilities and advise of medical evacuations contemplated. Secure instructions
- ❑ Coordinate with the Director of Emergency Operations in the EOC for transport requirements
- ❑ If contact with offsite medical facilities cannot be made, identify a Casualty Collection Point (CCP) location within the site area. Move casualties to that location using available means
- ❑ Provide casualty care personnel to the CCP. Ensure medical supplies, blankets, water and other necessary items are supplied to the CCP
- ❑ Coordinate with the Supply representative in the EOC for materials/supplies required
- ❑ Ensure that casualties are identified, tagged, and properly tracked as they are relocated or evacuated from areas of the site
- ❑ Provide identification of all casualties to the Employee Relations representative in the EOC
- ❑ Estimate future resource needs and give that information to appropriate representatives in the EOC
- ❑ Address the special needs of casualties who become mentally distressed
- ❑ Be prepared for aftershocks (earthquake event) and ensure casualties are properly protected in the CCP
- ❑ Assist the EOC in emergency planning as requested. Recommend priorities for medical or first aid support.

Fig. 11b-3. Example EOC checklist.

plan. A brief outline of this plan is included here for reference.

Short-Term Plan

- Priorities for restoration and recovery
- Employee considerations
- Back-up operations (e.g., emergency power, payroll, communications, data retrieval)
- Facilities inspections
- Replacement of damaged equipment and supplies
- Timelines for re-establishing essential operations.

Long-Term Plan

- Facilities reconstruction
- Financial planning
- Consolidation-relocation considerations
- Legal issues
- Identification of assistance programs.

Training and Exercises

Training for personnel who will be involved in either response or recovery efforts is as important as the plan. As noted earlier, the existence of a written plan does not signify the quality of physical preparedness at a site or the operational readiness of key personnel. The ability to respond effectively and recover from an emergency is directly related to the quantity and quality of training.

A brief outline of an overall facility training program for emergency preparedness is included below.

Employee and Management Orientation

Provide one-to-two-hour-long group sessions focused on the hazard assessment, orientation to the emergency plan, and expected employee actions.

Response Team Training

Conduct hands-on training using professionals to provide the various teams with sufficient background to *safely* take emergency-response actions if necessary

Management Team Workshops

Provide response training to discuss the plan in light of problem situations. These develop an awareness of what could happen in an emergency environment.

Exercises

EOC training may be conducted as *desk-top* exercises with problem scenarios, involving just the EOC, or as full-system exercises involving the EOC, emergency and support teams, employees and field emergency actions.

Chapter

12

Risk Management

FOREWORD: The Facility Manager's Perspective

Donald G. Eagling

Risk management is an important part of a comprehensive earthquake safety program. It is inherent in establishment of lateral force criteria for the design of buildings and equipment, including research facilities and operations. Building codes, such as the *Uniform Building Code* (UBC) (Ref. 1), establish *minimum* seismic requirements for life-safety and essentially provide protection against collapse. Damage control is not the prime objective. Many code provisions, such as limitations on story *drift* (deflection between successive stories based on prescribed lateral forces), have the effect of reducing damage, but the real earthquake will cause deflections much greater than *code deflections.* When damage control is an important consideration, design must account for these larger deflections. This protection is not inherent in the code. One approach is to analyze a structure with the objective of predicting or estimating the location and extent of damage that will probably result from a major earthquake. *In this way, additional attention to design detail can be applied specifically in the area of concern to "buy insurance" against damage for little extra cost; this is an example of good risk management.*

Liability is a legitimate concern in risk management. Often, however, this concern is translated into legalistic solutions, rather than practical solutions that do more to mitigate seismic hazards. For example, the code is generally not retroactive, so it is not legally incumbent upon the *responsible official* to upgrade an existing building to current standards. Furthermore, some earlier editions of the seismic code made it *legally* possible to design and construct hazardous buildings such as the Olive View Hospital, which was destroyed in the 1971 San Fernando, California earthquake or the apartment buildings destroyed in the 1994 Northridge, California, earthquake. Nonductile reinforced-concrete frame buildings such as the five-story Kaiser Permanente office building suffered partial collapse in the Northridge earthquake.

Presidential Executive Order 12941 (Ref. 2) adopted minimum standards for assessing the seismic safety of existing Federally owned and leased buildings and mitigating unacceptable risks in them. The Department of Energy (DOE) issued criteria for the evaluation, modification, or upgrade of existing facilities in DOE-STD-1020-94 (Ref. 3) which also covers new construction for natural phenomena hazards. In essence these new regulations, unlike the seismic provisions of the UBC, are retroactive.

A decision not to review an existing building because it was once designed to code is a legalistic solution to avoid liability, but it does nothing to mitigate seismic hazards that may exist. This legalistic position under the protective umbrella of the code is becoming

more difficult to assume because the engineering profession is now much more aware of hazardous buildings that have been *built to code.* Also, in recent years, courts have made substantial monetary awards to occupants of poor buildings injured in earthquakes. One cannot be certain to avoid liability by remaining ignorant of hazards. The legal issue may well be whether or not such a building is commonly known by members of the profession to be hazardous.

The risk of liability should be managed carefully when structural hazards are revealed as a result of seismic safety surveys and reports. For example, if a building is reported to be a collapse hazard, the official responsible for the safety of the occupants should take steps to mitigate the hazard. It is important to seek funds actively to abate the hazard and to inform the occupants that the building is deficient. While these steps will not guarantee immunity, failure to take them certainly increases liability.

The problems of funding rehabilitation work are usually difficult, and achieving solutions is time consuming. The longer hazards continue to exist, the greater the risk of liability becomes.

A public agency cannot legally go out of business; therefore, it cannot spend its available funds so heavily for rehabilitation that it cannot fulfill its prescribed missions. This fact weighs against liability, but does not provide immunity. In the event that a damaging earthquake results in litigation, the pertinent issue is: what funds were available to responsible officials and what were they used for? With this in mind, one approach to managing the risk of liability is to take basic risk-reducing steps that can be identified immediately (assuming that the hazard cannot be easily abated in a short time), then follow due process to find a permanent solution to the problem. Examples of emergency steps are:

- adding temporary supports
- reinforcing structural joints
- installing epoxy grouting
- removing potential hazards
- changing or reducing the occupancy loading.

Normally, emergency funding can be found for such purposes while the more time-consuming tasks of evolving a permanent solution and developing adequate funding take place. *The important point is that responsible action (within constraints) must take place if liability is to be minimized.*

It is likely that a seismic safety survey will turn up a number of structurally deficient buildings and facilities. This is a common result when such reviews are carried out. For years, many buildings were designed with nonductile, reinforced-concrete frames which were then permitted by code, but are now known to have poor seismic resistance. Many older buildings have no formal or predictable lateral-force-resisting system. Sometimes building alterations have reduced or destroyed the resistance incorporated in the original design. The point is, a seismic safety survey will likely present the responsible official with a multiplicity of hazards and risks to manage.

It is important to mitigate the risks on the basis of priority, but it is even more important for life safety not to get bogged down in a complex series of studies or a methodology that slows the process of abatement. A simplistic priority system based on due process and responsible professional judgment is sufficient. As with the Richter scale for measuring the magnitude of earthquakes, it is not as important that the result is accurate as it is that relative size (or priority) is easily and quickly established.

The same selection principle should be applied when seismic safety surveys are initially carried out. That is, the priority system for the sequence in which buildings and facilities are surveyed should be simplistic and direct. Obvious problems, possible collapse hazards, and high-risk facilities, such as those with hazardous dispersible contents, should be reviewed early. High-occupancy buildings and lifeline facilities also should be early on the list. It is important that the survey is not held back by an academic approach to the multiplicity of potential hazards and the complexity of the problem. *The recommendation is to keep the approach simple, rely on good professional judgment, and move forward expeditiously.*

One stumbling block to seismic safety is the ever-present concern for accurately estimating the intensity of the potential earthquake that a given site might experience. For various

political, academic, and psychological reasons, the immediate, basic need to find out whether a brace is indeed missing too often becomes secondary to guessing how big the earthquake is going to be. Commonly, the problems uncovered in a seismic survey have less to do with lateral force criteria to be applied than with obvious or simple design deficiencies such as missing links, brittle members or connections, lack of continuity, or just poor construction. When a building or facility is found deficient, the size of potential seismic input is only one of the considerations that may be brought to bear on corrective measures. Usually there is ample time to develop detailed lateral force criteria after the real problems are revealed. Often design criteria for strengthening an existing building are more dependent on the deficiency to be corrected than on seismic input.

The recommendation is: don't delay the seismic survey in order to study the potential seismicity of the site. Experience shows that this approach is not good risk management.

The design criteria for new buildings are rather well established in building codes, but this is not true for rehabilitation work. Here good risk management requires more careful consideration. The lateral force provisions in the code provide good guidelines for rehabilitation design, but often, lateral force resistance is only part of the problem. As discussed, brittleness, lack of continuity or redundancy, deflections, poor detailing, poor workmanship, and many other possible deficiencies may exist. From the standpoint of risk management, it is even more important than in the design of new facilities that the designers of rehabilitation work give particular attention to seismic structural diagnosis and criteria development.

Once it is determined that a building has a serious structural deficiency that must be corrected, another kind of problem is often present. The building may have other code deficiencies by current standards that are not central to the main hazard. That is, the main structural deficiency may be a collapse hazard, but the other code deficiencies may not present life-safety hazards. The question may then arise: will responsible officials (or the engineers who design the rehabilitation work) be placed in a position of liability if the design does not correct all of the deficiencies by current code standards? Often it is not economically feasible or even good risk management to correct everything. For example, funding may be better used to correct two collapse hazards rather than spend it all on one building to bring everything up to code.

The recommendation is to achieve the most life safety for the funding available, but mitigate possible liabilities for the design professional by careful due process. For example: a criteria board can be established consisting of professionally knowledgeable members such as the responsible rehabilitation designers, the plan checker (preferably an independent consultant), and the Building Official (the in-house person responsible for enforcing the code for design and construction).

The authority to set seismic criteria, not covered by the UBC (Ref. 1) or DOE-STD-1020 (Ref. 2), should then be officially delegated to this *expert* group. Often detailed criteria needed for seismic retrofit will not be available from building codes or regulations. Also, it may not be practical or possible to rid an existing building of non-conforming materials that are not life threatening from earthquake hazards.

Court judgments have held that the responsible authorities are immune from liability for acting in a discretionary manner if they have the legal authority and expert knowledge to do so. The authority must be properly delegated, and *knowledge* implies professional judgment in the practice involved. The engineer in responsible charge of the retrofit design will be reasonably protected by the due process involved, assuming that he or she fulfills his or her design-responsibilities satisfactorily.

California engineers generally believe the buildings they design will be subjected to a damaging earthquake during the lifetime of the structure. They do not consider the hazard to be one that in all likelihood will not occur. They are also aware that the earthquake is the real *master inspector.* If there is a gap in the lateral-force-resisting system, the consequences will be more serious than statistical. This point of view has a very positive effect upon risk management as applied to design. As an aside, it is interesting to note that the probability estimated for a magnitude 7 earthquake in the San Francisco Bay area is now based on the 30-year period commonly used for home mortgages.

In areas of the country where earthquakes are very rare events, it is difficult for engineers to take them seriously, particularly in the design

of conventional structures. It is a fact that extremely few buildings in the midwest and east have been designed for earthquake safety, even in those areas where it is well known that damaging earthquakes have occurred in the historical past. Many buildings are constructed of unreinforced unit masonry, one of the building types particularly susceptible to earthquake damage and collapse. A great deal of progress could be made by simply avoiding the use of unreinforced unit masonry in new construction.

In *earthquake country*, the choice of criteria for seismic design can be a relatively simple matter, if one believes that the great earthquake is imminent. In this case, the design earthquake can be taken relatively close to the maximum expected earthquake. This might correspond to 0.2g base shear using the static lateral force design approach or to 0.8g using ground spectra acceleration for dynamic analysis. This design approach is based on what is known as the *minimax decision*, because it minimizes the maximum losses in the future.

In areas of the country where the potential for a great earthquake exists but the probability is extremely low, the choice of criteria may be more difficult. The maximum expected earthquake is not a practical choice for the design of most conventional structures in such areas. A *more fundamental consideration relates to the decision of whether or not to design for seismic forces at all. However, considerable seismic resistance can be achieved for very little extra cost by simply applying the principles of static lateral force design and making sure that the system is continuous and ductile.* The insurance available in such a minimal approach is a true bargain in risk management. The lateral force factor to be used is of secondary importance. However, if the building code values are used, one has some assurance that this choice is properly coordinated with its other provisions.

Risk management deals primarily with a variety of nontechnical issues that must be carefully managed in a comprehensive earthquake safety program. The intent of this discussion is to provide facility managers with practical guidance through the maze of socio-political, legal, and economic risks that may impede the progress and success of such a program.

High on the list of effective risk-management techniques related to seismic safety is the so-called *third-party plan check* that is described elsewhere in this book. This independent plan check, together with proper field inspection of construction (whether new or rehabilitation work), is highly recommended as one of the best ways of ensuring seismic safety in structural design and construction. This important topic is discussed in detail in Chapter 12a, *Quality Assurance by Peer Review*.

In Chapter 12b, *Risk Management Analysis*, both practical and technical issues of risk management are discussed, providing techniques for dealing with the probabilistic nature of earthquakes and illustrating methods of relating hazards, mitigation costs, and probability to management decisions. Included are examples of the decision-tree analysis technique for diagramming and tracking the risk management decision process, forecasting rare events, and dealing with multiple hazards.

References

1. *Uniform Building Code*, International Conference of Building Officials, Whittier, California, 1994.

2. *Seismic Safety of Existing Federally Owned or Leased Buildings*, Presidential Executive Order 12941, Washington, D.C., December 1, 1994.

3. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, DOE-STD-1020-94, Washington D.C., April 1994.

Chapter

12a

Quality Assurance by Peer Review

Frank E. McClure

Introduction

DOE Order 5700.6C (Ref. 1), *Quality Assurance*, (QA) requires that design and construction of DOE facilities incorporate the necessary review requirements to ensure that established program quality assurance objectives are met in the design per se, construction contract documents, and actual construction. *Project Peer Review* (PPR) is one of the best ways to ensure that a structure can resist earthquakes.

The American Consulting Engineers Council (ACEC) and *American Society of Civil Engineers* (ASCE) have undertaken a very significant program to improve the quality of constructed projects. Their joint publication, *Project Peer Review Guidelines: 1990*, (Ref. 2) is an excellent reference for persons involved in PPR. It is intended to help implement the concepts presented in the ASCE *Manual of Professional Practice*, Number 73, *Quality in the Constructed Project*, 1990, Chapter 13, *Peer Review*.

The PPR process is intended to enhance the quality of a constructed project by providing an external review of design assumptions, project management, and final design documents. However, responsibility for the structural design remains with the design organization's *Engineer of Record* (EOR). The written agreement implementing the peer review should state this unambiguously.

PPR is an autonomous and objective review of a proposed project by qualified engineers who hold neither a personal interest or claim in the project, nor any conflicts of interest. The reviewer should be in no way beholden to the commissioning agency, hereafter referred to as the *client*, who engages and pays them, nor to the EOR.

PPR is a documented, critical review performed by peers who are independent of those who performed the work, but who have technical expertise and experience in similar projects at least equivalent to those who performed the original work. PPRs can vary in scope from just reviewing the general design criteria to making an in-depth, critical review and evaluation of design criteria, design, and construction contract documents. An in-depth review requires interpretation and mature judgment in addition to normal technical reviews for compliance with DOE Orders and Standards. The *peer review* has a specific purpose, scope, format, and duration, which should be documented and agreed to by all parties.

The purpose of PPRs is to ensure proper designs and procedures, evaluate new or especially innovative designs, and improve project quality. PPRs are not organizational, management, *constructability*, or *value-engineering* reviews.

Review Requirements in DOE Order 6430.1A

DOE Order 6430.1A, *General Design Criteria*, (Ref. 3) requires that facility designs incorporate the necessary QA provisions to ensure that established program and project objectives are satisfied. The assurance that project construction documents (drawings and specifications) conform to project design codes and standards is critically important in satisfying those objectives.

For buildings and other structures designed to resist earthquake forces, Order 6430.1A requires that an independent or peer review of the seismic design be made for facilities and buildings where a seismic event would risk life-safety or large economic loss. Reviews are required at two stages; the first at the end of preliminary design (Title I, preliminary drawings and specifications) and the second just before the final design (Title II, final working drawings and specifications) is complete. An additional review at 50% completion of Title II documents also is recommended. It is intended that this additional review preclude early incorporation of errors and/or omissions so that later in the final design process the EOR will not be reluctant to make the changes necessary to correct them.

DOE Order 6430.1A has QA requirements related to review of structural design calculations and construction documents (drawings and specifications). Specifically, Section 0140, *Quality Assurance*, requires that an adequate QA program provide four assurances:

- The design will satisfy program and project requirements
- That prepared drawings and construction specifications adequately address QA requirements
- Construction can be performed in accordance with the design
- Tests confirm the adequacy of the design and the quality of construction and manufactured components, where appropriate.

The Order also requires that provisions be made for review and checking of design calculations, drawings, and construction specifications by qualified personnel other than those responsible for the original design. To the extent practicable, and particularly in the case of innovative designs, designs must be reviewed by consultants competent in construction or manufacturing techniques to confirm practicability of construction or manufacture.

The QA requirements in DOE Order 6430.1A take on the aspects of *plan-checking* as discussed in the *Uniform Building Code* (UBC) and go beyond the usual scope of work for PPRs, as discussed previously.

Review Requirements in DOE STD-1020

DOE-STD-1020, *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities* (Ref. 4) which implements DOE Order 5480.28, requires that designers use special QA procedures and that their work be subject to independent peer review for facilities in *Performance Categories* PC-2 and above.

To achieve well-designed and constructed earthquake-resistant facilities or to evaluate the seismic vulnerability of existing facilities, it is necessary for designers to:

- Understand the seismic response of the facility
- Select and provide an appropriate structural system
- Provide seismic design detailing that allows tough ductile response and avoids premature failures caused by instability or low-ductility response
- Provide a material testing and construction inspection program that ensures that construction complies with the intentions of designers.

All DOE *structures, systems, and components* (SSCs) must be designed or evaluated using an earthquake engineering QA plan as required by DOE Order 5700.6C and DOE STD-1020. The level of rigor in such a plan should be consistent with designated performance categories and their performance goals.

For Performance Categories 1, 2, 3, and 4, QA plans should include a statement (on the design drawings) by the EOR explaining the earthquake design basis including:

- Description of the lateral-force-resisting system

- Definition of the earthquake loading used for design or evaluation.

Seismic design or evaluation calculations should be checked for numerical accuracy and for theory and assumptions. The calculations must be signed by the responsible engineer (EOR) who prepared the calculations, the engineers who checked numerical accuracy, and the engineers who checked the theory and assumptions.

For new construction, the EOR should specify a material testing and construction inspection program. In addition, design engineers should review all testing and inspection reports and periodically make site visits to observe compliance with plans and specifications. For certain circumstances, such as the placement of reinforcing steel and concrete for special ductile frame construction and welding steel moment-resisting joints, the EOR should arrange to provide a specially qualified inspector to continually inspect the construction and to certify compliance with design and specifications.

For Performance Categories 2, 3, and 4, DOE-STD-1020 requires that all aspects of the seismic design and evaluation include independent peer review. The seismic design or evaluation should include design philosophy, structural system, construction materials, design/ evaluation criteria used, and other factors pertinent to the seismic capacity of the facility.

The review need not provide a detailed check, but rather identify oversights, errors, conceptual deficiencies, and other potential problems that might affect facility performance during an earthquake.

The peer review is to be performed by independent, qualified personnel. Peer reviewers must not have been involved in the original design or evaluation. If they are from the same company or organization as the designer/ evaluator, they must not be part of the same project (or program) or be influenced by cost and schedule considerations. Individuals performing peer reviews must be licensed civil or mechanical engineers with five or more years of experience in seismic evaluations. It is very beneficial to have peer reviewers participate early in the project, such as the start of preliminary design so that rework can be minimized.

Peer Review at Lawrence Berkeley Laboratory

In the aftermath of the destructive 1971 San Fernando earthquake in southern California, the Facilities Engineering Department of the University of California's Lawrence Berkeley Laboratory (LBL) implemented an independent seismic design review procedure long before there were DOE PPR requirements. The stimulus for this independent seismic design review policy resulted from a 1971 seismic vulnerability study that showed that many buildings were not properly designed to resist earthquakes even though the University of California required designers to follow the seismic provisions of the UBC in effect at the time of construction.

The use of independent seismic review has proved to be prudent risk management as exemplified by the successful earthquake performance of public schools in California that had been constructed and approved under the procedures of the *California Field Act*. The Field Act requires, among other quality assurance procedures, an independent *plan check* of the structural design calculations and the construction documents by the Division of the State Architect.

At LBL, independent structural and seismic design reviews are made by consultants experienced in seismic design and evaluation of buildings and in field investigations of earthquake damage to buildings. These reviews are usually made in three stages for facilities having potential risk to life safety, facilities using hazardous materials, and facilities with a potentially large economic loss.

The first review is made at the end of preliminary design, or Title I, services. The second review is made at 50% completion of Title II construction documents and design calculations. The third review is made when the final design (Title II construction documents and design calculations) are about 95% complete, before bids are taken, so that resolution of the final comments can be incorporated in final bid documents. At the completion of the reviews, when comments and questions are resolved, the review engineer submits a simple report recording that the design and construction documents meet the intent of the project program and design criteria.

Structural and seismic design reviews cover design philosophy, criteria, framing systems, construction materials, and other factors pertinent to the seismic capability of the proposed facility. *Particularly important is the check for continuous load paths and the adequacy of their strength, stiffness, and ductility to transfer the seismic forces from the points of origin and application to the final points of resistance.*

In summary, peer review by an independent consultant or peer group need not provide a detailed check of the spacing of the reinforcing steel or numerical accuracy of design structural calculations. Rather, it is a review to identify oversights, errors, conceptual deficiencies, and other elements likely to cause problems during construction, after the building is completed, and during earthquakes. Peer reviews catch costly design mistakes in judgment, design criteria, and philosophy. This has been true at LBL for many projects, large and small, since 1971. For major facilities, an independent peer review can more than pay for itself by uncovering design deficiencies before bids are taken and construction starts.

Problem Areas With Project Peer Review

The potential liability of the peer reviewer is one of the first questions asked by all parties to the peer review process. Professional engineering societies adhere to the principle that responsibility for the design/evaluation must remain with the design organization's EOR. This is in keeping with laws regulating the licensing of engineers. Consequently, they strongly recommend that written agreements outlining the scope of the peer review and the roles and responsibilities of all parties be entered into prior to the start of peer review work. They have proposed sample contract indemnification clauses for the inclusion in the written agreement to limit the liability of peer reviewers.

The fact that the EOR's work will be subjected to the peer review process should be incorporated in the *request for proposal* (RFP) as part of the *Architect/Engineer* selection process. This is essential if the client is to avoid cost extras for work associated with peer review.

The ACEC has prepared several manuals covering peer review to guide clients (who authorize and pay for peer reviews), EORs, and peer reviewers in developing written agreements specifying the scope of review work and responsibilities of the parties to the review.

Unless a written agreement is executed defining the responsibilities of all the parties, clients may believe that peer reviewers are offering a *Good Housekeeping Seal of Approval* and accepting co-responsibility for the design with the EOR when they agree to review the project seismic design. If an adverse situation should arise, clients may attempt to hold both the EOR and the independent peer reviewer liable.

There is little legal history on the liability of the peer reviewers. However, it is reasonable to assume that in this litigious society, it is difficult to be truly free from liability. Reviewers should use prudence and good judgment, conduct their review appropriately, and keep good documentation.

There can be some *human problems* with the selection of independent peer reviewers, how the peer review process is handled, and the attitude and conduct of clients, EORs, and peer reviewers.

Clients should avoid engaging PRs who are too closely linked personally with the EOR through long-time personal relationships, participating in joint ventures, or working together on professional committees. There may be a reluctance to criticize a friend's design or point out omissions if there have been prior close personal relationships. These types of relationships preclude truly independent judgments by PRs.

Also, clients should avoid interchanging the roles of EORs and PRs on projects. This tends to encourage less rigorous peer review because PRs on a current project realize that EORs might become PRs on their future projects.

The review should be started early in the design process. Significant design errors or omissions discovered at completion of the construction drawings and specifications are often difficult and costly to correct. Consequently, EORs may defensively try to justify errors or omissions that, if found earlier, could have been easily corrected.

An unfortunate situation could arise in which PRs and EORs disagree, and EORs refuse to make the changes suggested by PRs. Clients can be left in the unenviable position of trying to sort out

which one is correct or try to force a compromise. In such cases, clients may be forced to engage another third-party reviewer to help them resolve the dispute. This situation can result in delays in meeting project deadlines, additional review costs, and increased costs for inflation. Every DOE site should have one qualified person who is delegated responsibility for enforcing the building code. This person, usually designated as the *Building Official* for the site, has the authority to resolve such issues. It is most important, however, that the EOR not be required to reduce safety factors as a result of peer review. This could negate the EOR's professional responsibility for the design. Under no circumstances should a PR supplant the original EOR whose work the PR has reviewed.

Potential *conflicts of interest* can arise. It is recognized that PRs should not take commissions to perform detailed *plan-checking* reviews on projects for which they were the project peer reviewers. Clients should be aware that PRs could unconsciously use the peer review process to diminish the professional reputation of a fellow competitor when they aggressively look for everything wrong with the EOR's design. Sometimes a PR attempts to show up the EOR by finding as many errors and omissions as possible in the EOR's design to show the EOR that the PR is a better engineer.

Generally, LBL has had no significant problems in any of these areas over the 25 years that independent plan checks have been used there. On the other hand, significant deficiencies have been corrected or the design has been significantly improved in over 90% of the projects reviewed. Plan checking fees have been nominal, usually less than 0.2% of construction cost.

The peer review should be a *friendly* review of a fellow design professional's work. The review is initially aimed at making every effort to ensure that serious errors and omissions are found and corrected. It should not be a *nit-picking* type of review.

References

1. DOE Order 5700.6C *Quality Assurance*, U.S. Department of Energy, Washington, D.C., 1991.

2. *Project Peer Review Guidelines*, American Consulting Engineers Council/American Society of Civil Engineers, New York, New York, 1990.

3. DOE Order 6430.1A, *General Design Criteria*, U.S. Department of Energy, Washington, D.C., 1989.

4. DOE-STD-1020-94, *Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities*, U.S. Department of Energy, Washington, D.C., 1994.

Chapter

12b

Risk Management Analysis

Jack R. Benjamin

Introduction

Risk management from the technical point of view is the formal process by which hazards are mitigated under the constraint that all acceptable mitigation measures cannot be accomplished instantaneously. In the simplest case, risk management may determine that an acceptable mitigation measure involves only following a check list to ensure that an important item in operation or maintenance is not forgotten. At the other extreme, there are important facilities that may be of questionable structural integrity—yet are subject to diverse human and natural hazards—and for which resources for hazard mitigation are not only limited but become available as a function of time. The problem in this latter case is that of obtaining the best allocation and expenditure of scarce resources at each instant of time.

Major technical concerns in risk management of important facilities include the uncertainties inherent in the hazards and effectiveness of any mitigation effort. It is common for hazards to be described in probabilistic terms by level and occurrence over a period of time. For example, the earthquake hazard may be described by an effective peak acceleration level that has a 10% probability of being exceeded in 50 years, or a wind hazard may be described in terms of a velocity with a return period of 100 years. In contrast, the effectiveness of any mitigation effort is traditionally described in deterministic terms. For example, the structure designed to the code should not collapse even under severe earthquake load and should sustain only minor structural damage during moderate ground shaking. These estimates of behavior are deterministic, because there is no estimate of the probability of different damage levels. Risk management, therefore, requires combining diverse types of forecasts, both deterministic and probabilistic.

The Decision Tree

The decision tree provides a useful device for diagramming and systematically keeping track of risk management decisions. The tree is the framework for evaluating alternative mitigation plans and, because the tree can be updated, it can continuously model the decision situation as a function of time.

A simple decision tree for two earthquake-related hazards is shown in Fig. 12b-1. Assume that the responsible official or panel of professionals is to decide on the risk management program. This *decision maker* is considered to be at Apex 1. With an available $100,000 expenditure, it is further assumed that only two different mitigation plans are acceptable. With Action A, all resources go to major structural

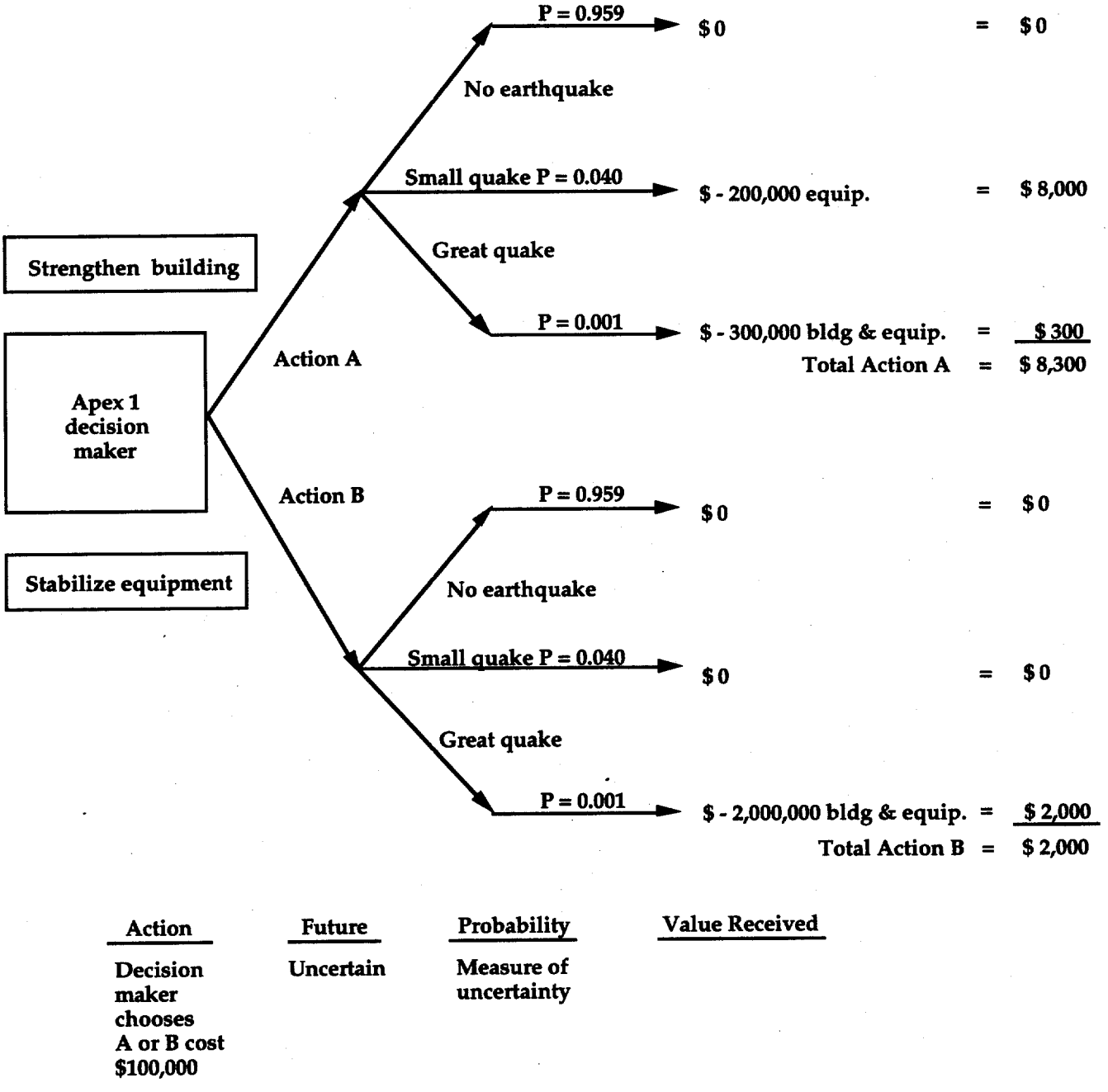


Fig. 12b-1. Risk management decision tree.

strengthening of the building, while with Action B all resources go to stabilizing critical equipment in the building. The equipment can be severely damaged by a low-level earthquake that the building survives. If the earthquake ground motion is very severe, the building structural system fails and the building collapses, destroying the critical equipment, even if it is stabilized. The earthquake that could cause the building to fail is a very rare event, while low-level earthquake ground motions frequently occur.

Now, if decision makers take Action A to strengthen the building and the future (one year)

includes the small earthquake, the equipment loss is shown to be $200,000. Similarly, if Action A is taken and there is a great earthquake, the total loss is $300,000, consisting of partial-building and complete-equipment damage. If no earthquake occurs, there is no loss or gain except that the $100,000 expenditure is a sunk cost.* In contrast, if the available funds are expended to stabilize the equipment (Action B), there is no loss with a small event, but a $2,000,000 loss (total loss of building and equipment) is estimated to be the consequence of very severe ground motion from a great earthquake.

Thus, the decision tree contains the consequence of taking an action and finding the future. The probabilities of occurrence of the hazard are noted on the tree so that the diagram contains all of the basic ingredients for the decision. The units of the consequence may be dollars or any other convenient and consistent measure of preference.

For simplicity, the *decision maker* in Fig. 12b-1 must choose either A or B. Action B reduces the possible loss from the occurrence of a frequent but small earthquake, while Action A reduces the worst possible loss from a great event. The optimum action, A or B, depends on the probabilities of occurrence of earthquake levels in any one year in the life of the facility. Two contrasting viewpoints exist in choosing Action A or B. First, if the $2,000,000 loss with a great earthquake is so large as to be completely unacceptable, the optimum action is A. This type of decision is called a *minimax decision* because it minimizes the maximum possible loss that can be experienced in the future. *This type of decision rule fits the case in which one of the possible losses is not acceptable, or the probability of occurrence of the level of hazard does not effectively influence the decision.*

In contrast, if the losses shown are severe but not catastrophic, the optimum decision can be determined by weighting the losses by the probabilities of occurrence and summing for each action. The optimum decision is then the one with the smallest weighted loss. This is known as the *expected-value* decision rule. Using this decision rule, the expected loss per year with Action A is 0.040 x $200,000 = $8,000 for the small earthquake and 0.001 x $300,000 = $300 for the great earthquake. The sum is thus $8,300 per year. With Action B, the expected loss is $2,000 per year, so that the optimum action is to stabilize the equipment and accept the small risk that the entire building with equipment could be a total loss in a great earthquake.

In most practical problems, a combination of the minimax and expected-value rules is employed. For example, if a third Action, C, is also possible with intermediate loss characteristics, the minimax rule could be used to eliminate Action B, and then the expected value rule used to choose between Actions A and C. The choice of decision rules is obviously at the discretion of the "decision maker."

Forecasting Rare Events

The most common frequency statistic employed with rare events is the return period, T. The return period is the average long-run time between events of the same description. That is, if the return period of the 1906 San Francisco earthquake is 200 years, over a million or so years, on the average, one such earthquake occurs each 200 years. The actual record would show considerable variability in the time between events, but the average time between events is 200 years. This does not mean that the next event is forecast for 1906 + 200 = 2106.

If the return period is 200 years and the event is equally likely to occur any year, the probability of occurrence in any one year is approximately $1/T = 1/200 = 0.005$. If the probability of occurrence in any one year is 0.005, the probability of nonoccurrence is obviously 1 - 0.005 = 0.995. The probability of nonoccurrence in any two years is then 0.995 x 0.995 = 0.990. Thus, the probability of at least one occurrence in these same two years is 1 - 0.990 = 0.010. The probability of nonoccurrence in 200 years of the 200-year event is $(0.995)^{200} = 0.37$, so that the probability of at least one occurrence is 1 - 0.37 = 0.63. Thus, the probability of occurrence of an event with a return period of T years in a time span of T years is approximately two-thirds.

The results of calculations of this type are given in Table 12b-1, in which return periods are related to the probability of occurrence in a given

*Note that to compare Actions A and B with the "do nothing" Action, in which the $100,000 expenditure is not made, requires that a future longer than one year be considered. It is assumed in this example that in the long run both Actions A and B are preferable to the "do-nothing" Action.

Table 12b-1. Return period data.

	Return Period in Years, T						
	10	20	50	100	200	500	1000
Approximate Annual Probability of Occurrence, p	0.1	0.05	0.02	0.01	0.005	0.002	0.001
Probability of Occurrence in T years	0.63	0.63	0.63	0.63	0.63	0.63	0.63
Number of Years, n, for which there is a 10% probability that the T Year event will be exceeded (90% probability of nonexceedance)	1	2	5	10*	21	53	105
Number of Years, n, for which there is a 20% probability that the T Year event will be exceeded (80% probability of nonexceedance)	2	4	11	22	45	111	223
Number of Years, n, for which there is a 50% probability that the T Year event will be exceeded (50% probability of nonexceedance)	7	14	34	69	138	346	693

Equation: $\text{Probability of Exceedance} = 1 - (1-p)^n$

$$n = \frac{\log(\text{Probability of Nonexceedance})}{\log(1-p)}$$

Note: The event with a return period of 475 years has a probability of exceedance of 10% (nonexceedance of 90%) in 50 years.

*As an example, the probability is 90% that the largest event in 10 years will not exceed that with a return period of 100 years or probability is 90% for T = 100 years event will be satisfactory for 10 years.

time span, nonoccurrence in the same time span, and the probability that the largest event in a given time span will be the event with a return period of T years. The latter follows directly by defining the T-year event as the largest event of interest.

These same basic procedures for calculating occurrence probabilities apply to fires, high winds, accidents of all types including automobiles, and all other rare events that can only be classed and counted in a time reference.

Multiple Earthquake Hazards

One of the more common combinations of hazard events is that of earthquake followed by fire. It is not satisfactory, however, to assume that fire is certain after a major earthquake, because historical evidence shows that this combination, although more common than other combinations of events, is relatively rare. The actuarial data on the occurrence of fires do not apply to fires associated with earthquakes, because the latter are either too rare to materially influence the statistics or simply not treated as a separate class. However, a good physical knowledge of a facility aids in subjectively estimating the fire hazard related to earthquakes.

The simplest way to analyze possible multiple-hazard events is to use an event tree, which is the subset of the decision tree dealing with the uncertain future. An example of an event tree of the occurrence of fire and earthquake is shown in Fig. 12b-2. Beginning at the left apex and moving to the right, the branching shows the sequence of events that are possible in the time period of interest.

For the example of fire and earthquake hazards, the possible events are:*

- No fire and no earthquake
- Fire without earthquake
- Earthquake with subsequent fire
- Earthquake without subsequent fire.

*Note that it is possible to have a fire occur at one time and an earthquake occur at a different time during the same year. However, the probability of this type of multiple hazard actually occurring is assumed to be so small that it can be neglected in this analysis.

Because fires and earthquakes are rare events, the most likely probability is that of no earthquake and no fire. Assuming that the fire of concern, without earthquake, occurs on an average of one time a year per 1,000 laboratory buildings, the probability of occurrence in any given year is 0.001 for a specific laboratory building. If the earthquake of concern is the 100-year event, the probability of occurrence in a given year is 0.01. If there is an earthquake, it is subjectively estimated by a knowledgeable professional that the probability of a subsequent fire is 0.2. Thus, the probability of earthquake and subsequent fire is then 0.01 x 0.2 = 0.002 so that the probability of earthquake and no subsequent fire is 0.008 (i.e., 0.01 x 0.8).

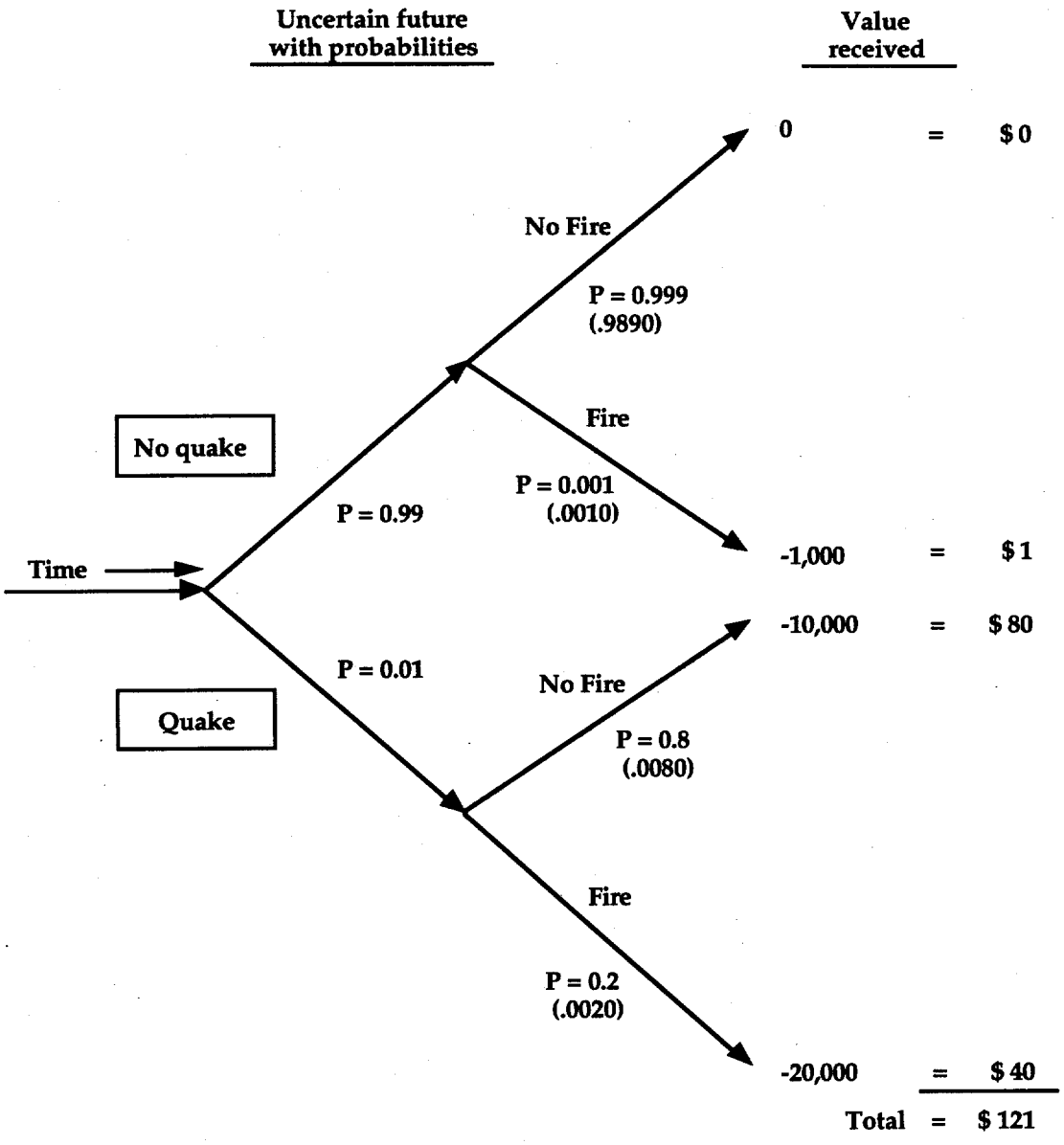
Assuming that losses associated with each event are estimated by analysis or judgment, the annual expected loss of earthquake and fire can be calculated. For example, if the loss caused by fire without earthquake is $ 1,000, the loss caused by earthquake without fire is $10,000, and the loss caused by earthquake with fire is $20,000, the total expected annual exposure for consideration in the mitigation plan (or for purchase of insurance) is:

Fire (no earthquake)	0.001 x 0.99 x 1,000	=	$ 1
Earthquake, no subsequent fire	0.008 x 10,000	=	80
Earthquake, subsequent fire	0.002 x 20,000	=	40
	Total annual exposure per building		$121

It is important to note that this weighting procedure assumes that the loss levels are within normal operational bounds and are acceptable. That is, if earthquake plus fire would result in total destruction (an abnormal and unacceptable outcome to some decision makers), while fire or earthquake by themselves do not eliminate long-range functional survival, the mitigation plan should take steps to prevent the possible losses resulting from total destruction.

Example of Analytical Risk Management

As an example of multiple-hazard risk management, assume that a risk management program is developed for a laboratory complex consisting of three buildings.



Total expected annual exposure from earthquake = $121

Fig. 12b-2. Event tree for fire and earthquake hazards.

Building A is an old masonry warehouse used to store laboratory supplies and spare equipment. It is likely to sustain total collapse in a major earthquake and moderate to total damage in a moderate earthquake. There is no fire hazard, unless the usage changes.

Building B is a modern one-story steel-frame structure housing very fragile laboratory equipment. Human occupancy is low level, but the fire hazard is high. There is a sprinkler system to prevent fire damage. In a major earthquake, the structure will sustain light

damage with light to severe equipment damage. A fire is certain to start; however, the sprinkler system was not originally designed to displace the same amount without breaking as was the building. Thus, it is estimated that the chance of the sprinkler system working following a major earthquake is only 25%. If the sprinklers do not work, then the equipment will be further damaged and the chance of building collapse is 50%. In a moderate earthquake, the types of hazards are similar to those in a major earthquake; however, the extent of possible damage is less, and the estimated probabilities of the possible damage states are different. For example, the probability of the sprinklers' functioning is estimated to be 50% (this probability could be increased if periodic maintenance were performed). If the sprinklers *do not work*, the chance of building collapse is estimated to be 20% because it is more likely that the fire department will be able to control the fire before collapse.

Building C is a two-story unreinforced concrete-block masonry structure with timber framing. It is used for offices and has a high level of human occupancy. The building has no sprinklers, but has a moderate fire hazard, caused by a gas leak or inadvertent trash fire, etc. In a major earthquake, there would be moderate to heavy structural damage, but collapses would be localized because of the many closely spaced partitions. If a fire should follow the earthquake after a break in a gas line, there would be time for evacuation with minimum human injury, but further structural damage to the point of total loss would very likely occur. During a moderate earthquake, structural damage would be light to moderate. The chance of fire following the earthquake is much less; however, the consequences could be either heavy or total structural damage. If a fire starts in *Building C* without an earthquake, the possible damage levels range from light to total depending on the arrival time of the fire department.

The return period of major earthquakes at the site is estimated at 200 years, while for moderate earthquakes it is 20 years. It is further estimated that a fire level causing damage to *Building B* occurs on an average of one time a year per 50 laboratory buildings of this type, and important fire losses in office buildings such as *Building C* have a return period of 100 years. The other conditional fire probabilities are estimated by responsible professionals either subjectively or by analysis. Costs of the different possible damage levels to the three buildings and their contents are listed in Table 12b-2.

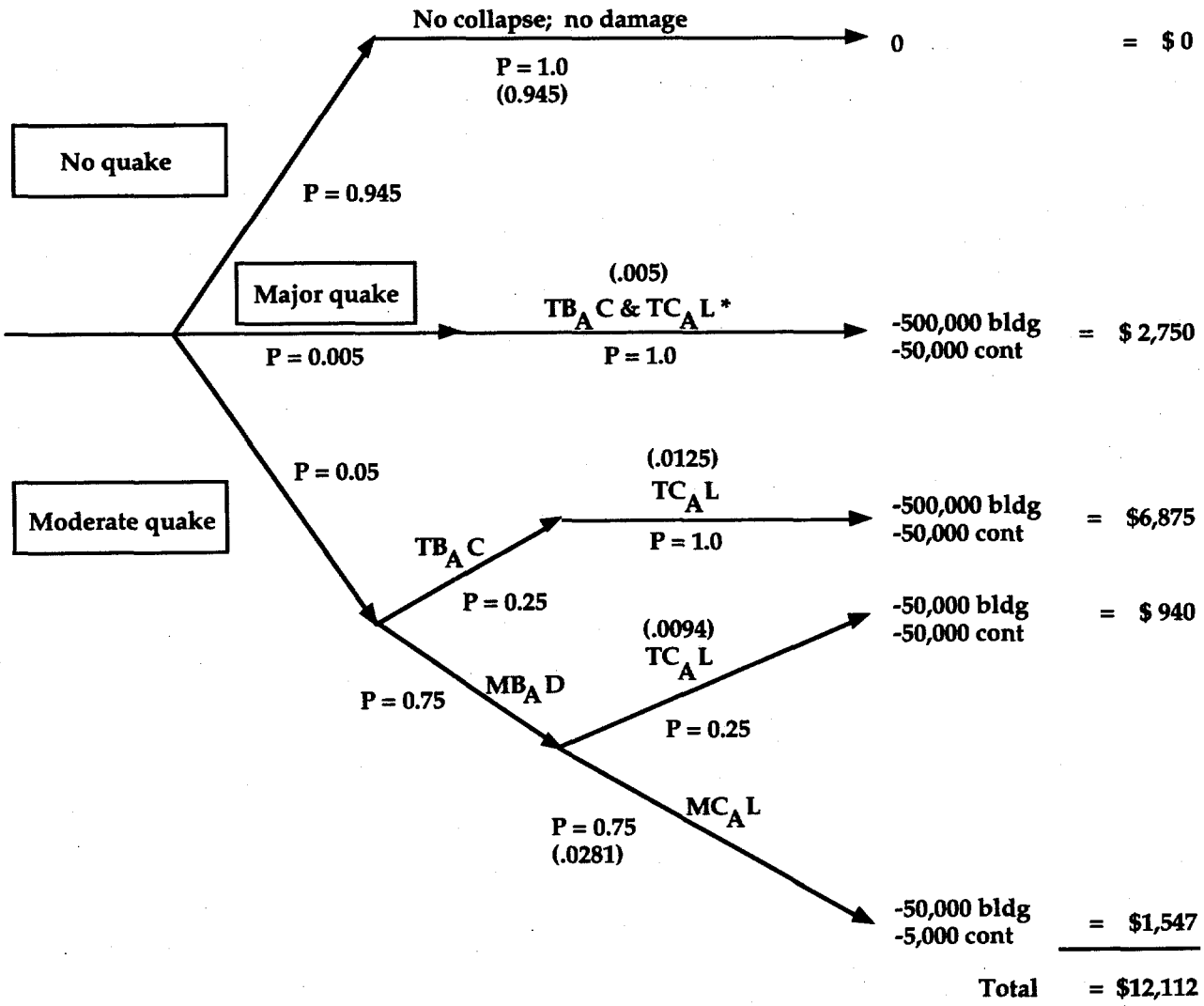
The first step in setting up the decision tree for the risk management program is to construct the event trees for each of the three buildings, realizing that these event trees will ultimately be merged to combine the hazard effects and consequences for all three buildings. Figs. 12b-3, 12b-4, and 12b-5 show the event trees for each building, respectively, along with the estimated costs of damage and estimated probabilities for each possible event.

Table 12b-2. Estimated costs of possible damage levels.

Building A		
Total Building A collapse (TB_AC)	=	—$500,000
Moderate Building A damage (MB_AD)	=	—$50,000
Total content loss (TC_AL)	=	—$50,000
Moderate content loss (MC_AL)	=	—$5,000
Building B		
Total Building B collapse (TB_BC)	=	—$1,000,000
Light Building B damage (LB_BD)	=	—$10,000
Severe equipment damage (SE_BD)	=	—$1,000,000
Moderate equipment damage (ME_BD)	=	—$100,000
Light equipment damage (LE_BD)	=	—$5,000
Building C		
Total Building C damage (TB_CD)	=	—$750,000
Moderate Building C damage (MB_CD)	=	—$200,000
Light Building C damage (LB_CD)	=	—$20,000
Lawsuits for injuries	=	—$1,000,000

Multiplying the estimated loss for each possible event branch by the probabilities along that branch and summing the products gives the expected annual loss for each building. These expected annual exposures are the basis for determining the cost-effectiveness of different mitigation alternatives.

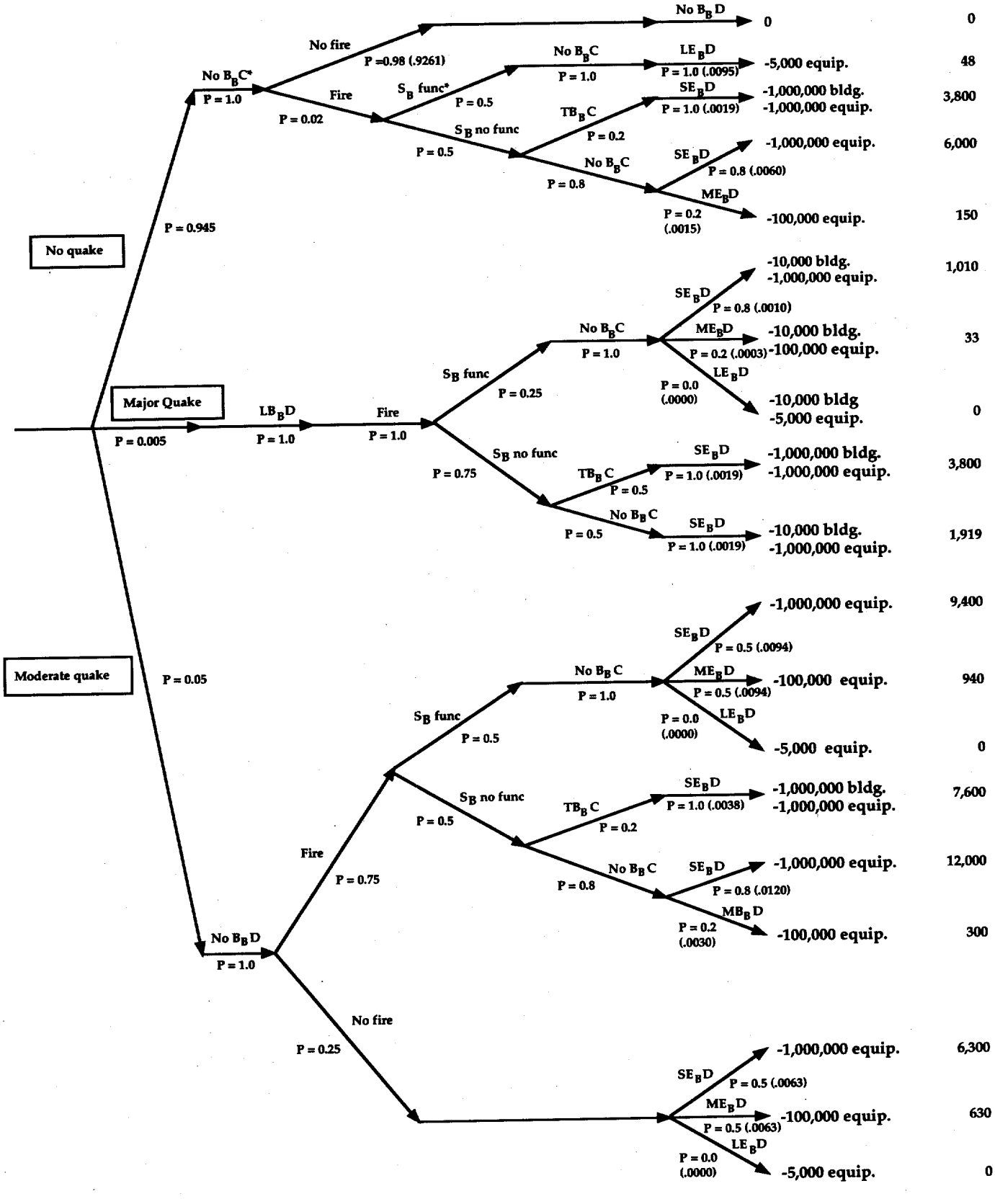
For example, considering no mitigation effort, the expected annual exposure for *Building B* from fire and earthquake hazards is $53,900. By providing a more aggressive maintenance and repair program for the sprinkler system in the building, the probabilities that the sprinkler system will function are increased to 80% for a fire following a moderate earthquake and 50% for a fire following a major earthquake. The probability that the sprinklers will function in a fire without an earthquake is also increased to 80%. The expected annual exposure with the improved maintenance and repair is thus $41,200, which is an expected annual savings of $12,700. If the annual cost of this maintenance and repair program is more than $12,700, then the mitigation is not cost-effective.



Expected annual exposure from earthquake = $ 12,100

***See Table 12b-3 for damage nomenclature**

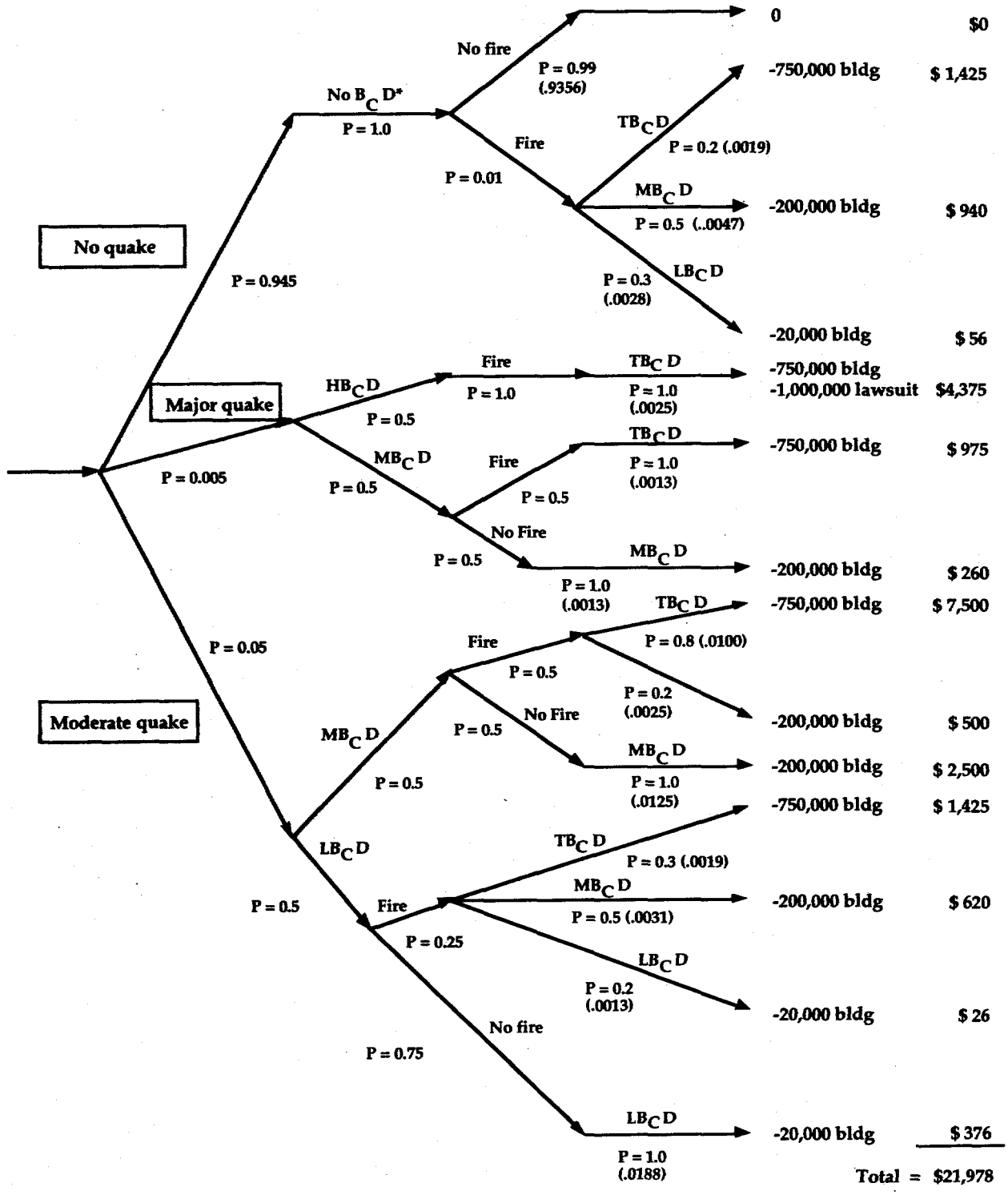
Fig. 12b-3. Event tree for Building A — No mitigation.



Expected annual exposure from earthquake and fire = $53,900

***See Table 12b-3 for damage nomenclature**

Fig. 12b-4. Event tree for Building B — No mitigation.



Expected annual exposure from earthquake and fire = $22,000

*See Table 12b-3 for damage nomenclature

Fig. 12b-5 . Event tree for Building C — No mitigation.

A second mitigation effort might be the modification of the equipment supports to lower the chance of damage during an earthquake.

Suppose that it is possible to lower the equipment damage level to moderate or light during an earthquake in which a fire does not start or the sprinkler system functions. Assuming that there is a 60% chance of moderate equipment damage during a major earthquake after which the sprinkler system functions, and a 80% chance of moderate damage following a moderate earthquake after which no fire starts or the sprinklers work, the expected annual exposure is reduced to $45,000, or an expected savings of $8,900. If equipment supports can be modified for less than $8,900/year, then this alternative becomes cost effective. It may be, however, that modifications to the equipment supports reduce the functional value of the equipment, rendering this alternative unacceptable, in which case a high-priced insurance premium may be the only acceptable alternative. It is interesting to note that if both the sprinkler-maintenance program and the support-modification program are implemented, the expected annual exposure is $28,800, or an expected savings of $25,100/year, which is greater than the sum of the savings considered independently.

Each of the three building hazard event trees can be used separately in the preceding manner, if each has an annual mitigation budget of its own. However, if mitigation alternatives for the three buildings are in competition for the funding available, then the event trees must be combined into a single event tree that encompasses all possible outcomes for all three buildings on an annual basis. Only in this manner can mitigation alternatives for one building be compared with those for another building, or with composite mitigation efforts for all three buildings.

The preceding examples illustrate both the complexities inherent in earthquake-related risk management problems and the systematic methodology for rationally evaluating and selecting mitigation alternatives that optimize the use of available funds.

Table 12b-3. Damage nomenclature.

Building A	
TB_AC	Total Building A collapse
MB_AD	Moderate Building A damage
TC_AL	Total content loss, Building A
MC_AL	Moderate content loss, Building A
Bldg.	Building
Cont.	Content
Building B	
TB_BC	Total Building B collapse
NoB_BC	No Building B collapse
LB_BD	Light Building B damage
NoB_BD	No Building B damage
S_B Func.	Sprinkler system functions, Building B
SB_B No Func.	Sprinkler system does not function, Building B
SE_BD	Severe equipment damage, Building B
ME_BD	Moderate equipment damage, Building B
LE_BD	Light equipment damage, Building B
Ept.	Equipment
Building C	
TB_CD	Total Building C damage
HB_CD	Heavy Building C damage
MBCD	Moderate Building C damage
LB_CD	Light Building C damage
NoB_CD	No Building C damage

Publisher of the

***Uniform Building Code*™**

Chapter

13

Model Code Related Services

FOREWORD: Facility Manager's Perspective

Donald G. Eagling

The *Uniform Building Code* (UBC), a widely recognized *model code,* is prescribed by the Department of Energy (DOE) for seismic design of buildings. It includes well-developed provisions for designing buildings for lateral force resistance to dynamic loads produced by earthquakes. Not everything one needs to know for seismic design is in the UBC, but in the hands of competent earthquake engineers it is a very effective document.

The UBC is published by the *International Conference of Building Officials* (ICBO), a nonprofit organization owned and controlled by its Class A Members who are *building officials* from UBC member cities, counties, states, or any governmental unit that has jurisdiction over building construction. Facility managers for DOE sites who are responsible for seeing that the UBC requirements are met in the design, construction, and modification of buildings and facilities may become Class A Members of the ICBO, providing they have a *population* to protect. Class A membership automatically entitles members to one vote in the code-change process.

The 1994 UBC consists of three volumes:

- *Administrative, Fire- and Life-Safety, and Field Inspection Provisions*
- *Structural Engineering Design Provisions*
- *Material, Testing and Installation Standards.*

In addition to the UBC, the ICBO publishes the *Uniform Mechanical Code* (UMC) and *UMC Standards*, the *Building Standards Magazine*, and many other useful documents.

The ICBO maintains an expert technical staff to provide plan-checking and code-interpretation services for members desiring such assistance. The ICBO has neither control nor legal authority over any federal, state, county, city, or other jurisdiction. The UBC is written so that it can be legally adopted by any jurisdiction as a statute governing construction. The *Building Official* (Class A Member) of a jurisdiction has the legal responsibility for enforcing it. The ICBO does not. The ICBO serves and supports *building officials*. It is the prime resource for advice and recommendations about code issues.

Suppliers and vendors who wish to have their products or systems (such as metal decking) listed by *ICBO Evaluation Services, Inc.*, (ICBOES) must have their products tested and certified by quality-control agencies as meeting ICBOES acceptance criteria. A complete report must then be submitted to ICBOES for review and approval prior to issuance of a *Research Report* by ICBOES. Products must be re-examined regularly to maintain this approval.

ICBOES *Evaluation Reports*, a publication that reports product tests and specifies how they must be applied and what values can be used in a given design, are sent to Class A Members bimonthly. Designers and managers who do not have access to these reports do not know the proper allowable values to use in seismic design; similarly, they do not have current listings of the products that are approved by the ICBOES. Manufacturers offering ICBO-recognized products for sale in periodical advertising and other sales literature must include ICBOES-listed design values. Vendors of products that have not been listed by the ICBOES obviously have no such restriction and may list design values that are not acceptable to the ICBOES.

If the ICBO staff is used for plan checking, Class A Members doing so receive a reviewed set of plans, plus a report setting forth specific ICBO-recommended corrections. It is up to Class A Members having jurisdiction whether or not the advice is acted upon. The ICBO does a thorough job of plan checking, covering all aspects of code application.

In 1994, the cost to join the ICBO and receive the *Uniform Building Code, Building Standards Magazine*, and the ICBO *Evaluation Reports* was $85 per year for most DOE sites. The fee depends on the population of the facility. Advice and interpretations (by phone or letter) are free. The cost for plan checking is reasonable; for example, the ICBO fee was about $1,770 in 1994 for a one-million-dollar construction contract, with a sliding scale producing proportionately lower fees as the project cost increases (e.g., for a 10-million dollar construction contract, the plan-check fee in 1994 was $11,770). If one is responsible for enforcing the use of the UBC and is not a Class A member, one does not have all the data needed to do the job properly. The *Evaluation Reports* and other published material provided by the *Conference* (ICBO) are of great technical importance to *Building Officials* as well as to designers.

Often provisions of the UBC are necessarily generalized. For example, the code does not provide a direct interpretation for the height to be used in seismic calculations for a building that is taller on one end than on the other. The formulas provided by building codes apply generally to flat building sites. As mentioned, advice and code interpretation are available from the technical staff of the ICBO to assist Building Officials in such special situations. Consulting with the staff is, of course, not mandatory, but may prove extremely helpful.

One last point is that the so-called third-party plan check, an independent design review, is a highly effective and inexpensive tool for seismic safety. DOE Order 6430.1A specifies that "An independent review of the seismic design shall be made for facilities and buildings where a seismic event can have a potential risk to operator lives, to public safety, or of large economic loss." This specification covers most buildings with occupants. It is not necessary to use ICBO for this purpose, although the ICBO staff does an excellent job. For fast turnaround, milestone reviews, and close-coupled interaction, it may be more practical to use independent consulting engineers. However, it is vitally important to employ competent, experienced earthquake engineers, not ones who are inexperienced or must rely entirely upon building codes for direction. For best results, the plan check should be performed by engineers with actual experience in earthquake damage surveys. If this is not practical, then it is advisable to choose professionals who have carefully studied earthquake damage reports and are competent in structural engineering for lateral force design. In any case, reviewers should be thoroughly familiar with the so-called *Bluebook*, the *Recommended Lateral Force Requirements and Commentary*, published by the *Structural Engineers Association of California*. (SEAOC). This is the reference document used by the ICBO to interpret the lateral force provisions of the UBC.

The *Uniform Building Code* (UBC) is only one of several good model codes published by nonprofit organizations engaged in the study and advancement of standards and criteria for safe building construction throughout the United States. The ICBO home office is located in southern California, in *earthquake country*, so the UBC is particularly sensitive to lateral force design. Practicing structural and geotechnical engineers who seek to improve seismic safety have generally pursued this goal through the advancement of the lateral force provisions of the UBC.

It is important to understand that the ICBO has no jurisdiction per se. It publishes the UBC and provides technical support services. The UBC becomes a legal document only after it is adopted in a statute or regulation by a governmental agency such as a state, county, municipality, or special district having legal

jurisdiction. Often the agency having jurisdiction adopts the code with exceptions or modifications.

Generally, most facility managers are not sufficiently familiar with the use of model codes, how they are changed, how to interpret code provisions, and how to use the consulting services that model codes provide. In particular, they need to know more about the ICBO simply because the *Uniform Building Code* is the model code specified for the seismic design and construction of DOE buildings in DOE Order 6430.1A and followed by DOE-STD-1020 for Performance Categories 1 and 2 corresponding to UBC *Standard Occupancy Structures* and *Essential Facilities*, respectively.

The UBC has been followed in DOE-STD-1020 for Performance Categories 1 and 2 because it is believed that more earthquake engineers are familiar with this code than other model building codes. The *Interagency Committee on Seismic Safety in Construction* (ICSSC), has concluded, however, that the following *seismic provisions* are equivalent as long as a site specific ground motion is incorporated.

- *The 1991 Uniform Building Code (UBC) the International Conference of Building Officials (ICBO).*
- *1991 National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for the Development of Seismic Regulations for New Buildings*, FEMA 222.
- *1992 Supplement to the National Building Officials and Code Administrators International (BOCA).*
- *1992 Amendment to the Standard Building Code by the Southern Building Code Congress International (SBCCI).*

The NEHRP *Recommended Provisions* do not constitute a model building code. There are, however, written so that they can be incorporated as a seismic component of a building code.

An important aspect of model codes that users should understand is that the provisions of an individual model code are complementary and interdependent. That is, many provisions work on the premise that other related provisions of the same code will also be complied with. Generally, one should not combine or co-mingle the provisions of different building codes without full understanding of the consequences.

The major model code organizations have professional staffs that are valuable ongoing resources to member building officials, material suppliers, and designers. In contrast, other organizations that provide valuable reference documents, such as NEHRP, the *Structural Engineers Association of California* (SEAOC), the *American Society of Civil Engineers* (ASCE), and the *American National Standards Institute* (ANSI), rely primarily on committees and pro bono contributors who are neither continuously nor easily available to users. The professional services available from model code organizations are very important to those who must enforce the code.

Chapter 13a provides an overview of *The World of Buildings Codes*, including historical background data and major reference documents.

Chapter 13b provides a detailed description of the ICBO, including its goals, services, and publications as viewed by the ICBO itself. Of particular interest for those who must interpret the UBC are two publications:

- *Recommended Lateral Force Requirements and Commentary*, Seismology Committee, Structural Engineers Association of California (the *Blue Book*), Sacramento, California.
- *Handbook to the Uniform Building Code, An illustrative commentary*, International Conference of Building Officials, 5360 Workman Mill Road, Whittier, California 90601.

Chapter

13a

The World of Building Codes

Diana Todd

Executive Order 12699 requires that federal agencies use appropriate seismic design and construction standards for the construction of all new buildings owned, leased, assisted, or regulated by the federal government. The order requires that *nationally recognized private sector standards and practices* be used, unless such standards are found to be inadequate for agency use. Local building codes may be used if they are determined to be adequate. The Interagency Committee on Seismic Safety in Construction (ICSSC), in its Recommended Practice 2.1 -A, *Guidelines and Procedures for Implementation of the Executive Order on Seismic Safety of New Building Construction*, recommends the use of building codes that are substantially equivalent to the *National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for the Development of Seismic Regulations for New Buildings*. The NEHRP Recommended Provisions themselves are not a building code, but rather a resource document.

This section describes how local codes, model codes, national standards, and seismic design resource documents are related. Fig. 13a - 1 shows the flow of information among researchers and investigators, resource documents, national standards, and model and local codes.

In the United States, the authority to adopt and enforce building codes is delegated to state, county, and local jurisdictions. Approximately 40,000 jurisdictions adopt and enforce building codes, but most jurisdictions adopt one of three major model building codes rather than develop independent codes.

The term *building code* refers to a legally adopted and enforced statute. Model codes developed specifically for adoption by legal jurisdictions are not themselves building codes, but rather models that can be used to create legal building codes.

Model codes are developed to cover all aspects of building design and construction. In addition to model building codes, which cover primarily structural and architectural concerns, there are also mechanical, fire, plumbing, and other model codes. For the most part, the three major model building codes incorporate the same national standards, although significant differences exist in requirements for environmental forces such as wind, snow, and seismic loads. Each code also has its own format for organizing requirements, and each includes some specific provisions that are unique.

These are the three major model codes:

- The Uniform Building Code, published by the International Conference of Building Officials (ICBO)

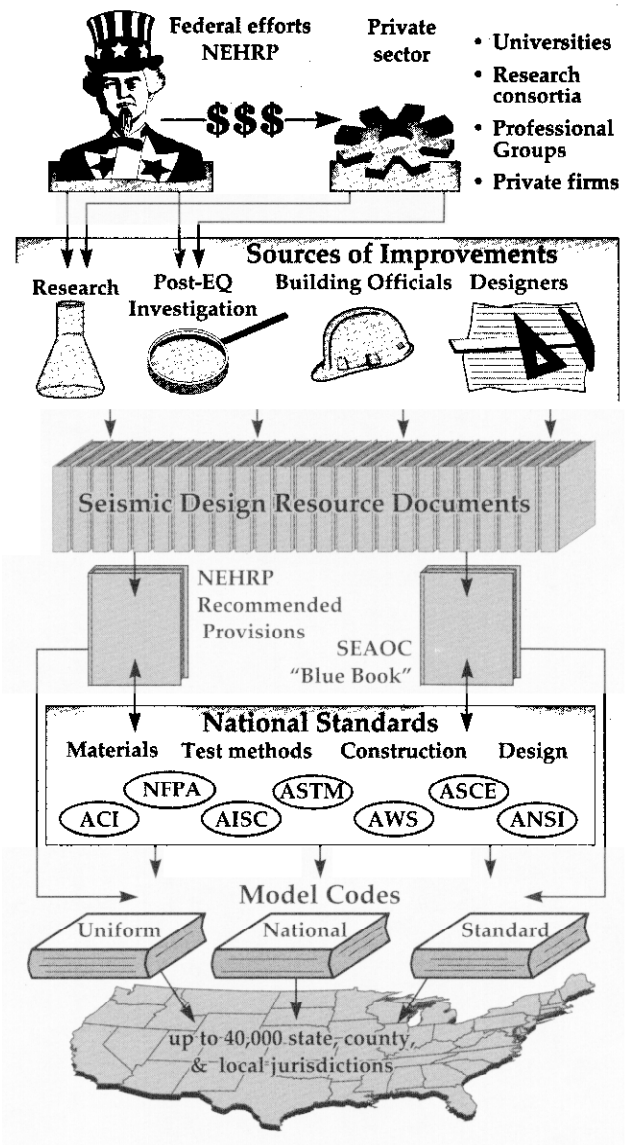


Fig. 13a-1. Linkages among the organizations and documents that constitute the building-code community.

- The *National Building Code*, published by Building Officials and Code Administrators International (BOCA)

- The *Standard Building Code*, published by Southern Building Code Congress International (SBCCI).

These codes are known colloquially by several terms. ICBO's model code is known as the UBC or the Uniform code. The BOCA model code is referred to as BOCA, the National code, or the BOCA National code. The model code of the SBCCI was formerly referred to primarily as the Southern code, but the term Standard code is coming into more common use. In this section the terms Uniform code, National code, and Standard code are used.

Each of the model codes can be adopted and applied anywhere in the country, but each is used largely on a regional basis (see Fig. 13a-2). The Uniform code is used predominately in the western half of the country, the National code is used generally in the Midwest and Northeast, and the Standard code is used mostly in the Southeast. New York and Wisconsin use their own codes.

A fourth model code that is used throughout the country is published by the Council of American Building Officials (CABO). CABO represents the three major model code organizations, and publishes the One- and Two-Family Dwelling Code, colloquially known as the 1 and 2 Family Code or the CABO code. This prescriptive code, made up largely of tables and drawings, is meant to be applied by home builders for simple residential buildings that do not require the design expertise of architects or engineers.

National standards include design requirements for materials, such as the American Concrete Institute's *Building Code Requirements for Reinforced Concrete* (ACI 318); the American

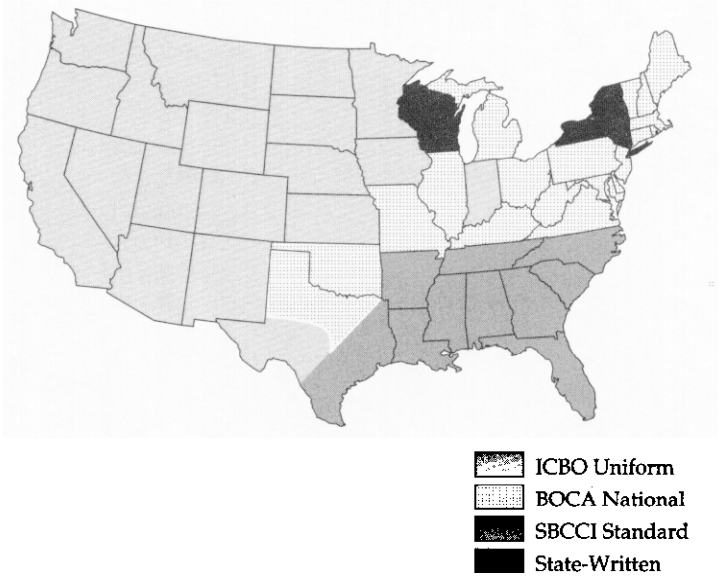


Fig. 13a-2. Regions where the three model building codes are predominantly used and states that write their own codes.

Institute of Steel Construction's *Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings* (AISC); the American Society of Civil Engineers standards, *Building Code Requirements for Masonry Structures* (ASCE 5) and *Specifications for Masonry Structures* (ASCE 6); and the National Forest Products Association's *Design Values for Wood Construction*. Testing, inspection, and construction standards developed by the American Society for Testing and Materials (ASTM) and organizations such as the American Welding Society (AWS) also are in this category. The American Society of Civil Engineers has recently taken over promulgation of *Minimum Design Loads for Buildings and Other Structures*. This document, formerly known as American National Standards Institute ANSI A58.1, is now published as ASCE 7.

Two resource documents currently exist for seismic design: the *NEHRP Recommended Provisions* and the Structural Engineers Association of California (SEAOC) *Recommended Lateral Force Requirements and Commentary*, or the *Blue Book*. The *NEHRP Recommended Provisions* were first published in 1985. Updated editions were issued in 1988, 1991 and 1994, and the document continues to be revised on a three-year cycle. The SEAOC *Blue Book* has been published since 1959, and is revised periodically. These documents serve as resources for the model codes. Information is exchanged in all directions among the resource documents, the national standards, and the model codes.

Improvements to seismic provisions in the resource documents and the national standards, and thus to the model and locally enforced codes, come from research results, postearthquake investigation, and feedback from designers and building officials. Federal efforts under the NEHRP and efforts from the private sector (many of them federally funded) lead to research and investigation results that improve seismic design and construction standards.

Until the *NEHRP Recommended Provisions* were published, the SEAOC *Blue Book* was the only document that methodically attempted to incorporate research and investigation results into a comprehensive seismic-design document. A new edition of the *Blue Book* was typically adopted almost verbatim into the Uniform code during its regular update cycle. ANSI A58.1 also rapidly incorporated advances in seismic design presented in the *Blue Book*. The National and Standard codes adopted the updated design recommendations from ANSI A58.1. (See Fig. 13a-3).

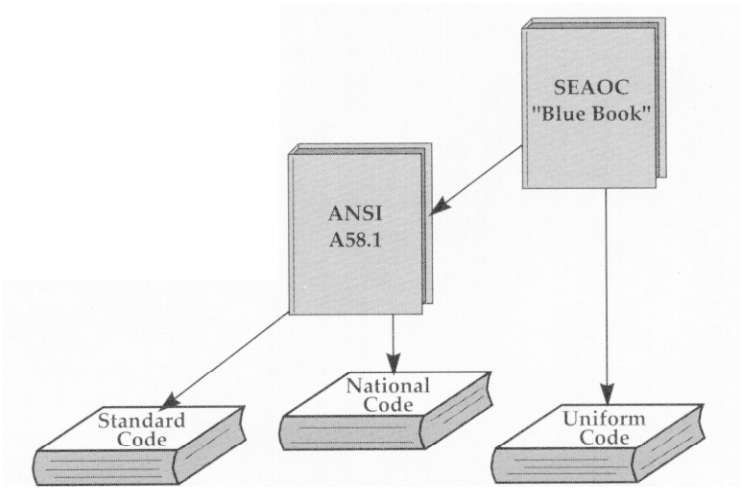


Fig. 13a-3. Pattern of seismic code improvement-pre-1980's.

Today the *NEHRP Recommended Provisions* document constitutes a second resource that provides model codes with up-to-date seismic design and construction recommendations. While the *NEHRP Recommended Provisions* and the SEAOC *Blue Book* both incorporate the same research and investigation results, the documents differ because the *NEHRP Recommended Provisions* use ultimate-strength design, while the *Blue Book* uses allowable-stress design. The two also differ in how they address the issues of building occupancy or importance. The National and Standard codes have adopted the format and requirements of the *NEHRP Recommended Provisions* in their 1992 supplements and additions. ASCE 7 is currently considering adoption of the *NEHRP Recommended Provisions*. (See Fig. 13a-4).

The *NEHRP Recommended Provisions* and the SEAOC *Blue Book* are both excellent documents that complement each other and provide a resource to continue to improve seismic safety.

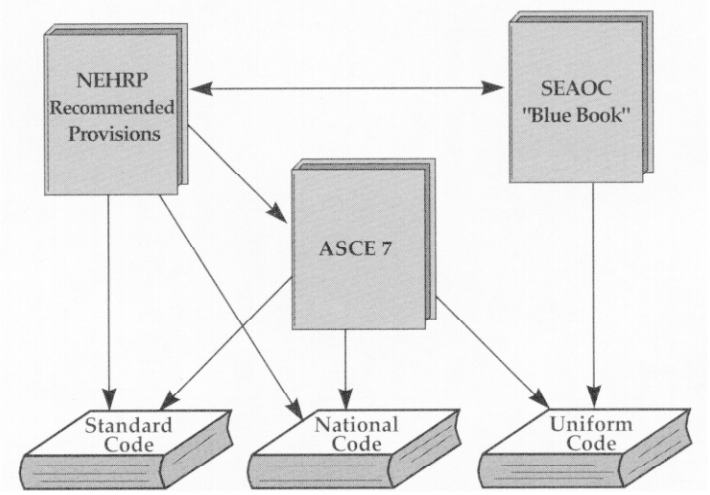


Fig. 13a-4. Current sources of seismic code improvement.

Chapter

13b

The International Conference of Building Officials

Douglas W. Thornburg

Introduction

To administer a comprehensive earthquake-safety program, the facility manager of a DOE facility must establish procedures, make decisions, offer guidance, and take specific actions to ensure code compliance and seismic safety in buildings and other facilities. *The International Conference of Building Officials* (ICBO) offers general and technical services to assist in meeting those objectives.

Organization and Objectives

The ICBO is a nonprofit service organization, owned and controlled by its member cities, counties, states, and other government units. The conference has six main objectives:

1. To investigate and promote the principles underlying safety in the construction, occupancy, and location of buildings and structures.
2. To research, develop, recommend, and promote uniform regulations, legislation, and enforcement pertaining to all phases of building construction.
3. To develop, maintain, and promote the adoption of the *Uniform Building Code* (UBC) and other uniform codes and related documents that are designed to advance the cause of uniformity in regulations for the construction, alteration, conservation, maintenance, preservation, or repair of buildings and structures.
4. To advise and assist in the administration of building laws and ordinances, the development of management and enforcement programs, and related activities.
5. To research, develop, and publish educational materials relating to uniform building construction procedures and practices.

6. To advance the professional skills of those engaged in the administration and enforcement of building laws.

The primary operating control of the ICBO is vested in its Class A Members, the governmental units or departments engaged in the administration or formulation of laws and ordinances relating to building construction. Each Class A Member is required to designate a representative to act for the member in conference affairs. For most cities and counties, the designee is that individual who has the legal responsibility for the administration of the adopted building codes, laws, and ordinances; usually, this is the *Building Official*. For other governmental units, the designee is the person responsible for administration of the code-compliance program. These representatives act for their jurisdictions in matters of code revisions and maintenance, and they are the members eligible to vote, hold office, and serve on committees of the *Conference* (ICBO).

Since the founding of ICBO in 1922, expansion of the *Conference* and adoption of the UBC by an increasing number of governmental bodies in ever-widening areas of the nation have led to a tremendous increase in the number of services and activities.

The organization is directed through a Board of Directors and Officers elected from the Class A membership. It operates through a headquarters staff based in Whittier, California, and regional offices in Kansas City, Missouri; Austin, Texas; Bellevue, Washington; Indianapolis, Indiana; and Pleasanton, California. These offices are designed to provide full services in the realms of plan checking, code consultation and interpretation, education, and participation in regional activities. Each Class A Member, regardless of population of his or her jurisdiction, has equal voting privileges on changes to the UBC and all corporate operations of the *Conference*. The UBC and its related documents are maintained current through an annual review process, with new editions published every three years. All code and related activities of the *Conference* are conducted in an open forum that permits all segments of the industry the full opportunity to be heard and participate in the processes.

Membership

Membership in the *Conference* is open to all governmental units as well as all other segments of the building-construction industry. There are eight primary classifications of members. In many instances, DOE operating agencies qualify for Class A Membership. The eight classes of membership are as follows:

1. *Class A* - Governmental units or departments engaged in the administration or formulation of laws or ordinances relating to building construction.

2. *Governmental Individual* - Individuals responsible for the enforcement or administration of laws and ordinances relating to building construction.

3. *Certified* - Individuals who maintain a current certificate under at least one of the ICBO certification categories.

4. *Chapter* - Associations or groups of city or county officials engaged in the administration or formulation of laws or ordinances related to building construction.

5. *Professional* - Individuals or firms engaged in the practice of architecture, engineering, inspection, construction, research, or related activities.

6. *Associate* - Firms interested in the objectives of the conference.

7. *Subscribing* - Trade associations or groups of firms interested in the objectives of the conference.

8. *Honorary* - Bestowed upon an individual or organization that has rendered outstanding and meritorious service to further the objectives of ICBO.

Copies of two types of membership application forms are located at the end of this chapter. The first is for *Class A Membership* applicants and the second covers applicants for any of the other membership classes.

Services

ICBO Evaluation Services, Inc., a wholly owned subsidiary corporation of ICBO, maintains a full-time staff of registered civil, structural, mechanical, and fire-protection engineers for the purpose of evaluating new materials, products, and construction systems. A number of the evaluations deal with lateral-force-resisting systems and other seismic-safety-oriented determinations (shear and tension values of anchor bolts for anchoring structures or machinery, for example). Class A Members receive a complete file of the current evaluations and bimonthly supplements that keep the file up to date.

A plans examination service is provided by a staff of registered civil, structural, and fire-protection engineers. This service provides a *third-party review* by an engineering staff with experience in earthquake engineering and related requirements of the *Code* (UBC).

A staff of engineers and experienced construction experts is available to assist members in the interpretation and application of the UBC and other *Conference* publications. This assistance is available by letter or through telephone consultation. Interpretations of common or unusual interest are published in each issue of the *Building Standards* magazine.

Educational seminars are sponsored by various organizations throughout the country for training and development of building officials, plan examiners, and inspectors. All members are kept fully informed on vital issues affecting their work through a constant flow of information disseminated primarily through the bimonthly *Building Standards* magazine and the other monthly newsletters.

Publications

One of the primary functions of ICBO is the publication and maintenance of the UBC and its related documents. From its early and continuing recognition of the importance of proper seismic safety design, the UBC has established a position of leadership across the nation for its maintenance of objective and responsive regulations that address this problem.

The seismic design requirements of the UBC are based on studies made by the Seismology Committee of the *Structural Engineers Association of California* (SEAOC) over the years—most specifically since 1957. In that year, the Board of Directors of SEAOC gave specific direction to the Seismology Committee to initiate studies leading to code provisions that would be available to ICBO or any other code-writing body that desired to use them. They are based on the observed fact that earthquake ground motion and the response of buildings and structures thereto is a problem in structural dynamics, even though the actual provisions in the code are expressed as so-called *equivalent* static forces. Beginning with the 1988 UBC, provisions were added to the code addressing irregular features in structures. The measures for addressing irregular features include, but are not limited to, performing dynamic analysis for seismic force distribution. Minimum requirements that must be satisfied when dynamic analysis is performed are also prescribed.

These provisions have evolved and changed through the years based on actual experience during earthquakes. The resulting studies of damage and the interpretations and conclusions drawn therefrom have led to revised code provisions. Furthermore, the provisions are based on research from universities specializing in studying earthquake-resistant design and specifically the ductility of structural framing systems.

At present, the UBC seismic design provisions are based on a preference for specially detailed moment-resisting frame[1] systems that can absorb large amounts of energy within acceptable limits of inelastic deformation as the primary seismic resistance. However, the provisions also recognize that for

[1] Because of the poor performance of the steel moment frame girder-to-column connections during the Northridge earthquake, an emergency code change was recommended by the Seismology Committee of SEAOC to ensure that these connections satisify seismic demands. The ability to meet these demands can be satisified by approved cyclic test results or calculations, but demonstration by testing is preferred.

low and intermediate-height buildings, shear walls and braced frames may provide the primary resistance and, furthermore, give a slight bonus to those systems by using the detailed moment-resisting frame as a *back-up* system, or second line of resistance. The *Code* also provides for possible effects caused by the depth and type of soil at the site, the importance of the building as related to its function, and the geographic location of the structure as it relates to exposure to earthquake damage.

In addition to recognizing the dynamic nature of the earthquake problem and the desirability of ductility in the framing system, the seismic design provisions of the UBC require that the building structure and all of its components be tied together in one cohesive unit and that a logical load-resistance path be maintained continuously through the structure so that all loads in the structure can be adequately resisted and delivered to the ground.

Experience has shown that merely establishing a level of lateral force resistance for structures is not effective without making certain that the load-resisting system is continuous, adequately connected, and anchored, to perform properly during a seismic incident. In recognition of this important fact, the UBC incorporates provisions to ensure the integrity of related features in the structure, such as fire-protection elements, building exits, storage racks, machinery, and other equipment.

Every three years, a new edition of each of the various ICBO codes is published, incorporating advances in seismic, structural, fire, and life-safety design and keeping pace with changes in building construction technology. In each of the two intervening years, a supplement is issued containing all changes approved at the most recent annual business meeting, plus an analysis of these changes. In this manner, the codes are maintained as a *living document.*

As mentioned in the foreword to this chapter, in addition to the *Uniform Building Code*, the *Conference* publishes the *Uniform Mechanical Code, Uniform Code for the Abatement of Dangerous Buildings, Uniform Zoning Code, Uniform Code for Building Conservation*, and additional educational and technical reference materials.

Short courses, video training tapes, textbooks, and manuals on building department administration, field inspection of buildings and structures, and plan review also are available. Building construction and design offices frequently use these services when there is a need for supplemental training of personnel.

Potential Benefits

Managers of DOE facilities who are responsible for building construction (and code compliance) should consider *Individual* or *Agency mem*bership in the ICBO as an integral part of an overall seismic safety program so that the *in-house* staff may receive the benefits of the available ICBO services. When dealing with design of new facilities, evaluation or rehabilitation of existing facilities, and other aspects of a professional seismic safety program, there is a continuing need for an *on-call* technical service capability for plan review, evaluation, and interpretation offered by the conference.

Applying for ICBO Membership

Copies of two types of ICBO membership application forms, complete with descriptions of the classes of membership and benefits, are presented on the next four pages.

INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS

5360 Workman Mill Road • Whittier, **California** 90601-2298 • (310) 692-4226 • FAX (310) 692-3853
6130 Stoneridge Mall Road, Suite 120 • Pleasanton, **California** 94588 • (510) 734-3080 • FAX (510) 463-3295
7998 Georgetown Road, Suite 900 • Indianapolis, **Indiana** 46268 • (317) 879-1677 • FAX (317) 879-0966
6738 N.W. Tower Drive • Kansas City, **Missouri** 64151 • (816) 741-2241 • FAX (816) 741-9475
9300 Jollyville Road, Suite 101 • Austin, **Texas** 78759-7455 • (512) 794-8700 • FAX (512) 343-9116
2122-112th Avenue, N.E., Suite B-300 • Bellevue, **Washington** 98004 • (206) 451-9541 • FAX (206) 637-8939

Publisher of the
Uniform Building Code™

MEMBERSHIP APPLICATION

MEMBERSHIP NUMBER
(To be assigned by ICBO)

APPLICANT COMPLETE SECTIONS **1** THROUGH **5**. **PLEASE TYPE OR PRINT CLEARLY.**

1 MEMBERSHIP CATEGORY

- ❑ CLASS A *(governmental jurisdictions only) (Population less than 10,000)* $ 85.00
- ❑ CLASS A *(governmental jurisdictions only) (Population 10,000 and over)* 195.00

2

_____
(MEMBERSHIP IS ON BEHALF OF JURISDICTION)

❑ MR.
❑ MS. _____
CLASS A DESIGNEE

_____
TITLE

_____
STREET *(STREET ADDRESS IS REQUIRED FOR U.P.S.)*

_____
CITY STATE ZIP + 4

_____
P.O. BOX (IF APPLICABLE) CITY ZIP + 4

(_____) _____
TELEPHONE COUNTY *(FOR SALES TAX PURPOSES)*

(_____) _____
MESSAGE PHONE

(_____) _____
FAX NUMBER

X _____
(Signature of applicant or designated representative of jurisdiction) Date

3 I WANT TO RECEIVE THE FOLLOWING:

(PLEASE CHECK)

Code Monographs (Part III):
- ❑ Suggested Revisions
- ❑ Code Development Committee Reports
- ❑ Code Change Agenda

- ❑ Evaluation Reports Listing by Products (Part II)
- ❑ Membership Roster
- ❑ Annual Supplement
- ❑ *Building Standards* magazine/newsletter (Part 1)
- ❑ Annual Conference Notices
- ❑ Evaluation Reports

4 INDICATE METHOD OF PAYMENT BELOW:

❑ VISA ❑ MASTERCARD ❑ CHECK *(Make payable to ICBO)* $_____

CARD NO. ____ ____ ____ ____ - ____ ____ ____ ____ - ____ ____ ____ ____ - ____ ____ ____ ____ EXP. DATE ____ ____ - ____ ____
Mo. Yr.

NAME AS IT APPEARS ON CARD _____

SIGNATURE _____ DATE _____
(Please sign above if credit card is used.)

5 THIS SECTION MUST BE COMPLETED BY THE ADMINISTRATOR OR ELECTED OFFICIAL WHO SUPERVISES THE BUILDING OFFICIAL AND/OR THE BUILDING SAFETY FUNCTION.

I, _____ / _____ ,
(Name of administrator or elected official who supervises the building official and/or the building safety function.) *Title*

hereby designate _____, whose title is

_____, as the Class A member designee who shall be the official

representative of _____ for the purpose of voting, executing written consents and

committee membership. _____ _____
(Population of jurisdiction) *(Signature of administrator or elected official)*

For ICBO use only

❑ 100 S 94	❑ 105 S 94	❑ 119 S 94	❑ 217 L 00
❑ 101 S 94	❑ 106 S 94	❑ 120 S 94	❑ 219 L 00
❑ 102 S 94	❑ 108 S 94	❑ 401 S 94	❑ 231 S 94
❑ 103 S 94	❑ 117 S 94	❑ 601 R 78	❑ 110 S ___
❑ 104 S 94	❑ 118 S 94	❑ 216 S 94	

CLASS A MEMBERSHIP

This classification of membership is restricted to governmental units or agencies engaged in the administration or formulation of laws and ordinances relating to building construction. In no case shall a governmental unit be entitled to more than one Class A membership, except as it has separate agencies engaged in the above activities, in which case the board of directors may classify such separate agencies as members.

All members are subject to classification by and approval of the board of directors. Only Class A member designees and designated employees of Class A members shall be entitled to vote on any matter, whether as a committee member or otherwise. Each Class A member shall have one vote and one vote only on any given matter. However, for the purpose of bringing up matters for discussion, all members may make and second motions. All members shall be entitled to participate in meetings and discussions. Except as otherwise provided by the Bylaws, any member may be appointed to a committee as a nonvoting member.

Class A members shall, when admitted to membership, designate in writing the individual who is to act as official representative for the purpose of voting, executing written consents, and for committee membership. Class A members may, in addition, designate, in writing, individuals employed by that Class A member to serve on standing committees, and to vote on other matters of business which may be assigned by the board of directors. Said designations may be changed in writing from time to time. The Class A member may also designate in writing an individual to act as a proxy in the place and stead of the Class A designee in all respects as set forth above except for matters arising out of committee membership.

MEMBERSHIP BENEFITS

CLASS A MEMBERS may initially elect to receive all Conference services, including a gratis subscription to the Conference periodical, *Building Standards*™, code development monographs and one copy of each of the following: *Uniform Building Code*™, Volumes 1, 2 and 3 (soft cover), *Uniform Mechanical Code*™; *Uniform Housing Code*™; *Uniform Code for Building Conservation*™; *Uniform Code for the Abatement of Dangerous Buildings*™; *Uniform Sign Code*™; *Uniform Fire Code*™; *Uniform Fire Code Standards*™; *Uniform Building Security Code*™; *Uniform Administrative Code*™; *Uniform Zoning Code*™; *Model Program for Special Inspectors*; and *The ICBO Code Development Process.* In addition, they receive ICBO Evaluation Service Reports and supplements as they are published, a copy of each new edition of the *Uniform Building Code,* Volume 1 (soft cover) and annual supplements in the years between republication, a current copy of the *Membership Roster,* a current copy of the *Certification Roster,* and meeting notices. Class A members may also receive either telephone consultations on, or written interpretations of, the Uniform Codes published by ICBO.

(Members in all categories are entitled to purchase ICBO publications at member discounts.)

INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS

5360 Workman Mill Road • Whittier, **California** 90601-2298 • (310) 692-4226 • FAX (310) 692-3853
6130 Stoneridge Mall Road, Suite 120 • Pleasanton, **California** 94588 • (510) 734-3080 • FAX (510) 463-3295
7998 Georgetown Road, Suite 900 • Indianapolis, **Indiana** 46268 • (317) 879-1677 • FAX (317) 879-0966
6738 N.W. Tower Drive • Kansas City, **Missouri** 64151 • (816) 741-2241 • FAX (816) 741-9475
9300 Jollyville Road, Suite 101 • Austin, **Texas** 78759-7455 • (512) 794-8700 • FAX (512) 343-9116
2122-112th Avenue, N.E., Suite B-300 • Bellevue, **Washington** 98004 • (206) 451-9541 • FAX (206) 637-8939

Publisher of the
***Uniform Building Code*™**

MEMBERSHIP APPLICATION

MEMBERSHIP NUMBER
(To be assigned by ICBO)

APPLICANT COMPLETE SECTIONS **1** THROUGH **4**. PLEASE TYPE OR PRINT CLEARLY

A separate application is available from ICBO for Class A membership.

1 MEMBERSHIP CATEGORY

(✓) Check the membership category for which you are applying. (See membership category descriptions and benefits on back.)

❑ GOVERNMENTAL INDIVIDUAL	$ 60.00
❑ CERTIFIED *(information in section 2 will also update certification records)*	35.00
❑ PROFESSIONAL *(If membership is on behalf of company, payment must be paid by company check.)*	90.00
❑ ASSOCIATE	300.00
❑ SUBSCRIBING	500.00
❑ STUDENT	20.00
❑ RETIRED	20.00

I WANT TO RECEIVE THE FOLLOWING:

- ❑ *Building Standards* magazine/newsletter (Part 1)
- ❑ Membership Roster *(Not available to student members)*
- ❑ Annual Conference Notices *(Not available to student members)*

2

_____
MEMBERSHIP IS ON BEHALF OF (SELF, COMPANY, JURISDICTION)

❑ MR.
❑ MS. _____
REPRESENTATIVE/CONTACT PERSON

_____
TITLE

_____
STREET *(STREET ADDRESS IS REQUIRED FOR U.P.S.)*

May we publish this information

_____ _____ _____ YES ☐ NO ☐
CITY STATE ZIP + 4

_____ _____ _____ ☐ ☐
P.O. BOX (IF APPLICABLE) CITY ZIP + 4

(_____) _____ ☐ ☐
TELEPHONE COUNTY *(FOR SALES TAX PURPOSES)*

(_____) _____ ☐ ☐
MESSAGE PHONE

(_____) _____ ☐ ☐
FAX NUMBER

X _____
(Signature of applicant) Date

3 I WANT TO RECEIVE THESE ADDITIONAL BENEFITS:

(Governmental Individual, Professional, Associate, and Subscribing members only)

(PLEASE CHECK)

Code Monographs (Part III):

- ❑ Suggested Revisions
- ❑ Code Development Committee Reports
- ❑ Code Change Agenda

- ❑ Evaluation Reports Listing by Products (Part II)
- ❑ Annual Supplement

4 INDICATE METHOD OF PAYMENT BELOW:

❑ VISA ❑ MasterCard ❑ CHECK *(Make payable to ICBO)* $_____

CARD NO. __ __ __ __ - __ __ __ __ - __ __ __ __ - __ __ __ __

EXP. DATE __ __ - __ __
Mo. Yr.

NAME AS IT APPEARS ON CARD _____

SIGNATURE _____ DATE _____
(Please sign above if credit card is used.)

For ICBO use only

❑ 100 S 94	❑ 601 R 78
❑ 103 S 94	❑ 216 S 94
❑ 103 L 94	❑ 110 S ___
❑ 401 S 94	❑ 404 P 00

Rev.6/94

MEMBERSHIP CLASSES*

1. GOVERNMENTAL INDIVIDUAL. Full- or part-time employees of jurisdictions eligible for Class A membership who are involved in the enforcement and administration of laws and ordinances relating to building construction.

2. CHAPTER MEMBER.	An association or group of Class A member designees, or others engaged in the administration or formulation of laws and ordinances relating to building construction, together with any associated interests, and who subscribe to the objectives of the Conference.
3. CERTIFIED MEMBER.	An individual who maintains a current certificate under at least one of the International Conference of Building Officials' certification categories.
4. PROFESSIONAL MEMBER.	An individual or firm, incorporated or unincorporated, engaged in the practice of architecture, engineering, inspection, research, testing, construction, or related activities.
5. ASSOCIATE MEMBER.	A firm or corporation interested in the objectives of the Conference.
6. SUBSCRIBING MEMBER.	An association or group of firms or corporations interested in the objectives of the Conference.
7. HONORARY MEMBER.	An individual who has rendered outstanding and meritorious services in the furtherance of the objectives of the Conference, and who shall be proposed by the board of directors and confirmed by a majority vote at the annual business meeting.
8. STUDENT MEMBER.	Any individual enrolled in classes or a course of study occupying at least twelve (12) hours of classroom instruction per week.
9. RETIRED MEMBER.	Any former designated representative of a Class A member, any former representative of any other membership class or any former individual member who is retired.

All members are subject to classification by and approval of the board of directors. Only Class A member designees and designated employees of Class A members shall be entitled to vote on any matter, whether as a committee member or otherwise. Each Class A member shall have one vote and one vote only on any given matter. However, for the purpose of bringing up matters for discussion, all members may make and second motions. All members shall be entitled to participate in meetings and discussions. Except as otherwise provided by the Bylaws, any member may be appointed to a committee as a non-voting member.

Class A members shall, when admitted to membership, designate in writing the individual who is to act as official representative for the purpose of voting, executing written consents, and for committee membership. Class A members may, in addition, designate, in writing, individuals employed by that Class A member to serve on standing committees, and to vote on other matters of business which may be assigned by the board of directors. Said designations may be changed in writing from time to time. The Class A member may also designate in writing an individual to act as a proxy in the place and stead of the Class A designee in all respects as set forth above except for matters arising out of committee membership.

***A separate application is available from ICBO for Class A membership.**

MEMBERSHIP BENEFITS

GOVERNMENTAL INDIVIDUAL MEMBERS may elect to receive a subscription to the Conference periodical, *Building Standards,* code development monographs, Volume 1 (soft cover) of the current or forthcoming edition of the *Uniform Building Code* and annual supplements in the years between republication, the current *Membership Roster,* and meeting notices.

CHAPTER MEMBERS receive meeting notices and such other services as authorized by the board of directors.

CERTIFIED MEMBERS may elect to receive a subscription to the Conference periodical, *Building Standards,* the current *Certification Roster,* the current *Membership Roster,* and meeting notices.

PROFESSIONAL MEMBERS may elect to receive a subscription to the Conference periodical, *Building Standards,* code development monographs, Volume 1 (soft cover) of the current or forthcoming edition of the *Uniform Building Code* and annual supplements in the years between republication, the current *Membership Roster,* and meeting notices.

ASSOCIATE MEMBERS may elect to receive a subscription to the Conference periodical, *Building Standards,* code development monographs, Volume 1 (soft cover) of the current or forthcoming edition of the *Uniform Building Code* and annual supplements in the years between republication, ICBO Evaluation Service Reports and supplements as they are published, the current *Membership Roster,* and meeting notices.

SUBSCRIBING MEMBERS may elect to receive two of each of the following: the Conference periodical, *Building Standards,* code development monographs, the current or forthcoming Volume 1 (soft cover) edition of the *Uniform Building Code* and annual supplements in the years between republication, ICBO Evaluation Service Reports and supplements as they are published, the current *Membership Roster,* and meeting notices.

HONORARY MEMBERS may elect to receive a gratis subscription to the Conference periodical, *Building Standards,* code development monographs, Volume 1 (soft cover) of the current or forthcoming edition of the *Uniform Building Code* and annual supplements in the years between republication, the current *Membership Roster,* and meeting notices.

STUDENT MEMBERS may elect to receive a subscription to the Conference periodical, *Building Standards.*

RETIRED MEMBERS may elect to receive a subscription to the Conference periodical, *Building Standards,* the current *Membership Roster,* and meeting notices.

bers *tegories* *member*

Appendix A

Biographical Sketches for Authors of the Text and the Preface

Jack R. Benjamin

Dr. Benjamin is a leader in the application of probabilistic methods and decision theory in civil engineering. He is Professor Emeritus of Civil Engineering at Stanford University. Dr. Benjamin is author of *Statically Indeterminate Structures*, a book in the McGraw-Hill Engineering Series, and is co-author of *Probability, Statistics, and Decision for Civil Engineers*, the standard text and reference on probabilistic methods in civil engineering.

Much of his work has dealt with extreme and unusual (high-hazard) loading conditions, the development of rational, probability-based design criteria, and safety analyses using decision, event, and fault-tree techniques. In addition, he has had extensive structural engineering experience for more than 30 years, both as a consultant and in his own design practice. A registered Civil Engineer in California, Dr. Benjamin is Chairman of the Board of Jack R. Benjamin and Associates, Inc., Consulting Engineers, Palo Alto, California.

Donald G. Eagling

Mr. Eagling has 46 years experience in civil, structural, and geotechnical engineering, planning and construction. For 24 years he was in charge of engineering, construction management, operations, and maintenance for facilities at the University of California's Lawrence Berkeley Laboratory (LBL).

Mr. Eagling is a registered Civil Engineer in California and a member of the Earthquake Engineering Research Institute (EERI); he was also a Class A Member of the International Conference of Building Officials (ICBO) and a member of the Structural Engineers Association of Northern California, representing LBL. He holds a Master of Engineering degree from the University of Michigan, where he majored in structures and soils engineering.

In 1990, Mr. Eagling received the U.S. Department of Energy's Distinguished Associate Award in recognition of his leadership in seismic hazard mitigation.

John J. Earle

Mr. Earle is a registered Civil and Structural Engineer in California, with SOH and Associates, Structural Engineers, San Francisco, California. He has extensive structural analysis and design experience and is active in seismic code and educational work. He was the principal project engineer for all work done by SOH and Associates involving structural analysis and design of the earthquake-resisting systems incorporated into existing buildings, radiation shielding blocks, and other facilities and equipment at the Lawrence Berkeley Laboratory. Much of his work includes seismic design of schools, laboratories, and other facilities.

Mr. Earle is past chairman of both the Structural Technical Group and the Professional Development Committee of the San Francisco Section of the American Society of Civil Engineers as well as past chairman of the Continuing Education Committee and Director of the Structural Engineers Association of Northern California. He has organized several seminars on seismic design and code applications.

John M. Eidinger

Mr. Eidinger is President of G&E Engineering Systems, Inc., Oakland, and a registered Structural Engineer. He received his B.S. in Civil Engineering from the Massachusetts Institute of Technology (M.I.T.), M.S. and M.Engineering in Structural Engineering and Structural Mechanics and M.B.A. from the University of California at Berkeley. He is currently Chairman of the American Society of Civil Engineers (ASCE) Technical Council of Lifeline Earthquake Engineering Water and Sewage Committee.

He has 20 years' experience in the seismic evaluation and design of electric, water and transportation lifeline systems. He has been involved with a variety of seismic and risk studies for the U.S. Department of Energy's Hanford facilities.

Mr. Eidinger has been active in the development and implementation of procedures for the earthquake risk analysis of lifeline systems, and in the projection of possible consequences from fire following earthquake. He has performed earthquake evaluations of most public transit agencies in the San Francisco Bay area; assessed the seismic performance of several water and wastewater utilities; conducted analyses of the San Francisco Oakland Bay Bridge, developed the design for a new salt water fire fighting system for the City of Berkeley; and developed models of fire following earthquakes for many of the cities in the San Francisco Bay area.

Mr. Eidinger is a principal author of the National Institute of Building Sciences' *Standardized Earthquake Loss Estimation Methodology* for lifelines and fire following earthquakes.

Eric Elsesser

Eric Elsesser is founder (1960) and president of Forell/Elsesser Engineers, Inc., San Francisco. He holds B.S and M.S. degrees in Civil Engineering and Structural Engineering (respectively) from Stanford University. He has 39 years experience in the analysis, design, and construction of earthquake resistant structures and has been in responsible charge for the design of more than 1000 projects.

Mr. Elsesser has been in the forefront of the analysis and design of seismic isolation and energy dissipation systems for both new and retrofitted buildings. Under his leadership, Forell/Elsesser Engineers designed the seismic isolation retrofit for the Salt Lake City and County Building, a 4-story structure originally constructed about 1894 of unreinforced brick and sandstone. This unprecedented project was the first use of seismic isolation for retrofit of a major building in the United States. Currently, Forell/Elsesser's experience includes 8 other major projects utilizing seismic isolation and/or energy dissipation techniques including the retrofit of the historic 18-story Oakland City Hall which is currently the tallest building in the world to be seismically isolated.

Mr. Elsesser has been very active in professional organizations. He is a past president of the Structural Engineers Association of Northern California and past Director of the Structural Engineers Association of California. He was chair of the American Society of Civil Engineer's (National) Dynamic Effects Committee and the Seismic Effects Committee, a member of the Governor's Board of Inquiry for the 1989 (Loma Prieta) earthquake damage to freeways, an Advisory Panel Member for the National Science Foundation's Seismic Engineering Program and a member of the Advisory Panel for the Office of Science and Technology Policy, Office of the President.

Harold M. Engle, Jr.

Harold Engle, Jr. is the senior partner of Engle and Engle, Structural Engineers of Ross, California. He received a B.S. in Civil Engineering from the University of Southern California, and is a registered Civil and Structural Engineer with over 30 years' experience in structural design and seismic rehabilitation of California public schools, institutional and commercial buildings, and hospitals as well as U.S. Department of Energy (DOE) facilities. Engle & Engle was the lead structural firm for the original seismic safety evaluation of facilities at the Lawrence Berkeley Laboratory (LBL) and has since performed numerous peer-review plan checks and seismic upgrades both for LBL and the Lawrence Livermore National Laboratory (LLNL).

Mr. Engle's father, Harold M. Engle, Sr. (1898-1977), whose 58-year career in civil and structural engineering included 19 years on the Field Act Advisory Board to the State Architect (California) for the design of public school buildings, founded the firm in 1926. Additionally, he was Consulting Structural Engineer for the Pacific Fire Rating Bureau (now Insurance Services, Inc.) and the Factory Insurance Association and developed procedures for rating buildings for earthquake insurance rates.

Harold Engle, Jr. has been a member of the firm since 1961 and Senior Partner and Principal Structural Engineer since 1975. He is a member of the Structural Engineer's Association of Northern California (SEAONC), the Consulting Engineers Association of California (SEAC), the Seismological Society of America and the LBL Seismic Safety Committee. He is a consultant to San Francisco Bay Area governmental agencies for seismic rehabilitation and has provided seismic ratings for more than 310 buildings and other structures at LLNL and LBL.

Ronald P. Gallagher

Mr. Gallagher is President of R. P. Gallagher Associates, Inc., San Francisco, and a registered Structural Engineer. He received his BS and MS degrees in Structural Engineering from the University of California, Berkeley. In 1994-95, he was President of the Structural Engineers Association Northern California (SEAONC).

He has over 25 years experience in structural and earthquake engineering design of buildings and seismic vulnerability assessments of many existing facilities including DOE nuclear facilities at Hanford, Savannah River, INEL, and Oak Ridge.

Mr. Gallagher performed damage reconnaissance surveys after many notable earthquakes, including the 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge gaining valuable insight into the performance of buildings, structures, nonstructural elements and building contents. He was a member of the Structural Engineers Association of California (SEAOC) State Seismology Committee for five years and chairperson in 1991-92. The SEACO *Bluebook* which the Committee maintains and develops, is the basis for the seismic requirements in the *Uniform Building Code* (UBC) and other codes.

Mr. Gallagher was the principal author of ATC-20, *Procedures for Postearthquake Building Safety Evaluation*, which was widely used by San Francisco Bay Area building departments after the 1989 Loma Prieta earthquake. ATC-20 has been adopted for damage assessments by the State of California. Following the Loma Prieta earthquake, he was retained by the California Seismic Safety Commission to review damage assessment and posting of buildings throughout the region of damage and to make recommendations for improvement.

Mr. Gallagher has helped many organizations establish damage assessment programs including Boeing, the Pacific Gas and Electric Company, and the U.S. Postal Service. He has trained over 3,000 people in damage assessment techniques, including building officials, structural engineers, and disaster workers, as well as staff for organizations such as Bechtel, Chevron, Lockheed, and Los Angeles County.

Terence P. Haney

Mr. Haney is owner of the Terence Haney Company, a Los Angeles consulting firm working in the fields of public-policy issues, inter-governmental programs and private sector contingency planning. He has a BA degree in Sociology from the University of St. Thomas, St. Paul, Minnesota and has 35 years experience in emergency planning, and the development of emergency management systems and training programs. For 20 years he was a senior systems analyst and program manager for the System Development Corporation (SDC) and for 8 years, Principal Consultant to the Southern California FIRESCOPE program which includes development and field testing for the Incident Command System (ICS) and the Multi-Agency Coordinations Systems (MACS) now adopted by all California fire services, as well as many enforcement and public safety agencies in other states.

Since then Mr. Haney has continued developing operational response plans and training programs for state and local governments and private sector clients. He has assisted the Governor's Office of Emergency Services in the development of the southern California Earthquake Response Plan, the San Francisco Bay Area Catastrophic Earthquake Plan, and the State's Multihazard Functional Guidance development. He was a principal in the design of the State's southern California Coordination Center and its procedures documentation. Mr. Haney also was controller for Response 87 and 89, major federal, state and local earthquake simulation exercises, to test emergency response plans for southern California and the San Francisco Bay Area.

James R. Hill

James Hill (retired) managed Natural Phenomena Hazards (NPH) Safety Programs for the U.S. Department of Energy (DOE), Office of Environment, Health and Safety in Washington, D.C. As well, he was appointed DOE's first Seismic Safety Coordinator by the Secretary of Energy. He holds a B.S. in Chemical Engineering from Montana State University where he also fulfilled a National Science Foundation Fellowship in graduate engineering studies. He has over 40 years experience in natural hazards reduction.

Mr. Hill directed DOE's NPH mitigation programs from 1975 until he retired in 1996. His efforts have led to the development and establishment of greatly improved, comprehensive protection of workers, the public, and the environment against earthquakes, extreme winds, tornadoes, and flood hazards to DOE facilities nationwide. This broad protection encompasses the evaluation and retrofit of existing facilities as well as planning, design, construction, and operation of new facilities. Under his leadership centers for research and development were established at the Lawrence Livermore National Laboratory and the Oak Ridge National Laboratory to spearhead activities in collaboration with universities, private sector professionals and other federal agencies.

Mr. Hill has conducted numerous (DOE) nationwide conferences, seminars, workshops, and training sessions on NPH mitigation and emergency preparedness and provided support for national and world conferences on earthquake engineering. He represented DOE on the Interagency Committee on Seismic Safety in Construction and was chairperson of the National Research Council Building Research Board Committee on Civil and Structural Engineering. He also served on the U.S.-Japan Panel on Wind and Seismic effects and on the Federal Committee for Natural Disaster Reduction.

James J. Johnson

Dr. James Johnson is Division President of EQE International in San Francisco. He holds a B.C.E. in Civil Engineering from the University of Minnesota and an M.S. and Ph.D. in Civil Engineering from the University of Illinois. He is a registered Civil Engineer in California, and has taught Earthquake Engineering of Major Facilities at the University of California at Berkeley.

Dr. Johnson has contributed to over 40 technical reports and journal articles covering seismic analysis, fragility and risk assessments, soil-structure interaction (SSI) and related computer programs. He has developed, verified, maintained, and extensively applied several large computer programs to perform stress and seismic analysis. Among these is MODSAP, a general purpose finite element program with special capability in the dynamic analysis of structures with localized nonlinearities.

Dr. Johnson has extensive theoretical and practical experience in SSI analysis of major facilities and has written a comprehensive assessment of the state-of-the-art of SSI. He has performed SSI analyses of a wide variety of surface and embedded structures using simplified to sophisticated substructure methods and linear and non-linear finite element techniques.

Dr. Johnson is chairperson of the Working Group on Input to Secondary Systems of the American Society of Civil Engineers (ASCE) Nuclear Structures and Materials Committee, Dynamic Analysis Committee, and a member of the ASCE Committee on Nuclear Standards, Seismic Analysis of Safety Class Structures.

Frank E. McClure

Frank McClure is a consulting structural engineer specializing in earthquake engineering, seismic vulnerability assessments and retrofit of buildings. He received a B.S. in Civil Engineering from the University of California, Berkeley, and is a California registered Civil and Structural Engineer and Architect, with 50 years' experience, 34 in private practice and 16 in the public domain where he held positions as Chief Structural Engineer for the Lawrence Berkeley Laboratory (LBL) and University Engineer for the University of California Systemwide Administration.

In 1955, Mr. McClure founded his private practice in Oakland, California which later grew into the firm of Frank E. McClure and David L. Messinger, Consulting Structural Engineers, providing design services for public, industrial and commercial buildings and specializing in earthquake engineering. He joined the University in 1976 and retired from LBL in 1991 to return to private practice.

Mr. McClure, who is an Honorary Member and past President of the Earthquake Engineering Research Institute (EERI), has investigated numerous earthquakes in the United States and abroad and served as an earthquake damage evaluation consultant to the insurance industry and federal agencies. Also, he served as a member of the Scientific Advisory Committee, National Center for Earthquake Engineering Research (NCEER); the National Research Council, Committee on Earthquake Engineering; U.S. Department of Energy (DOE) Natural Phenomena Hazards Panel; Seismology Code Development Committee, International Conference of Building Officials (ICBO); the State of California, Field Act Advisory Board to the Division of the State Architect; and Advisory Panels for the California Seismic Safety Commission.

Mr. McClure received the U.S. Department of Energy's Distinguished Associate Award in recognition of his contributions toward earthquake hazard mitigation.

Robert C. Murray

Robert C. Murray is Project Leader of the Geologic and Atmospheric Hazards Project for the Lawrence Livermore National Laboratory (LLNL) in Livermore, California. He holds BS and MS degrees in Civil and Structural Engineering from the University of California at Berkeley, a Doctor of Engineering degree from the University of California at Davis and is a registered Civil Engineer in California.

Since 1975 Dr. Murray has provided the primary technical support for the Department of Energy's (DOE) Natural Phenomena Hazards Mitigation Program directed by DOE's James R. Hill. As well, Dr. Murray has managed numerous natural phenomena projects for the Nuclear Regulatory Commission (NRC) since 1974 and has managed the development of blast resistant criteria for the U.S. Air Force. His projects included the revision of criteria for geological siting of commercial nuclear power plants and seismic hazard assessments of U.S. sites, the development of natural phenomena hazards (NPH) standards and guidelines for DOE facilities throughout the United States as well as analyses and evaluations of nuclear processing plants and operating reactors.

Dr. Murray and his staff, with the support of specialized consultants, have developed a major system of orders, standards, guidance documents and design manuals which have been adopted by DOE to carry out the policies of its NPH mitigation program covering earthquakes, tornadoes/extreme winds, and floods.

Dr. Murray is currently chairman of the American Society of Civil Engineers (ASCE) main committee on Dynamic Analysis of Nuclear Structures Committee and Chairman of the working group for Revision of ASCE 4-86 Standard on Seismic Analysis.

Maurice S. Power

Mr. Power is a Principal Engineer of Geomatrix Consultants, Inc., San Francisco, and a registered Civil and Geotechnical Engineer in California. He received his B.S. in Civil Engineering from Stanford University and his M.S. in Geotechnical Engineering from the University of Calfornia, Berkeley.

Mr. Power has 31 years of experience in consulting geotechnical engineering practice, specializing in earthquake engineering applications: analyzing ground response, developing design earthquake ground motions, evaluating potential for liquefaction and other seismic-geologic hazards, analyzing soil-structure interaction, and evaluating the seismic stability of slopes and dams. He has conducted studies for nuclear facilities, buildings, bridges, dams, pipelines and utility systems, port facilities, offshore structures, base-isolated structures, and other facilities. He has also conducted regional evaluations of seismic hazards.

Mr. Power has been heavily involved in developing and updating seismic code provisions and guidelines documents for seismic ground motions, seismic-geologic hazards evaluations and seismic foundation engineering. He has served as chairman of the Foundation and Seismic Zonation Committees, as well as Treasurer and member of the Board of Directors of the Structural Engineers Association of Northern California (SEAONC). He is a member of the Foundation Committee of the Building Seismic Safety Council (BSSC). He was Chairman of the Organizing Committee of the NCEER/SEAONC/BSSC 1992 workshop that developed new provisions for site effects on ground motions that have been adopted into the 1994 *National Earthquake Hazard Reduction Program (NEHRP) Provisions* for buildings. Mr. Power has authored more than 50 technical papers on topics in earthquake engineering.

Daniel Shapiro

Mr. Shapiro is a registered Civil and Structural Engineer and a principal of SOH & Associates, Structural Engineers in San Francisco which he founded in 1965 after 15 years experience as an engineer and partner in other firms. Since graduation from the University of California at Berkeley, he has devoted most of his career to designing buildings to resist seismic forces and, through his professional society work, to developing codes and regulations for earthquake safety.

Mr. Shapiro chaired the Retrofit of Existing Buildings Committee of the Building Seismic Safety Council (BSSC) and is currently project director for the National Earthquake Hazard Reduction Program (NEHRP) project to develop national guidelines for the seismic rehabilitation of existing buildings which is sponsored by the Federal Emergency Management Agency (FEMA).

Although his firm has a general structural engineering practice, it has developed an expertise in the seismic rehabilitation of existing buildings and received national recognition for its work in this field. Mr. Shapiro has conducted seminars and workshops and taught university-level courses on the seismic design of buildings.

Roland L. Sharpe

A leading authority in the field of seismic safety, Mr. Sharpe has 46 years experience in structural and earthquake engineering design and analysis. He was technical director in charge of design of physical plant facilities for the Stanford Linear Accelerator Center, a principal consultant to the U.S. Atomic Energy Commission on the seismic safety of 20 nuclear power plants, and Project Director for the Applied Technology Council ATC 3-06 seismic design provisions project. He has investigated damage that occurred in several earthquakes including the October 1989 Loma Prieta and January 1994 Northridge, California earthquakes, and the January 1995 Kyogo-ken Nanbu (Japan) earthquake.

Mr. Sharpe has authored numerous professional papers, engineering analyses and research reports on: recommendations concerning the behavior of structural systems under dynamic loadings; recommendations for shape of earthquake response spectra; seismic design of structures; and the seismic design of restraints/anchorage for electrical and mechanical equipment and data processing systems.

Mr. Sharpe was project director for the National Science Foundation (NSF)/National Institute of Standards and Technology (NIST) project to develop comprehensive seismic design recommendations for use throughout the United States. He was chair of the Design Committee for the 1985 *National Earthquake Hazard Reduction Program (NEHRP) Provisions* as well as for the 1988 and 1991 updates. More recently he was appointed to the President's NEHRP Advisory Committee to the Federal Emergency Management Agency (FEMA), U.S. Geological Survey (USGS), NSF, and NIST on planning and implementation of the *National Earthquake Loss Reduction Program* which will report to Congress.

He is a Life Member of Structural Engineers Association of Northern California (SEAONC) and American Concrete Institute (ACI), and an Honorary Member of American Society of Civil Engineers (ASCE) and the Japan Structural Consultants Association. He holds B.S.E. and M.S.E degrees in Civil and Structural Engineering from the University of Michigan and is a registered Civil and Structural Engineer in California.

Karl V. Steinbrugge

Professor Steinbrugge has been active in the fields of structural and earthquake engineering for more than 40 years. He is a licensed Civil and Structural Engineer in the State of California. He was Professor of Structural Design at the University of California at Berkeley, College of Environmental Design, from 1950 until his retirement in 1978. Concurrently, he was Manager of the Earthquake Department of the Pacific Fire Rating Bureau in San Francisco (later the Insurance Services Office). His work included engineering investigations of potential earthquake damage to structures as well as field studies of earthquakes and their effects. The results of these investigations were published as reports.

He was the first chairman of the California State Seismic Safety Commission (1975-80) and president of the Earthquake Engineering Research Institute (1968-70). On two occasions he served in the Executive Office of the President in Washington, D.C., helping to formulate earthquake hazard reduction policy. He has served as consultant to many local, state, and federal agencies, including the Federal Emergency Management Agency, U.S. Geological Survey, and the California Division of Mines and Geology. He has published more than 100 articles and is a respected authority in his field. One of his best-known works is *Earthquakes, Volcanoes, and Tsunamis: An Anatomy of Hazards.* In retirement, he serves as consultant to government and the private sector on earthquake hazard evaluation.

James L. Stratta

The late Mr. Stratta had been a consulting Civil and Structural Engineer since his graduation from the University of California at Berkeley in 1943. In 1952 he formed a partnership for architecture and engineering services with special emphasis on seismic analysis and design. He began his private consulting practice in 1978, and served as an international consultant in earthquake engineering and the diagnosis of major structural failures.

Mr. Stratta served in 1962 as president of the Structural Engineers Association of Northern California (SEAONC), and in 1967 as president of the Consulting Engineers Association of California. He was a director of the American Consulting Engineers Council, 1968, and a member of the Earthquake Engineering Research Institute. He was a fellow of both the American Society of Civil Engineers and the American Consulting Engineers Council. He authored or co-authored numerous reports covering the effects of the world's damaging earthquakes.

Mr. Stratta firmly believed that engineers should see earthquake damage in the field. Consequently, he organized a series of seminars in 1977 called *Learning from Earthquakes*. This collection of first-hand accounts by *Expert Quake Chasers* was published by the SEAONC in 1986. He also believed that principals of firms should be responsible for connection details and *constructibility*. This was the underlying theme of his 1985 book on designing buildings to resist earthquakes, published by McGraw Hill entitled *Manual of Seismic Design*.

Sam W. Swan

Mr. Swan is vice president of EQE Inc., San Francisco and a registered Mechanical and Nuclear Engineer in California. He holds an MS degree in mechanical engineering and the Degree of Engineer from Stanford University. He has been a principal investigator of earthquakes for EQE under the sponsorship of the Electric Power Research Institute (EPRI), the National Center for Earthquake Engineering Research (NCEER) and the Earthquake Engineering Research Institute (EERI). His primary focus has been the effects of earthquakes on industrial facilities and lifelines. Mr. Swan has investigated over 20 damaging earthquakes and compiled the results into a computerized data base which provides detailed information about the seismic performance of several thousand items of electrical and mechanical equipment from 120 sites in regions of strong shaking. He has used this experience and data to perform seismic hazard reviews and vulnerability studies for numerous nuclear and conventional power plants, industrial facilities and utility systems.

Mr. Swan provided the primary technical basis for the U.S. nuclear industry's resolution of Nuclear Regulatory Commission (NRC) Safety Issue A-46, the seismic qualification of critical equipment in operating nuclear plants. This same technical basis has been incorporated into standards developed by the U.S. Department of Energy (DOE), the Institute of Electrical and Electronics Engineers (IEEE), and the American Society of Mechanical Engineers (ASME). He is currently serving on the committee for revision of IEEE Standard 693, seismic design of electrical substations.

Mr. Swan's primary interest is in the seismic vulnerability of lifelines for power, water, communications, fuel and transportation systems.

Douglas W. Thornburg

Mr. Thornburg has more than 16 years' experience in management and technical and educational aspects of building regulation, including plan review, field inspection, supervision, and training. He served the City of Wichita, Kansas, as building inspector, plans examiner, and codes administrator. In the latter position, he was responsible for management of all inspection operations.

A graduate of Kansas State University with a Bachelor of Architecture degree, Mr. Thornburg is a registered Architect and member of the American Institute of Architects. Following four years in private construction/design work and five years with the City of Wichita, he began working as a training and development specialist with the International Conference of Building Officials (ICBO), publishers of the Uniform Building Code (UBC). Now as Principal Staff Architect, Mr. Thornburg is involved in a variety of activities, including the ICBO seminar, video training, and certification programs.

Diana R. Todd

Diana Todd was a Research Structural Engineer with the National Institute of Standards and Technology (NIST) in Gaithersburg, Maryland. She holds a B.S. degree in English from Dartmouth College and an M.S. degree in Civil Engineering from the University of Colorado. Prior to joining the research staff at NIST, Ms. Todd was a structural engineer with J.R. Harris & Company in Denver where she helped develop teaching materials to introduce engineers, architects, and building officials to the *National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for the Development of Seismic Regulations for New Buildings.*

Ms. Todd worked with the Interagency Committee on Seismic Safety in Construction (ICSSC) to develop seismic evaluation and rehabilitation standards for federally owned and leased buildings. Previously, she helped the ICSSC develop guidelines for implementing the President's Executive Order 12699, *Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction.* As part of that project, she managed a study comparing the seismic provisions of the major model building codes to the *NEHRP Provisions* for new buildings.

Peter I. Yanev

Peter Yanev is Chairman of the Board and co-founder of EQE International in San Francisco. He holds a B.S. in Civil/Structural Engineering from the University of California at Berkeley and an M.S. in Structural/Earthquake Engineering from the Massachusetts Insititute of Technology at Cambridge. He is a registered Civil Engineer in California. His experience includes project management, consultation, and research in earthquake engineering and the seismic design, analysis, evaluation and testing of power generating facilities, and on-site investigations of more than 34 destructive earthquakes worldwide.

Mr. Yanev has written and presented more than 110 technical papers and is the author of *Peace of Mind in Earthquake Country,* a best selling book on earthquake engineering for the public. He is actively engaged in the dissemination of information to the public about earthquakes and earthquake hazards through numerous lectures, media appearances and writing for the San Francisco Chronicle.

Mr. Yanev has been a primary spokesperson and technical consultant for the Electrical Power Research Institute's (EPRI) Seismic Qualification Utility Group (SQUG) since its birth in 1981. He initially proposed the use of seismic experience data as an alternative to conventional seismic evaluation. Accepted by the U.S. Nuclear Regulatory Commission (NRC), this concept has become the basis of the SQUG program to resolve seismic safety issues for operating nuclear power plants.

Mr. Yanev has participated in walkdown inspections of more than 40 nuclear power plants worldwide. He has been a consultant to the U.S. Department of Energy (DOE), the U.S. Nuclear Regulatory Commission (NRC), the U.S. National Science Foundation (NSF), the Federal Emergency Management Agency (FEMA), the International Atomic Energy Agency (IAEA), the Advisory Committee on Reactor Safeguards (ACRS), and other licensing and industry standards groups such as IEEE, ASCE and ASME.

Appendix B

Bibliography of Documents Produced for the Department of Energy Natural Phenomena Hazards Safety Program

Hazard Estimates

Coats, D. W., and R. C. Murray (1984), Natural Phenomena Hazard Modeling Project: Seismic Hazard Models for Department of Energy Sites, LLNL Report UCRL-53582, Rev. 1.

Coats, D. W., and R. C. Murray (1984), Natural Phenomena Hazard Modeling Project: Extreme Wind/Tornado Hazard Models for Department of Energy Sites, LLNL Report UCRL-53526, Rev. 1.

Savy, J. B., and R. C. Murray (1988), Natural Phenomena Hazards Modeling Project: Flood Hazard Models for Department of Energy Sites, LLNL Report UCRL-53851.

McCann, M. W., and A. C. Boissonnade (1988), Preliminary Flood Hazard Estimates for Screening Department of Energy Sites, LLNL Report UCRL-21045.

McCann, M. W., and A. C. Boissonnade (1988), Probabilistic Flood Hazard Assessment for the N Reactor, Hanford, Washington, LLNL Report UCRL-21069.

State of Practice Manuals

Eagling, D. G. (1983) Seismic Safety Guide, Lawrence Berkeley Laboratory, Report LBL-9143.

Eagling, D.G. (1996), Department of Energy Seismic Safety Manual — A Practical Guide for Facility Managers and Earthquake Engineers, LLNL Report UCRL-MA-125085.

ED2 (1985), Suspended Ceiling System Survey and Seismic Bracing Recommendations, LLNL Report UCRL-15714.

EQE Inc. (1986), Practical Equipment Seismic Upgrade and Strengthening Guidelines, LLNL Report UCRL-15815.

McDonald, J. R. (1988), Structural Details for Wind Design, LLNL Report UCRL-21131.

EQE Engineering (1991), Structural Concepts and Details for Seismic Design, LLNL Report UCRL-CR-106554.

DOE NPH Order and Standards

Kennedy, R. P., and S. A. Short (1994), Basis for Seismic Provisions of DOE-STD-1020-94, LLNL Report UCRL-CR-111478, BNL Report - BNL 52418.

U.S. Department of Energy (January 1993), Natural Phenomena Mitigation, DOE Order 5480.28, Washington, D.C.

U.S. Department of Energy (April 1994), Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities, DOE-STD-1020-94.

U.S. Department of Energy (July 1993), Natural Phenomena Hazards, Performance Categorization Guidelines for Structures, Systems, and Components, DOE-STD-1021-93.

U.S. Department of Energy (March 1994), Natural Phenomena Hazards Site Characterization Criteria, DOE-STD-1022-94.

U.S. Department of Energy (September 1995), Natural Phenomena Hazards Assessment Criteria, DOE-STD-1023-95.

U.S. Department of Energy (December 1992), Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites, DOE-STD-1024-92.

Seismic Evaluation Procedure

Eder, S. J., M. W. Eli, and M. W. Salmon (1993), Walkthrough Screening Evaluation Field Guide, LLNL Report UCRL-ID-115714, Rev. 2.

Salmon, M. W., and R. P. Kennedy (1994), Meeting Performance Goals by the Use of Experience Data, LLNL Report UCRL-CR-120813.

Murray, R. C., J. K. Kimball, D. J. Guzy, J. R. Hill, (December 1994), Use of Experience Data for Seismic Evaluations at Department of Energy Facilities, presented at 5th Symposium on Current Issues Related to Nuclear Power Plant Structures, Equipment and Piping.

Seismic Evaluation Procedure for Equipment in U.S. Department of Energy Facilities, LLNL Report UCRL-ID-122109, (October 1996).

Other

Eli, M. W., S. C. Sommer, T. R. Roche, K. L. Merz (1995), The January 17, 1994 Northridge Earthquake: Effects on Selected Industrial Facilities and Lifelines, LLNL Report UCRL-ID-120522, Rev 0.

Chokshi, N. C., et al. (1996), The Hyogo-Ken Nanbu Earthquake of January 17, 1995—NRC/DOE Reconnaissance Report, NUREG Draft.

Harben, P. E. (1991), Earthquake Alert System Feasibility Study, LLNL, Report UCRL-LR-109625.

Technology Transfer Package on Seismic Base Isolation, (1994) Volume I Workshop Notes, Volume II, Short Course Notes, Volume III Reference Material, CONF 9205413.

McDonald, J. R. (1994), Tornado Missile Criteria for DOE Facilities, Draft LLNL Report.

Lawrence Livermore National Laboratory, Phenomenal News.

UCRL-MI-105101-90-1	October 1990
UCRL-MI-105101-91-1	January 1991
UCRL-MI-105101-91-2	April 1991
UCRL-MI-105101-91-3	July 1991
UCRL-MI-105101-91-4	October 1991
UCRL-MI-105101-92-1	January 1992
UCRL-MI-105101-92-2	April 1992
UCRL-MI-105101-92-3	July 1992
UCRL-MI-105101-92-4	October 1992
UCRL-MI-105101-93-1	January 1993
UCRL-MI-105101-93-2	April 1993
UCRL-MI-105101-93-3	July 1993
UCRL-MI-105101-93-4	December 1993
UCRL-MI-105101-94-1	April 1994
UCRL-MI-105101-94-2	August 1994
UCRL-MI-105101-94-3	December 1994.

Heiken, G., M. Murphy, W. Hackett, and W. Scott (1995), Volcanic Hazards and Energy Infrastructure — United States, LANL Report with Map, LA-UR 95-1087.

Hasbrouck, R. T., and K. C. Majumdar (1995), Development of a Guidance Document for Lightning Protection of DOE Facilities, Draft LLNL Report.

Conference Proceedings

Lawrence Livermore National Laboratory (1985), Proceedings, DOE Natural Phenomena Hazards Mitigation Conference, LLNL Report CONF-8510118.

Lawrence Livermore National Laboratory (1989), Proceedings, Second DOE Natural Phenomena Hazards Mitigation Conference, LLNL Report CONF-89010192.

Lawrence Livermore National Laboratory (1991), Proceedings, Third DOE Natural Phenomena Hazards Mitigation Conference, LLNL Report CONF-9110122.

Lawrence Livermore National Laboratory (1993), Proceedings, Fourth DOE Natural Phenomena Hazards Mitigation Conference, LLNL Report CONF-9310102.

Lawrence Livermore National Laboratory (1995) Proceedings, Fifth DOE Natural Phenomena Hazards Mitigation Symposium.

Mini Courses

Kennedy, R. P., and S. A. Short (1985), Seismic Analysis, LLNL Report UCRL-15742.

Nicoletti, J. P., L. E. Malik, and P. G. Griffin (1985), Seismic Design, LLNL Report UCRL-15744.

Mc Donald, J. R. (1985), Extreme Winds and Tornadoes: An Overview, LLNL Report UCRL-15745.

McDonald, J. R. (1985), Extreme Winds and Tornadoes: Design and Evaluation of Buildings and Structures, LLNL Report UCRL-15747.

McCann, M. W., And J. J. Owen (1985), Overview of Flood Considerations, LLNL Report UCRL-15743.

Yanev, P. I., S. Hom, C. A. Kircher, and N. D. Bailey (1985), Emergency Preparedness, LLNL Report UCRL-15746.

COMMENT FORM

SEISMIC SAFETY MANUAL — A PRACTICAL GUIDE FOR FACILITY MANAGERS AND EARTHQUAKE ENGINEERS

We would appreciate your comments on the use of this document. Comments and corrections are welcomed at your earliest convenience.

Address all comments to:

Robert C. Murray, L-224
LAWRENCE LIVERMORE NATIONAL LABORATORY
P. O. Box 808
Livermore, California 94550

(510) 422-0308
(510) 423-2163 (Fax)

Comments: