EARTHQUAKE LOSS ESTIMATION METHODOLOGY

HAZUS®99

TECHNICAL MANUAL

Developed by:

Federal Emergency Management Agency
Washington, D.C.

Through a cooperative agreement with:

National Institute of Building Sciences
Washington, D.C.

Preface

Earthquakes pose a threat to life and property in 45 states and territories. As the United States has become more urbanized, more frequent smaller earthquakes in the 6.5 to 7.5 Magnitude range now have the potential of causing damage equal to or exceeding the estimated $40 billion from the 1994 Northridge earthquake. Earthquakes in urban areas, such as Kobe, Japan and Izmit, Turkey, are grim reminders of the kind of damage that may result from larger earthquakes, like the San Francisco event of 1906 and eastern events that occurred in New Madrid in 1811-12.

The Federal Emergency Management Agency is committed to mitigation as a means of reducing damages and the social and economic impacts from earthquakes. FEMA, under a Cooperative Agreement with the National Institute of Building Sciences, has developed HAZUS®99 (HAZUS® stands for "Hazards U.S."), the second edition of the standard, nationally-applicable methodology for assessing earthquake risk. Significant enhancements have been added to HAZUS®99, particularly, a disaster response application to facilitate the use of HAZUS® in the immediate post-disaster environment. HAZUS®99 and the preceding edition of the earthquake loss estimation methodology, HAZUS®97, represent the dedicated efforts of more than 130 nationally-recognized earthquake and software professionals.

HAZUS is an important component of FEMA's *Project Impact*, a national movement to create safe and disaster-resistant communities. FEMA is making HAZUS® available to all states and communities, including the almost 200 now participating in *Project Impact*, and the private sector. Communities find HAZUS® to be a valuable tool in promoting a broader understanding of potential earthquake losses and in helping to build a community consensus for disaster loss prevention and mitigation.

Since the first release of HAZUS®, FEMA has been expanding the capability of HAZUS® by initiating loss estimation models for flood and hurricane hazards. Preview versions of these flood and hurricane models are being readied for release in 2002.

I am pleased to disseminate this manual to state and local users.

Michael J. Armstrong
Associate Director for Mitigation
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Foreword

The work that provided the basis for this publication was supported by funding from the Federal Emergency Management Agency (FEMA) under a cooperative agreement with the National Institute of Building Sciences. The substance and findings of that work are dedicated to the public. NIBS is solely responsible for the accuracy of the statements and interpretations contained in this publication. Such interpretations do not necessarily reflect the views of the Federal Government.

The National Institute of Building Sciences (NIBS) is a non-governmental, non-profit organization, authorized by Congress to encourage a more rational building regulatory environment, to accelerate the introduction of existing and new technology into the building process and to disseminate technical information.

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MESSAGE TO USERS

HAZUS is designed to produce loss estimates for use by state, regional and local governments in planning for earthquake loss mitigation, emergency preparedness and response and recovery. The methodology deals with nearly all aspects of the built environment, and with a wide range of different types of losses. The methodology has been tested against the experience from several past earthquakes and against the judgment of experts. Subject to several limitations noted below, HAZUS has been judged capable of producing results that are credible for the intended purposes.

Uncertainties are inherent is any such loss estimation methodology. They arise in part from incomplete scientific knowledge concerning earthquakes and their effect upon buildings and facilities, and in part from the approximations and simplifications necessary for comprehensive analyses. The possible range of uncertainty, possibly a factor or two or more, is best evaluated by conducting multiple analyses, varying certain of the input parameters to which losses are most sensitive. This *User's Manual* gives guidance concerning the planning of such sensitivity studies.

Users should be aware of the following specific limitations:

> HAZUS is most accurate when applied to a class of buildings or facilities, and least accurate if applied to a particular building or facility.
>
> Accuracy of losses associated with lifelines may be less than for losses associated with the general building stock.
>
> Based on several initial abbreviated tests, the losses from small magnitude (less than M 6.0) earthquakes appear to be overestimated.
>
> Uncertainty related to the characteristics of ground motion in the Eastern U.S. is high. Conservative treatment of this uncertainty may lead to overestimation of losses in this area, both for scenario events and when using probabilistic ground motion.
>
> Pilot and calibration studies have as yet not provided an adequate test concerning the possible extent and effects of landslides and the performance of water systems.
>
> The indirect economic loss module is new and experimental. While output from pilot studies has generally been credible, this module requires further testing.

HAZUS should be regarded as a work in progress. Additional improvements and increased confidence will come with further experience in using HAZUS. To assist us in further improving HAZUS, users are invited to submit comments on methodological and software issues by letter, fax or e-mail to:

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What is New in HAZUS99?

- The ground motion model has been revised by implementing new algorithms for calculating the distance to the fault rupture plane and accounting for earthquakes that rupture across multiple fault segments. New attenuation functions have been added for Hawaii (Munson & Thurber) and the Eastern United States (Lawrence Livermore National Lab). Details of these changes are included in Chapter 4 of the *Technical Manual*.

- A new bridge model based on the nonlinear performance of bridges has been implemented along with a revised bridge classification scheme and updated national bridge inventory. Details of these changes are included in Chapter 7 of the Technical Manual.

- For the probabilistic analysis of building damage, revised fragility curves have been added that are compatible with the USGS probabilistic ground motion maps. These new fragility curves, however, are still under review by the Earthquake Committee. In addition, **HAZUS99** now has the capability to automatically compute annualized loss estimates for buildings. Details of these changes are included in Chapters 5 and 16 of the *Technical Manual*.

- HAZUS99 now includes a network analysis model for potable water systems. Although the model is fully functional, the results generated are still under review by the Utility Lifeline Subcommittee. Details of these changes are included in Chapter 8 of the *Technical Manual*.

- The indirect economic loss model has been improved to accommodate weekly and monthly inputs in the first two years after an earthquake event. Details of these changes are included in Chapter 16 of the *Technical Manual*.

- **HAZUS99** includes a new application that can directly link **HAZUS** with Tri-NET. This capability will allow **HAZUS** to monitor Tri-NET and to automatically create a study region and execute the analysis when an earthquake is broadcast. In addition, **HAZUS99** response and recovery capabilities have been enhanced with the addition of a “ground truthing” option. This special feature allows users to incorporate observed damage information for use in post-event operational response. Details of these changes are included in Chapter 9 and 12 of the *User’s Manual*.

- **HAZUS99** has been optimized for greater speed.

- In addition to several new summary reports, a comprehensive summary report of analysis results has been added. The report, about 20 pages in length, contains text and tabular data about the study region, the earthquake scenario selected, and the results.

- The capability to save and recall map workspaces has been added.

- Several databases in HAZUS99 have been added: updated USGS probabilistic ground motion maps and US source maps, a revised hospital database, a new national bridge inventory, an updated hazardous material site database and a new national railroad track database.

Chapter 1
Introduction to the FEMA Loss Estimation Methodology

1.1 Background

The Technical Manual describes the methods for performing earthquake loss estimation. It is based on a multi-year project to develop a nationally applicable methodology for estimating potential earthquake losses on a regional basis. The project has being conducted for the National Institute of Building Science (NIBS) under a cooperative agreement with the Federal Emergency Management Agency (FEMA).

The primary purpose of the project is to develop guidelines and procedures for making earthquake loss estimates at a regional scale. These loss estimates would be used primarily by local, state and regional officials to plan and stimulate efforts to reduce risks from earthquakes and to prepare for emergency response and recovery. A secondary purpose of the project is to provide a basis for assessing nationwide risk of earthquake losses.

The methodology development and software implementation has been performed by a team of earthquake loss experts composed of earth scientists, engineers, architects, economists, emergency planners, social scientists and software developers. The Earthquake Committee has provided technical direction and review of work with guidance from the Project Oversight Committee (POC), a group representing user interests in the earthquake engineering community.

1.2 Technical Manual Scope

The scope of the *Technical Manual* includes documentation of all methods and data that are used by the methodology. Loss estimation methods and data are obtained from referenced sources tailored to fit the framework of the methodology, or from new methods and data developed when existing methods and data were lacking or were not current with the state of the art.

The *Technical Manual* is a comprehensive, highly technical collection of methods and data covering a broad range of topics and disciplines, including earth science, seismic/structural engineering, social science and economics. The *Technical Manual* is written for readers who are expected to have some degree of expertise in the technical topic of interest, and may be inappropriate for readers who do not have this background.

As described in Chapter 2, a separate *User Manual* describes the earthquake loss estimation methodology in non-technical terms and provides guidance to users in the application of the methodology. The methodology software is implemented using Geographical Information System (GIS) software as described in the *Technical Manual*.

1.3 Technical Manual Organization

The *Technical Manual* contains sixteen chapters. Chapter 2 describes the overall framework of the methodology and provides background on the approach developed used to meet the project's objectives. Chapter 3 discusses inventory data, including classification schemes of different systems, attributes required to perform damage and loss estimation, and the data supplied with the methodology. Sources and methods of collection of inventory data are not covered in Chapter 3, but may be found in the *User Manual*.

Chapters 4 through 16 cover, respectively, each of thirteen major components or subcomponents (modules) of the methodology. Each of the major components and subcomponents are described in Chapter 2. A flowchart is provided in Chapter 2 as a "road map" of the relationships between modules of the methodology. This flowchart is repeated at the beginning of each chapter with the module of interest high-lighted to show input from and output to other modules of the methodology.

Chapter 2
Overall Approach and Framework of Methodology

This chapter describes the overall approach used by the developers to meet the objectives of the project, the components and subcomponents of earthquake loss estimation and their relationship within the framework of methodology.

2.1 Vision Statement

The overall approach for the project is based on the following "vision" of the earthquake loss estimation methodology.

> *The earthquake loss estimation methodology will provide local, state and regional officials with the tools necessary to plan and stimulate efforts to reduce risk from earthquakes and to prepare for emergency response and recovery from an earthquake. The methodology will also provide the basis for assessment of nationwide risks of earthquake loss.*
>
> *The methodology can be used by a variety of users with needs ranging from simplified estimates that require minimal input to refined calculations of earthquake loss. The methodology may be implemented using either geographical information system (GIS) technology provided in a software package or by application of the theory documented in a Technical Manual. An easily understood User Manual will guide implementation of the methodology by either technical or non-technical users.*

The vision of earthquake loss estimation requires a methodology that is both flexible, accommodating the needs of a variety of different users and applications, and able to provide the uniformity of a standardized approach. The framework of the methodology includes each of the components shown in Figure 2-1: Potential Earth Science Hazard (PESH), Inventory, Direct Physical Damage, Induced Physical Damage, Direct Economic/Social Loss and Indirect Economic Loss. As indicated by arrows in the figure, modules are interdependent with output of some modules acting as input to others. In general, each of the components will be required for loss estimation. However, the degree of sophistication and associated cost will vary greatly by user and application. It is therefore necessary and appropriate that components have multiple levels (multiple modules) of detail or precision when required to accommodate user needs.

Framing the earthquake loss estimation methodology as a collection of modules permits adding new modules (or improving models/data of existing modules) without reworking the entire methodology. Improvements may be made to adapt modules to local or regional needs or to incorporate new models and data. The modular nature of the methodology permits a logical evolution of the methodology as research progresses and the state-of-the-art advances.

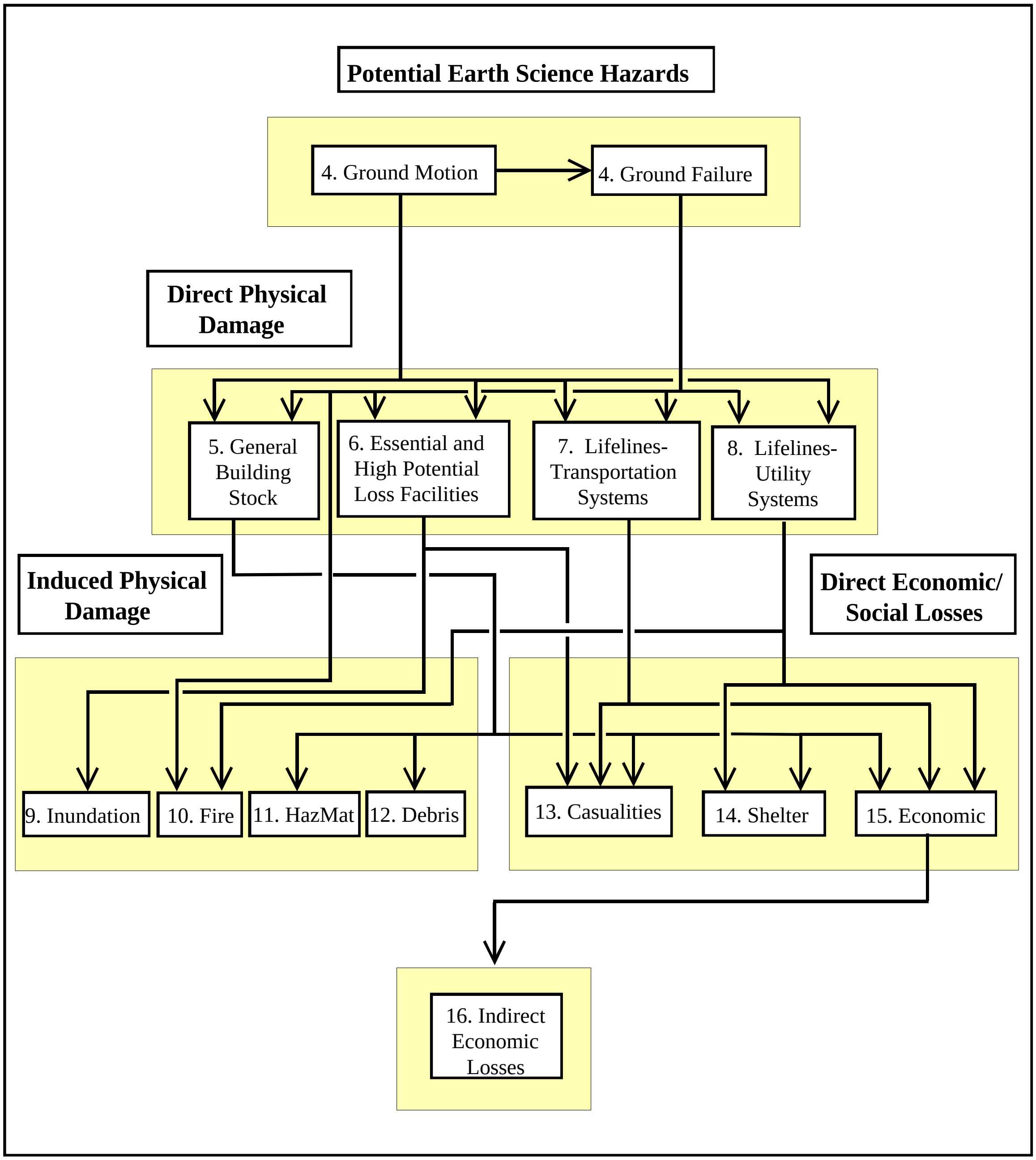

Figure 2.1 Flowchart of the Earthquake Loss Estimation Methodology.

Most users will implement the methodology using the GIS-based software application provided by NIBS. After initial inventory entry, the program will run efficiently on desktop computer. The GIS technology provides a powerful tool for displaying outputs and permits users to "see" the effects of different earthquake scenarios and assumptions. A *User Manual* will guide users in program manipulation, input of new data, and changes to existing data.

Certain users may not wish to use the software application, or may want to augment the results with supplementary calculations. In such cases, users can refer to the *Technical Manual* for a complete description of models and data of each module. The *Technical Manual* is useful to technical experts, such as those engineers and scientists that have conducted previous earthquake loss studies, but might be inappropriate for non-technical users.

Both technical and non-technical users are guided in the application of the methodology by the *User Manual*, which addresses important implementation issues, such as:

(1) Selection of scenario earthquakes and PESH inputs
(2) Selection of appropriate methods (modules) to meet different user needs
(3) Collection of required inventory data, i.e., how to obtain necessary information
(4) Costs associated with inventory collection and methodology implementation
(5) Presentation of results including appropriate terminology, etc.
(6) Interpretation of results including consideration of model/data uncertainty.

The three project deliverables are shown in Figure 2.2.

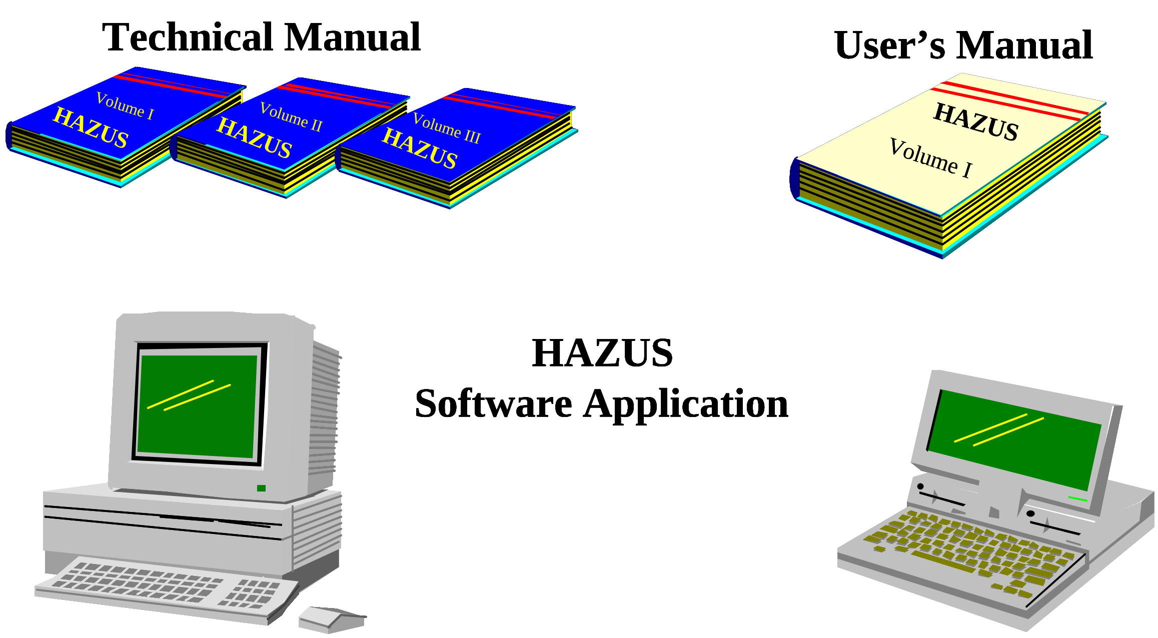

Figure 2.2 Project Deliverables.

2.2 Project Objectives

The development of an earthquake loss estimation methodology has been defined by the eight General Objectives outlined in the NIBS/FEMA "Task Plan for Tasks 2 and 5," October 18, 1993. The following sections summarize the approach taken to meet each objective.

Accommodation of User Needs

The methodology utilizes a modular approach with different modules addressing different user needs. This approach avoids the need to decide on who is the designated user. The needs of most, if not all, users are accommodated by the flexibility of a modular approach.

The GIS technology permits easy implementation by users on desktop computers. The visual display and interactive nature of a GIS application provides an immediate basis for exchange of information and dialog with end-users of the results. The *User Manual* provides appropriate terminology and definitions, and user-oriented descriptions of the loss estimation process.

State-of-the-Art

The methodology incorporates available state-of-the-art models in the earthquake loss estimation methodology. For example, ground shaking hazard and related damage functions are described in terms of spectral response rather than MMI. Modules include damage loss estimators not previously found in most studies, such as induced damage due to fire following earthquake and indirect economic loss. A nationally applicable scheme is developed for classifying buildings, structures and facilities.

Balance

The methodology permits users to select methods (modules) that produce varying degrees of precision. The *User Manual* provides guidance to users regarding the selection of modules that are appropriate for their needs and which have a proper balance between different components of earthquake loss estimation.

Flexibility in Earthquake Demand

The methodology incorporates both deterministic (scenario earthquake) and probabilistic descriptions of spectral response. Alternatively, the proposed methodology accepts user-supplied maps of earthquake demand. The software application is structured to also accept externally supplied maps of earthquake ground shaking.

"Uncertainty" in earthquake demand due to spatial variability of ground motion is addressed implicitly by the variability of damage probability matrices (DPM's) or fragility curves. Uncertainty in earthquake demand due to temporal variability (i.e., earthquake recurrence rate) or uncertainty in the magnitude of earthquake selected for scenario event may be readily evaluated by the users.

Once the data is input into the software application, any number of scenario events can be evaluated. The *User Manual* provides guidance for the consideration of uncertainty, including that associated with earthquake demand.

Uses of Methodology Data

The *User Manual* provides recommendations for collecting inventory data that will permit use of the data for non-earthquake purposes. Inventory information will come from databases supplied with the methodology and/or collected in databases compatible with the software. Such data will be available to users for other applications.

Accommodation of Different Levels of Funding

The methodology includes modules that permit different levels of inventory collection and associated levels of funding. For example, the methodology permits simplified (Default Data Analysis) estimates of damage and loss, using primarily default data supplied with the software application. These estimates of damage/loss do not require extensive inventory collection and can be performed on a modest budget. More precise damage/loss (User-Supplied Data Analysis) estimates require more extensive inventory information at additional cost to the user. The *User Manual* provides guidance to users regarding trade-offs in cost and accuracy of results.

Standardization

The methodology includes standard methods for:

(1) Inventory data collection based on census tract areas
(2) Using database maps of soil type, ground motion, ground failure, etc.
(3) Classifying occupancy of buildings and facilities
(4) Classifying building structure type
(5) Describing damage states
(6) Developing building damage functions
(7) Grouping, ranking and analyzing lifelines
(7) Using technical terminology
(8) Providing output.

Non-Proprietary

The methodology includes only non-proprietary loss estimation methods. The software application is non-proprietary to the extent permitted by the GIS-software suppliers.

2.3 Description of Loss Estimation Methodology

The earthquake loss estimation methodology is an improvement over existing regional loss estimation methodologies, since it more completely addresses regional impacts of earthquakes that have been omitted or at best discussed in a qualitative manner in previous studies. Examples of these impacts are service outages for lifelines, estimates of fire ignitions and fire spread, potential for a serious hazardous materials release incident, and indirect economic effects. In addition, strength of this methodology is the ability to readily display inputs and outputs on GIS-based maps that can be overlaid. By overlaying maps the user is able to experiment with different scenarios and ask "what if" questions.

As discussed in Section 2.1, the methodology is modular, with different modules interacting in the calculation of different losses. Figure 2.1 shows each of the modules and the flow of information among them. It can be seen that, because of the complexity of earthquake damage and loss estimation, the model is complex. One advantage of the modularity of the methodology is that it enables users to limit their studies to selected losses. For example, a user may wish to ignore induced physical damage when computing direct economic and social losses. This would eliminate the lower left portion of the flow diagram along with corresponding input requirements. A limited study may be desirable for a variety of reasons, including budget and inventory constraints, or the need to obtain answers to very specific questions.

The methodology has been developed with as much capability as possible. However, there are certain areas where methods are limited. For example, the methodology calculates potential exposure to flood (e.g., dam break) or fire (following earthquake) in terms of the fraction of a geographical area that may be flooded or burned, but does not have methods for rigorous calculation of damage or loss due to flooding or fire. Consequently, these two potential contributors to the total loss would not be included in estimates of economic loss, casualties or loss of shelter.

A limiting factor in performing a study and quality of the inventory is the associated cost. Collection of inventory is without question the most costly part of performing the study. Furthermore, many municipalities have limited budgets for performing an earthquake loss estimation study. Thus, the methodology is structured to accommodate different users with different levels of resources.

While most users will develop a local inventory that best reflects the characteristics of their region, such as building types and demographics, the methodology is capable of producing crude estimates of losses based on a minimum of local input. Of course, the quality and uncertainty of the results is related to the detail of the inventory and the economic and demographic data provided. Crude estimates would most likely be used only as initial estimates to determine where more detailed analyses would be warranted.

At the other end of the spectrum, a user may wish to make detailed assessments of damage to and service outages for lifelines. Detailed analyses of lifelines require cooperation and input from utilities and transportation agencies. Lifeline systems require an understanding of the interactions between components and the potential for alternative pathways when certain components fail. Thus, without cooperation of utilities, the user is limited in the quality of analysis that can be performed.

The proposed loss estimation methods are capable of providing estimates of damage to and service outages for lifelines with a minimum of cooperation from lifeline operators. These estimates, of course, will have a great deal of uncertainty associated with them. However, they will be useful for planning purposes and for an initial estimate to determine where more detailed analyses would be warranted. Many lifeline operators perform their own detailed earthquake loss studies that incorporate detailed models of their systems.

Three types of analysis are defined to describe implementation of the methodology by users with different needs and resources. These types and their definitions are somewhat arbitrary, and the boundaries between the three types are not well defined. The three types are defined as follows:

Default Data Analysis: This is the simplest type of analysis requiring minimum effort by the user as it is based mostly on input provided with the methodology (e.g. census information, broad regional patterns of seismic code adoption and earthquake resistance of classes of construction, etc.). The user is not expected to have extensive technical knowledge. While the methods require some user-supplied input to run, the type of input required could be gathered by contacting government agencies or by referring to published information. At this level, estimates will be crude, and will likely be appropriate only as initial loss estimates to determine where more detailed analyses are warranted.

Some components of the methodology cannot be performed in a Default Data Analysis since they require more detailed inventory than that provided with the methodology. The following are not included in the Default Data Analysis: damage/loss due to liquefaction, landslide or surface fault rupture; damage/loss due to tsunamis, seiche or dam failure. At this level, the user has the option (not required) to enter information about hazardous substances and emergency facilities. One week to a month would be required to collect relevant information depending on the size of the region and the level of detail the user desires.

User-Supplied Data Analysis: This type of analysis will be the most commonly used. It requires more extensive inventory data and effort by the user than Default Data Analysis. The purpose of this type is to provide the user with the best estimates of earthquake damage/loss that can be obtained using the standardized methods of analysis included in the methodology. It is likely that the user will need to employ consultants to assist in the implementation of certain methods. For example, a local geotechnical engineer would likely be required to define soil and ground conditions.

All components of the methodology can be performed at this level and loss estimates are based on locally (user) developed inventories. At this level, there are standardized methods of analysis included in the software, but there is no standardized User-Supplied Data Analysis study. As the user provides more complete data, the quality of the analysis and results improve. Depending on the size of the region and the level of detail desired by the user, one to six months would be required to obtain the required input for this type of analysis.

Advanced Data and Models Analysis: This type incorporates results from engineering and economic studies carried out using methods and software not included within the methodology. At this level, one or more technical experts would be needed to acquire data, perform detailed analyses, assess damage/loss, and assist the user in gathering more extensive inventory. It is anticipated that at this level there will be extensive participation by local utilities and owners of special facilities. There is no standardized Advanced Data and Models Analysis study. The quality and detail of the results depend upon the level of effort. Six months to two years would be required to complete an Advanced Data and Models Analysis.

To summarize, User-Supplied Data Analysis and Advanced Data and Models Analysis represent a broad range of analyses, and the line between one type of analysis and another is fuzzy. The above definitions are provided to understand the scope and flexibility of the methodology, not to limit its application. The primary limit on the type of analysis will be the user's ability to provide required data.

Even with perfect data, which can never be obtained, the methodology would not be able to precisely estimate earthquake loss. Simply put, predictive methods are approximate and will often have large amounts of uncertainty associated with damage and loss estimates. A discussion of uncertainty and guidance for users performing earthquake loss estimation is provided in the *User Manual*.

Chapter 3
Inventory Data: Collection and Classification

3.1. Introduction

This chapter describes the classification of different buildings and lifeline systems, data and attributes required for performing damage and loss estimation, and the data supplied with the methodology. The different systems covered in this chapter include buildings and facilities, transportation systems, utility systems, and hazardous material facilities. In addition, census data, county business patterns, and indirect economic data are discussed. Sources and methods of collecting inventory data can be found in the User's Manual.

Required input data include both default data (data supplied with the methodology) and data that must be supplied by the user. Data supplied with the methodology include default values of classification systems (i.e., mapping relationships) and default databases (e.g., facility location, census information, and economic factors). Default data are supplied to assist the user that may not have the resources to develop inventory data and may be superseded by better information when the user can obtain such for the study region of interest.

3.2. Direct Damage Data - Buildings and Facilities

This section deals with the general building stock, essential facilities, and high potential loss facilities.

3.2.1. General Building Stock

The general building stock includes residential, commercial, industrial, agricultural, religious, government, and educational buildings. The damage state probability of the general building stock is computed at the centroid of the census tract. The entire composition of the general building stock within a given census tract is lumped at the centroid of the census tract. The inventory information required for the analysis to evaluate the probability of damage to occupancy classes is the relationship between the specific occupancy class and the model building types. This can be computed directly from the specific occupancy class square footage inventory.

3.2.1.1. Classification

The purpose of a building inventory classification system is to group buildings with similar damage/loss characteristics into a set of pre-defined building classes. Damage and loss prediction models can then be developed for model building types which represent the average characteristics of the total population of buildings within each class.

The building inventory classification system used in this methodology has been developed to provide an ability to differentiate between buildings with substantially different damage and loss characteristics. The following primary parameters affecting building damage and loss characteristics were given consideration in developing the building inventory classification system.

- Structural parameters affecting structural capacity and response
  - Basic structural system (steel moment frame)
  - Building height (low-rise, mid-rise, high-rise)
  - Seismic design criteria (seismic zone) (Refer to Chapter 5)
- Nonstructural elements affecting nonstructural damage
- Occupancy (affecting casualties, business interruption and contents damage)
- Regional building practices (Refer to Chapter 5)
- Variability of building characteristics within the classification

To account for these parameters, the building inventory classification system consists of a two-dimensional matrix relating building structure (model building) types grouped in terms of basic structural systems and occupancy classes.

The basic model building types are based on FEMA-178 (FEMA, 1992) building classes. Building height subclasses were added to reflect the variation of typical building periods and other design parameters with building height. Mobile homes, which are not included in the FEMA-178 classification, were also added. A listing of structural building types, with corresponding labels, descriptions, and heights, is provided in Table 3.1.

The general building stock is also classified based on occupancy. The occupancy classification is broken into general occupancy and specific occupancy classes. For the methodology, the general occupancy classification system consists of seven groups (residential, commercial, industrial, religion/nonprofit, government, education and lifelines). There are 28 specific occupancy classes. The building occupancy classes are given in Table 3.2, where the general occupancy classes are identified in boldface. The distribution of specific occupancies classes within each general occupancy class can be computed for each census tract based on the occupancy square footage inventory (Section 3.6). These relationships are in a form shown in Table 3A.1 of Appendix 3A.

Table 3.1: Building Structure (Model Building) Types

No.	Label	Description	Height			
			Range		Typical	
			Name	Stories	Stories	Feet
1 2	W1 W2	**Wood, Light Frame (≤ 5,000 sq. ft.)** **Wood, Commercial and Industrial (> 5,000 sq. ft.)**		1 - 2 All	1 2	14 24
3 4 5	S1L S1M S1H	**Steel Moment Frame**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
6 7 8	S2L S2M S2H	**Steel Braced Frame**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
9	S3	**Steel Light Frame**		All	1	15
10 11 12	S4L S4M S4H	**Steel Frame with Cast-in-Place Concrete Shear Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
13 14 15	S5L S5M S5H	**Steel Frame with Unreinforced Masonry Infill Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
16 17 18	C1L C1M C1H	**Concrete Moment Frame**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
19 20 21	C2L C2M C2H	**Concrete Shear Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
22 23 24	C3L C3M C3H	**Concrete Frame with Unreinforced Masonry Infill Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
25	PC1	**Precast Concrete Tilt-Up Walls**		All	1	15
26 27 28	PC2L PC2M PC2H	**Precast Concrete Frames with Concrete Shear Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
29 30	RM1L RM2M	**Reinforced Masonry Bearing Walls with Wood or Metal Deck Diaphragms**	Low-Rise Mid-Rise	1-3 4+	2 5	20 50
31 32 33	RM2L RM2M RM2H	**Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
34 35	URML URM M	**Unreinforced Masonry Bearing Walls**	Low-Rise Mid-Rise	1 - 2 3+	1 3	15 35
36	MH	**Mobile Homes**		All	1	10

Table 3.2: Building Occupancy Classes

Label	Occupancy Class	Example Descriptions
	Residential	
RES1	Single Family Dwelling	House
RES2	Mobile Home	Mobile Home
RES3	Multi Family Dwelling	Apartment/Condominium
RES4	Temporary Lodging	Hotel/Motel
RES5	Institutional Dormitory	Group Housing (military, college), Jails
RES6	Nursing Home	
	Commercial	
COM1	Retail Trade	Store
COM2	Wholesale Trade	Warehouse
COM3	Personal and Repair Services	Service Station/Shop
COM4	Professional/Technical Services	Offices
COM5	Banks	
COM6	Hospital	
COM7	Medical Office/Clinic	
COM8	Entertainment & Recreation	Restaurants/Bars
COM9	Theaters	Theaters
COM10	Parking	Garages
	Industrial	
IND1	Heavy	Factory
IND2	Light	Factory
IND3	Food/Drugs/Chemicals	Factory
IND4	Metals/Minerals Processing	Factory
IND5	High Technology	Factory
IND6	Construction	Office
	Agriculture	
AGR1	Agriculture	
	Religion/Non/Profit	
REL1	Church/Non-Profit	
	Government	
GOV1	General Services	Office
GOV2	Emergency Response	Police/Fire Station/EOC
	Education	
EDU1	Grade Schools	
EDU2	Colleges/Universities	Does not include group housing

3.2.1.2. Specific Occupancy-to-Model Building Type Mapping

Default mapping schemes for specific occupancy classes (except for RES1) to model building types by floor area percentage are provided in Tables 3A.2 through 3A.16 of Appendix 3A. Table 3A.2 through 3A.10 provide the suggested mappings for the Western U.S. buildings and are based on information provided in ATC-13 (1985). Tables 3A.11 through 3A.16 provide the mapping for buildings in the rest of the United States and are based on proprietary insurance data, opinions of a limited number of experts, and inferences drawn from tax assessors records. Table 3C.1 in Appendix 3C provides regional classification of the states. Table 3A.17 through 3A.21 provide model building distribution for the specific occupancy class “RES1” on a state-by-state basis. Tables 3A.2 through 3A.10 provide the mapping based on the height of buildings and the age of construction. The user must provide, for census tracts on the west coast, the proportion of buildings in low, mid, and high rise categories, and the proportion of buildings in the three categories according to age (pre- 1950, 1950-1970, and post 1970). These proportions are used to compute a weighted sum of matrices in Table 3A.2 through Table 3A.10 to arrive at the default specific occupancy class to model building type mapping. For the rest of the United States, Tables 3A.11 through 3A.16 provides the mapping based on the height of buildings only and the user must provide the proportion of buildings in low-, mid-, and high-rise categories to compute the default specific occupancy class to model building type mapping. The default mapping provided in Tables 3A.2 through 3A.16 should be considered as a guide: Accurate mapping may be developed based on the particular building type distribution within in the study region.

3.2.2. Essential Facilities

Essential facilities are those facilities that provide services to the community and should be functional after an earthquake. Essential facilities include hospitals, police stations, fire stations and schools. The damage state probabilities for essential facilities are determined on a site-specific basis (i.e., the ground motion parameters are computed at the location of the facility). The purpose of the essential facility module is to determine the expected loss of functionality for these critical facilities. Economic losses associated with these facilities are computed as part of the analysis of the general building stock (general building stock occupancy classes 12, 26, 27 and 28). The data required for the analysis include mapping of essential facility’s occupancy classes to model building types or a combination of essential facilities building type, design level and construction quality factor. In addition, the number of beds for each hospital and the number of fire trucks at each fire station are required. The fire truck information is used as input for the fire following earthquake analysis (Chapter 10).

3.2.2.1. Classification

The essential facilities are also classified based on the building structure type and occupancy class. The building structure types of essential facilities are the same as those for the general building stock presented in Table 3.1. The occupancy classification is broken into general occupancy and specific occupancy classes. For the methodology, the general occupancy classification system consists of three groups (medical care, emergency response, and schools). Specific occupancy consists of nine classes. The occupancy classes are given in Table 3.3, where the general occupancy classes are identified in boldface. Relationships between specific and general occupancy classes are in a form shown in Table 3B.1 of Appendix 3B.

Table 3.3: Essential Facilities Classification

Label	Occupancy Class	Description
	Medical Care Facilities	
EFHS	Small Hospital	Hospital with less than 50 Beds
EFHM	Medium Hospital	Hospital with beds between 50 & 150
EFHL	Large Hospital	Hospital with greater than 150 Beds
EFMC	Medical Clinics	Clinics, Labs, Blood Banks
	Emergency Response	
EFFS	Fire Station	
EFPS	Police Station	
EFEO	Emergency Operation Centers	
	Schools	
EFS1	Grade Schools	Primary/ High Schools
EFS2	Colleges/Universities	

3.2.2.2. Occupancy to Model Building Type Relationship

Default mapping of essential facility occupancy classes to model building types is provided in Tables 3B.2 through 3B.16 of Appendix 3B. For the regional designation of a particular state, refer to Table 3C.1 in Appendix C. The default mapping of specific occupancy to model building type mapping is based on general building stock occupancy classes 12, 26, 27 and 28.

3.2.3. High Potential Loss Facilities

High potential loss facilities are facilities that are likely to cause heavy earthquake losses if damaged. For this methodology, high potential loss (HPL) facilities include nuclear power plants, dams, and some military installations. The inventory data required for HPL facilities include the geographical location (latitude and longitude) of the facility. Damage and loss estimation calculation for high potential loss facilities are not performed as part of the methodology.

3.2.3.1. Classification

Three types of HPL facilities are identified in the methodology (dams, nuclear power facilities and military installations) are shown in Table 3.4. The dam classification is based on the National Inventory of Dams (NATDAM) database (FEMA, 1993).

Table 3.4: High Potential Loss Facilities Classification

Label	Description
	Dams
HPDE	Earth
HPDR	Rock fill
HPDG	Gravity
HPDB	Buttress
HPDA	Arch
HPDU	Multi-Arch
HPDC	Concrete
HPDM	Masonry
HPDS	Stone
HPDT	Timber Crib
HPDZ	Miscellaneous
	Nuclear Power Facilities
HPNP	Nuclear Power Facilities
	Military Installations
HPMI	Military Installations

3.3. Direct Damage Data - Transportation Systems

The inventory classification scheme for lifeline systems separates components that make up the system into a set of pre-defined classes. The classification system used in this methodology was developed to provide an ability to differentiate between varying lifeline system components with substantially different damage and loss characteristics. Transportation systems addressed in the methodology include highways, railways, light rail, bus, ports, ferries and airports. The classification of each of these transportation systems is discussed in detail in the following sections. The inventory data required for the analysis of each system is also identified in the following sections.

For some transportation facilities, classification of the facility is based on whether the equipment is anchored or not. Anchored equipment in general refers to equipment designed with special seismic tie-downs or tiebacks, while unanchored equipment refers to equipment designed with no special considerations other than the manufacturer's normal requirements. While some vibrating components, such as pumps, are bolted down regardless of concern for earthquakes, as used here “anchored” means all components have been engineered to meet seismic criteria which may include bracing (e.g., pipe or stack bracing) or flexibility requirements (e.g., flexible connections across separation joints) as well as anchorage.

3.3.1. Highway Systems

A highway transportation system consists of roadways, bridges and tunnels. The inventory data required for analysis include the geographical location, classification, and replacement cost of the system components. The analysis also requires the length of each highway segment.

3.3.1.1. Classification

The classes of highway system components are presented in Table 3.6. For more details on how to classify these components, refer to section 7.1.5 of Chapter 7.

Table 3.6: Highway System Classification

Label	Description
	Highway Roads
HRD1	Major Roads
HRD2	Urban Roads
	Highway Bridges
HWB1	Major Bridge - Length > 150m (Conventional Design)
HWB2	Major Bridge - Length > 150m (Seismic Design)
HWB3	Single Span – (Not HWB1 or HWB2) (Conventional Design)
HWB4	Single Span – (Not HWB1 or HWB2) (Seismic Design)
HWB5	Concrete, Multi-Column Bent, Simple Support (Conventional Design), Non-California (Non-CA)
HWB6	Concrete, Multi-Column Bent, Simple Support (Conventional Design), California (CA)
HWB7	Concrete, Multi-Column Bent, Simple Support (Seismic Design)
HWB8	Continuous Concrete, Single Column, Box Girder (Conventional Design)
HWB9	Continuous Concrete, Single Column, Box Girder (Seismic Design)
HWB10	Continuous Concrete, (Not HWB8 or HWB9) (Conventional Design)
HWB11	Continuous Concrete, (Not HWB8 or HWB9) (Seismic Design)
HWB12	Steel, Multi-Column Bent, Simple Support (Conventional Design), Non-California (Non-CA)
HWB13	Steel, Multi-Column Bent, Simple Support (Conventional Design), California (CA)
HWB14	Steel, Multi-Column Bent, Simple Support (Seismic Design)
HWB15	Continuous Steel (Conventional Design)
HWB16	Continuous Steel (Seismic Design)
HWB17	PS Concrete Multi-Column Bent, Simple Support - (Conventional Design), Non-California
HWB18	PS Concrete, Multi-Column Bent, Simple Support (Conventional Design), California (CA)
HWB19	PS Concrete, Multi-Column Bent, Simple Support (Seismic Design)
HWB20	PS Concrete, Single Column, Box Girder (Conventional Design)
HWB21	PS Concrete, Single Column, Box Girder (Seismic Design)
HWB22	Continuous Concrete, (Not HWB20/HWB21) (Conventional Design)
HWB23	Continuous Concrete, (Not HWB20/HWB21) (Seismic Design)
HWB24	Same definition as HWB12 except that the bridge length is less than 20 meters
HWB25	Same definition as HWB13 except that the bridge length is less than 20 meters
HWB26	Same definition as HWB15 except that the bridge length is less than 20 meters and Non-CA
HWB27	Same definition as HWB15 except that the bridge length is less than 20 meters and in CA
HWB28	All other bridges that are not classified (including wooden bridges)
	Highway Tunnels
HTU1	Highway Bored/Drilled Tunnel
HTU2	Highway Cut and Cover Tunnel

3.3.2. Railways

A railway transportation system consists of tracks, bridges, tunnels, stations, and fuel, dispatch and maintenance facilities. The inventory data required for analysis include the geographical location, classification and replacement cost of the facilities, bridges, tunnels, and track segments. The analysis also requires the length of the railway segments.

3.3.2.1.Classification

The various classes of railway system components are presented in Table 3.7. For more details on how to classify these components refer to section 7.2 of Chapter 7.

Table 3.7: Railway System Classification

Label	Description
	Railway Tracks
RTR1	Railway Tracks
	Railway Bridges
RBR1	Rail Bridge - Seismically Designed/Retrofitted
RBR2	Rail Bridge - Conventionally Designed
	Railway Urban Station
RST1L	Rail Urban Station, Reinforced Concrete Shear Walls (C2L)
RST2L	Rail Urban Station, Braced Steel Frame (S2L)
RST3L	Rail Urban Station, Moment Resisting Steel Frame (S1L)
RST4L	Rail Urban Station, Steel Frame & URM (S5L)
RST5L	Rail Urban Station, Precast Concrete Tilt-up (PC1L)
RST6L	Rail Urban Station, Reinforced Concrete Frame & URM (C3L)
RST7L	Rail Urban Station, Wood (W1)
RST1M	Rail Urban Station, Reinforced Concrete Shear Walls (C2L)
RST2M	Rail Urban Station, Braced Steel Frame (S2L)
RST3M	Rail Urban Station, Moment Resisting Steel Frame (S1L)
RST4M	Rail Urban Station, Steel Frame & URM (S5L)
RST5M	Rail Urban Station, Precast Concrete Tilt-up (PC1L)
RST6M	Rail Urban Station, Reinforced Concrete Frame & URM (C3L)
RST7M	Rail Urban Station, Wood (W1)
RST1H	Rail Urban Station, Reinforced Concrete Shear Walls (C2L)
RST2H	Rail Urban Station, Braced Steel Frame (S2L)
RST3H	Rail Urban Station, Moment Resisting Steel Frame (S1L)
RST4H	Rail Urban Station, Steel Frame & URM (S5L)
RST5H	Rail Urban Station, Precast Concrete Tilt-up (PC1L)
RST6H	Rail Urban Station, Reinforced Concrete Frame & URM (C3L)
RST7H	Rail Urban Station, Wood (W1)

H = high, M = moderate, L = low seismic design level.

Table 3.7 Cont.: Railway System Classification

Label	Description
	Railway Tunnels
RTU1	Rail Bored/Drilled Tunnel
RTU2	Rail Cut and Cover Tunnel
	Railway Fuel Facility
RFF1	Rail Fuel Facility w/ Anchored Tanks, w/ Back-Up (BU) Power
RFF2	Rail Fuel Facility w/ Anchored Tanks, w/o Back-Up (BU) Power
RFF3	Rail Fuel Facility w/ Unanchored Tanks, w/ Back-Up (BU) Power
RFF4	Rail Fuel Facility w/ Unanchored Tanks, w/o Back-Up (BU) Power
RFF5	Rail Fuel Facility w/ Buried Tanks
	Railway Dispatch Facility
RDF1	Rail Dispatch Facility w/ Anchored Sub-Component, w/ Back-Up (BU) Power
RDF2	Rail Dispatch Facility w/ Anchored Sub-Component, w/o BU Power
RDF3	Rail Dispatch Facility w/ Unanchored Sub-Component, w/ BU Power
RDF4	Rail Dispatch Facility w/ Unanchored Sub-Component, w/o BU Power
	Railway Maintenance Facility
RMF1L	Rail Maintenance Facility, Reinforced Concrete Shear Walls (C2L)
RMF2L	Rail Maintenance Facililty, Braced Steel Frame (S2L)
RMF3L	Rail Maintenance Facility, Moment Resisting Steel Frame (S1L)
RMF4L	Rail Maintenance Facility, Steel Frame & URM (S5L)
RMF5L	Rail Maintenance Facility, Precast Concrete Tilt-up (PC1)
RMF6L	Rail Maintenance Facility, Reinforced Concrete Frame & URM (C3L)
RMF7L	Rail Maintenance Facility, Wood (W1)
RMF1M	Rail Maintenance Facility, Reinforced Concrete Shear Walls (C2L)
RMF2M	Rail Maintenance Facililty, Braced Steel Frame (S2L)
RMF3M	Rail Maintenance Facility, Moment Resisting Steel Frame (S1L)
RMF4M	Rail Maintenance Facility, Steel Frame & URM (S5L)
RMF5M	Rail Maintenance Facility, Precast Concrete Tilt-up (PC1)
RMF6M	Rail Maintenance Facility, Reinforced Concrete Frame & URM (C3L)
RMF7M	Rail Maintenance Facility, Wood (W1)
RMF1H	Rail Maintenance Facility, Reinforced Concrete Shear Walls (C2L)
RMF2H	Rail Maintenance Facililty, Braced Steel Frame (S2L)
RMF3H	Rail Maintenance Facility, Moment Resisting Steel Frame (S1L)
RMF4H	Rail Maintenance Facility, Steel Frame & URM (S5L)
RMF5H	Rail Maintenance Facility, Precast Concrete Tilt-up (PC1)
RMF6H	Rail Maintenance Facility, Reinforced Concrete Frame & URM (C3L)
RMF7H	Rail Maintenance Facility, Wood (W1)

H = high, M = moderate, L = low seismic design level.

3.3.3. Light Rail

Like railways, light rail systems are composed of tracks, bridges, tunnels, and facilities. The major difference between the two is with regards to power supply, where light rail systems operate with DC power substations. The inventory data required for analysis include the classification, geographical location, and replacement cost of facilities, bridges, tunnels, and tracks. In addition, the analysis requires the track length.

3.3.3.1. Classification

Table 3.8 describes the various classes of light rail system components. For more details on how to classify these components refer to section 7.3 of Chapter 7.

Table 3.8: Light Rail System Classification

Label	Description
	Light Rail Tracks
LTR1	Light Rail Track
	Light Rail Bridges
LBR1	Light Rail Bridge - Seismically Designed/Retrofitted
LBR2	Light Rail Bridge - Conventionally Designed
	Light Rail Tunnels
LTU1	Light Rail Bored/Drilled Tunnel
LTU2	Light Rail Cut and Cover Tunnel
	DC Substation
LDC1	Light Rail DC Substation w/ Anchored Sub-Components
LDC2	Light Rail DC Substation w/ Unanchored Sub-Components
	Dispatch Facility
LDF1	Light Rail Dispatch Facility w/ Anchored Sub-Comp., w/ Back-Up (BU) Power
LDF2	Light Rail Dispatch Facility w/ Anchored Sub-Comp., w/o BU Power
LDF3	Light Rail Dispatch Facility w/ Unanchored Sub-Comp., w/ BU Power
LDF4	Light Rail Dispatch Facility w/ Unanchored Sub-Comp., w/o BU Power
	Maintenance Facility
LMF1L	Maintenance Facility, Reinforced Concrete Shear Walls (C2L)
LMF2L	Maintenance Facility, Braced Steel Frame (S2L)
LMF3L	Maintenance Facility, Moment Resisting Steel Frame (S1L)
LMF4L	Maintenance Facility, Steel Frame & URM (S5L)
LMF5L	Maintenance Facility, Precast Concrete Tilt-up (PC1)
LMF6L	Maintenance Facility, Reinforced Concrete Frame & URM (C3L)
LMF7L	Maintenance Facility, Wood (W1)
LMF1M	Maintenance Facility, Reinforced Concrete Shear Walls (C2L)
LMF2M	Maintenance Facility, Braced Steel Frame (S2L)
LMF3M	Maintenance Facility, Moment Resisting Steel Frame (S1L)
LMF4M	Maintenance Facility, Steel Frame & URM (S5L)
LMF5M	Maintenance Facility, Precast Concrete Tilt-up (PC1)
LMF6M	Maintenance Facility, Reinforced Concrete Frame & URM (C3L)
LMF7M	Maintenance Facility, Wood (W1)
LMF1H	Maintenance Facility, Reinforced Concrete Shear Walls (C2L)
LMF2H	Maintenance Facility, Braced Steel Frame (S2L)
LMF3H	Maintenance Facility, Moment Resisting Steel Frame (S1L)
LMF4H	Maintenance Facility, Steel Frame & URM (S5L)
LMF5H	Maintenance Facility, Precast Concrete Tilt-up (PC1)
LMF6H	Maintenance Facility, Reinforced Concrete Frame & URM (C3L)
LMF7H	Maintenance Facility, Wood (W1)

H = high, M = moderate, L = low seismic design level.

3.3.4. Bus System

A bus transportation system consists of urban stations, fuel facilities, dispatch facilities and maintenance facilities. The inventory data required for bus systems analysis include the geographical location, classification, and replacement cost of bus system facilities.

3.3.4.1. Classification

Table 3.9 describes the various classes of bus system components. For more details on how to classify these components refer to section 7.4 of Chapter 7.

Table 3.9: Bus System Classification

Label	Description
	Bus Urban Station
BPT1L	Bus Urban Station, Reinforced Concrete Shear Walls (C2L)
BPT2L	Bus Urban Station, Braced Steel Frame (S2L)
BPT3L	Bus Urban Station, Moment Resisting Steel Frame (S1L)
BPT4L	Bus Urban Station, Steel Frame & URM (S5L)
BPT5L	Bus Urban Station, Precast Concrete Tilt-up (PC1)
BPT6L	Bus Urban Station, Reinforced Concrete Frame & URM (C3L)
BPT7L	Bus Urban Station, Wood (W1)
BPT1M	Bus Urban Station, Reinforced Concrete Shear Walls (C2L)
BPT2M	Bus Urban Station, Braced Steel Frame (S2L)
BPT3M	Bus Urban Station, Moment Resisting Steel Frame (S1L)
BPT4M	Bus Urban Station, Steel Frame & URM (S5L)
BPT5M	Bus Urban Station, Precast Concrete Tilt-up (PC1)
BPT6M	Bus Urban Station, Reinforced Concrete Frame & URM (C3L)
BPT7M	Bus Urban Station, Wood (W1)
BPT1H	Bus Urban Station, Reinforced Concrete Shear Walls (C2L)
BPT2H	Bus Urban Station, Braced Steel Frame (S2L)
BPT3H	Bus Urban Station, Moment Resisting Steel Frame (S1L)
BPT4H	Bus Urban Station, Steel Frame & URM (S5L)
BPT5H	Bus Urban Station, Precast Concrete Tilt-up (PC1)
BPT6H	Bus Urban Station, Reinforced Concrete Frame & URM (C3L)
BPT7H	Bus Urban Station, Wood (W1)
	Bus Fuel Facility
BFF1	Bus Fuel Facility w/ Anchored Tanks, w/ Back-Up (BU) Power
BFF2	Bus Fuel Facility w/ Anchored Tanks, w/o BU Power
BFF3	Bus Fuel Facility w/ Unanchored Tanks, w/ BU Power
BFF4	Bus Fuel Facility w/ Unanchored Tanks, w/o BU Power
BFF5	Bus Fuel Facility w/ Buried Tanks

H = high, M = moderate, L = low seismic design level.

Table 3.9 Cont.: Bus System Classification

Label	Description
	Bus Dispatch Facility
BDF1	Bus Dispatch Facility w/ Anchored Sub-Component, w/ BU Power
BDF2	Bus Dispatch Facility w/ Anchored Sub-Component, w/o BU Power
BDF3	Bus Dispatch Facility w/ Unanchored Sub-Component, w/ BU Power
BDF4	Bus Dispatch Facility w/ Unanchored Sub-Component, w/o BU Power
	Bus Maintenance Facility
BMF1L	Bus Maintenance Facilities, Reinforced Concrete Shear Walls (C2L)
BMF2L	Bus Maintenance Facilities, Braced Steel Frame (S2L)
BMF3L	Bus Maintenance Facilities, Moment Resisting Steel Frame (S1L)
BMF4L	Bus Maintenance Facilities, Steel Frame & URM (S5L)
BMF5L	Bus Maintenance Facilities, Precast Concrete Tilt-up (PC1)
BMF6L	Bus Maintenance Facilities, Reinforced Concrete Frame & URM (C3L)
BMF7L	Bus Maintenance Facilities, Wood (W1)
BMF1M	Bus Maintenance Facilities, Reinforced Concrete Shear Walls (C2L)
BMF2M	Bus Maintenance Facilities, Braced Steel Frame (S2L)
BMF3M	Bus Maintenance Facilities, Moment Resisting Steel Frame (S1L)
BMF4M	Bus Maintenance Facilities, Steel Frame & URM (S5L)
BMF5M	Bus Maintenance Facilities, Precast Concrete Tilt-up (PC1)
BMF6M	Bus Maintenance Facilities, Reinforced Concrete Frame & URM (C3L)
BMF7M	Bus Maintenance Facilities, Wood (W1)
BMF1H	Bus Maintenance Facilities, Reinforced Concrete Shear Walls (C2L)
BMF2H	Bus Maintenance Facilities, Braced Steel Frame (S2L)
BMF3H	Bus Maintenance Facilities, Moment Resisting Steel Frame (S1L)
BMF4H	Bus Maintenance Facilities, Steel Frame & URM (S5L)
BMF5H	Bus Maintenance Facilities, Precast Concrete Tilt-up (PC1)
BMF6H	Bus Maintenance Facilities, Reinforced Concrete Frame & URM (C3L)
BMF7H	Bus Maintenance Facilities, Wood (W1)

H = high, M = moderate, L = low seismic design level.

3.3.4.2.Ports and Harbors

Port and harbor transportation systems consist of waterfront structures, cranes/cargo handling equipment, warehouses and fuel facilities. The inventory data required for ports and harbors analysis include the geographical location, classification and replacement cost of the port and harbor system facilities.

3.3.4.3. Classification

Table 3.10 describes the various classes of port and harbor transportation system components. For more details on how to classify these components refer to section 7.5 of Chapter 7.

Table 3.10: Port and Harbor System Classification

Label	Description
	Waterfront Structures
PWS1	Waterfront Structures
	Cranes/Cargo Handling Equipment
PEQ1	Stationary Port Handling Equipment
PEQ2	Rail Mounted Port Handling Equipment
	Warehouses
PWH1L	Port Warehouses, Reinforced Concrete Shear Walls (C2L)
PWH2L	Port Warehouses, Braced Steel Frame (S2L)
PWH3L	Port Warehouses, Moment Resisting Steel Frame (S1L)
PWH4L	Port Warehouses, Steel Frame & URM (S5L)
PWH5L	Port Warehouses, Precast Concrete Tilt-Up (PC1)
PWH6L	Port Warehouses, Reinforced Concrete Frame & URM (C3L)
PWH7L	Port Warehouses, Wood (W1)
PWH1M	Port Warehouses, Reinforced Concrete Shear Walls (C2L)
PWH2M	Port Warehouses, Braced Steel Frame (S2L)
PWH3M	Port Warehouses, Moment Resisting Steel Frame (S1L)
PWH4M	Port Warehouses, Steel Frame & URM (S5L)
PWH5M	Port Warehouses, Precast Concrete Tilt-Up (PC1)
PWH6M	Port Warehouses, Reinforced Concrete Frame & URM (C3L)
PWH7M	Port Warehouses, Wood (W1)
PWH1H	Port Warehouses, Reinforced Concrete Shear Walls (C2L)
PWH2H	Port Warehouses, Braced Steel Frame (S2L)
PWH3H	Port Warehouses, Moment Resisting Steel Frame (S1L)
PWH4H	Port Warehouses, Steel Frame & URM (S5L)
PWH5H	Port Warehouses, Precast Concrete Tilt-Up (PC1)
PWH6H	Port Warehouses, Reinforced Concrete Frame & URM (C3L)
PWH7H	Port Warehouses, Wood (W1)
	Fuel Facility
PFF1	Port Fuel Facility w/ Anchored Tanks, w/ Back-Up (BU) Power
PFF2	Port Fuel Facility w/ Anchored Tanks, w/o BU Power
PFF3	Port Fuel Facility w/ Unanchored Tanks, w/ BU Power
PFF4	Port Fuel Facility w/ Unanchored Tanks, w/o BU Power
PFF5	Port Fuel Facility w/ Buried Tanks

H = high, M = moderate, L = low seismic design level.

3.3.4.4.Ferry

A ferry transportation system consists of waterfront structures, passenger terminals, fuel facilities, dispatch facilities and maintenance facilities. The inventory data required for ferry systems analysis include the geographical location, classification and replacement cost of ferry system facilities.

3.3.4.5. Classification

Table 3.11 describes the various classes of ferry transportation system components. For more details on how to classify these components refer to section 7.6 of Chapter 7.

Table 3.11: Ferry System Classification

Label	Description
	Water Front Structures
FWS1	Ferry Waterfront Structures
	Ferry Passenger Terminals
FPT1L	Passenger Terminals, Reinforced Concrete Shear Walls (C2L)
FPT2L	Passenger Terminals, Braced Steel Frame (S2L)
FPT3L	Passenger Terminals, Moment Resisting Steel Frame (S1L)
FPT4L	Passenger Terminals, Steel Frame & URM (S5L)
FPT5L	Passenger Terminals, Precast Concrete Tilt-up (PC1)
FPT6L	Passenger Terminals, Reinforced Concrete Frame & URM (C3L)
FPT7L	Passenger Terminals, Wood (W1)
FPT1M	Passenger Terminals, Reinforced Concrete Shear Walls (C2L)
FPT2M	Passenger Terminals, Braced Steel Frame (S2L)
FPT3M	Passenger Terminals, Moment Resisting Steel Frame (S1L)
FPT4M	Passenger Terminals, Steel Frame & URM (S5L)
FPT5M	Passenger Terminals, Precast Concrete Tilt-up (PC1)
FPT6M	Passenger Terminals, Reinforced Concrete Frame & URM (C3L)
FPT7M	Passenger Terminals, Wood (W1)
FPT1H	Passenger Terminals, Reinforced Concrete Shear Walls (C2L)
FPT2H	Passenger Terminals, Braced Steel Frame (S2L)
FPT3H	Passenger Terminals, Moment Resisting Steel Frame (S1L)
FPT4H	Passenger Terminals, Steel Frame & URM (S5L)
FPT5H	Passenger Terminals, Precast Concrete Tilt-up (PC1)
FPT6H	Passenger Terminals, Reinforced Concrete Frame & URM (C3L)
FPT7H	Passenger Terminals, Wood (W1)
	Ferry Fuel Facility
FFF1	Ferry Fuel Facility w/ Anchored Tanks, w/ Back-Up (BU) Power
FFF2	Ferry Fuel Facility w/ Anchored Tanks, w/o BU Power
FFF3	Ferry Fuel Facility w/ Unanchored Tanks, w/ BU Power
FFF4	Ferry Fuel Facility w/ Unanchored Tanks, w/o BU Power
FFF5	Ferry Fuel Facility w/ Buried Tanks
	Ferry Dispatch Facility
FDF1	Ferry Dispatch Facility w/ Anchored Sub-Comp., w/ BU Power
FDF2	Ferry Dispatch Facility w/ Anchored Sub-Comp., w/o BU Power
FDF3	Ferry Dispatch Facility w/ Unanchored Sub-Comp., w/ BU Power
FDF4	Ferry Dispatch Facility w/ Unanchored Sub-Comp., w/o BU Power

H = high, M = moderate, L = low seismic design level.

Table 3.11 Cont.: Ferry System Classification

Label	Description
	Ferry Maintenance Facility
FMF1L	Piers and Dock Facilities, Reinforced Concrete Shear Walls (C2L)
FMF2L	Piers and Dock Facilities, Braced Steel Frame (S2L)
FMF3L	Piers and Dock Facilities, Moment Resisting Steel Frame (S1L)
FMF4L	Piers and Dock Facilities, Steel Frame & URM (S5L)
FMF5L	Piers and Dock Facilities, Precast Concrete Tilt-up (PC1)
FMF6L	Piers and Dock Facilities, Reinforced Concrete Frame & URM (C3L)
FMF7L	Piers and Dock Facilities, Wood (W1)
FMF1M	Piers and Dock Facilities, Reinforced Concrete Shear Walls (C2L)
FMF2M	Piers and Dock Facilities, Braced Steel Frame (S2L)
FMF3M	Piers and Dock Facilities, Moment Resisting Steel Frame (S1L)
FMF4M	Piers and Dock Facilities, Steel Frame & URM (S5L)
FMF5M	Piers and Dock Facilities, Precast Concrete Tilt-up (PC1)
FMF6M	Piers and Dock Facilities, Reinforced Concrete Frame & URM (C3L)
FMF7M	Piers and Dock Facilities, Wood (W1)
FMF1H	Piers and Dock Facilities, Reinforced Concrete Shear Walls (C2L)
FMF2H	Piers and Dock Facilities, Braced Steel Frame (S2L)
FMF3H	Piers and Dock Facilities, Moment Resisting Steel Frame (S1L)
FMF4H	Piers and Dock Facilities, Steel Frame & URM (S5L)
FMF5H	Piers and Dock Facilities, Precast Concrete Tilt-up (PC1)
FMF6H	Piers and Dock Facilities, Reinforced Concrete Frame & URM (C3L)
FMF7H	Piers and Dock Facilities, Wood (W1)

H = high, M = moderate, L = low seismic design level.

3.3.5. Airports

An airport transportation system consists of control towers, runways, terminal buildings, parking structures, fuel facilities, and maintenance and hangar facilities. The inventory data required for airports analysis include the geographical location, classification and replacement cost of airport facilities.

3.3.5.1. Classification

Table 3.12 describes the various classes of airport system components. For more details on how to classify these components refer to section 7.7 of Chapter 7.

Table 3.12: Airport System Classification

Label	Description
	Airport Control Towers
ACT1L	Airport Control Tower, Reinforced Concrete Shear Walls (C2L)
ACT2L	Airport Control Tower, Braced Steel Frame (S2L)
ACT3L	Airport Control Tower, Moment Resisting Steel Frame (S1L)
ACT4L	Airport Control Tower, Steel Frame & URM (S5L)
ACT5L	Airport Control Tower, Precast Concrete Tilt-up (PC1)
ACT6L	Airport Control Tower, Reinforced Concrete Frame & URM (C3L)
ACT7L	Airport Control Tower, Wood (W1)
ACT1M	Airport Control Tower, Reinforced Concrete Shear Walls (C2L)
ACT2M	Airport Control Tower, Braced Steel Frame (S2L)
ACT3M	Airport Control Tower, Moment Resisting Steel Frame (S1L)
ACT4M	Airport Control Tower, Steel Frame & URM (S5L)
ACT5M	Airport Control Tower, Precast Concrete Tilt-up (PC1)
ACT6M	Airport Control Tower, Reinforced Concrete Frame & URM (C3L)
ACT7M	Airport Control Tower, Wood (W1)
ACT1H	Airport Control Tower, Reinforced Concrete Shear Walls (C2L)
ACT2H	Airport Control Tower, Braced Steel Frame (S2L)
ACT3H	Airport Control Tower, Moment Resisting Steel Frame (S1L)
ACT4H	Airport Control Tower, Steel Frame & URM (S5L)
ACT5H	Airport Control Tower, Precast Concrete Tilt-up (PC1)
ACT6H	Airport Control Tower, Reinforced Concrete Frame & URM (C3L)
ACT7H	Airport Control Tower, Wood (W1)
	Airport Terminal Buildings
ATB1L	Airport Terminal Building, Reinforced Concrete Shear Walls (C2L)
ATB2L	Airport Terminal Building, Braced Steel Frame (S2L)
ATB3L	Airport Terminal Building, Moment Resisting Steel Frame (S1L)
ATB4L	Airport Terminal Building, Steel Frame & URM (S5L)
ATB5L	Airport Terminal Building, Precast Concrete Tilt-up (PC1)
ATB6L	Airport Terminal Building, Reinforced Concrete Frame & URM (C3L)
ATB7L	Airport Terminal Building, Wood (W1)
ATB1M	Airport Terminal Building, Reinforced Concrete Shear Walls (C2L)
ATB2M	Airport Terminal Building, Braced Steel Frame (S2L)
ATB3M	Airport Terminal Building, Moment Resisting Steel Frame (S1L)
ATB4M	Airport Terminal Building, Steel Frame & URM (S5L)
ATB5M	Airport Terminal Building, Precast Concrete Tilt-up (PC1)
ATB6M	Airport Terminal Building, Wood (W1)
ATB7M	Airport Terminal Building, Reinforced Concrete Frame & URM (C3L)
ATB1H	Airport Terminal Building, Reinforced Concrete Shear Walls (C2L)
ATB2H	Airport Terminal Building, Braced Steel Frame (S2L)
ATB3H	Airport Terminal Building, Moment Resisting Steel Frame (S1L)
ATB4H	Airport Terminal Building, Steel Frame & URM (S5L)
ATB5H	Airport Terminal Building, Precast Concrete Tilt-up (PC1)
ATB6H	Airport Terminal Building, Wood (W1)
ATB7H	Airport Terminal Building, Reinforced Concrete Frame & URM (C3L)
ATBU1	Airport Terminal Building w/Unknown Structure Type

H = high, M = moderate, L = low seismic design level.

Table 3.12 Cont.: Airport System Classification

Label	Description
	Airport Parking Structures
APS1L	Airport Parking Structure, Reinforced Concrete Shear Walls (C2L)
APS2L	Airport Parking Structure, Braced Steel Frame (S2L)
APS3L	Airport Parking Structure, Moment Resisting Steel Frame (S1L)
APS4L	Airport Parking Structure, Steel Frame & URM (S5L)
APS5L	Airport Parking Structure, Precast Concrete Tilt-up (PC1)
APS6L	Airport Parking Structure, Reinforced Concrete Frame & URM (C3L)
APS1M	Airport Parking Structure, Reinforced Concrete Shear Walls (C2L)
APS2M	Airport Parking Structure, Braced Steel Frame (S2L)
APS3M	Airport Parking Structure, Moment Resisting Steel Frame (S1L)
APS4M	Airport Parking Structure, Steel Frame & URM (S5L)
APS5M	Airport Parking Structure, Precast Concrete Tilt-up (PC1)
APS6M	Airport Parking Structure, Reinforced Concrete Frame & URM (C3L)
APS1H	Airport Parking Structure, Reinforced Concrete Shear Walls (C2L)
APS2H	Airport Parking Structure, Braced Steel Frame (S2L)
APS3H	Airport Parking Structure, Moment Resisting Steel Frame (S1L)
APS4H	Airport Parking Structure, Steel Frame & URM (S5L)
APS5H	Airport Parking Structure, Precast Concrete Tilt-up (PC1)
APS6H	Airport Parking Structure, Reinforced Concrete Frame & URM (C3L)
	Fuel Facilities
AFF1	Airport Fuel Facility w/ Anchored Tanks, w/ Back-Up (BU) Power
AFF2	Airport Fuel Facility w/ Anchored Tanks, w/o BU Power
AFF3	Airport Fuel Facility w/ Unanchored Tanks, w/ Back-Up (BU) Power
AFF4	Airport Fuel Facility w/ Unanchored Tanks, w/o BU Power
AFF5	Airport Fuel Facility w/ Buried Tanks
	Airport Maintenance & Hangar Facility
AMF1L	Airport Maintenance & Hangar Facility, Reinforced Concrete Shear Walls (C2L)
AMF2L	Airport Maintenance & Hangar Facility, Braced Steel Frame (S2L)
AMF3L	Airport Maintenance & Hangar Facility, Moment Resisting Steel Frame (S1L)
AMF4L	Airport Maintenance & Hangar Facility, Steel Frame & URM (S5L)
AMF5L	Airport Maintenance & Hangar Facility, Precast Concrete Tilt-up (PC1)
AMF6L	Airport Maintenance & Hangar Facility, Reinforced Concrete Frame & URM (C3L)
AMF7L	Airport Maintenance & Hangar Facility, Wood (W1)
AMF1M	Airport Maintenance & Hangar Facility, Reinforced Concrete Shear Walls (C2L)
AMF2M	Airport Maintenance & Hangar Facility, Braced Steel Frame (S2L)
AMF3M	Airport Maintenance & Hangar Facility, Moment Resisting Steel Frame (S1L)
AMF4M	Airport Maintenance & Hangar Facility, Steel Frame & URM (S5L)
AMF5M	Airport Maintenance & Hangar Facility, Precast Concrete Tilt-up (PC1)
AMF6M	Airport Maintenance & Hangar Facility, Reinforced Concrete Frame & URM (C3L)
AMF7M	Airport Maintenance & Hangar Facility, Wood (W1)
AMF1H	Airport Maintenance & Hangar Facility, Reinforced Concrete Shear Walls (C2L)
AMF2H	Airport Maintenance & Hangar Facility, Braced Steel Frame (S2L)
AMF3H	Airport Maintenance & Hangar Facility, Moment Resisting Steel Frame (S1L)
AMF4H	Airport Maintenance & Hangar Facility, Steel Frame & URM (S5L)
AMF5H	Airport Maintenance & Hangar Facility, Precast Concrete Tilt-up (PC1)
AMF6H	Airport Maintenance & Hangar Facility, Reinforced Concrete Frame & URM (C3L)
AMF7H	Airport Maintenance & Hangar Facility, Wood (W1)

Table 3.12 Cont.: Airport System Classification

Label	Description
	Airport Runways
ARW1	Airport Runway
	Airport Facilities - Others
AFO1	Gliderport, Seaport, Stolport, Ultralight or Baloonport Facilities
AFH1	Heliport Facilities

3.4. Direct Damage Data - Lifeline Utility Systems

Lifeline utility systems include potable water, waste water, oil, natural gas, electric power and communication systems. This section describes the classification of lifeline utility system and their components, and data required to provide damage and loss estimates.

3.4.1. Potable Water System

A potable water system consists of pipelines, water treatment plants, wells, storage tanks and pumping stations. The inventory data required for potable water systems analysis include the geographical location and classification of system components. The analysis also requires the replacement cost for facilities and the repair cost for pipelines.

3.4.1.1. Classification

Table 3.13 describes the various classes of potable water system components. For more details on how to classify these components refer to section 8.1 of Chapter 8.

Table 3.13: Potable Water System Classification

Label	Description
	Pipelines
PWP1	Brittle Pipe
PWP2	Ductile Pipe
	Water Treatment Plants
PWT1	Small WTP with Anchored Components < 50 MGD
PWT2	Small WTP with Unanchored Components < 50 MGD
PWT3	Medium WTP with Anchored Components 50-200 MGD
PWT4	Medium WTP with Unanchored Components 50-200 MGD
PWT5	Large WTP with Anchored Components > 200 MGD
PWT6	Large WTP with Unanchored Components > 200 MGD
	Wells
PWE1	Wells
	Water Storage Tanks (Typically, 0.5 MGD to 2 MGD)
PST1	On Ground Anchored Concrete Tank
PST2	On Ground Unanchored Concrete Tank
PST3	On Ground Anchored Steel Tank
PST4	On Ground Unanchored Steel Tank
PST5	Above Ground Steel Tank
PST6	On Ground Wood Tank
PST7	Buried Concrete Tank
	Pumping Plants
PPP1	Small Pumping Plant with Anchored Equipment < 10 MGD
PPP2	Small Pumping Plant with Unanchored Equipment < 10 MGD
PPP3	Medium/Large Pumping Plant with Anchored Equipment ≥ 10 MGD
PPP4	Medium/Large Pumping Plant with Unanchored Equipment ≥10 MGD

3.4.2. Waste Water

A waste water system consists of pipelines, waste water treatment plants and lift stations. The inventory data required for waste water systems analysis include the geographical location and classification of system components. The analysis also requires the replacement cost for facilities and the repair cost for pipelines.

3.4.2.1. Classification

Table 3.14 describes the various classes of waste water system components. For more details on how to classify these components refer to section 8.2 of Chapter 8.

Table 3.14: Waste Water System Classification

Label	Description
	Buried Pipelines
WWP1	Brittle Pipe
WWP2	Ductile Pipe
	Waste Water Treatment Plants
WWT1	Small WWTP with Anchored Components < 50 MGD
WWT2	Small WWTP with Unanchored Components < 50 MGD
WWT3	Medium WWTP with Anchored Components 50-200 MGD
WWT4	Medium WWTP with Unanchored Components 50-200 MGD
WWT5	Large WWTP with Anchored Components > 200 MGD
WWT6	Large WWTP with Unanchored Components > 200 MGD
	Lift Stations
WLS1	Small Lift Stations with Anchored Components < 10 MGD
WLS2	Small Lift Stations with Unanchored Components < 10 MGD
WLS3	Medium/Large Lift Stations with Anchored Components ≥ 10 MGD
WLS4	Medium/Large Lift Stations with Unanchored Components ≥ 10 MGD

3.4.3. Oil Systems

An oil system consists of pipelines, refineries, pumping plants and tank farms. The inventory data required for oil systems analysis include the geographical location and classification of system components. The analysis also requires the replacement cost for facilities and the repair cost for pipelines.

3.4.3.1. Classification

Table 3.15 describes the various classes of oil system components. For more details on how to classify these components refer to section 8.3 of Chapter 8.

Table 3.15: Oil System Classification

Label	Description
	Pipelines
OIP1	Welded Steel Pipe with Gas Welded Joints
OIP2	Welded Steel Pipe with Arc Welded Joints
	Refineries
ORF1	Small Refinery with Anchored Equipment < 100,000 lb./day
ORF2	Small Refinery with Unanchored Equipment < 100,000 lb./day
ORF3	Medium/Large Refinery with Anchored Equipment ≥ 100,000 lb./day
ORF4	Medium/Large Refinery with Unanchored Equipment ≥100,000 lb./day
	Pumping Plants
OPP1	Pumping Plant with Anchored Equipment
OPP2	Pumping Plant with Unanchored Equipment
	Tank Farms
OTF1	Tank Farms with Anchored Tanks
OTF2	Tank Farms with Unanchored Tanks

3.4.4. Natural Gas Systems

A natural gas system consists of pipelines and compressor stations. The inventory data required for natural gas systems analysis include the geographical location and classification of system components. The analysis also requires the replacement cost for facilities and the repair cost for pipelines.

3.4.4.1. Classification

Table 3.16 describes the various classes of natural gas system components. For more details on how to classify these components refer to section 8.4 of Chapter 8.

Table 3.16: Natural Gas System Classification

Label	Description
	Buried Pipelines
NGP1	Welded Steel Pipe with Gas Welded Joints
NGP2	Welded Steel Pipe with Arc Welded Joints
	Compressor Stations
NGC1	Compressor Stations with Anchored Components
NGC2	Compressor Stations with Unanchored Components

3.4.5. Electric Power

An electric power system consists of substations, distribution circuits, generation plants and transmission towers. The inventory data required for electric power systems analysis include the geographical location, classification and replacement cost of the facilities.

3.4.5.1. Classification

Table 3.17 describes the various classes of electric power system components. For more details on how to classify these components refer to section 8.5 of Chapter 8.

Table 3.17: Electric Power System Classification

Label	Description
	Transmission Substations
ESS1	Low Voltage (115 KV) Substation with Anchored Components
ESS2	Low Voltage (115 KV) Substation with Unanchored Components
ESS3	Medium Voltage (230 KV) Substation with Anchored Components
ESS4	Medium Voltage (230 KV) Substation with Unanchored Components
ESS5	High Voltage (500 KV) Substation with Anchored Components
ESS6	High Voltage (500 KV) Substation with Unanchored Components
	Distribution Circuits
EDC1	Distribution Circuits with Seismically Designed Components
EDC2	Distribution Circuits with Standard Components
	Generation Plants
EPP1	Small Power Plants with Anchored Components < 100 MW
EPP2	Small Power Plants with Unanchored Components < 100 MW
EPP3	Medium/Large Power Plants with Anchored Components ≥ 100 MW
EPP4	Medium/Large Power Plants with Unanchored Components ≥100 MW

3.4.6. Communication

In the loss estimation methodology, a communication system consists of telephone central offices. The inventory data required for communication systems analysis include the geographical location and the classification. The analysis also requires the replacement cost of the facilities.

3.4.6.1. Classification

Table 3.18 describes the various classes of central offices. For more details on how to classify these components refer to section 8.6 of Chapter 8.

Table 3.18: Communication Classification

Label	Description
	Central Offices
CCO1	Central Offices with Anchored Components , w/ Back-Up (BU) Power
CCO2	Central Offices with Anchored Components , w/o BU Power
CCO3	Central Offices with Unanchored Components , w/ BU Power
CCO4	Central Offices with Unanchored Components , w/o BU Power
	Stations or Transmitters
CBR1	AM or FM radio stations or transmitters
CBT1	TV stations or transmitters
CBW1	Weather stations or transmitters
CBO1	Other stations or transmitters

3.5. Hazardous Materials Facilities

Hazardous material facilities contain substances that can pose significant hazards because of their toxicity, radioactivity, flammability, explosiveness or reactivity. Significant casualties or property damage could occur form a small number or even a single hazardous materials release induced by an earthquake, and the consequence of an earthquake-caused release can vary greatly according to the type and quantity of substance released, meteorological conditions and timeliness and effectiveness of emergency response. Similarly to the case of critical faculties with a potential for high loss, such as large dams, the methodology does not attempt to estimate losses caused by earthquake which caused hazardous materials releases. Thus, the hazardous materials module of **HAZUS** is limited to inventory data concerning the location and nature of hazardous materials located at various sites. Section 11.1.2 describes the scheme used to define the degree of danger of hazardous materials.

3.6. Direct Economic and Social Loss

In this section, information related to inventory data required to determine direct economic and social loss is presented. The two main databases used to determine direct economic and social loss are demographic and building square footage databases.

3.6.1. Demographics Data

The census data are used to estimate direct social loss due to displaced households, casualties due to earthquakes, and the estimation quality of building space (square footage) for certain occupancy classes. The Census Bureau collects and publishes statistics about the people of the United States based on the constitutionally required census every 10 years, which is taken in the years ending in "0" (e.g., 1990). The

Bureau's population census data describes the characteristics of the population including age, income, housing and ethnic origin.

The census data were processed for all of the census tracts in the United States, and 29 fields of direct importance to the methodology were extracted and stored. These fields are shown in Table 3.19 and are supplied as default information with the methodology. The population information is aggregated to a census tract level. Census tracts are divisions of land that are designed to contain 2500-8000 inhabitants with relatively homogeneous population characteristics, economic status and living conditions. Census tract divisions and boundaries change only once every ten years. Census tract boundaries never cross county boundaries, and all the area within a county is contained within one or more census tracts. This characteristic allows for a unique division of land from country to state to county to census tract. Each Census tract is identified by a unique 11 digit number. The first two digits represent the tract's state, the next three digits represent the tract's county, while the last 6 digits identify the tract within the county. For example, a census tract numbered 10050505800 would be located in Delaware (10) in Sussex County (050).

Table 3.19: Demographics Data Fields and Usage

	Module Usage			
Description of Field	**Shelter**	**Casualty**	**Occupancy Class**	**Lifelines**
Total Population in Census Tract	*	*		*
Total Household in Census Tract	*			*
Total Number of People in General Quarter	*			
Total Number of People < 16 years old	*	*		
Total Number of People 16-65 years old	*			
Total Number of People > 65 years old	*			
Total Number of People - White	*			
Total Number of People - Black	*			
Total Number of People - Native American	*			
Total Number of People - Asian	*			
Total Number of People - Hispanic	*			
Total # of Households with Income < $10,000	*			
Total # of Households with Income $10 - $15K	*			
Total # of Households with Income $15 - $25K	*			
Total # of Households with Income $25 - $35K	*			
Total # of Households with Income > $35,000	*			
Total in Residential Property during Day		*		
Total in Residential Property at Night		*		
Total Working Population in Commercial Industry		*		
Total Working Population in Industrial Industry		*		
Total Commuting at 5 PM		*		
Total Owner Occupied - Single Household Units	*		*	
Total Owner Occupied - Multi-Household Units	*		*	
Total Owner Occupied - Multi-Household Structure	*		*	
Total Owner Occupied - Mobile Homes	*		*	
Total Renter Occupied - Single Household Units	*		*	
Total Renter Occupied - Multi-Household Units	*		*	
Total Renter Occupied - Multi-Household Structure	*		*	
Total Renter Occupied - Mobile Homes	*		*	
Total Vacant - Single Household Units			*	
Total Vacant - Multi-Household Units			*	
Total Vacant - Multi-Household Structure			*	
Total Vacant - Mobile Homes			*	
Structure Age <40 years			*	
Structure Age >40 years			*	

3.6.2. Default Occupancy Class Square Foot Inventory

The default square footage estimates for occupancy classes RES1, 2,3,5, are based on census data on the number of dwelling units or the number of people for that occupancy class. Table 3.20 provides the conversion factors for these occupancy classes. These conversion factors are obtained from expert opinion and modifications to ATC-13 values. The conversion factors were also calibrated against tax assessors data for region-specific counties. The square foot estimates are calculated using the following expression:

$$SFI = UD * CF \tag{3-1}$$

where,

SFI = building square footage for an occupancy class
UD = unit of data for that occupancy class
CF = conversion factor for that occupancy class (Table 3.20)

The building square footage estimates for the remaining occupancy classes were obtained using a building square footage inventory database purchased from the Dun and Bradstreet Company (D&B). The square footage information was classified based on Standard Industrial Code (SIC) and provided at a census tract resolution. The SIC codes were mapped to NIBS occupancy classes using the mapping scheme provided in Table 3.20. There is no default information for occupancy class COM10.

3.7. Indirect Economic Data

The indirect economic data refers to the post-earthquake change in the demand and supply of products, change in employment and change in tax revenues. The user can specify the levels of potential increase in imports and exports, supply and product inventories and unemployment rates.

Table 3.20: Mapping of Standard Industrial Codes, Conversion Factors to Estimate Occupancy Square Footage and Square Footage Per Occupancy Class

Label	Occupancy Class	Source of Data: Census – Unit of Data	Source of Data: Census – Conversion Factor	Source of Data: Dun and Bradstreet – SIC Code	Square Footage Per Occupancy Type
	Residential				
RES1	Single Family Dwelling	# of Units	1500 sq. ft./unit		1,500
RES2	Mobile Home	# of Units	1000 sq. ft./unit		1,000
RES3	Multi Family Dwelling	# of Units	1000 sq. ft./unit		16,000
RES4	Temporary Lodging			70	50,000
RES5	Institutional Dormitory	# in Group Quarters	700 sq. ft./person		30,000
RES6	Nursing Home			8051, 8052, 8059	45,000
	Commercial				
COM1	Retail Trade			52, 53, 54, 55, 56, 57, 59	14,000
COM2	Wholesale Trade			42, 50, 51	35,000
COM3	Personal/Repair Services			72,75,76,83,88	12,000
COM4	Prof./Technical Services			40, 41, 44, 45, 46, 47, 49, 61, 62, 63, 64, 65, 67, 73, 78 (except 7832), 81, 87, 89	35,000
COM5	Banks			60	22,000
COM6	Hospital			8062, 8063, 8069	95,000
COM7	Medical Office/Clinic			80 (except 8051, 8052, 8059, 8062, 8063, 8069)	12,000
COM8	Entertainment & Rec.			48, 58, 79, (except 7911), 84	13,000
COM9	Theaters			7832, 7911	17,000
COM10	Parking				9,000
	Industrial				
IND1	Heavy			22, 24, 26, 32, 34, 35 (except 3571, 3572), 37	50,000
IND2	Light			23, 25, 27, 30, 31, 36 (except 3671, 3672, 3674), 38, 39	20,000
IND3	Food/Drugs/Chemicals			20, 21, 28, 29	21,000
IND4	Metals/Minerals Processing.			10, 12, 13, 14, 33	16,000
IND5	High Technology			3571, 3572, 3671, 3672, 3674	17,000
IND6	Construction			15, 16, 17	19,000
	Agriculture				
AGR1	Agriculture			01, 02, 07, 08, 09	14,000
	Religion/Non/Profit				
REL1	Church/ N.P. Offices			86	15,000
	Government				
GOV1	General Services			43, 91, 92 (except 9221, 9224), 93, 94, 95, 96, 97	25,000
GOV2	Emergency Response			9221, 9224	10,000
	Education				
EDU1	Schools			82 (except 8221, 8222)	20,000
EDU2	Colleges/Universities			8221, 8222	25,000

3.8. References

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APPENDIX 3A
General Building Stock

Table 3A.1: Distribution Percentage of Floor Area for Specific Occupancy Classes within each General Occupancy Class♣

			General Occupancy Class						
Specific Occupancy Class			RES	COM	IND	AGR	REL	GOV	EDU
No.	Label	Occupancy Class	1	2	3	4	5	6	7
1	RES1	Single Family Dwelling	♣						
2	RES2	Mobile Home	♣						
3	RES3	Multi Family Dwelling	♣						
4	RES4	Temporary Lodging	♣						
5	RES5	Institutional Dormitory	♣						
6	RES6	Nursing Home	♣						
7	COM1	Retail Trade		♣					
8	COM2	Wholesale Trade		♣					
9	COM3	Personal and Repair Services		♣					
10	COM4	Professional/Technical		♣					
11	COM5	Banks		♣					
12	COM6	Hospital		♣					
13	COM7	Medical Office/Clinic		♣					
14	COM8	Entertainment & Recreation		♣					
15	COM9	Theaters		♣					
16	COM10	Parking		♣					
17	IND1	Heavy			♣				
18	IND2	Light			♣				
19	IND3	Food/Drugs/Chemicals			♣				
20	IND4	Metals/Minerals Processing			♣				
21	IND5	High Technology			♣				
22	IND6	Construction			♣				
23	AGR1	Agriculture				100			
24	REL1	Church					100		
25	GOV1	General Services						♣	
26	GOV2	Emergency Response						♣	
27	EDU1	Schools							♣
28	EDU2	Colleges/Universities							♣

♣ The relative distribution varies by census tract and is computed directly from the specific occupancy class square footage inventory. For Agriculture (AGR) and Religion (REL) there is only one specific occupancy class, therefore the distribution is always 100%.

Table 3A.2: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, Pre-1950, West Coast* (after ATC-13, 1985)

No.	Specific Occup. Class	Model Building Type															
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34	36
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML	MH
1	RES1	For State-Specific "Res1" Distribution, Refer to Table 3A.17															
2	RES2																100
3	RES3	73		1	1	1		6		3	3			1		9	2
4	RES4	34		2	1	2	1	19		16	3			4		18	
5	RES5	20		5	1		1			28	18			6		21	
6	RES6	45				10		5		10				20		10	
7	COM1		22	2		6	3	20		17	1			6		23	
8	COM2		8	3		4	2	41		18	1	3		5	2	13	
9	COM3		28	1	1	3		18		7		1		8		33	
10	COM4		27	2	1	3		19		15				7		26	
11	COM5		27	2	1	3		19		15				7		26	
12	COM6		8	5	2	11		11		27	2	1		27		6	
13	COM7		25	5	2	10		10		15	2	1		20		10	
14	COM8		8	12	1	2	3	16		27	4			5	1	21	
15	COM9		5	20	7			15		20	3			10		20	
16	COM10				8		8	18		43	7		1	6	3	6	
17	IND1		3	29	13	2	2	15		14	7	1		4	2	8	
18	IND2		4	14	8	22	1	18		16	1	1		2		13	
19	IND3		1	18	8	3	3	20		22		2		3		20	
20	IND4		2	24	12	7	2	13		16		2		2	6	14	
21	IND5			21	5	5		3		35	2	10	2	15		2	
22	IND6		32	3	2	10		18		8	7					13	7
23	AGR1	56		3	2	14		2		9					1	13	
24	REL1	22		8		2		21		15	5			8		19	
25	GOV1		9	8	1	3	4	12		42	4			6		11	
26	GOV2	45					2			37				3		13	
27	EDU1	11		6		3	3	21		21	4			9		22	
28	EDU2	2		5	10		5	15		20				20	5	18	

* Refer to Table 3C.1 for states' classifications.

Table 3A.3: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, 1950-1970 , West Coast* (after ATC-13, 1985)

No.	Specific Occup. Class	Model Building Type															
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34	36
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML	MH
1	RES1	For State-Specific "Res1" Distribution, Refer to Table 3A.18															
2	RES2																100
3	RES3	72		1	2	2		1		6	2			8		3	3
4	RES4	55		1	2	2	2	3		11	2			18	1	3	
5	RES5	39		3	3		1	8		16	6			18	1	5	
6	RES6	70				3	1	1		5				20			
7	COM1		34	3	1	3	2	4		13	5	10	1	18	2	4	
8	COM2		12	4	5	5	3	3		18		22	1	19	4	4	
9	COM3		12	3	5	5	2	3		23	4	12	1	22	4	4	
10	COM4		34	3	3	1	2	3		17	5	3		23	4	2	
11	COM5		34	3	3	1	2	3		17	5	3		23	4	2	
12	COM6		32	5	2	4	3			16	6			28	4		
13	COM7		46	13	1	3	3			9				20		5	
14	COM8		13	17	12	3	3			13	6			30	3		
15	COM9		10	10	30			5		10		5		30			
16	COM10			5	8		20			34			5	20	6	2	
17	IND1		10	25	30	3			7	14				9	2		
18	IND2		8	5	14	17	4			10	5	22	3	12			
19	IND3			14	16	6	1		5	17		28	1	10	2		
20	IND4			18	25	9			11	10		7		15	3		2
21	IND5			4	9	3	2		4	20		35	3	15	4		1
22	IND6		30		1	15				7		4		20	3		20
23	AGR1	51		4	8	12				2		10		11	2		
24	REL1	20		4	1	3	3			24		4		37	4		
25	GOV1		21	6	3	2	2			26	5	4	2	27	2		
26	GOV2	50								13		7		20	10		
27	EDU1	25		3	4	5	4			20		4	2	29	4		
28	EDU2	5		2	12		5			20				50	6		

***** Refer to Table 3C.1 for states' classifications.

Table 3A.4: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, Post-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occup. Class	Model Building Type															
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34	36
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML	MH
1	RES1	For State-Specific "Res1" Distribution, Refer to Table 3A.19															
2	RES2																100
3	RES3	73				2	3			6	1		1	9			5
4	RES4	53		3		2	3		4	13				20	2		
5	RES5	33		3	3		6		5	24				23	3		
6	RES6	70								5		5		20			
7	COM1		26	9	1	2	1		6	10	1	15	5	21	3		
8	COM2		8	4	1	3	4		2	12		41	3	19	3		
9	COM3		13	3	2	2	3		3	13		20	5	34	2		
10	COM4		35	3	2	1	3		4	15		8	3	24	2		
11	COM5		35	3	2	1	3		4	15		8	3	24	2		
12	COM6		31	6	1	1	7		4	13		7		28	2		
13	COM7		47	16			5		4	6		2		20			
14	COM8		4	23	8	1	3		2	15		4	1	32	7		
15	COM9		5	27	20					12		4		27	5		
16	COM10			8	8		6		3	49		3	13	7	3		
17	IND1		11	19	28	3	2		1	9		11	3	11	1		1
18	IND2		3	13	9	6	3			10		41	3	12			
19	IND3		2	15	10	5	3			12		28	7	18			
20	IND4		1	26	18	5	4		1	11	1	12	5	15	1		
21	IND5		1	12	8	2	3			10		38	7	17	1		1
22	IND6		30	4	6	11				8		16	6	14			5
23	AGR1	40		8	11	8				3		11	1	15	1		2
24	REL1	23		12	3	1	6			26		1	3	22	3		
25	GOV1		8	15	4	3	7		2	32			4	16	9		
26	GOV2	40		3	7		23			10			7	3	7		
27	EDU1	24		9	6	1	5		3	16	3	4	3	21	5		
28	EDU2	5		10	10		5			20		5		40	5		

***** Refer to Table 3C.1 for states' classifications.

Table 3A.5: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, Pre-1950, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type										
		4	**7**	**11**	**14**	**17**	**20**	**23**	**27**	**30**	**32**	**35**
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
3	RES3	15	4	5		1	19	25		8		23
4	RES4	18	4	12		1	20	20		8		17
5	RES5	16	1	5			40	20				18
6	RES6	20		5			35	20		10		10
7	COM1	8	6	3			21	34		11	1	16
8	COM2	8					27	53		5		7
9	COM3	18					22	42		5		13
10	COM4	25	7	10		2	22	16		9		9
11	COM5	25	7	10		2	22	16		9		9
12	COM6	18	4	6		1	35	19		8		9
13	COM7	20	5	5			30	20		10		10
14	COM8	25		20			40	5				10
15	COM9	30		10			40	10				10
16	COM10		10	5		2	55	18		3	2	5
17	IND1											
18	IND2			10			5	75				10
19	IND3	32	3	1		1	14	41		3		5
20	IND4	25	3	1			9	52				10
21	IND5	35	10				30	5		20		
22	IND6						20	80				
23	AGR1						25	75				
24	REL1						10	90				
25	GOV1	30	15	5		3	23	10		4		10
26	GOV2											
28	EDU2	10		20			60	3		5		2

***** Refer to Table 3C.1 for states' classifications.

Table 3A.6: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, 1950-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occup. Class	Model Building Type										
		4	**7**	**11**	**14**	**17**	**20**	**23**	**27**	**30**	**32**	**35**
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
3	RES3	10	15	6		4	37		1	21	6	
4	RES4	9	24	9		5	34	1		14	4	
5	RES5	6	1	11		9	45			18	10	
6	RES6	15	10	15		5	25			25	5	
7	COM1	7	25	5		3	31			22	7	
8	COM2	21	3			2	34		1	34	5	
9	COM3	10	3				28			54	5	
10	COM4	17	18	9		9	18		2	23	4	
11	COM5	17	18	9		9	18		2	23	4	
12	COM6	14	10	14		5	23		3	23	8	
13	COM7	15	10	15		5	25			25	5	
14	COM8	5		28			52			10	5	
15	COM9	5		30			50			10	5	
16	COM10	5	8	8		7	39		8	18	7	
17	IND1		10	20			40			20	10	
18	IND2		15	10			50			20	5	
19	IND3	11	4	10		30	20		1	15	9	
20	IND4					100						
21	IND5	10	5	13			32			30	10	
22	IND6											
23	AGR1											
24	REL1						80			10	10	
25	GOV1	15	6	15		11	28		2	18	5	
26	GOV2	5	10	10		5	60				10	
28	EDU2	20		15		5	35			15	10	

***** Refer to Table 3C.1 for states' classifications.

Table 3A.7: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, Post-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type										
		4	**7**	**11**	**14**	**17**	**20**	**23**	**27**	**30**	**32**	**35**
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
3	RES3	9	23	8		10	28		7	12	3	
4	RES4	16	28	8		11	18		3	13	3	
5	RES5	9	10	11		16	34		4	11	5	
6	RES6	25	10	15		10	35			5		
7	COM1	34	9	3		12	17		5	15	5	
8	COM2	20	17			15	10		8	15	15	
9	COM3	11	17	3		10	17		12	17	13	
10	COM4	37	10	12		9	15		3	9	5	
11	COM5	37	10	12		9	15		3	9	5	
12	COM6	25	9	15		10	33		1	6	1	
13	COM7	25	10	15		10	35			5		
14	COM8		10			90						
15	COM9		10			90						
16	COM10	4	8	3		4	66		8	6	1	
17	IND1											
18	IND2											
19	IND3	62	5	1		23	4		1	3	1	
20	IND4	100										
21	IND5	18	14	3		34	13		5	10	3	
22	IND6											
23	AGR1											
24	REL1		5			90					5	
25	GOV1	25	11	15		22	12		4	9	2	
26	GOV2	25	20	35			20					
28	EDU2	20	5	10		25	25			10	5	

* Refer to Table 3C.1 for states' classifications.

Table 3A.8: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, Pre-1950, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
3	RES3	39	1	2		8	24	23	3	
4	RES4	45	3	3		8	20	18	3	
5	RES5	15	5	10			30	40		
10	COM4	47	10	4		1	21	16	1	
11	COM5	47	10	4		1	21	16	1	
12	COM6	56	9	1		1	24	8	1	
13	COM7									
16	COM10									
23	AGR1									
25	GOV1	53	5	5		3	30	3	1	
28	EDU2	5	5	35			40	15		

***** Refer to Table 3C.1 for states' classifications.

Table 3A.9: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, 1950-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
3	RES3	30	21	6		13	24		3	3
4	RES4	48	10	9		12	19		1	1
5	RES5	20	15	25		30	5			5
10	COM4	40	26	18		6	7		1	2
11	COM5	40	26	18		6	7		1	2
12	COM6	35	27	17		4	15		1	1
13	COM7									
16	COM10									
23	AGR1									
25	GOV1	46	13	22		10	8			1
28	EDU2	35	20	20		25				

***** Refer to Table 3C.1 for states' classifications.

Table 3A.10: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, Post-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
3	RES3	44	6	5		18	20		5	2
4	RES4	56	10	6		16	9		2	1
5	RES5	25	18	20		37				
10	COM4	56	10	14		14	5		1	
11	COM5	54	10	15		15	5		1	
12	COM6	45	6	19		13	17			
13	COM7									
16	COM10									
23	AGR1									
25	GOV1	52	14	14		14	6			
28	EDU2	30	10	10		50				

* Refer to Table 3C.1 for states' classifications.

Table 3A.11: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, Mid-West*

No.	Specific Occup. Class	Model Building Type															
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34	36
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML	MH
1	RES1	For State-Specific “Res1” Distribution, Refer to Table 3A.20															
2	RES2																100
3	RES3	75												2		23	
4	RES4	50												3	2	45	
5	RES5	20							4	13	2	22	4	2		33	
6	RES6	90														10	
7	COM1		30	2	4	11	6	7		5		5		2		28	
8	COM2		10	2	4	11	6	7	2	10	2	14	2	2		28	
9	COM3		30	2	4	11	6	7		5		5		2		28	
10	COM4		30	2	4	11	6	7		5		5		2		28	
11	COM5		30	2	4	11	6	7		5		5		2		28	
12	COM6				2	4	2	2	6	21	4	33	6	2		18	
13	COM7		30	2	4	11	6	7		5		5		2		28	
14	COM8		30	2	4	11	6	7		5		5		2		28	
15	COM9			2	6	14	8	10	4	13	2	22	4			15	
16	COM10			2	4	11	6	7	6	21	4	33	6				
17	IND1			5	10	25	13	17	2	7	2	12	2			5	
18	IND2		10	2	4	11	6	7	2	10	2	14	2	3		27	
19	IND3		10	2	4	11	6	7	2	10	2	14	2	3		27	
20	IND4			5	10	25	13	17	2	7	2	12	2			5	
21	IND5		10	2	4	11	6	7	2	10	2	14	2	2		28	
22	IND6		30	2	4	11	6	7		5		5		2		28	
23	AGR1		10	2	4	11	6	7	2	10	2	14	2	2		28	
24	REL1	30			3	5	3	4		5		5		2	2	41	
25	GOV1		15	14	21				7	6		4		3		30	
26	GOV2		14	7	17				4	12					3	43	
27	EDU1		10	5	12				5	7				11		50	
28	EDU2		14	6	12			2	8	11					10	37	

***** Refer to Table 3C.1 for states’ classifications.

Table 3A.12: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, Mid-West*

No.	Specific Occupancy Class	Model Building Type										
		4	7	11	14	17	20	23	27	30	32	35
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
3	RES3		10	7	3	14	39		7		2	18
4	RES4		10	7	3	14	37	2	7		2	18
5	RES5					25	62	2	11			
6	RES6											
7	COM1	3	20	16	6	11	27	2	5		2	8
8	COM2		7	3		14	37	2	7		3	27
9	COM3	3	20	16	6	11	27	2	5		2	8
10	COM4	3	20	16	6	11	27	2	5		2	8
11	COM5	3	20	16	6	11	27	2	5		2	8
12	COM6	3	20	16	6	12	30	2	6			5
13	COM7	3	20	16	6	11	27	2	5		2	8
14	COM8	3	20	16	6	11	27	2	5		2	8
15	COM9											
16	COM10	2	14	10	4	17	43	2	8			
17	IND1											
18	IND2		7	3		14	37	2	7		3	27
19	IND3		7	3		14	37	2	7		3	27
20	IND4											
21	IND5		7	3		14	37	2	7		3	27
22	IND6											
23	AGR1		7	3		14	37	2	7		3	27
24	REL1	3	20	16	6	11	27	2	5		2	8
25	GOV1	20	24			11	9				5	31
26	GOV2											
28	EDU2	7	14			9	13				13	44

* Refer to Table 3C.1 for states' classifications.

Table 3A.13: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, Mid-West*

No.	Specific Occup. Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
3	RES3	3	13	4		16	44	7	7	6
4	RES4	3	13	4		16	44	7	7	6
5	RES5					26	74			
10	COM4	7	29	9		12	32	4	4	3
11	COM5	7	29	9		12	32	4	4	3
12	COM6	7	29	9		13	36	2	2	2
13	COM7	7	29	9		12	32	4	4	3
16	COM10	5	19	6		18	52			
23	AGR1	2	6	2		16	44	11	11	8
25	GOV1									
28	EDU2									

* Refer to Table 3C.1 for states' classifications.

Table 3A.14: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, East Coast*

No.	Specific Occup. Class	Model Building Type															
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34	36
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML	MH
1	RES1	For State-Specific “Res1” Distribution, Refer to Table 3A.21															
2	RES2																100
3	RES3	62			3				2	2				5	4	22	
4	RES4	48		5	4			4	8	4		3	3	3	3	15	
5	RES5	7		7	6			6	17	6	3	8	6	5	5	24	
6	RES6	22		11	8			8	8	3	2	4	3	5	4	22	
7	COM1		14	20	15	5		16	3	2		2		4	2	17	
8	COM2		10	21	15	7		16	3	2		2		3	4	17	
9	COM3		25	7	5	11		5	3	2		2		6	4	30	
10	COM4		26	11	8	4		9	4	2		3		5	4	24	
11	COM5		13	13	9	13		10	5	3		2	2	5	3	22	
12	COM6		2	22	15			18	10	4	2	5	4	3	2	13	
13	COM7		24	10	7	15		8	3	2		3		4	4	20	
14	COM8		19	19	13	6		15	3	2		2		3	3	15	
15	COM9		5	20	13	12	2	16	7	2		3	3	3	2	12	
16	COM10			10	7			8	30	11	6	14	12			2	
17	IND1		5	22	15	4	2	17	7	3		3	3	3	3	13	
18	IND2		10	15	9	15		11	5	3		2	2	4	5	19	
19	IND3		7	25	18	3		19	4	2		2	2	3	2	13	
20	IND4		7	26	19	3		20	3	2		2		2	3	13	
21	IND5		5	25	17	3	2	20	7	3		3	3		2	10	
22	IND6		10	21	14	7	2	16	5	2		2	2	2	3	14	
23	AGR1		48	8	6	12		7	2					3	2	12	
24	REL1	36		4	4			3	2	2		2		7	6	34	
25	GOV1		7	24	16	3		19	5	3		2	1	3	3	13	
26	GOV2		8	16	11	4		13	8	3	2	4	3	4	5	19	
27	EDU1		13	17	13			13	5	3		2	2	5	5	22	
28	EDU2		4	18	13			14	8	3	2	4	3	5	4	22	

***** Refer to Table 3C.1 for states' classifications.

Table 3A.15: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, East Coast*

No.	Specific Occupancy Class	Model Building Type										
		4	7	11	14	17	20	23	27	30	32	35
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
3	RES3	3	4			6	3		14		13	57
4	RES4	9	12		3	18	9	2	11		7	29
5	RES5	7	10		3	23	11	3	12		5	26
6	RES6											
7	COM1	23	29	2	8	5	3		5		5	20
8	COM2	23	30	3	8	4	3		5		5	19
9	COM3	10	13		3	5	4		11		10	44
10	COM4	14	19	2	5	7	4		9		7	33
11	COM5	15	21	2	6	8	5		8		6	29
12	COM6	21	27	2	8	12	6	2	7		2	13
13	COM7	15	20	2	5	7	4		9		6	32
14	COM8	22	30	3	8	5	3		5		5	19
15	COM9											
16	COM10	10	13		3	38	17	6	11			2
17	IND1											
18	IND2	22	28	2	8	10	5	2	6		3	14
19	IND3	25	32	3	9	6	4		4		3	14
20	IND4											
21	IND5	24	32	3	9	9	6		5		2	10
22	IND6											
23	AGR1	19	25	2	7	4	2		7		6	28
24	REL1	5	9		2	4	3		12		12	53
25	GOV1	24	30	3	9	7	5		5		3	14
26	GOV2											
28	EDU2	17	23	2	6	10	5	2	8		4	23

* Refer to Table 3C.1 for states' classifications.

Table 3A.16: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, East Coast*

No.	Specific Occup. Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
3	RES3	8	21	8		34	17	2	5	5
4	RES4	8	21	8		34	17	2	5	5
5	RES5	6	16	6		40	20	3	5	4
10	COM4	15	36	15		15	8		2	9
11	COM5	15	36	15		15	8		2	9
12	COM6	14	35	14		17	8	2	2	8
13	COM7	15	38	15		14	8		2	8
16	COM10	5	12	5		43	21	4	6	4
23	AGR1	7	4	18		20	42			9
25	GOV1									
28	EDU2									

* Refer to Table 3C.1 for states' classifications.

Table 3A.17: Distribution Percentage of Floor Area for Model Building Types within "RES1" Building Occupancy Class, Pre-1950, West Coast

State FIPS*	State Abbreviation	State	Model Building Type					
			1	9	13	19	29	34
			W1	S3	S5L	C2L	RM1L	URML
02	AK	Alaska	99			1		
04	AZ	Arizona	60				25	16
06	CA	California	85	4	3		5	3
08	CO	Colorado	76				15	9
15	HI	Hawaii	92			1	4	3
16	ID	Idaho	95				3	2
30	MT	Montana	98				1	1
35	NM	New Mexico	74				16	10
32	NV	Nevada	97				2	1
41	OR	Oregon	99				1	
49	UT	Utah	82				11	7
53	WA	Washington	98				1	1
56	WY	Wyoming	92				5	3

* State FIPS are two digit unique number representative of each state and US territory. Refer to Table 3C.1 of Appendix C for a complete list of State FIPS.

Table 3A.18: Distribution Percentage of Floor Area for Model Building Types within "RES1" Building Occupancy Class, 1950-1970, West Coast

State FIPS	State Abbreviation	State	Model Building Type					
			1	9	13	19	29	34
			W1	S3	S5L	C2L	RM1L	URML
02	AK	Alaska	99			1		
04	AZ	Arizona	60				36	4
06	CA	California	85	5	1		8	1
08	CO	Colorado	76				21	3
15	HI	Hawaii	92			1	6	1
16	ID	Idaho	95				4	1
30	MT	Montana	98				2	
35	NM	New Mexico	74				23	3
32	NV	Nevada	97				3	
41	OR	Oregon	99				1	
49	UT	Utah	82				16	2
53	WA	Washington	98				2	
56	WY	Wyoming	92				7	1

Table 3A.19: Distribution Percentage of Floor Area for Model Building Types within "RES1" Building Occupancy Class, Post-1970, West Coast

State FIPS	State Abbreviation	State	Model Building Type					
			1	9	13	19	29	34
			W1	S3	S5L	C2L	RM1L	URML
02	AK	Alaska	99			1		
04	AZ	Arizona	60				40	
06	CA	California	83	5	3		9	
08	CO	Colorado	76				24	
15	HI	Hawaii	92			1	7	
16	ID	Idaho	95				5	
30	MT	Montana	98				2	
35	NM	New Mexico	74				26	
32	NV	Nevada	97				3	
41	OR	Oregon	99				1	
49	UT	Utah	82				18	
53	WA	Washington	98				2	
56	WY	Wyoming	92				8	

Table 3A.20: Distribution Percentage of Floor Area for Model Building Types within "RES1" Building Occupancy Class, Mid-West

State FIPS	State Abbreviation	State	Model Building Type 1 W1	19 C2L	34 URML
05	AR	Arkansas	87		13
19	IA	Iowa	92		8
17	IL	Illinois	77	1	22
18	IN	Indiana	80		20
20	KS	Kansas	91		9
21	KY	Kentucky	88		12
22	LA	Louisiana	89		11
26	MI	Michigan	86		14
27	MN	Minnesota	95	1	4
29	MO	Missouri	76		24
28	MS	Mississippi	94		6
38	ND	North Dakota	98		2
31	NE	Nebraska	89	1	10
39	OH	Ohio	76		24
40	OK	Oklahoma	71		29
46	SD	South Dakota	97		3
47	TN	Tennessee	90		10
48	TX	Texas	100		
55	WI	Wisconsin	90		10

Table 3A.21: Distribution Percentage of Floor Area for Model Building Types within "RES1" Building Occupancy Class, East Coast

State FIPS	State Abbreviation	State	Model Building Type 1 W1	19 C2L	34 URML
01	AL	Alabama	95		5
09	CT	Connecticut	96		4
11	DC	District of Columbia	21	3	76
10	DE	Delaware	71	1	28
12	FL	Florida	25	5	70
13	GA	Georgia	93		7
25	MA	Massachusetts	96		4
24	MD	Maryland	71	1	28
23	ME	Maine	99		1
37	NC	North Carolina	90		10
33	NH	New Hampshire	97	1	2
34	NJ	New Jersey	91		9
36	NY	New York	85	1	14
42	PA	Pennsylvania	66		34
44	RI	Rhode Island	98		2
45	SC	South Carolina	92		8
51	VA	Virginia	75		25
50	VT	Vermont	96	2	2
54	WV	West Virginia	72		28

APPENDIX 3B
Essential Facilities

Table 3B.1: Distribution Percentage of Floor Area for Specific Occupancy Classes within each General Occupancy Class

Specific Occupancy Class			General Occupancy Class		
			Medical Care	Emergency Response	Schools
No.	Label	Occupancy Class	1	2	3
1	EFHS	Small Hospital	X		
2	EFHM	Medium Hospital	X		
3	EFHL	Large Hospital	X		
4	EFMC	Medical Clinics	X		
5	EFFS	Fire Station		X	
6	EFPS	Police Station		X	
7	EFEO	Emergency Operation Centers		X	
8	EFS1	Grade Schools			X
9	EFS2	Colleges/ Universities			X

Table 3B.2: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, Pre-1950, West Coast* (after ATC-13, 1985)

No.	Specific Occup. Class	Model Building Type														
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML
1	EFHS		8	5	2	11		11		27	2	1		27		6
2	EFHM		8	5	2	11		11		27	2	1		27		6
3	EFHL		8	5	2	11		11		27	2	1		27		6
4	EFMC		8	5	2	11		11		27	2	1		27		6
5	EFFS	45					2			37				3		13
6	EFPS	45					2			37				3		13
7	EFEO	45					2			37				3		13
8	EFS1	11		6		3	3	21		21	4			9		22
9	EFS2	2		5	10		5	15		20				20	5	18

Table 3B.3: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, 1950-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occup. Class	Model Building Type														
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML
1	EFHS		32	5	2	4	3			16	6			28	4	
2	EFHM		32	5	2	4	3			16	6			28	4	
3	EFHL		32	5	2	4	3			16	6			28	4	
4	EFMC		32	5	2	4	3			16	6			28	4	
5	EFFS	50								13		7		20	10	
6	EFPS	50								13		7		20	10	
7	EFEO	50								13		7		20	10	
8	EFS1	25		3	4	5	4			20		4	2	29	4	
9	EFS2	5		2	12		5			20				50	6	

***** Refer to Table 3C.1 for states' classifications.

Table 3B.4: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, Post-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occup. Class	Model Building Type														
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML
1	EFHS		31	6	1	1	7		4	13		7		28	2	
2	EFHM		31	6	1	1	7		4	13		7		28	2	
3	EFHL		31	6	1	1	7		4	13		7		28	2	
4	EFMC		31	6	1	1	7		4	13		7		28	2	
5	EFFS	40		3	7		23			10			7	3	7	
6	EFPS	40		3	7		23			10			7	3	7	
7	EFEO	40		3	7		23			10			7	3	7	
8	EFS1	24		9	6	1	5		3	16	3	4	3	21	5	
9	EFS2	5		10	10		5			20		5		40	5	

Table 3B.5: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, Pre-1950, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type										
		4	7	11	14	17	20	23	27	30	32	35
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
1	EFHS	18	4	6		1	35	19		8		9
2	EFHM	18	4	6		1	35	19		8		9
3	EFHL	18	4	6		1	35	19		8		9
4	EFMC	18	4	6		1	35	19		8		9
5	EFFS											
6	EFPS											
7	EFEO											
9	EFS2	10		20			60	3		5		2

* Refer to Table 3C.1 for states' classifications.

Table 3B.6: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, 1950-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type										
		4	7	11	14	17	20	23	27	30	32	35
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
1	EFHS	14	10	14		5	23		3	23	8	
2	EFHM	14	10	14		5	23		3	23	8	
3	EFHL	14	10	14		5	23		3	23	8	
4	EFMC	14	10	14		5	23		3	23	8	
5	EFFS	5	10	10		5	60				10	
6	EFPS	5	10	10		5	60				10	
7	EFEO	5	10	10		5	60				10	
9	EFS2	20		15		5	35			15	10	

Table 3B.7: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, Post-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type										
		4	7	11	14	17	20	23	27	30	32	35
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
1	EFHS	25	9	15		10	33		1	6	1	
2	EFHM	25	9	15		10	33		1	6	1	
3	EFHL	25	9	15		10	33		1	6	1	
4	EFMC	25	9	15		10	33		1	6	1	
5	EFFS	25	20	35			20					
6	EFPS	25	20	35			20					
7	EFEO	25	20	35			20					
9	EFS2	20	5	10		25	25			10	5	

* Refer to Table 3C.1 for states' classifications.

Table 3B.8: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, Pre-1950, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
1	EFHS	56	9	1		1	24	8	1	
2	EFHM	56	9	1		1	24	8	1	
3	EFHL	56	9	1		1	24	8	1	
4	EFMC	56	9	1		1	24	8	1	
9	EFS2	5	5	35			40	15		

Table 3B.9: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, 1950-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
1	EFHS	35	27	17		4	15		1	1
2	EFHM	35	27	17		4	15		1	1
3	EFHL	35	27	17		4	15		1	1
4	EFMC	35	27	17		4	15		1	1
9	EFS2	35	20	20		25				

Table 3B.10: Distribution Percentage of Floor Area, for Model Building Types within Each Building Occupancy Class, High Rise, Post-1970, West Coast* (after ATC-13, 1985)

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
1	EFHS	45	6	19		13	17			
2	EFHM	45	6	19		13	17			
3	EFHL	45	6	19		13	17			
4	EFMC	45	6	19		13	17			
9	EFS2	30	10	10		50				

* Refer to Table 3C.1 for states' classifications.

Table 3B.11: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, Mid-West*

No.	Specific Occup. Class	Model Building Type														
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML
1	EFHS		30	2	4	11	6	7		5		5		2		28
2	EFHM				2	4	2	2	6	21	4	33	6	2		18
3	EFHL				2	4	2	2	6	21	4	33	6	2		18
4	EFMC		30	2	4	11	6	7		5		5		2		28
5	EFFS		14	7	17				4	12					3	43
6	EFPS		14	7	17				4	12					3	43
7	EFEO		14	7	17				4	12					3	43
8	EFS1		10	5	12				5	7				11		50
9	EFS2		14	6	12			2	8	11					10	37

Table 3B.12: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, Mid-West*

No.	Specific Occupancy Class	Model Building Type										
		4	7	11	14	17	20	23	27	30	32	35
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
1	EFHS	3	20	16	6	11	27	2	5		2	8
2	EFHM	3	20	16	6	12	30	2	6			5
3	EFHL	3	20	16	6	12	30	2	6			5
4	EFMC	3	20	16	6	11	27	2	5		2	8
5	EFFS											
6	EFPS											
7	EFEO											
9	EFS2	7	14			9	13				13	44

***** Refer to Table 3C.1 for states' classifications.

Table 3B.13: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, Mid-West*

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
1	EFHS	7	29	9		12	32	4	4	3
2	EFHM	7	29	9		13	36	2	2	2
3	EFHL	7	29	9		13	36	2	2	2
4	EFMC	7	29	9		12	32	4	4	3
7	EFEO									
9	EFS2									

Table 3B.14: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Low Rise, East Coast*

No.	Specific Occup. Class	Model Building Type														
		1	2	3	6	9	10	13	16	19	22	25	26	29	31	34
		W1	W2	S1L	S2L	S3	S4L	S5L	C1L	C2L	C3L	PC1	PC2L	RM1L	RM2L	URML
1	EFHS		24	10	7	15		8	3	2		3		4	4	20
2	EFHM		2	22	15			18	10	4	2	5	4	3	2	13
3	EFHL		2	22	15			18	10	4	2	5	4	3	2	13
4	EFMC		24	10	7	15		8	3	2		3		4	4	20
5	EFFS		8	16	11	4		13	8	3	2	4	3	4	5	19
6	EFPS		8	16	11	4		13	8	3	2	4	3	4	5	19
7	EFEO		8	16	11	4		13	8	3	2	4	3	4	5	19
8	EFS1		13	17	13			13	5	3		2	2	5	5	22
9	EFS2		4	18	13			14	8	3	2	4	3	5	4	22

* Refer to Table 3C.1 for states' classifications.

Table 3B.15: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, Mid Rise, East Coast*

No.	Specific Occupancy Class	Model Building Type										
		4	7	11	14	17	20	23	27	30	32	35
		S1M	S2M	S4M	S5M	C1M	C2M	C3M	PC2M	RM1M	RM2M	URMM
1	EFHS	15	20	2	5	7	4		9		6	32
2	EFHM	21	27	2	8	12	6	2	7		2	13
3	EFHL	21	27	2	8	12	6	2	7		2	13
4	EFMC	15	20	2	5	7	4		9		6	32
5	EFFS											
6	EFPS											
7	EFEO											
9	EFS2	17	23	2	6	10	5	2	8		4	23

Table 3B.16: Distribution Percentage of Floor Area for Model Building Types within Each Building Occupancy Class, High Rise, East Coast*

No.	Specific Occupancy Class	Model Building Type								
		5	8	12	15	18	21	24	28	33
		S1H	S2H	S4H	S5H	C1H	C2H	C3H	PC2H	RM2H
1	EFHS	15	38	15		14	8		2	8
2	EFHM	14	35	14		17	8	2	2	8
3	EFHL	14	35	14		17	8	2	2	8
4	EFMC	15	38	15		14	8		2	8
7	EFEO									
9	EFS2									

- Refer to Table 3C.1 for states' classifications.

APPENDIX 3C

States' Classifications

Table 3C.1: Regional Distribution of States

State Fips	State Abbreviation	State Name	Group
02	AK	Alaska	West
01	AL	Alabama	East
05	AR	Arkansas	Mid-West
04	AZ	Arizona	West
06	CA	California	West
08	CO	Colorado	West
09	CT	Connecticut	East
11	DC	District of Columbia	East
10	DE	Delaware	East
12	FL	Florida	East
13	GA	Georgia	East
15	HI	Hawaii	West
19	IA	Iowa	Mid-West
16	ID	Idaho	West
17	IL	Illinois	Mid-West
18	IN	Indiana	Mid-West
20	KS	Kansas	Mid-West
21	KY	Kentucky	Mid-West
22	LA	Louisiana	Mid-West
25	MA	Massachusetts	East
24	MD	Maryland	East
23	ME	Maine	East
26	MI	Michigan	Mid-West
27	MN	Minnesota	Mid-West
29	MO	Missouri	Mid-West
28	MS	Mississippi	Mid-West
30	MT	Montana	West
37	NC	North Carolina	East
38	ND	North Dakota	Mid-West
31	NE	Nebraska	Mid-West
33	NH	New Hampshire	East
34	NJ	New Jersey	East
35	NM	New Mexico	West
32	NV	Nevada	West
36	NY	New York	East
39	OH	Ohio	Mid-West
40	OK	Oklahoma	Mid-West
41	OR	Oregon	West
42	PA	Pennsylvania	East
44	RI	Rhode Island	East

Table 3C.1(cont.): Regional Distribution of States

State Fips	State Abbreviation	State Name	Group
45	SC	South Carolina	East
46	SD	South Dakota	Mid-West
47	TN	Tennessee	Mid-West
48	TX	Texas	Mid-West
49	UT	Utah	West
51	VA	Virginia	East
50	VT	Vermont	East
53	WA	Washington	West
55	WI	Wisconsin	Mid-West
54	WV	West Virginia	East
56	WY	Wyoming	West
60	AS	American Samoa	West
66	GU	Guam	West
69	MR	Northern Mariana Islands	West
72	PR	Puerto Rico	East
78	VI	Virgin Islands	East

Chapter 4
Potential Earth Science Hazards (PESH)

Potential earth science hazards (PESH) include ground motion, ground failure (i.e., liquefaction, landslide and surface fault rupture) and tsunami/seiche. Methods for developing estimates of ground motion and ground failure are discussed in the following sections. Tsunami/seiche can be included in the Methodology in the form of user-supplied inundation maps as discussed in Chapter 9. The Methodology, highlighting the PESH component, is shown in Flowchart 4.1.

4.1 Ground Motion

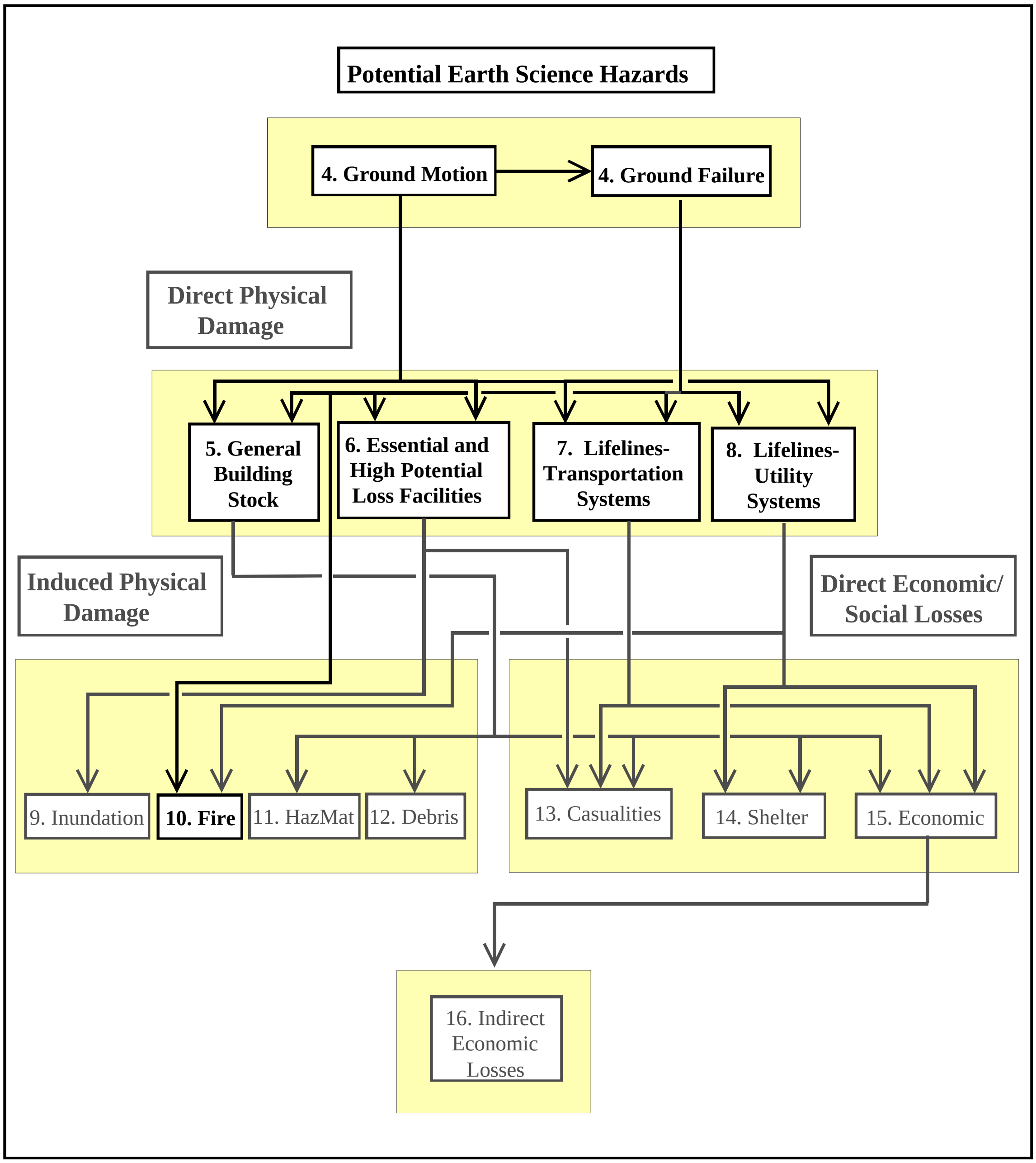
4.1.1 Introduction

Ground motion estimates are generated in the form of GIS-based contour maps and location-specific seismic demands stored in relational databases. Ground motion is characterized by: (1) spectral response, based on a standard spectrum shape, (2) peak ground acceleration and (3) peak ground velocity. The spatial distribution of ground motion can be determined using one of the following methods or sources:

- Deterministic ground motion analysis (Methodology calculation)
- USGS probabilistic ground motion maps (maps supplied with **HAZUS**)
- Other probabilistic or deterministic ground motion maps (user-supplied maps)

Deterministic seismic ground motion demands are calculated for user-specified scenario earthquakes (Section 4.1.2.1). For a given event magnitude, attenuation relationships (Section 4.1.2.3) are used to calculate ground shaking demand for rock sites (Site Class B), which is then amplified by factors (Section 4.1.2.4) based on local soil conditions when a soil map is supplied by the user. The attenuation relationships provided with the Methodology for Western United States (WUS) sites are based on Boore, Joyner & Fumal (1993, 1994a, 1994b), Campbell and Bozorgnia (1994), Munson and Thurber (1997), Sadigh, Chang, Abrahamson, Chiou and Power (1993) and Youngs, Chiou, Silva and Humphrey (1997). For sites in the Central and Eastern United States (CEUS), the attenuation relationships are based on Frankel et al. (1996), Savy (1998) and Toro, Abrahamson and Schneider (1997).

In the Methodology's probabilistic analysis procedure, the ground shaking demand is characterized by spectral contour maps developed by the United States Geological Survey (USGS) as part of Project 97 project (Frankel et. al, 1996). The Methodology includes maps for eight probabilistic hazard levels: ranging from ground shaking with a 39% probability of being exceeded in 50 years (100 year return period) to the ground shaking with a 2% probability of being exceeded in 50 years (2500 year return period). The USGS maps describe ground shaking demand for rock (Site Class B) sites, which the Methodology amplifies based on local soil conditions.



Flowchart 4.1: Ground Motion and Ground Failure Relationship to other Modules of the Earthquake Loss Estimation Methodology

User-supplied peak ground acceleration (PGA) and spectral acceleration contour maps may also be used with **HAZUS** (Section 4.1.2.1). In this case, the user must provide all contour maps in a pre-defined digital format (as specified in the). As stated in Section 4.1.2.1, the Methodology assumes that user-supplied maps include soil amplification.

4.1.1.1 Form of Ground Motion Estimates / Site Effects

Ground motion estimates are represented by: (1) contour maps and (2) location-specific values of ground shaking demand. For computational efficiency and improved accuracy, earthquake losses are generally computed using location-specific estimates of ground shaking demand. For general building stock the analysis has been simplified so that ground motion demand is computed at the centroid of a census tract. However, contour maps are also developed to provide pictorial representations of the variation in ground motion demand within the study region. When ground motion is based on either USGS or user-supplied maps, location-specific values of ground shaking demand are interpolated between PGA, PGV or spectral acceleration contours, respectively.

Elastic response spectra (5% damping) are used by the Methodology to characterize ground shaking demand. These spectra all have the same “standard” format defined by a PGA value (at zero period) and spectral response at a period of 0.3 second (acceleration domain) and spectral response at a period of 1.0 second (velocity domain). Ground shaking demand is also defined by peak ground velocity (PGV).

4.1.1.2 Input Requirements and Output Information

For computation of ground shaking demand, the following inputs are required:

- **Scenario Basis** - The user must select the basis for determining ground shaking demand from one of three options: (1) a deterministic calculation, (2) probabilistic maps, supplied with the Methodology, or (3) user-supplied maps. For deterministic calculation of ground shaking, the user specifies a scenario earthquake magnitude and location. In some cases, the user may also need to specify certain source attributes required by the attenuation relationships supplied with the Methodology.
- **Attenuation Relationship** - For deterministic calculation of ground shaking, the user selects an appropriate attenuation relationship from those supplied with the Methodology. Attenuation relationships are based on the geographic location of the study region (Western United States vs. Central Eastern United States) and on the type of fault for WUS sources. WUS regions include locations in, or west of, the Rocky Mountains, Hawaii and Alaska. Figure 4-1 shows the regional separation of WUS and CEUS locations as defined in Project 97 (Frankel et al., 1996). The designation of states as WUS or CEUS as specified in the Methodology is found in Table 3C.1. For WUS sources, the attenuation functions predict ground shaking

based on source type, including: (1) strike-slip faults, (2) reverse-slip faults, (3) deep faults (> 50 km) and (4) Cascadia subduction zone sources. The Methodology provides "default" combinations of attenuation functions for the WUS and CEUS, respectively, following the theory developed by the USGS for the 48 contiguous states in Project 97 (Frankel et al., 1996), for Alaska (Frankel, 1997), and Hawaii (Klein et al., 1998).

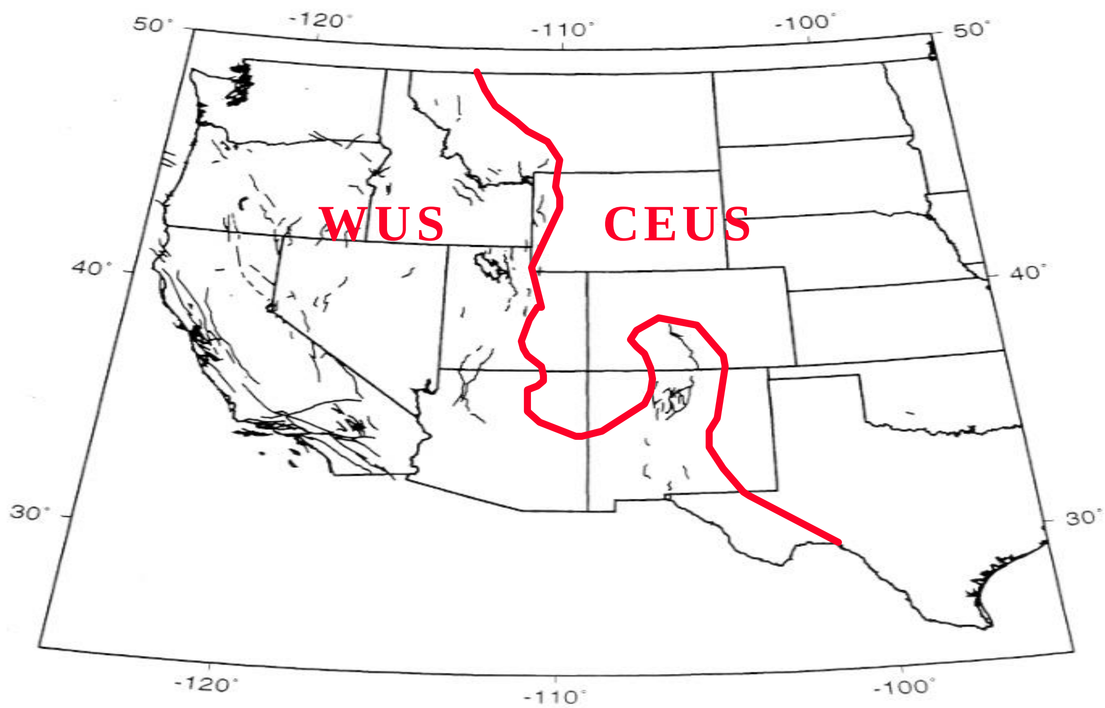


Figure 4.1 Boundaries Between WUS and CEUS Locations as Defined in Project 97.

- **Soil Map -** The user may supply a detailed soil map to account for local site conditions. This map must identify soil type using a scheme that is based on, or can be related to, the site class definitions of the 1997 *NEHRP Provisions* (Section 4.1.2.4), and must be in pre-defined digital format (as specified in the *User's Manual*). In the absence of a soil map, **HAZUS** will amplify the ground motion demand assuming Site Class D soil at all sites. However; a user may specify a soil map on a census tract basis using **HAZUS** (see Section 6.8 of the *User's Manual*).

4.1.2 Description of Methods

The description of the methods for calculating ground shaking is divided into four separate areas:

- Basis for ground shaking (Section 4.1.2.1)
- Standard shape of response spectra (Section 4.1.2.2)
- Attenuation of ground shaking (Section 4.1.2.3)
- Amplification of ground shaking - local site conditions (Section 4.1.2.4)

4.1.2.1 Basis for Ground Shaking

The methodology supports three options as the basis for ground shaking:

- Deterministic calculation of scenario earthquake ground shaking
- Probabilistic seismic hazard maps (USGS)
- User-supplied seismic hazard maps

Deterministic Calculation of Scenario Earthquake Ground Shaking

For deterministic calculation of the scenario event, the user specifies the location (e.g., epicenter) and magnitude of the scenario earthquake. The Methodology provides three options for selection of an appropriate scenario earthquake location. The user can either: (1) specify an event based on a database of WUS seismic sources (faults), (2) specify an event based on a database of historical earthquake epicenters, or (3) specify an event based on an arbitrary choice of the epicenter. These options are described below.

Seismic Source Database (WUS Fault Map)

For the WUS, the Methodology provides a database of seismic sources (fault segments) developed by the USGS, the California Department of Mines and Geology (CDMG) and the Nevada Bureau of Mines and Geology (NBMG). The user accesses the database map (using **HAZUS**) and selects a magnitude and epicenter on one of the identified fault segments. The database includes information on fault segment type, location, orientation and geometry (e.g., depth, width and dip angle), as well as on each fault segment's seismic potential (e.g., maximum moment).

The Methodology computes the expected values of surface and subsurface fault rupture length. Fault rupture length is based on the relationship of Wells and Coppersmith (1994) given below:

$$\log_{10}(L) = a + b \cdot M \tag{4-1}$$

where: L is the rupture length (km)

M is the moment magnitude of the earthquake

Table 4.1 Regression Coefficients of Fault Rupture Relationship of Wells and Coppersmith (1994)

Rupture Type	Fault Type	a	b
Surface	Strike Slip	-3.55	0.74
	Reverse	-2.86	0.63
	All	-3.22	0.69
Subsurface	Strike Slip	-2.57	0.62
	Reverse	-2.42	0.58
	All	-2.44	0.59

Fault rupture is assumed to be of equal length on each side of the epicenter, provided the calculated rupture length is available in both directions along the specified fault segment. If the epicenter location is less than one-half of the rupture length from an end point of the fault segment (e.g., the epicenter is located at or near an end of the fault segment), then fault rupture length is truncated so that rupture does not extend past the end of the fault segment. If the calculated rupture length exceeds the length of the fault segment, then the entire fault segment is assumed to rupture between its end points, unless the fault is connected to other fault segments. In the case where multiple faults segments share common endpoints (i.e. the segments are connected), the methodology provides the user with the ability to create an earthquake rupture across multiple segments.

Historical Earthquake Database (Epicenter Map)

The Methodology software provides a database of historical earthquakes developed from the Global Hypocenter Database available from the National Earthquake Information Center (NEIC, 1992), which contains reported earthquakes from 300 BC to 1990. The database has been sorted to remove historical earthquakes with magnitudes less than 5.0. The user accesses the database via **HAZUS** and selects a historical earthquake epicenter which includes location, depth and magnitude information.

For the WUS, the attenuation relationships require the user to specify the type and orientation of the fault associated with the selected epicenter. The Methodology computes the expected values of surface and subsurface fault rupture length using Equation (4-1). Fault rupture is assumed to be of equal length on each side of the epicenter. For the CEUS, the attenuation relationships depend on the hypocentral distance (Frankel et al., 1996 & Savy, 1998) or closest horizontal distance to the epicenter (Toro et al., 1997).

Arbitrary Event

Under this option, the user specifies a scenario event magnitude and arbitrary epicenter (using **HAZUS**). For the WUS, the user must also supply the type and orientation of the fault associated with the arbitrary epicenter. The Methodology computes the fault rupture length based on Equation (4-1) and assumes fault rupture to be of equal length on each side of the epicenter. For the CEUS the user must supply the depth of the hypocenter.

Probabilistic Seismic Hazard Maps (USGS)
The Methodology includes probabilistic seismic hazard contour maps developed by the USGS for Project 97. The USGS maps provide estimates of PGA and spectral acceleration at periods of 0.3 second and 1.0 second, respectively. Ground shaking estimates are available for eight hazard levels: ranging from the ground shaking with a 39% probability of being exceeded in 50 years to ground shakeing with a 2% probability of being exceeded in 50 years. In terms of mean return periods, the hazard levels range from 100 years to 2500 years.

User-Supplied Seismic Hazard Maps
The Methodology allows the user to supply PGA and spectral acceleration contour maps of ground shaking in a pre-defined digital format (as specified in the User's Manual). This option permits the user to develop a scenario event that could not be described adequately by the available attenuation relationships, or to replicate historical earthquakes (e.g., 1994 Northridge Earthquake). The maps of PGA and spectral acceleration (periods of 0.3 and 1.0 second) must be provided. The Methodology software assumes these ground motion maps include soil amplification, thus no soil map is required.

Should only PGA contour maps be available, the user can develop the other required maps based on the spectral acceleration response factors given in Table 4.2 (WUS) and Table 4.3 (CEUS).

4.1.2.2 Standard Shape of the Response Spectra

The Methodology characterizes ground shaking using a standardized response spectrum shape, as shown in Figure 4.2. The standardized shape consists of four parts: peak ground acceleration (PGA), a region of constant spectral acceleration at periods from zero seconds to T_{AV} (seconds), a region of constant spectral velocity at periods from T_{AV} to T_{VD} (seconds) and a region of constant spectral displacement for periods of T_{VD} and beyond.

In Figure 4.2, spectral acceleration is plotted as a function of spectral displacement (rather than as a function of period). This is the format of response spectra used for evaluation of damage to buildings (Chapter 5) and essential facilities (Chapter 6). Equation (4-2) may be used to convert spectral displacement (inches), to period (seconds) for a given value of spectral acceleration (units of g), and Equation (4-3) may be used to convert spectral acceleration (units of g) to spectral displacement (inches) for a given value of period.

$$T = 0.32\sqrt{\frac{S_D}{S_A}} \qquad (4\text{-}2)$$

$$S_D = 9.8 \cdot S_A \cdot T^2 \qquad (4\text{-}3)$$

The region of constant spectral acceleration is defined by spectral acceleration at a period of 0.3 second. The constant spectral velocity region has spectral acceleration proportional to 1/T and is anchored to the spectral acceleration at a period of 1 second. The period, T_{AV}, is based on the intersection of the region of constant spectral acceleration and constant spectral velocity (spectral acceleration proportional to 1/T). The value of T_{AV} varies depending on the values of spectral acceleration that define these two intersecting regions. The constant spectral displacement region has spectral acceleration proportional to $1/T^2$ and is anchored to spectral acceleration at the period, T_{VD}, where constant spectral velocity transitions to constant spectral displacement.

The period, T_{VD}, is based on the reciprocal of the corner frequency, f_c, which is proportional to stress drop and seismic moment. The corner frequency is estimated in Joyner and Boore (1988) as a function of moment magnitude (**M**). Using Joyner and Boore's formulation, the period T_{VD}, in seconds, is expressed in terms of the earthquake's moment magnitude as shown by the following Equation (4-4):

$$T_{VD} = 1/f_c = 10^{\frac{(M-5)}{2}} \tag{4-4}$$

When the moment magnitude of the scenario earthquake is not known (e.g., when using USGS maps or user-supplied maps), the period T_{VD} is assumed to be 10 seconds (i.e., moment magnitude is assumed to be **M** = 7.0).

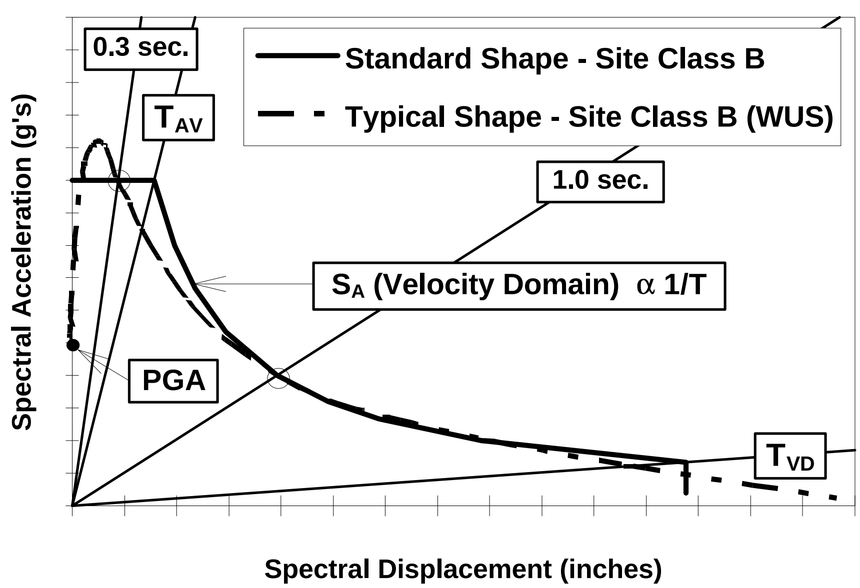


Figure 4.2 Standardized Response Spectrum Shape

Using a standard response spectrum shape simplifies calculation of response needed in estimating damage and loss. In reality, the shape of the spectrum will vary depending on whether the earthquake occurs in the WUS or CEUS, whether it is a large or moderate size event and whether the site is near or far from the earthquake source. However, the differences between the shape of an actual spectrum and the standard spectrum tend to be significant only at periods less than 0.3 second and at periods greater than T_{VD}, which do not significantly affect the Methodology's estimation of damage and loss.

The standard response spectrum shape (with adjustment for site amplification) represents all site/source conditions, except for site/source conditions that have strong amplification at periods beyond 1 second. Although relatively rare, strong amplification at periods beyond 1 second can occur. For example, strong amplification at a period of about 2 seconds caused extensive damage and loss to taller buildings in parts of Mexico City during the 1985 Michoacan earthquake. In this case, the standard response spectrum shape would tend to overestimate short-period spectral acceleration and to underestimate long-period (i.e., greater than 1-second) spectral acceleration.

Inferred Ground Shaking Hazard Information

Certain ground shaking hazard information is inferred from other ground shaking hazard information when complete hazard data is not available. Inferred data includes the following:

- Peak ground velocity (PGV) is inferred from 1-second spectral acceleration response
- Spectral acceleration response is inferred from the peak ground acceleration (PGA)
- 0.3-second spectral acceleration response is inferred from 0.2-second response

PGV Inferred from 1-Second Spectral Response

Unless supplied by the user (i.e., as user-supplied PGV maps), peak ground velocity (inches per second) is inferred from 1-second spectral acceleration, S_{A1} (units of g), using Equation (4-5).

$$PGV = \left(\frac{386.4}{2p} \cdot S_{A1} \right) / 1.65 \tag{4-5}$$

The factor of 1.65 in the denominator of Equation (4-5) represents the amplification assumed to exist between peak spectral response and PGV. This factor is based on the median spectrum amplification, as given in Table 2 of Newmark and Hall (1982) for a 5%-damped system whose period is within the velocity-domain region of the response spectrum.

Spectral Acceleration Response Inferred from Peak Ground Acceleration (PGA)

When a user has maps of PGA only, short-period spectral acceleration, S_{AS}, maps are developed from PGA, and 1.0-second spectral acceleration, S_{A1}, is inferred from short-period spectral acceleration, S_{AS}, based on the factors given in Table 4.2 for WUS rock (Site Class B) locations and in Table 4.3 for CEUS rock (Site Class B) locations.

Table 4.2 Spectral Acceleration Response Factors - WUS Rock (Site Class B)

Closest Distance to Fault Rupture	S_{AS}/PGA given Magnitude, M:				S_{AS}/S_{A1} given Magnitude, M:			
	≤ 5	6	7	≥ 8	≤ 5	6	7	≥ 8
≤ 10 km	1.4	1.8	2.1	2.1	5.3	3.7	3.1	1.8
20 km	1.5	2.0	2.1	2.0	5.0	3.5	2.5	1.7
40 km	1.6	2.1	2.2	2.0	4.6	3.3	2.3	1.6
≥ 80 km	1.3	1.8	2.1	2.0	4.1	3.1	2.1	1.5

Table 4.3 Spectral Acceleration Response Factors - CEUS Rock (Site Class B)

Hypocentral Distance	S_{AS}/PGA given Magnitude, M:				S_{AS}/S_{A1} given Magnitude, M:			
	≤ 5	6	7	≥ 8	≤ 5	6	7	≥ 8
≤ 10 km	0.9	1.2	1.5	2.1	8.7	4.2	3.1	2.3
20 km	1.0	1.3	1.4	1.6	8.1	4.0	3.0	2.7
40 km	1.2	1.4	1.6	1.6	7.3	3.7	2.8	2.6
≥ 80 km	1.5	1.7	1.8	1.9	6.5	3.3	2.5	2.4

The factors given in Tables 4.2 and 4.3 are based on the default combinations of attenuation WUS and CEUS functions, described in the next section. These factors distinguish between small-magnitude and large-magnitude events and between sites that are located at different distances from the source (i.e., closest distance to fault rupture for the WUS and distance to the hypocenter for the CEUS). The ratios of S_{AS}/S_{A1} and S_{AS}/PGA define the standard shape of the response spectrum for each of the magnitude/distance combinations of Tables 4.2 and 4.3.

Tables 4.2 and 4.3 require magnitude and distance information to determine spectrum amplification factors. This information would likely be available for maps of observed earthquake PGA, or scenario earthquake PGA, but is not available for probabilistic maps of PGA, since these maps are aggregated estimates of seismic hazard due to different event magnitudes and sources.

0.3-Second Spectral Acceleration Response Inferred from 0.2-Second Response

Some of the probabilistic maps developed by the USGS for Project 97, estimate short-period spectral response for a period of 0.2 second. Spectral response at a period of 0.3 second is calculated by dividing 0.2-second response by a factor of 1.1 for WUS locations and by dividing 0.2-second response by a factor of 1.4 for CEUS locations.

The factors describing the ratio of 0.2-second and 0.3-second response are based on the default combinations of WUS and CEUS attenuation functions, described in the next section, and the assumption that large-magnitude events tend to dominate seismic hazard at most WUS locations and that small-magnitude events tend to dominate seismic hazard at most CEUS locations.

4.1.2.3 Attenuation of Ground Shaking

Ground shaking is attenuated with distance from the source using relationships provided with the Methodology. These relationships define ground shaking for rock (Site Class B) conditions based on earthquake magnitude and other parameters. These relationships are used to estimate PGA and spectral demand at 0.3 and 1.0 seconds, and with the standard response spectrum shape (described in Section 4.1.2.2) fully define 5%-damped demand spectra at a given location.

The Methodology provides five WUS and three CEUS attenuation functions. The WUS relationships should be used for study regions located in, or west of, the Rocky Mountains, Hawaii and Alaska. The CEUS attenuation relationships should be used for the balance of the continental United States and Puerto Rico. Table 3C.1 defines the distribution of states for the WUS and CEUS.

<u>Western United States Attenuation Relationships</u>

The WUS attenuation relationships provided with the Methodology are based on:

- Boore, Joyner & Fumal (1993, 1994a, 1994b) - shallow crustal earthquakes
- Sadigh, Chang, Abrahamson, Chiou, and Power (1993) - shallow crustal earthquakes
- Campbell and Bozorgnia (1994) - shallow crustal earthquakes (PGA only)
- Munson and Thurber (1997) - Hawaiian earthquakes (PGA only)
- Youngs, Chiou, Silva and Humphrey (1997) - deep and subduction zone earthquakes

<u>Boore, Joyner and Fumal (1993, 1994a, 1994b)</u>

The Boore, Joyner and Fumal (1993, 1994a, 1994b) attenuation relationships predict PGA and spectral acceleration for different site conditions. In the Methodology, the Boore, Joyner and Fumal (BJF 1994) relationship, given in Equation (4-6), predicts the mean value of ground shaking for a site with a shear wave velocity of V_S = 760 m/sec. A shear wave velocity of 760 m/sec is the minimum value of shear wave velocity that defines Site Class B conditions (see Table 4.9), and is the same velocity used by the USGS (Project 97) to develop hazard maps for rock sites (Site Class B).

$$\begin{aligned} log_{10}(SD) &= B_{SA} + a_{SS} \cdot G_{SS} + a_{RS} \cdot G_{RS} + b(\mathbf{M}-6) + c(\mathbf{M}-6)^2 + d\left(\sqrt{r^2+h^2}\right) \\ &\quad + e[\ log_{10}\left(\sqrt{r^2+h^2}\right)] + f\left(2.881 - log_{10}V_B\right) \end{aligned} \qquad (4\text{-}6)$$

where:	SD	is mean of the seismic demand (PGA or spectral acceleration (S_A) in units of g)
	M	is the moment magnitude of the earthquake
	r	is the horizontal distance, in km, from the site to the closest point on the surface projection of fault rupture (see Figure 4.3)
	B_{SA}	is a factor converting spectral velocity (cm/sec) to spectral acc. (g)

a_{SS}, a_{RS} are coefficients for strike-slip and reverse-slip faults, respectively, as given in Table 4.4*

G_{SS}, G_{RS} are fault-type flags: G_{SS} =1 for strike-slip faults, 0 otherwise; G_{RS} = 1 for reverse-slip/thrust faults, 0 otherwise*

b, c, d, e, f are coefficients given in Table 4.4

h is the value of a 'fictitious' depth that is determined by the regression methods and varies by period. It should not be confused with measures of depth of the top edge of the fault rupture (Y_D) that is used in other attenuation relationships

V_B is the value of effective shear wave velocity for WUS rock sites (Site Class B) given in Table 4.4

* Oblique faults are categorized as strike slip if the rake angle is within 30^o of horizontal; otherwise, they are defined as reverse slip. The Methodology uses the strike slip relationship for normal slip earthquakes.

Table 4.4 Boore, Joyner and Fumal (1994) Coefficients - WUS Attenuation

Period	B_{SA}	a_{SS}	a_{RS}	b	c	e	f	h	V_B
Spectral Coefficients (5%-Damped Response Spectra)									
0.3	-1.670	1.930	2.019	0.334	-0.070	-0.893	-0.401	5.94	2130
1.0	-2.193	1.701	1.755	0.450	-0.014	-0.798	-0.698	2.90	1410
Peak Ground Acceleration Coefficients									
0.0	0.0	-0.136	-0.051	0.229	0.000	-0.778	-0.371	5.57	1400

Values of coefficients: B_{SA}, a_{SS}, a_{RS}, b, c, d, e, f, h, and V_B for prediction of 5%-damped response of the random horizontal component of ground shaking are given in Table 4.4.

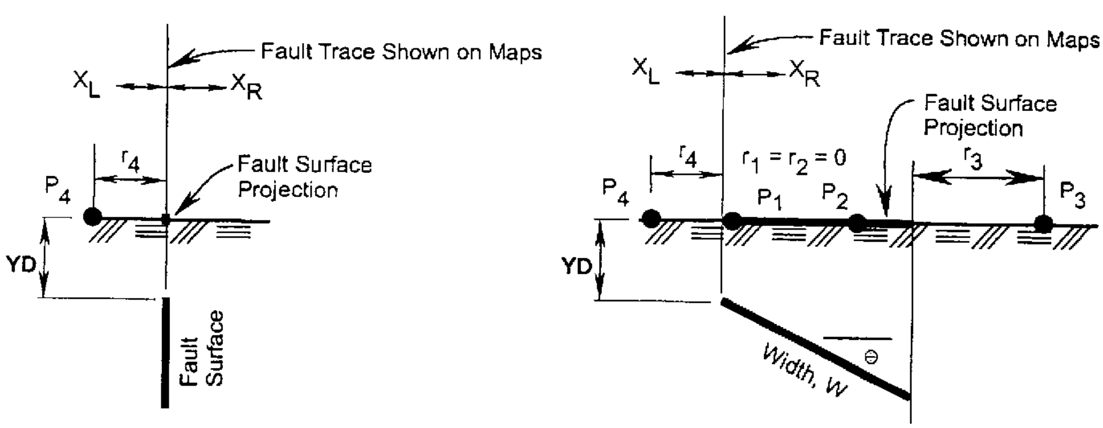


Figure 4.3 Measure of distance for vertical and dipping faults used in Boore Joyner & Fumal (1994) and Munson & Thurber (1997) attenuation relationships.

BJF 1994 limits the magnitude range of Equation (4-6) to 5.5 ≤ **M** ≤ 7.7. BJF 1994 also limits the applicability of Equation (4-6) to source-to-site distances of less than 100 kilometers. In the Methodology, seismic demand for distances greater than 100 kilometers is based on direct substitution of distance into the attenuation relationship (Equations 4-6). The Methodology does not use Equation (4-6) for **M** > 7.7.

Munson & Thurber (1997)

The Munson and Thurber (1997) attenuation relationship predicts PGA for earthquakes for the Island of Hawaii. In the Methodology, the relationship given in Equation (4-10) is used to predict the mean value of PGA for Site Class B.

$$\begin{aligned} log_{10}(SD) &= -1.804 + 0.387(\mathbf{M}-6) - 0.00256\left(\sqrt{r^2+11.29^2}\right) \\ &\quad - log_{10}\left(\sqrt{r^2+11.29^2}\right) \end{aligned} \qquad (4\text{-}7)$$

where: SD is mean of the PGA in units of g
M is the moment magnitude of the earthquake
r is the horizontal distance, in km, from the site to the closest point on the surface projection of fault rupture (see Figure 4.3)

For the Methodology to remain consistent with the USGS approach (Klein et al., 1998), the attenuation relationship for magnitudes greater than 7.0 is modified. From M = 7.0-7.7, the magnitude term becomes 0.316*(7.0) + 0.216*(M-7.0). For M > 7.7, a magnitude term is set to a constant value equal to 0.316*(7.0) + 0.216*(7.7-7.0).

Sadigh, Chang, Abrahamson, Chiou, and Power (1993)

The Sadigh, Chang, Abrahamson, Chiou and Power attenuation relationship (Sadigh 1993) predicts peak ground acceleration and 5%-damped spectral acceleration for rock sites (Site Class B). The relationship is given in Equation (4-8) for events of magnitude **M** < 6.5 and in Equation (4-9) for events of magnitude **M** ≥ 6.5.

M < 6.5:

$$\begin{aligned} ln(SD) &= a_{SS} \cdot G_{SS} + a_{RS} \cdot G_{RS} + 1.0\mathbf{M} + b(8.5-\mathbf{M})^{2.5} \\ &\quad + c\, ln\left[R + exp(1.29649 + 0.25 \cdot \mathbf{M})\right] + f \cdot \ln(R+2) \end{aligned} \qquad (4\text{-}8)$$

M ≥ 6.5:

$$ln(SD) = a_{SS} \cdot G_{SS} + a_{RS} \cdot G_{RS} + 1.1\mathbf{M} + b(8.5 - \mathbf{M})^{2.5} + c\, ln\left[R + exp(-0.48451 + 0.524\mathbf{M})\right] + f \cdot \ln(R + 2) \tag{4-9}$$

where:

SD	is the mean value of the seismic demand, PGA or spectral acceleration (S_A) in g
M	is the moment magnitude of the earthquake
R	is the distance, in km, to the closest point on the fault rupture surface (see Figure 4.4)
a_{SS}, a_{RS}	are coefficients for strike-slip and reverse-slip/thrust faults, respectively, as given in Table 4.5*
G_{SS}, G_{RS}	are fault-type flags: G_{SS} =1 for strike-slip faults, 0 otherwise; G_{RS} = 1 for reverse/thrust faults slip, 0 otherwise*
b, c, f	are coefficients given in Table 4.5

* Oblique faults are categorized as strike slip if the rake angle is within 30^o of horizontal; otherwise, they are defined as reverse slip. The Methodology uses the strike slip relationship for normal slip earthquakes.

Table 4.5 Sadigh et al. (1993) Coefficients - WUS Attenuation

Period	**a_{SS}**	**a_{RS}**	**b**	**c**
Earthquake Magnitude, M < 6.5				
PGA	-0.624	-0.442	0.0	-2.100
0.3	-0.057	0.125	-0.017	-2.028
1.0	-1.705	-1.523	-0.055	-1.800
Earthquake Magnitude, M ≥ 6.5				
PGA	-1.274	-1.092	0.0	-2.100
0.3	-0.707	-0.525	-0.017	-2.028
1.0	-2.355	-2.173	-0.055	-1.800

Sadigh 1993 limits the applicability of Equations 4-7 and 4-8 to earthquake magnitudes **M** ≤ 8.0. In the Methodology, seismic demand for magnitudes **M** > 8.0 is based on the Equations 4-7 and 4-8 predictions for **M** = 8.0.

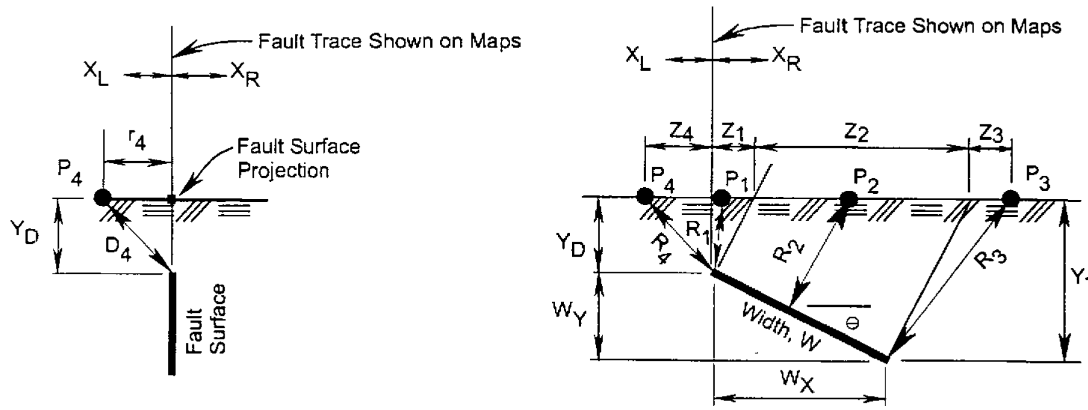


Figure 4.4 Measure of distance for vertical and dipping faults used in Sadigh et al. (1993) attenuation relationships.

Campbell and Bozorgnia (1994)

The Campbell and Bozorgnia (1994) attenuation relationship predicts mean values of PGA for source-to-site distances less than 60 kilometers. The Campbell and Bozorgnia 1994 relationship is given in Equation (4-10) for soft rock site conditions. Soft rock conditions are used by the Methodology for prediction of PGA at rock sites (Soil Class B) in the WUS.

$$\begin{aligned} ln(SD) &= -3.512 + 0.904 \quad - 1.328\, ln\sqrt{R^2 + [0.149 exp(0.647\mathbf{M})]^2} \\ &+ [1.125 - 0.112 ln(R) - 0.0957\mathbf{M}] \cdot G_{RS} + [0.440 - 0.171 ln(R)] \end{aligned} \qquad (4\text{-}10)$$

where:
- SD is mean value of the peak ground acceleration (g)
- **M** is the moment magnitude of the earthquake
- R is the closest distance, in km, to zone of seismogenic rupture on the fault (see Figure 4.5)
- G_{RS} is a fault type flag: G_{RS} = 1 for reverse-slip faults, 0 otherwise*

* Oblique faults are categorized as strike slip if the rake angle is within 30^o of horizontal; else they are defined as reverse slip. The Methodology uses the strike slip relationship for normal slip earthquakes.

The distance R (see Figure 4.5) is measured as the closest distance from the site to the zone of the seismogenic rupture. This definition assumes that fault rupture in the softer sediments of the upper 4 km of the fault is primarily non-seismogenic. The minimum depth is represented as Y_R in Figure 4.5. In the Methodology, Y_R is assumed to be a constant of 5 km. As shown in the figure, if Y_D is less than Y_R, distances are measured beginning with the 5 km depth. For Y_D greater than Y_R, distances are measured from the closest point on the fault.

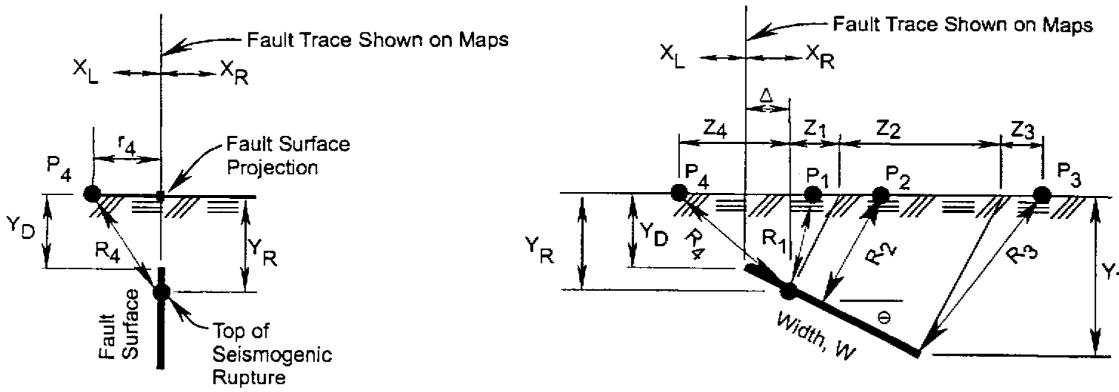


Figure 4.5 Measure of distance for vertical and dipping faults used in the Campbell & Bozorgnia (1994) attenuation relationship.

<u>Youngs, Chiou, Silva and Humphrey (1997)</u>

The Youngs, Chiou, Silva and Humphrey attenuation relationship (Youngs 1997) predicts PGA and spectral response at rock sites (Site Class B) for subduction zone earthquakes, differentiating between events which occur at the interface of subducting and overriding plates, *interface earthquakes*, and deep events which occur within the subducting plate, *intraslab earthquakes*. *Interface* earthquakes are typically high-angle normal faulting events. *Intraslab* events, as distinguished from shallow crustal earthquakes that occur in the upper 20 to 25 km of the crust, are relatively deep, shallow-angle thrust events. The attenuation relationships are valid for earthquakes of $\mathbf{M} \geq 5$ and for site-to-rupture surface distances of 10 to 500 km. The Youngs 1997 relationship is given in Equation (4-11).

$$\begin{aligned} ln(SD) = {} & a_{IF} \cdot G_{IF} + a_{IS} \cdot G_{IS} + 1.414\,\mathbf{M} + b(10 - \mathbf{M})^3 \\ & + c \cdot ln(R + 1.782e^{0.554\mathbf{M}}) + 0.00607H \end{aligned} \qquad (4\text{-}11)$$

where:	SD	is the mean value of seismic demand, PGA or spectral acceleration (S_A) in g
	M	is the moment magnitude of the earthquake
	R	is the distance, in km, to the closest point on the fault rupture surface (see Figure 4.4)
	a_{IF}, a_{IS}	are coefficients for *interface* and *intraslab* events, respectively, as given in Table 4.6
	G_{IF}, G_{IS}	are source-type flags: G_{IF} = 1 for *interface* events, 0 otherwise; G_{IS} = 1 for *intraslab* events, 0 otherwise
	H	is the focal depth (depth to the hypocenter), in km

Table 4.6 Youngs et al. (1997) Coefficients - WUS Attenuation

Period	a_{IF}	a_{IS}	b	c
PGA	0.2418	0.6264	0.0	-2.552
0.3	0.4878	0.8724	-0.0036	-2.454
1.0	-1.494	-1.1096	-0.0064	-2.234

Default Combination of Attenuation Functions - WUS

The Methodology provides a default combination of WUS attenuation functions based on the theory developed by the USGS for Project 97. The weighting rules for default combinations of attenuation functions are summarized below.

- **Western US Shallow Crustal Events**
  Peak Ground Acceleration:
    one-third BJF 1994 relationship
    one-third Sadigh 1993 relationship
    one-third Campbell & Bozorgnia 1994 relationship
    (for r > 60 km, one-half BJF 1994 and one-half Sadigh 1993; for **M** > 7.7 BJF 1994 is not used)
  Spectral Acceleration:
    one-half BJF 1994 relationship
    one-half Sadigh 1993 relationship
    (for **M** > 7.7 only Sadigh 1993 is used)

- **Deep Events (e.g., Puget Sound Earthquakes > 50 km in depth):**
    Youngs 1997 - *Intraslab* relationship

- **Cascadia Subduction Zone**:
    one-half Youngs 1997 - *interface* relationship
    one-half Sadigh 1993 - reverse-slip relationship
    (only Youngs 1997 is used for magnitudes greater than **M** = 8.0)

- **Hawaiian Events (M<7.0)**
  Peak Ground Acceleration:
    one-fourth BJF 1994 relationship
    one-fourth Sadigh 1993 relationship

one-fourth Campbell & Bozorgnia 1994 relationship
one-fourth Munson & Thurber 1997 relationship

Spectral Acceleration:

0.3 Seconds

one-third BJF 1994 relationship
one-third Sadigh 1993 relationship
one-third 2.5*(Munson & Thurber 1997 relationship)

1.0 Seconds

one-half BJF 1994 relationship
one-half Sadigh 1993 relationship

- **Hawaiian Events (M≥7.0)**

Peak Ground Acceleration:

one-half Sadigh 1993 relationship
one-half Munson & Thurber 1997 relationship

Spectral Acceleration:

0.3 Seconds

one-half Sadigh 1993 relationship
one-half 2.5*(Munson & Thurber 1997 relationship)

1.0 Seconds

Sadigh 1993 relationship

Eastern United States Attenuation Relationships

The Central and Eastern U.S. (CEUS) attenuation relationships provided with the Methodology are based on:

- Frankel et al. (Appendix C, Frankel et al., 1996)
- Toro, Abrahamson and Schneider (1997)
- Lawrence Livermore National Laboratory (Savy, 1998)

For the Eastern United States, the ground shaking attenuation relationships for PGA and spectral acceleration demand are derived from theoretical models, as described in Frankel et al. (1996), Toro, Abrahamson and Schneider (1997) and Savy (1998). The Frankel et al. (1996) attenuation relationship was developed specifically for Project 97. The Toro, Abrahamson and Schneider (1997) relationship was obtained from a paper submitted for publication to *Earthquake Spectra.* This paper summarizes work of a 1993 study performed by the authors for the Electric Power Research Institute (Toro et al., 1997). Savy (1998) describes the SSHAC expert elicitation methodology used by Lawrence Livermore National Laboratory to develop an attenuation model for hard rock sites in the Eastern United States.

Frankel et al. (1996)

The Frankel et al. attenuation relationship (Frankel 1996) predicts PGA and 0.3-second and 1.0-second spectral acceleration response based on simulations of a random vibration stochastic model. Appendix 4A includes tables of mean demand values as published in Frankel et al., 1996, resulting from averaging multiple simulations. Linear interpolation was used to calculate ground motion values for certain magnitudes and distances. These values predict demand for specific event magnitudes ranging from **M** = 5.0 to **M** = 8.0 and hypocentral distances ranging from 10 km to 350 km.

The user must specify the hypocentral depth for the Methodology to calculate the hypocentral distance. If not provided by the user, the Methodology assumes a hypocentral depth of 10 km, consistent with the theory of Project 97. Similarly, the Methodology limits the hypocentral distance to a minimum value of 10 km, and limits predicted values of PGA to 1.5g and predicted values of 0.3-second spectral acceleration to 3.75g, consistent with Project 97 theory.

Toro, Abrahamson & Schneider (1997)

The Toro, Abrahamson and Schneider (1997) attenuation relationship predicts PGA, and spectral acceleration for hard rock sites (Site Class A) in the CEUS. For use in the Methodology, the Toro 1997 attenuation relationship includes the following modifications:

- a factor (F_{AB}) is added to increase hard rock (Site Class A) predictions to a level that represents Site Class B (rock) conditions, based on the theory of Project 97
- the hypocentral distance term, R_M, is adjusted (i.e., R_M is replaced by R_M + $0.089e^{0.6\mathbf{M}}$) to model the saturation effect of extended ruptures on near-fault ground-motion, based on private communication with the authors and previous work by Toro and McGuire (1991)

The Toro 1997 relationship is given in Equation (4-12) with the modified hypocentral distance defined by Equation (4-13).

$$\begin{aligned} ln(SD) &= a + b(\mathbf{M}-6) + c(\mathbf{M}-6)^2 - d \cdot ln\left(R_{\mathbf{M}}\right) \\ &\quad - (e-d)\, max\left[ln\left(\frac{R_{\mathbf{M}}}{100}\right), 0\right] - f \cdot R_{\mathbf{M}} + F_{AB} \end{aligned} \tag{4-12}$$

$$R_{\mathbf{M}} = \sqrt{r^2 + h^2} + 0.089\exp(0.6\mathbf{M}) \tag{4-13}$$

where: SD is the mean value of the seismic demand, PGA or spectral acceleration (S_A) in g

M	is the moment magnitude of the earthquake
r	is the closest horizontal distance to the fault rupture (km)
a,b,c,d,e,f,h	are coefficients given in Table 4.7
F_{AB}	is a factor converting predicted PGA and spectral response values from hard rock (Site Class A) to Site Class B (rock) conditions, based on the theory of Frankel et al. 1996.

Table 4.7 Toro 1997 Coefficients - CEUS Attenuation

Period	a	b	c	d	e	f	h	F_{AB}
0.3	1.40	0.945	-0.05	0.955	0.61	0.0038	7.3	ln(1.72)
1.0	0.09	1.42	-0.20	0.90	0.49	0.0023	6.8	ln(1.34)
PGA	2.20	0.81	0.00	1.27	1.16	0.0021	9.4	ln(1.52)

Toro et al. (1997) provides coefficients for spectral acceleration response at periods of 0.2 and 0.4 second, but not at a period of 0.3 second. Coefficients given in Table 4.7 for 0.3-second spectral acceleration response are based on linear interpolation between coefficients for spectral acceleration response at 0.2 and 0.4 seconds.

<u>Lawrence Livermore National Laboratory (Savy, 1998)</u>

The Lawrence Livermore National Laboratory attenuation relationship (Savy, 1998) predicts peak ground acceleration and 5%-damped spectral acceleration for hard rock sites. The relationship is given in Equation (4-14) for the entire range of magnitudes. The coefficients in Table 4.8 are established for two magnitude ranges.

$$ln(SD) = a \cdot + b \cdot (\mathbf{M} - 6.25) + c(8.5 - \mathbf{M})^{2.} + \left[d + e(\mathbf{M} - 6.25)\right] * ln\left[\sqrt{(R)^{2.} + (h)^{2.}}\right] + e * F \qquad (4\text{-}14)$$

where:

SD	is the mean value of the seismic demand, PGA or spectral acceleration (S_A) in g
M	is the moment magnitude of the earthquake
R	is the distance, in km, from the site to the fault rupture (assmed to be hypocentral distance)
F	is the source mechanism flag: F = 0 for strike-slip faults; F = 1 for normal faults
a, b, c, d,e,h	are coefficients given in Table 4.8

The LLNL attenuation relationship provides coefficients for spectral acceleration response at periods of 0.1 and 0.4 second, but not at a period of 0.3 second. The values computed for the 0.3-second spectral acceleration response are based on linear interpolation between coefficients for spectral acceleration response at 0.1 and 0.4 seconds. Since the current version of **HAZUS** does not distinguish between source mechanisms for earthquakes in the Central and Eastern United States, the values computed for PGA and spectral acceleration response are the average between the strike slip and normal mechanisms.

Table 4.8 Lawrence Livermore National Laboratory Coefficients – CEUS Attenuation (Savy, 1998)

Period	a	b	c	d	e	e	h
Earthquake Magnitude, M < 6.25							
PGA	3.267	0.294	0.000	-1.446	0.146	0.015	9.2
0.1	3.580	0.294	-0.008	-1.354	0.146	0.021	9.1
0.4	2.349	0.294	-0.072	-1.138	0.146	0.065	7.7
1.0	1.464	0.294	-0.136	-1.061	0.146	-0.012	7.0
Earthquake Magnitude, M ≥ 6.25							
PGA	3.267	0.127	0.000	-1.446	0.146	0.015	9.2
0.1	3.580	0.127	-0.008	-1.354	0.146	0.021	9.1
0.4	2.349	0.127	-0.072	-1.138	0.146	0.065	7.7
1.0	1.464	0.127	-0.136	-1.061	0.146	-0.012	7.0

Default Combination of Attenuation Functions - CEUS

The Methodology provides a default combination of CEUS attenuation functions based on the theory developed by the USGS for Project 97. The Lawrence Livermore National Laboratory relationship was not used by the USGS in Project 97. The weighting rules for default combinations of attenuation functions are summarized below.

- Peak Ground Acceleration:
  one-half Frankel 1996 relationship
  one-half Toro 1997 relationship

- Spectral Acceleration:
  one-half Frankel 1996 relationship
  one-half Toro 1997 relationship

The default combination of CEUS attenuation functions predict significantly stronger ground shaking than the default combination of WUS attenuation functions for the same scenario earthquake (i.e., same moment magnitude and distance to source). For example, Figure 4.6 compares WUS and CEUS rock (Site Class B) response spectra (standard shape) for a magnitude **M** = 7.0 earthquake at 20 km from the source. As illustrated in

Figure 4.6, CEUS spectral demand is about 2.0 times WUS demand in the acceleration domain and between 1.5 to 2.0 times WUS demand in the velocity domain.

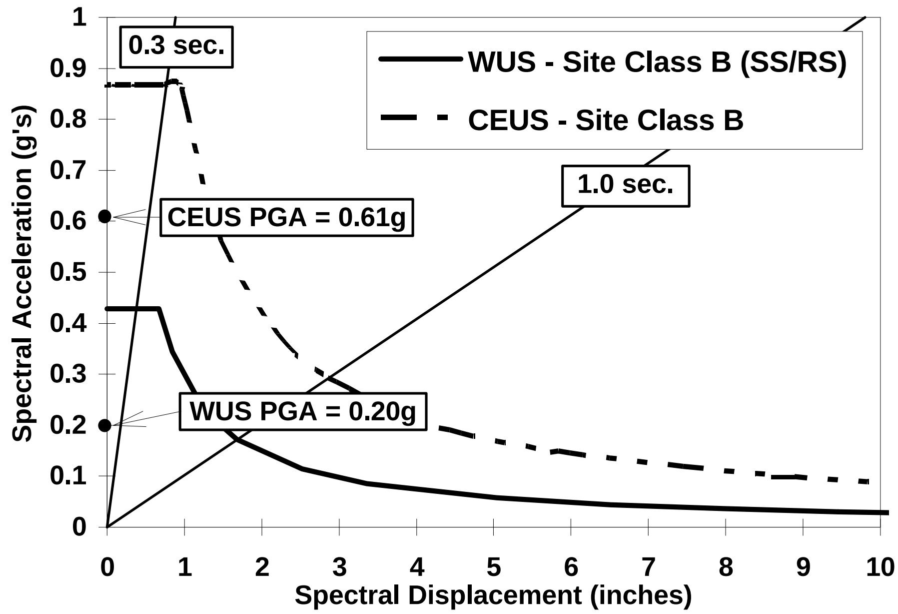


Figure 4.6 Example Comparison of WUS and CEUS Spectra - Site Class B (M = 7.0 at 20 km - Default Combination of Attenuation).

4.1.2.4 Amplification of Ground Shaking - Local Site Conditions

Amplification of ground shaking to account for local site conditions is based on the site classes and soil amplification factors proposed for the *1997 NEHRP Provisions* (which are essentially the same as the 1994 *NEHRP Provisions*, FEMA 222A, 1995). The *NEHRP Provisions* define a standardized site geology classification scheme and specify soil amplification factors for most site classes. The classification scheme of the NEHRP Provisions is based, in part, on the average shear wave velocity of the upper 30 meters of the local site geology, as shown in Table 4.9. Users (with geotechnical expertise) are required to relate the soil classification scheme of soil maps to the classification scheme shown in Table 4.9.

Table 4.9 Site Classes (from the 1997 *NEHRP Provisions*)

Site Class	Site Class Description	Shear Wave Velocity (m/sec)	
		Minimum	**Maximum**
A	**HARD ROCK** Eastern United States sites only	1500	
B	**ROCK**	760	1500
C	**VERY DENSE SOIL AND SOFT ROCK** Untrained shear strength $u_s \geq 2000$ psf ($u_s \geq 100$ kPa) or $N \geq 50$ blows/ft	360	760
D	**STIFF SOILS** Stiff soil with undrained shear strength 1000 psf $\leq u_s \leq 2000$ psf (50 kPa $\leq u_s \leq$ 100 kPa) or $15 \leq N \leq 50$ blows/ft	180	360
E	**SOFT SOILS** Profile with more than 10 ft (3 m) of soft clay defined as soil with plasticity index PI > 20, moisture content *w* > 40% and undrained shear strength $u_s < 1000$ psf (50 kPa) (N < 15 blows/ft)		180
F	**SOILS REQUIRING SITE SPECIFIC EVALUATIONS** **1. Soils vulnerable to potential failure or collapse under seismic loading:** e.g. liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. **2. Peats and/or highly organic clays** (10 ft (3 m) or thicker layer) **3. Very high plasticity clays:** (25 ft (8 m) or thicker layer with plasticity index >75) **4. Very thick soft/medium stiff clays:** (120 ft (36 m) or thicker layer)		

Soil amplification factors are provided in Table 4.10 for Site Classes A, B, C, D and E. No amplification factors are available for Site Class F, which requires special site-specific geotechnical evaluation and is not used in the Methodology.

Table 4.10 Soil Amplification Factors

Site Class B Spectral Acceleration	Site Class				
	A	**B**	**C**	**D**	**E**
Short-Period, S_{AS} (g)	**Short-Period Amplification Factor, F_A**				
≤ 0.25	0.8	1.0	1.2	1.6	2.5
0.50	0.8	1.0	1.2	1.4	1.7
0.75	0.8	1.0	1.1	1.2	1.2
1.0	0.8	1.0	1.0	1.1	0.9
≥ 1.25	0.8	1.0	1.0	1.0	0.8*
1-Second Period, S_{A1} (g)	**1.0-Second Period Amplification Factor, F_V**				
≤ 0.1	0.8	1.0	1.7	2.4	3.5
0.2	0.8	1.0	1.6	2.0	3.2
0.3	0.8	1.0	1.5	1.8	2.8
0.4	0.8	1.0	1.4	1.6	2.4
≥ 0.5	0.8	1.0	1.3	1.5	2.0*

* Site Class E amplification factors are not provided in the *NEHRP Provisions when* $S_{AS} > 1.0$ *or* $S_{A1} > 0.4$. Values shown with an asterisk are based on judgment.

The *NEHRP Provisions* do not provide soil amplification factors for PGA or PGV. The Methodology amplifies rock (Site Class B) PGA by the same factor as that specified in Table 4.10 for short-period (0.3-second) spectral acceleration, as expressed in Equation (4-15), and amplifies rock (Site Class B) PGV by the same factor as that specified in Table 4.10 for 1.0-second spectral acceleration, as expressed in Equations (4-16).

$$PGA_i = PGA \cdot F_{Ai} \tag{4-15}$$

$$PGV_i = PGV \cdot F_{Vi} \tag{4-16}$$

where:

- PGA_i is peak ground acceleration for Site Class i (in units of g)
- PGA is peak ground acceleration for Site Class B (in units of g)
- F_{Ai} is the short-period amplification factor for Site Class i, as specified in Table 4.10 for spectral acceleration, S_{AS}
- PGV_i is peak ground acceleration for Site Class i (in units of g)
- PGV is peak ground acceleration for Site Class B (in units of g)
- F_{Vi} is the 1-second period amplification factor for Site Class i, as specified in Table 4.10 for spectral acceleration, S_{A1}

Construction of Demand Spectra

Demand spectra including soil amplification effects are constructed at short-periods using Equation (4-17) and at long-periods using Equation (4-18). The period, T_{AV}, which defines the transition period from constant spectral acceleration to constant spectral velocity is a function of site class, as given in Equation (4-19). The period, T_{VD}, which defines the transition period from constant spectral velocity to constant spectral displacement is defined by Equation (4-4), and is not a function of site class.

$$S_{ASi} = S_{AS} \cdot F_{Ai} \tag{4-17}$$

$$S_{A1i} = S_{A1} \cdot F_{Vi} \tag{4-18}$$

$$T_{AVi} = \left(\frac{S_{A1}}{S_{AS}}\right)\left(\frac{F_{Vi}}{F_{Ai}}\right) \tag{4-19}$$

where:

- S_{ASi} is short-period spectral acceleration for Site Class i (in units of g)
- S_{AS} is short-period spectral acceleration for Site Class B (in units of g)
- F_{Ai} is the short-period amplification factor for Site Class i, as specified in Table 4.10 for spectral acceleration, S_{AS}
- S_{A1i} is 1-second period spectral acceleration for Site Class i (in units of g)
- S_{A1} is 1-second period spectral acceleration for Site Class B (in units of g)
- F_{Vi} is the 1-second period amplification factor for Site Class i, as specified in Table 4.10 for spectral acceleration, S_{A1}
- T_{AVi} is the transition period between constant spectral acceleration and constant spectral velocity for Site Class i (sec).

Figure 4.7 illustrates construction of response spectra for Site Class D (stiff soil) and E (soft soil) from Site Class B (rock) response spectra. These spectra represent response (of a 5%-damped, linear-elastic single-degree-of-freedom system) located at a WUS site, 20 km from a magnitude **M** = 7.0 earthquake, as predicted by the default combination of WUS attenuation relationships. Figure 4.7 shows the significance of soil type on site response (i.e., increase in site response with decrease in shear wave velocity) and the increase in the value of the transition period, T_{AV}, with decrease in shear wave velocity.

4.1.3 Guidance for Expert-Generated Ground Motion Estimation

Ground motion estimation is a sophisticated combination of earth science, engineering and probabilistic methods and should not be attempted by users, including local geotechnical engineers, who not have the proper expertise. It is assumed that any user sufficiently qualified to estimate ground motion would not need additional guidance.

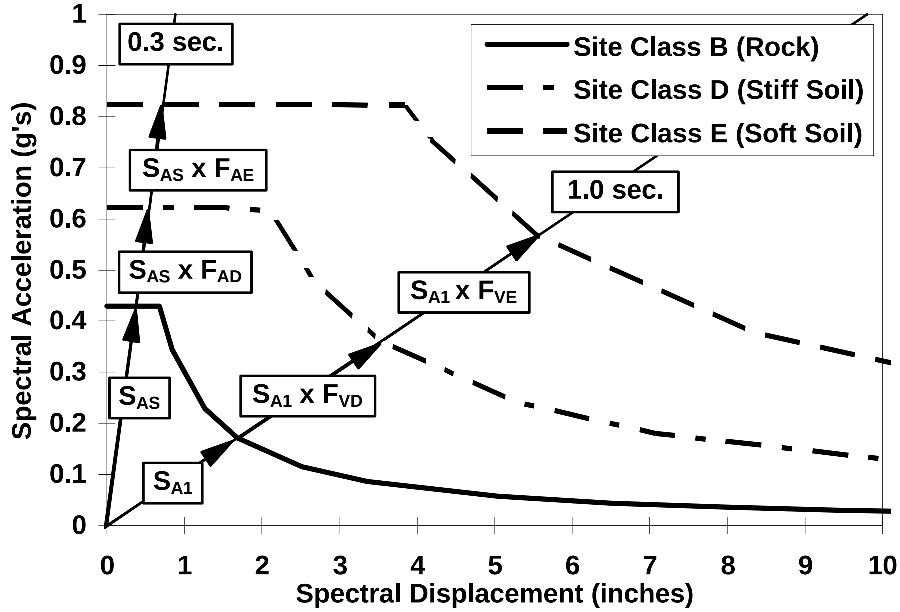


Figure 4.7 Example Construction of Site Class B, C and D Spectra - WUS (M = 7.0 at 20 km - Default Combination of Attenuation).

4.2 Ground Failure

4.2.1 Introduction

Three types of ground failure are considered: liquefaction, landsliding and surface fault rupture. Each of these types of ground failure are quantified by permanent ground deformation (PGD). Methods and alternatives for determining PGD due to each mode of ground failure are discussed below.

4.2.1.1 Scope

The scope of this section is to provide methods for evaluating the ground failure hazards of: (a) liquefaction, (b) landsliding, and (c) surface fault rupture. The evaluation of the hazard includes the probability of the hazard occurring and the resulting ground displacement.

4.2.1.2 Input Requirements and Output Information

__Input__

Liquefaction

- A geologic map based on the age, depositional environment, and possibly the material characteristics of the geologic units will be used with Table 4.11 to create a liquefaction susceptibility map
- Groundwater depth map is supplied with a default depth of 5 feet.

- Earthquake Moment Magnitude (**M**)

Landsliding

- A geologic map, a topographic map, and a map with ground water conditions will be used with Table 4.16 to produce a landslide susceptibility map
- Earthquake Moment Magnitude (**M**)

Surface Fault Rupture

- Location of the surface trace of a segment of an active fault that is postulated to rupture during the scenario earthquake

<u>Output</u>

Liquefaction, and Landsliding

- Aerial depiction map depicting estimated permanent ground deformations.

Surface Fault Rupture

- No maps are generated, only site-specific demands are determined.

4.2.2 Description of Methods

4.2.2.1 Liquefaction

4.2.2.1.1 Background

Liquefaction is a soil behavior phenomenon in which a saturated soil looses a substantial amount of strength due to high excess pore-water pressure generated by and accumulated during strong earthquake ground shaking.

Youd and Perkins (1978) have addressed the liquefaction susceptibility of various types of soil deposits by assigning a qualitative susceptibility rating based upon general depositional environment and geologic age of the deposit. The relative susceptibility ratings of Youd and Perkins (1978) shown in Table 4.11 indicate that recently deposited relatively unconsolidated soils such as Holocene-age river channel, flood plain, and delta deposits and uncompacted artificial fills located below the groundwater table have high to very high liquefaction susceptibility. Sands and silty sands are particularly susceptible to liquefaction. Silts and gravels also are susceptible to liquefaction, and some sensitive clays have exhibited liquefaction-type strength losses (Updike, et. al., 1988).

Permanent ground displacements due to lateral spreads or flow slides and differential settlement are commonly considered significant potential hazards associated with liquefaction.

4.2.2.1.2 Liquefaction Susceptibility

The initial step of the liquefaction hazard evaluation is to characterize the relative liquefaction susceptibility of the soil/geologic conditions of a region or subregion. Susceptibility is characterized utilizing geologic map information and the classification system presented by Youd and Perkins (1978) as summarized in Table 4.11. Large-scale (e.g., 1:24,000 or greater) or smaller-scale (e.g., 1:250,000) geologic maps are generally available for many areas from geologists at regional U.S. Geological Survey offices, state geological agencies, or local government agencies. The geologic maps typically identify the age, depositional environment, and material type for a particular mapped geologic unit. Based on these characteristics, a relative liquefaction susceptibility rating (e.g., very low to very high) is assigned from Table 4.11 to each soil type. Mapped areas of geologic materials characterized as rock or rock-like are considered for the analysis to present no liquefaction hazard.

Table 4.11 Liquefaction Susceptibility of Sedimentary Deposits (from Youd and Perkins, 1978)

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments when Saturated would be Susceptible to Liquefaction (by Age of Deposit)			
		< 500 yr Modern	Holocene < 11 ka	Pleistocene 11 ka - 2 Ma	Pre-Pleistocene > 2 Ma
(a) Continental Deposits					
River channel	Locally variable	Very High	High	Low	Very Low
Flood plain	Locally variable	High	Moderate	Low	Very Low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very Low
Marine terraces and plains	Widespread	---	Low	Very Low	Very Low
Delta and fan-delta	Widespread	High	Moderate	Low	Very Low
Lacustrine and playa	Variable	High	Moderate	Low	Very Low
Colluvium	Variable	High	Moderate	Low	Very Low
Talus	Widespread	Low	Low	Very Low	Very Low
Dunes	Widespread	High	Moderate	Low	Very Low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very Low	Very Low
Tuff	Rare	Low	Low	Very Low	Very Low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very Low	Very Low
Sebka	Locally variable	High	Moderate	Low	Very Low
(b) Coastal Zone					
Delta	Widespread	Very High	High	Low	Very Low
Esturine	Locally variable	High	Moderate	Low	Very Low
Beach					
High Wave Energy	Widespread	Moderate	Low	Very Low	Very Low
Low Wave Energy	Widespread	High	Moderate	Low	Very Low
Lagoonal	Locally variable	High	Moderate	Low	Very Low
Fore shore	Locally variable	High	Moderate	Low	Very Low
(c) Artificial					
Uncompacted Fill	Variable	Very High	---	---	---
Compacted Fill	Variable	Low	---	---	---

Liquefaction susceptibility maps produced for certain regions [e.g., greater San Francisco region (ABAG, 1980); San Diego (Power, et. al., 1982); Los Angeles (Tinsley, et. al., 1985); San Jose (Power, et. al., 1991); Seattle (Grant, et. al., 1991); among others] are also available and may alternatively be utilized in the hazard analysis.

4.2.2.1.3 Probability of Liquefaction

The likelihood of experiencing liquefaction at a specific location is primarily influenced by the susceptibility of the soil, the amplitude and duration of ground shaking and the depth of groundwater. The relative susceptibility of soils within a particular geologic unit is assigned as previously discussed. It is recognized that in reality, natural geologic deposits as well as man-placed fills encompass a range of liquefaction susceptibilities due to variations of soil type (i.e., grain size distribution), relative density, etc. Therefore, portions of a geologic map unit may not be susceptible to liquefaction, and this should be considered in assessing the probability of liquefaction at any given location within the unit. In general, we expect non-susceptible portions to be smaller for higher susceptibilities. This "reality" is incorporated by a probability factor that quantifies the proportion of a geologic map unit deemed susceptible to liquefaction (i.e., the likelihood of susceptible conditions existing at any given location within the unit). For the various susceptibility categories, suggested default values are provided in Table 4.12.

Table 4.12 Proportion of Map Unit Susceptible to Liquefaction

Mapped Relative Susceptibility	Proportion of Map Unit
Very High	0.25
High	0.20
Moderate	0.10
Low	0.05
Very Low	0.02
None	0.00

These values reflect judgments developed based on preliminary examination of soil properties data sets compiled for geologic map units characterized for various regional liquefaction studies (e.g., Power, et. al., 1992; Geomatrix, 1993).

As previously stated, the likelihood of liquefaction is significantly influenced by ground shaking amplitude (i.e., peak horizontal acceleration, PGA), ground shaking duration as reflected by earthquake magnitude, **M**, and groundwater depth. Thus, the probability of liquefaction for a given susceptibility category can be determined by the following relationship:

$$P\left[\mathrm{Liquefaction}_{SC}\right] = \frac{P\left[\mathrm{Liquefaction}_{SC} \middle| PGA = a\right]}{K_M \cdot K_w} \cdot P_{ml} \qquad (4\text{-}20)$$

where

$P\left[\text{Liquefaction}_{SC}\middle|PGA = a\right]$ is the conditional liquefaction probability for a given susceptibility category at a specified level of peak ground acceleration (See Figure 4.8)

K_M is the moment magnitude (**M**) correction factor (Equation 4-21)

K_w is the ground water correction factor (Equation 4-22)

P_{ml} proportion of map unit susceptible to liquefaction (Table 4.12)

Relationships between liquefaction probability and peak horizontal ground acceleration (PGA) are defined for the given susceptibility categories in Table 4.13 and also represented graphically in Figure 4.8. These relationships have been defined based on the state-of-practice empirical procedures, as well as the statistical modeling of the empirical liquefaction catalog presented by Liao, et. al. (1988) for representative penetration resistance characteristics of soils within each susceptibility category (See Section 4.2.3.2.3) as gleaned from regional liquefaction studies cited previously. Note that the relationships given in Figure 4.8 are simplified representations of the relationships that would be obtained using Liao, et al. (1988) or empirical procedures.

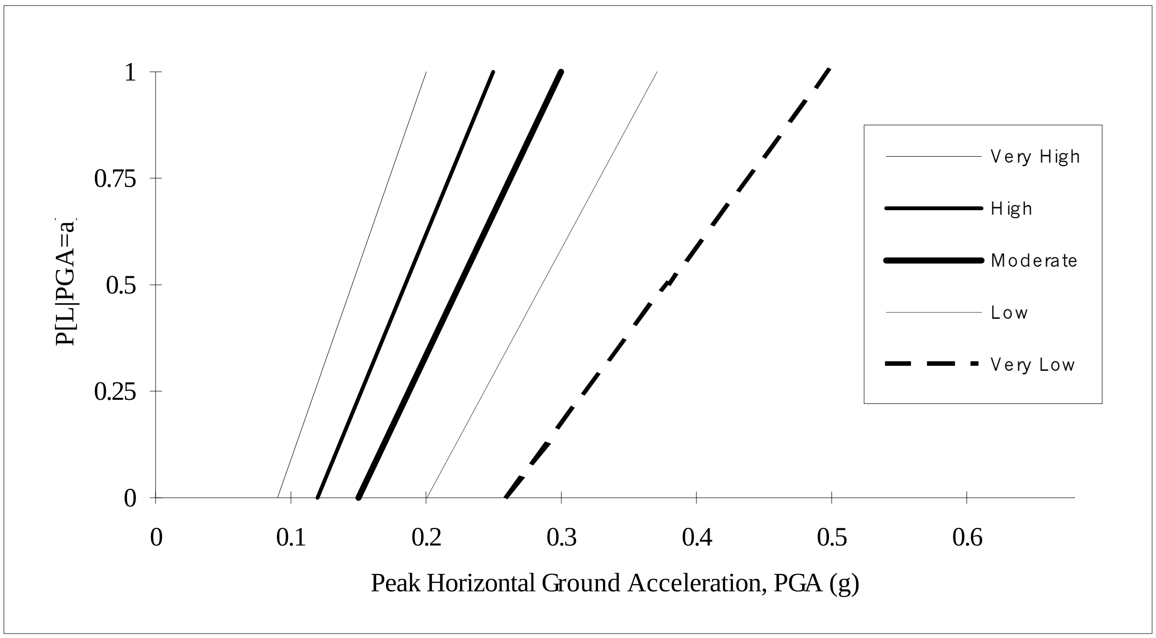


Figure 4.8 Conditional Liquefaction Probability Relationships for Liquefaction Susceptibility Categories (after Liao, et. al., 1988).

Table 4.13 Conditional Probability Relationship for Liquefaction Susceptibility Categories

Susceptibility Category	$P\left[\text{Liquefaction} \mid \text{PGA} = a\right]$
Very High	$0 \le 9.09\,a - 0.82 \le 1.0$
High	$0 \le 7.67a - 0.92 \le 1.0$
Moderate	$0 \le 6.67a - 1.0 \le 1.0$
Low	$0 \le 5.57a - 1.18 \le 1.0$
Very Low	$0 \le 4.16a - 1.08 \le 1.0$
None	0.0

The conditional liquefaction probability relationships presented in Figure 4.8 were developed for a **M** =7.5 earthquake and an assumed groundwater depth of five feet Correction factors to account for other moment magnitudes (**M**) and groundwater depths are given by Equations 4-21 and 4-22 respectively. These modification factors are well recognized and have been explicitly incorporated in state-of-practice empirical procedures for evaluating the liquefaction potential (Seed and Idriss, 1982; Seed, et. al., 1985; National Research Council, 1985). These relationships are also presented graphically in Figures 4.9 and 4.10. The magnitude and groundwater depth corrections are made automatically in the methodology. The modification factors can be computed using the following relationships:

$$K_m = 0.0027\mathbf{M}^3 - 0.0267\mathbf{M}^2 - 0.2055\mathbf{M} + 2.9188 \qquad (4\text{-}21)$$

$$K_w = 0.022 d_w + 0.93 \qquad (4\text{-}22)$$

where: K_m is the correction factor for moment magnitudes other than M=7.5;
K_w is the correction factor for groundwater depths other than five feet;
M represents the magnitude of the seismic event, and;
d_W represents the depth to the groundwater in feet.

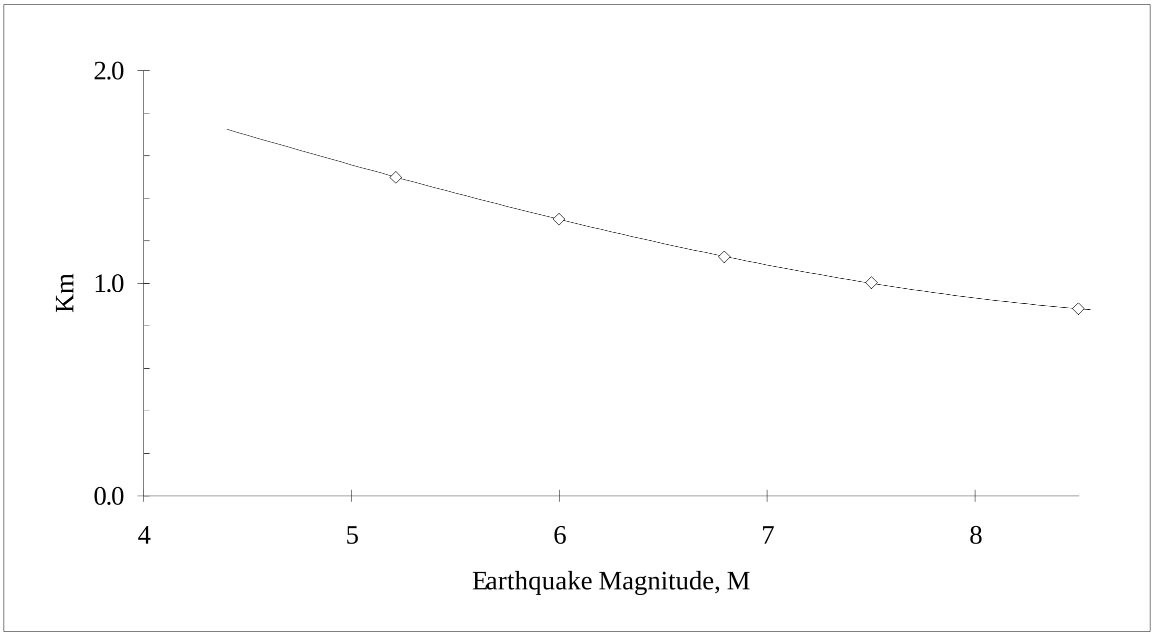


Figure 4.9 Moment Magnitude (M) Correction Factor for Liquefaction Probability Relationships (after Seed and Idriss, 1982).

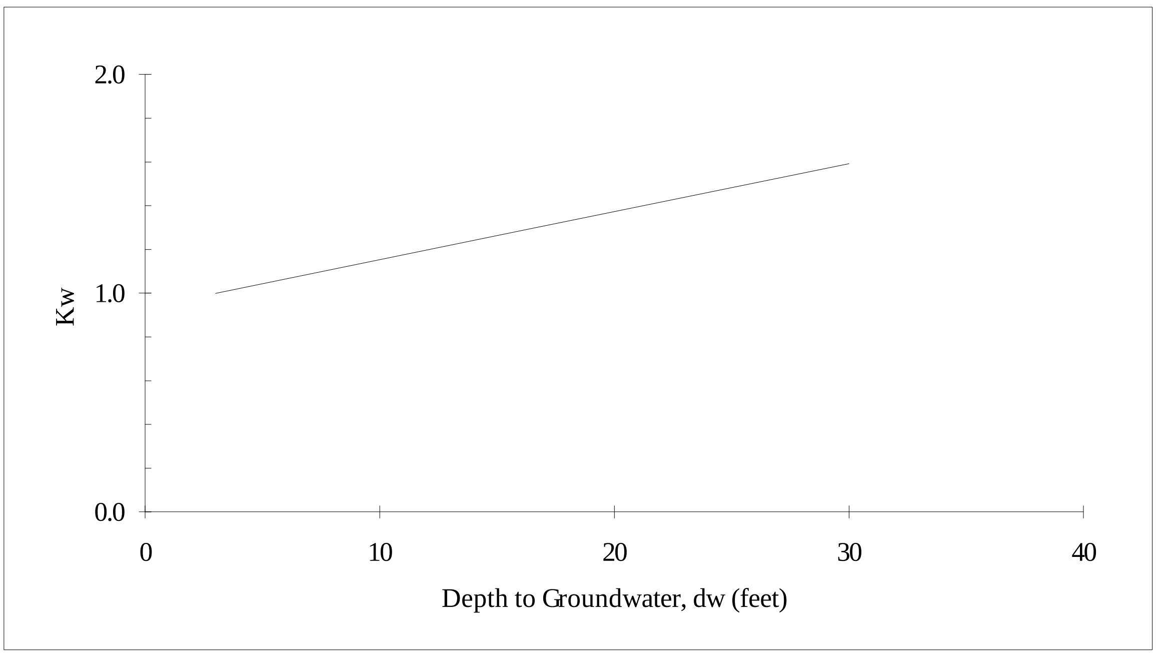


Figure 4.10 Ground Water Depth Correction Factor for Liquefaction Probability Relationships.

4.2.2.1.4 Permanent Ground Displacements

Lateral Spreading

The expected permanent ground displacements due to lateral spreading can be determined using the following relationship:

$$E\left[PGD_{SC}\right] = K_{\Delta} \cdot E\left[PGD \middle| \left(PGA / PL_{SC}\right) = a\right] \qquad (4\text{-}23)$$

where

$E\left[PGD \middle| \left(PGA / PL_{SC}\right) = a\right]$ is the expected permanent ground displacement for a given susceptibility category under a specified level of normalized ground shaking (PGA/PGA(t)) (Figure 4.11)

PGA(t) is the threshold ground acceleration necessary to induce liquefaction (Table 4.14)

K_{Δ} is the displacement correction factor given by Equation 4-24

This relationship for lateral spreading was developed by combining the Liquefaction Severity Index (LSI) relationship presented by Youd and Perkins (1987) with the ground motion attenuation relationship developed by Sadigh, et. al. (1986) as presented in Joyner and Boore (1988). The ground shaking level in Figure 4.11 has been normalized by the threshold peak ground acceleration PGA(t) corresponding to zero probability of liquefaction for each susceptibility category as shown on Figure 4.8. The PGA(t) values for different susceptibility categories are summarized in Table 4.14.

The displacement term, $E\left[PGD \middle| \left(PGA / PL_{SC}\right) = a\right]$, in Equation 4-23 is based on **M** = 7.5 earthquakes. Displacements for other magnitudes are determined by modifying this displacement term by the displacement correction factor given by Equation 4-24. This equation is based on work done by Seed & Idriss (1982). The displacement correction factor, K_{Δ}, is shown graphically in Figure 4.12.

$$K_{\Delta} = 0.0086\mathbf{M}^3 - 0.0914\mathbf{M}^2 + 0.4698\mathbf{M} - 0.9835 \qquad (4\text{-}24)$$

where **M** is moment magnitude.

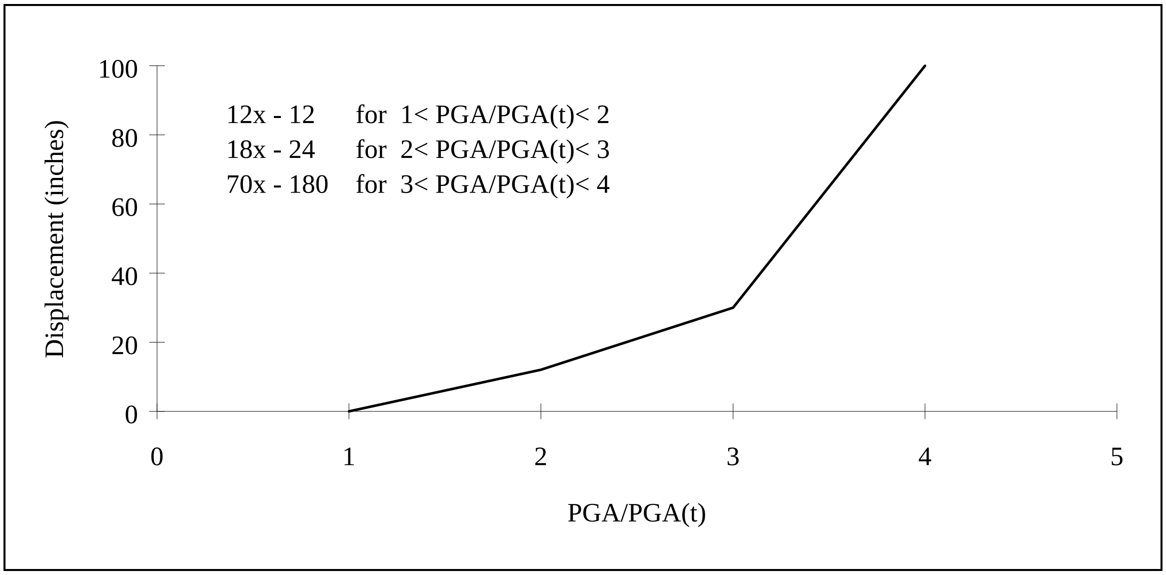


Figure 4.11 Lateral Spreading Displacement Relationship (after Youd and Perkins, 1978; Sadigh, et. al., 1986).

Table 4.14 Threshold Ground Acceleration (PGA(t)) Corresponding to Zero Probability of Liquefaction

Susceptibility Category	PGA(t)
Very High	0.09g
High	0.12g
Moderate	0.15g
Low	0.21g
Very Low	0.26g
None	N/A

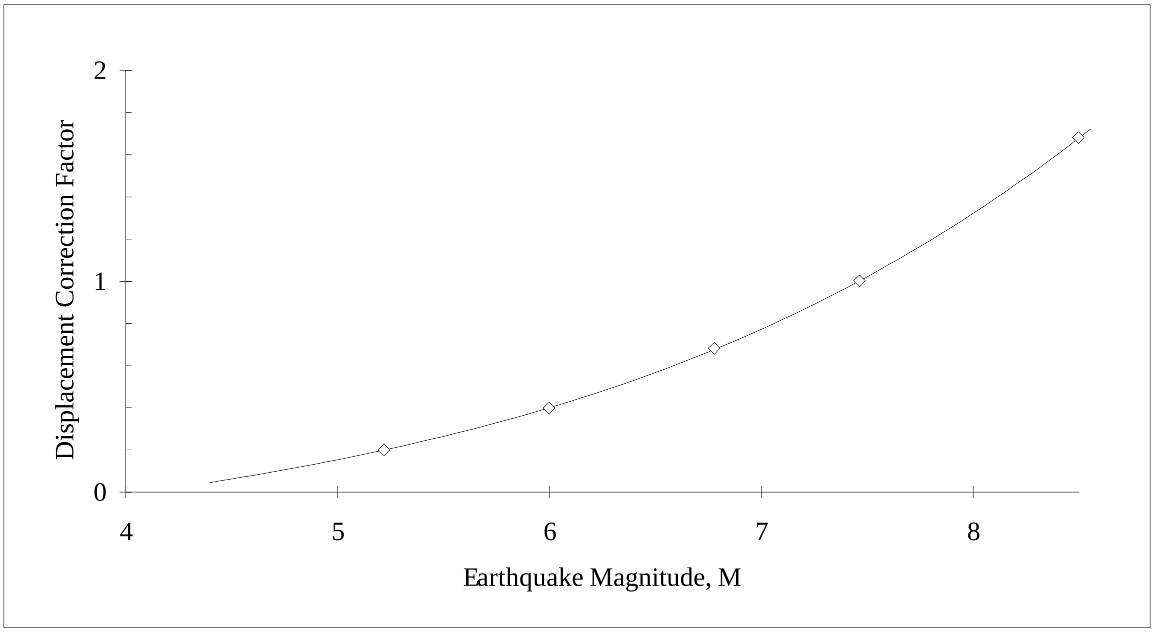


Figure 4.12 Displacement Correction Factor, K_Δ, for Lateral Spreading Displacement Relationships (after Seed & Idriss, 1982).

Ground Settlement

Ground settlement associated with liquefaction is assumed to be related to the susceptibility category assigned to an area. This assumption is consistent with relationships presented by Tokimatsu and Seed (1987) and Ishihara (1991) that indicate strong correlations between volumetric strain (settlement) and soil relative density (a measure of susceptibility). Additionally, experience has shown that deposits of higher susceptibility tend to have increased thicknesses of potentially liquefiable soils. Based on these considerations, the ground settlement amplitudes are given in Table 4.15 for the portion of a soil deposit estimated to experience liquefaction at a given ground motion level. The uncertainty associated with these settlement values is assumed to have a uniform probability distribution within bounds of one-half to two times the respective value. It is noted that the relationships presented by Tokimatsu and Seed (1987) and Ishihara (1991) demonstrate very little dependence of settlement on ground motion level given the occurrence of liquefaction. The expected settlement at a location, therefore, is the product of the probability of liquefaction (Equation 4-18) for a given ground motion level and the characteristic settlement amplitude appropriate to the susceptibility category (Table 4.15).

Table 4.15 Ground Settlement Amplitudes for Liquefaction Susceptibility Categories

Relative Susceptibility	Settlement (inches)
Very High	12
High	6
Moderate	2
Low	1
Very Low	0
None	0

4.2.2.2 Landslide

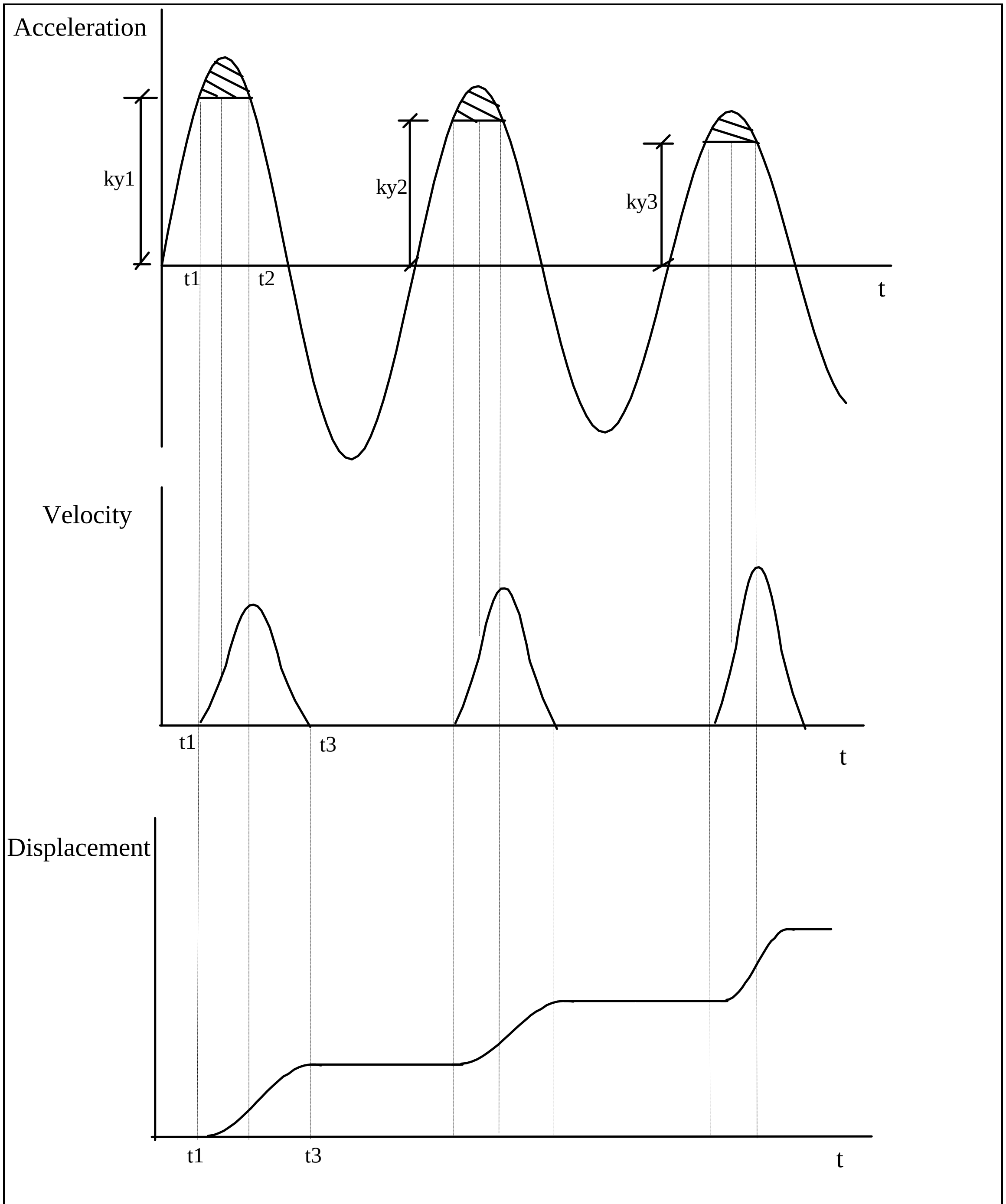
4.2.2.2.1 Background

Earthquake-induced landsliding of a hillside slope occurs when the static plus inertia forces within the slide mass cause the factor of safety to temporarily drop below 1.0. The value of the peak ground acceleration within the slide mass required to just cause the factor of safety to drop to 1.0 is denoted by the critical or yield acceleration a_c. This value of acceleration is determined based on pseudo-static slope stability analyses and/or empirically based on observations of slope behavior during past earthquakes.

Deformations are calculated using the approach originally developed by Newmark (1965). The sliding mass is assumed to be a rigid block. Downslope deformations occur during the time periods when the induced peak ground acceleration within the slide mass a_{is} exceeds the critical acceleration a_c. The accumulation of displacement is illustrated in Figure 4.13. In general, the smaller the ratio (below 1.0) of a_c to a_{is}, the greater is the number and duration of times when downslope movement occurs, and thus the greater is the total amount of downslope movement. The amount of downslope movement also depends on the duration or number of cycles of ground shaking. Since duration and number of cycles increase with earthquake magnitude, deformation tends to increase with increasing magnitude for given values of a_c and a_{is}.

4.2.2.2.2 Landslide Susceptibility

The landslide hazard evaluation requires the characterization of the landslide susceptibility of the soil/geologic conditions of a region or subregion. Susceptibility is



Note: Critical accelerations in figure are ky1, ky2 and ky3. In applications, critical acceleration is usually taken as a single value.

Figure 4.13 Integration of Accelerograms to Determine Downslope Displacements (Goodman and Seed, 1966).

characterized by the geologic group, slope angle and critical acceleration. The acceleration required to initiate slope movement is a complex function of slope geology, steepness, groundwater conditions, type of landsliding and history of previous slope performance. At the present time, a generally accepted relationship or simplified methodology for estimating a_c has not been developed.

The relationship proposed by Wilson and Keefer (1985) is utilized in the methodology. This relationship is shown in Figure 4.14. Landslide susceptibility is measured on a scale of I to X, with I being the least susceptible. The site condition is identified using three geologic groups and groundwater level. The description for each geologic group and its associated susceptibility is given in Table 4.16. The groundwater condition is divided into either dry condition (groundwater below level of the sliding) or wet condition (groundwater level at ground surface). The critical acceleration is then estimated for the respective geologic and groundwater conditions and the slope angle. To avoid calculating the occurrence of landsliding for very low or zero slope angles and critical accelerations, lower bounds for slope angles and critical accelerations are established. These bounds are shown in Table 4.17. Figure 4.14 shows the Wilson and Keefer relationships within these bounds.

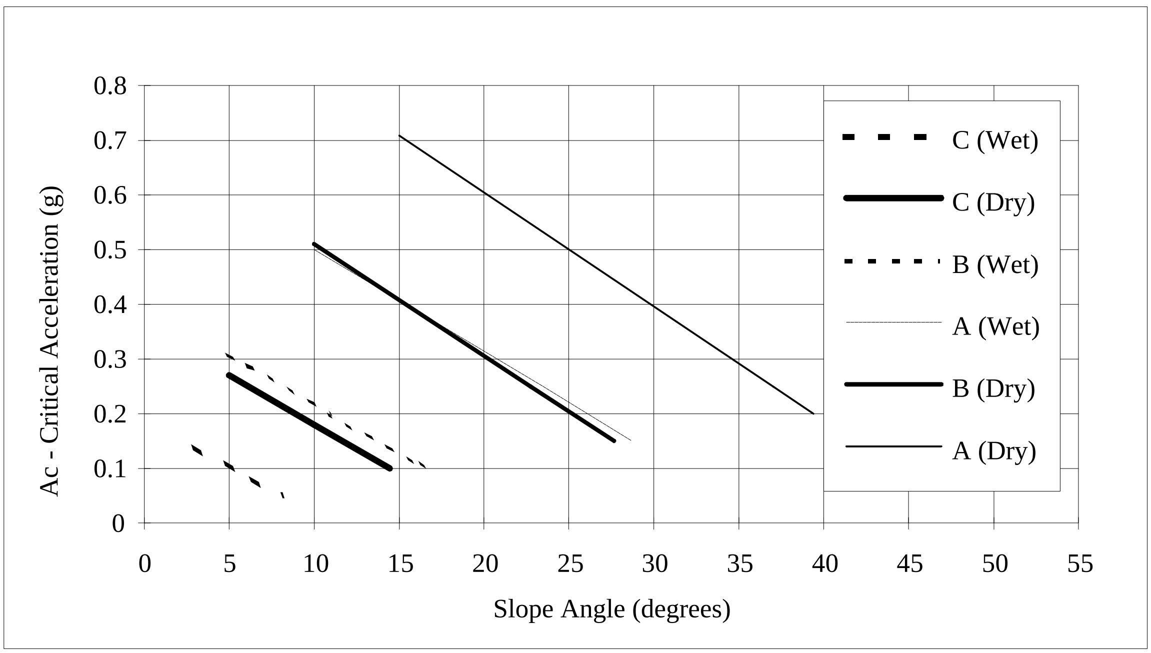


Figure 4.14 Critical Acceleration as a Function of Geologic Group and Slope Angle (Wilson and Keefer, 1985).

Table 4.16 Landslide Susceptibility of Geologic Groups

Geologic Group		Slope Angle, degrees					
		0-10	10-15	15-20	20-30	30-40	>40
(a) DRY (groundwater below level of sliding)							
A	**Strongly Cemented Rocks (crystalline rocks and well-cemented sandstone, $c' = 300$ psf, $\phi' = 35^o$)**	None	None	I	II	IV	VI
B	**Weakly Cemented Rocks and Soils (sandy soils and poorly cemented sandstone, $c' = 0$, $\phi' = 35^o$)**	None	III	IV	V	VI	VII
C	**Argillaceous Rocks (shales, clayey soil, existing landslides, poorly compacted fills, $c' = 0$ $\phi' = 20^o$)**	V	VI	VII	IX	IX	IX
(b) WET (groundwater level at ground surface)							
A	**Strongly Cemented Rocks (crystalline rocks and well-cemented sandstone, $c' = 300$ psf, $\phi' = 35^o$)**	None	III	VI	VII	VIII	VIII
B	**Weakly Cemented Rocks and Soils (sandy soils and poorly cemented sandstone, $c' = 0$, $\phi' = 35^o$)**	V	VIII	IX	IX	IX	X
C	**Argillaceous Rocks (shales, clayey soil, existing landslides, poorly compacted fills, $c' = 0$ $\phi' = 20^o$)**	VII	IX	X	X	X	X

Table 4.17 Lower Bounds for Slope Angles and Critical Accelerations for Landsliding Susceptibility

	Slope Angle, degrees		Critical Acceleration (g)	
Group	**Dry Conditions**	**Wet Conditions**	**Dry Conditions**	**Wet Conditions**
A	15	10	0.20	0.15
B	10	5	0.15	0.10
C	5	3	0.10	0.05

As pointed out by Wieczorek and others (1985), the relationships in Figure 4.14 are conservative representing the most landslide-susceptible geologic types likely to be found in the geologic group. Thus, in using this relationship further consideration must be given to evaluating the probability of slope failure as discussed in Section 4.2.2.2.3.

In Table 4.18, landslide susceptibility categories are defined as a function of critical acceleration. Then, using Wilson and Keefer's relationship in Figure 4.14 and the lower bound values in Table 4.17, the susceptibility categories are assigned as a function of geologic group, groundwater conditions, and slope angle in Table 4.16. Tables 4.16 and 4.18 thus define the landslide susceptibility.

Table 4.18 Critical Accelerations (a_c) for Susceptibility Categories

Susceptibility Category	None	I	II	III	IV	V	VI	VII	VIII	IX	X
Critical Accelerations (g)	None	0.60	0.50	0.40	0.35	0.30	0.25	0.20	0.15	0.10	0.05

4.2.2.2.3 Probability of Having a Landslide-Susceptible Deposit

Because of the conservative nature of the Wilson and Keefer (1985) correlation, an assessment is made of the percentage of a landslide susceptibility category that is expected to be susceptible to landslide. Based on Wieczorek and others (1985), this percentage is selected from Table 4.19 as a function of the susceptibility categories. Thus, at any given location, there is a specified probability of having a landslide-susceptible deposit, and landsliding either occurs or does not occur within susceptible deposits depending on whether the induced peak ground acceleration a_{is} exceeds the critical acceleration a_c.

Table 4.19 Percentage of Map Area Having a Landslide-Susceptible Deposit

Susceptibility Category	None	I	II	III	IV	V	VI	VII	VIII	IX	X
Map Area	0.00	0.01	0.02	0.03	0.05	0.08	0.10	0.15	0.20	0.25	0.30

4.2.2.2.4 Permanent Ground Displacements

The permanent ground displacements are determined using the following expression:

$$E[PGD] = E\left[d / a_{is}\right] \cdot a_{is} \cdot n \tag{4-25}$$

where

$E\left[d / a_{is}\right]$ is the expected displacement factor (Figure 4.16)
a_{is} is the induced acceleration (in decimal fraction of g's)
n is the number of cycles (Equation 4-26).

A relationship between number of cycles and earthquake moment magnitude (**M**) based on Seed and Idriss (1982) is shown in Figure 4.15 and can be expressed as follows:

$$n = 0.3419\mathbf{M}^3 - 5.5214\mathbf{M}^2 + 33.6154\mathbf{M} - 70.7692 \tag{4-26}$$

The induced peak ground acceleration within the slide mass, a_{is}, represents the average peak acceleration within the entire slide mass. For relatively shallow and laterally small slides, a_{is} is not significantly different than the induced peak ground surface acceleration

a_i. For deep and large slide masses a_{is} is less than a_i. For many applications a_{is} may be assumed equal to the accelerations predicted by the peak ground acceleration attenuation relationships being used for the loss estimation study. Considering also that topographic amplification of ground motion may also occur on hillside slopes (which is not explicitly incorporated in the attenuation relationships), the assumption of a_{is} equal to a_i may be prudent. The user may specify a ratio a_{is}/a_i less than 1.0. The default value is 1.0.

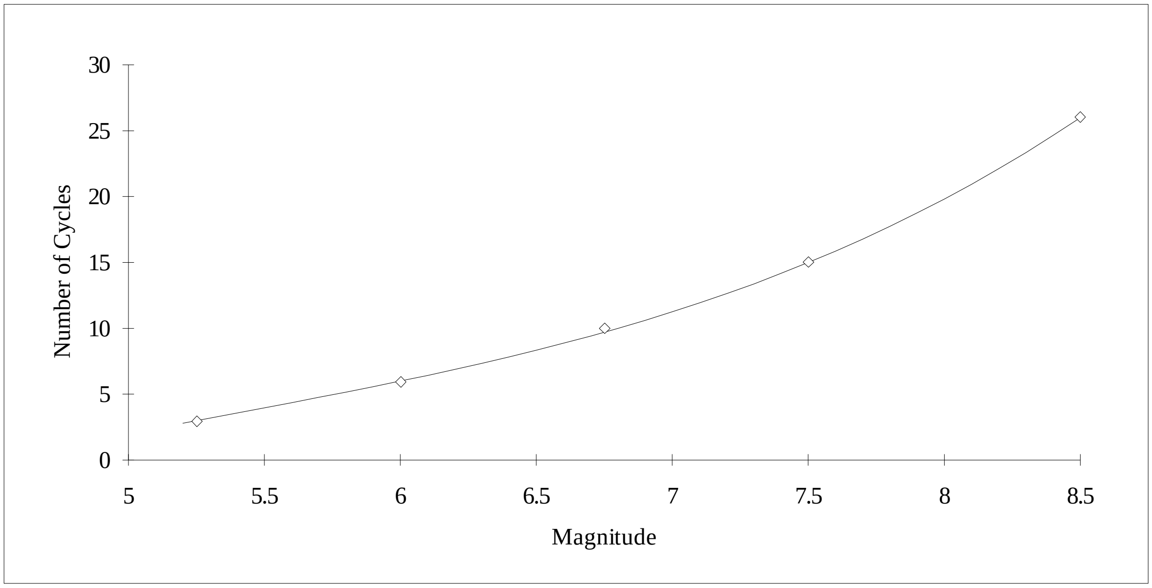


Figure 4.15 Relationship between Earthquake Moment Magnitude and Number of Cycles.

A relationship derived from the results of Makdisi and Seed (1978) is used to calculate downslope displacements. In this relationship, shown in Figure 4.16, the displacement factor d/a_{is} is calculated as a function of the ratio a_c/a_{is}. For the relationship shown in Figure 4.16, the range in estimated displacement factor is shown and it is assumed that there is a uniform probability distribution of displacement factors between the upper and lower bounds.

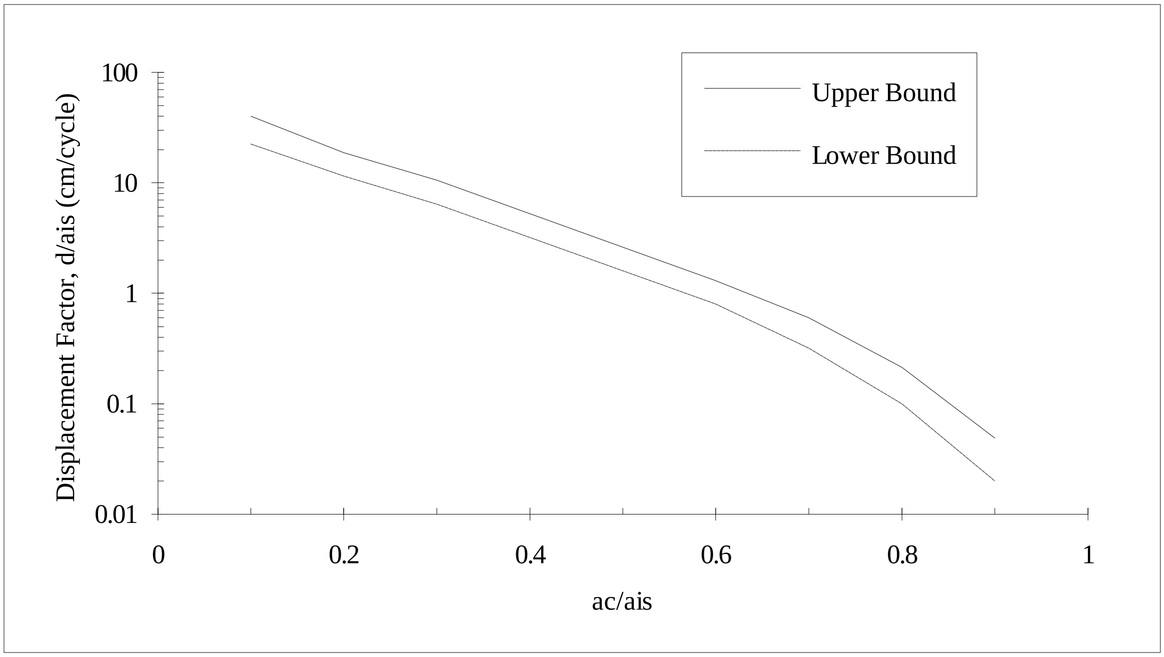


Figure 4.16 Relationship between Displacement Factor and Ratio of Critical Acceleration and Induced Acceleration.

4.2.2.3 Surface Fault Rupture

4.2.2.3.1 Permanent Ground Displacements

The correlation between surface fault displacement and earthquake moment magnitude (**M**) developed by Wells and Coppersmith (1994) is used. The maximum displacement is given by the relationship shown in Figure 4.17. It is assumed that the maximum displacement can potentially occur at any location along the fault, although at the ends of the fault, displacements must drop to zero. The relationship developed by Wells and Coppersmith based on their empirical data set for all types of faulting (strike slip, reverse and normal) is used. It is considered that this relationship provides reasonable estimates for any type of faulting for general loss estimation purposes. The uncertainty in the maximum displacement estimate is incorporated in the loss estimation analysis. The log of the standard deviation of estimate is equal to 0.35 which is equivalent to a factor of about 2 in the displacement estimate at the plus-or-minus one standard deviation level.

The median maximum displacement (MD) is given by the following relationship:

$$\log(\mathrm{MD}) = -5.26 + 0.79(\mathbf{M}) \tag{4-27}$$

where **M** is moment magnitude.

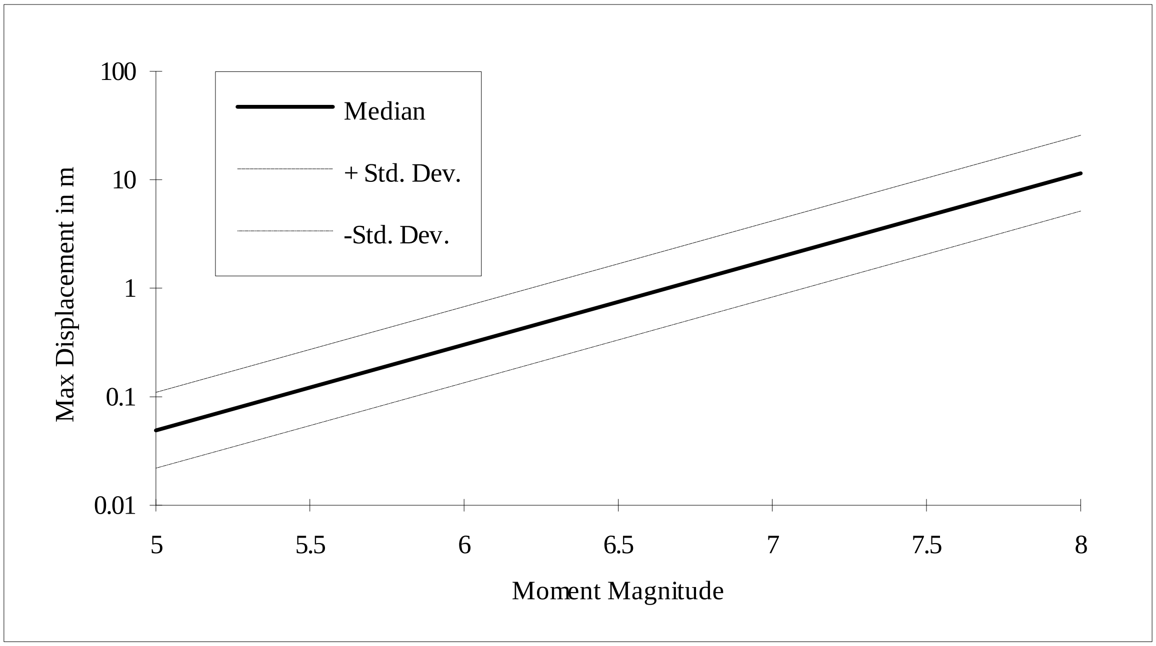


Figure 4.17 Relationship for Estimating Maximum Surface Fault Displacement.

It has been observed that displacements along a fault vary considerably in amplitude from zero to the maximum value. Wells and Coppersmith found that the average displacement along the fault rupture segment was approximately equal to one-half the maximum displacement. This is equivalent to a uniform probability distribution for values of displacement ranging from zero to the maximum displacement. As a conservative estimate, a uniform probability distribution from one-half of the maximum fault displacement to the maximum fault displacement is incorporated in the loss estimation methodology for any location along the fault rupture.

4.2.3 Guidance for Expert-Generated Ground Failure Estimation

This section provides guidance for users who wish to use more refined methods and data to prepare improved estimates of ground failure. It is assumed that such users would be geotechnical experts with sufficient expertise in ground failure prediction to develop site-specific estimates of PGD based on regional/local data.

4.2.3.1 Input Requirements and Output Information

4.2.3.1.1 Liquefaction

Input

- A map delineating areas of equal susceptibility (i.e., similar age, deposition, material properties, and ground water depth)
- Probability distribution of susceptibility variation within each area

- Relationships between liquefaction probability and ground acceleration for each susceptible area
- Maps delineating topographic conditions (i.e., slope gradients and/or free-face locations) and susceptible unit thicknesses
- Relationships between ground displacements (i.e., lateral spreading and settlement), and ground acceleration for each susceptible unit, including probability distribution for displacement; they may vary within a given susceptible unit depending on topographic and liquefied zone thickness conditions

Output

- Contour maps depicting liquefaction hazard and associated potential ground displacements

4.2.3.1.2 Landsliding

Input

- A map depicting areas of equal critical or yield acceleration a_c (i.e., the values of peak ground acceleration within the slide mass required to just initiate landsliding, that is, reduce the factor of safety to 1.0 at the instant of time a_c occurs)
- The probability distribution for a_c within each area
- The ratio between induced peak ground surface acceleration, a_i, and the peak ground acceleration within the slide mass a_{is} (note: could be a constant ratio or could vary for different areas). The value $a_{is}/a_i \leq 1$. The default ratio is 1.0
- Relationships between landslide displacement d induced acceleration a_{ic} and initial or yield acceleration a_c including the probability distribution for d. Different relationships can be specified for different areas. The default relationship between the displacement factor d/a_{is} and a_c/a_{is} is shown in Figure 4.16

Output

- Contour maps depicting landsliding hazard and permanent ground displacements

4.2.3.1.3 Surface Fault Rupture

Input

- Predictive relationship for the maximum amount of fault displacement
- Specification of regions of the fault having lower maximum displacements
- Specifying other than the default relationship for the probability distribution between minimum and maximum amounts of fault rupture displacement

Output

- Amount of fault displacement at locations along the fault trace

4.2.3.2 Liquefaction

4.2.3.2.1 Background

The key for the user in defining analysis inputs is understanding the interrelationship among factors that significantly influence occurrence of liquefaction and associated ground displacement phenomena.

During earthquake ground shaking, induced cyclic shear creates a tendency in most soils to change volume by rearrangement of the soil-particle structure. In loose soils, this volume change tendency is to compact or densify the soil structure. For soils such as fine sands, silts and clays, permeability is sufficiently low such that undrained conditions prevail and no or insignificant volume change can occur during the ground shaking. To accommodate the volume decrease tendency, the soil responds by increases of pore-water pressure and corresponding decreases of intergranular effective stress. The relationship between volume change tendency and pore-water increase is described by Martin, et. al. (1975). Egan and Sangrey (1978) discuss the relationship among compressibility characteristics, the potential amount of pore-water pressure generation and the subsequent loss of strength in various soil materials. In general, more compressible soils such as plastic silts or clays do not generate excess pore-water pressure as quickly or to as large an extent as less compressible soils such as sands. Therefore, silty and clayey soils tend to be less susceptible than sandy soils to liquefaction-type behaviors. Even within sandy soils, the presence of finer-grained materials affects susceptibility as is reflected in the correlations illustrated in Figure 4.18 prepared by Seed, et. al. (1985) for use in simplified empirical procedures for evaluating liquefaction potential.

Excess pore-water pressure generation and strength loss potential are also highly dependent on the density of the soil, as may also be inferred from Figure 4.18. Density characteristics of soils in a deposit, notably sandy and silty soils, are reflected in penetration resistance measured, for example, during drilling and sampling an exploratory boring. Using penetration resistance data to help assess liquefaction hazard due to an earthquake is considered a reasonable engineering approach (Seed and Idriss, 1982; Seed, et. al., 1985; National Research Council, 1985), because many of the factors affecting penetration resistance affect the liquefaction resistance of sandy and silty soils in a similar way and because state-of-practice liquefaction evaluation procedures are based on actual performance of soil deposits during worldwide historical earthquakes (e.g., Figure 4.18).

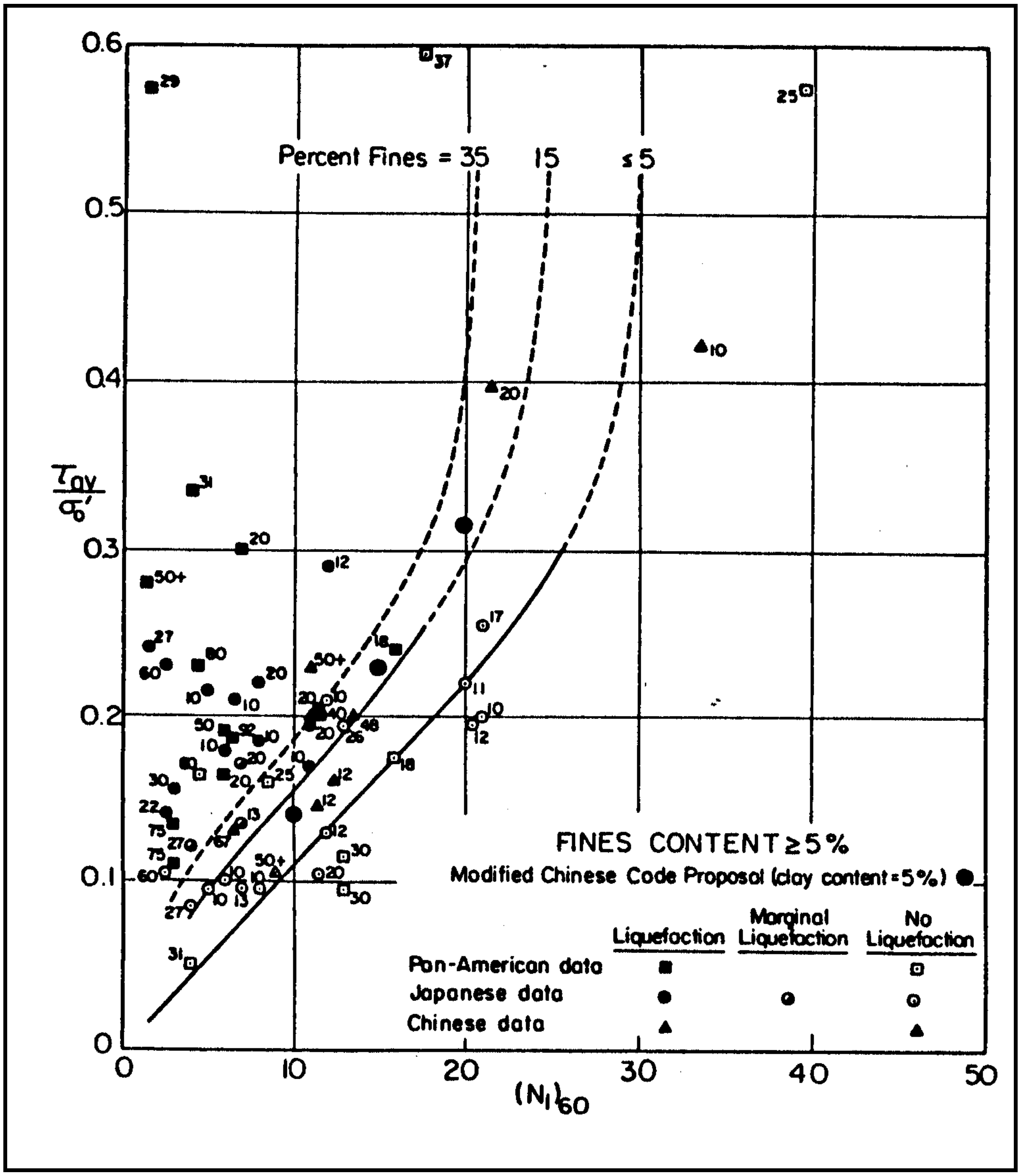


Figure 4.18 Relationship between Cyclic Stress Ratio causing Liquefaction and $(N_1)_{60}$ values (M=7.5) (Seed et al., 1985).

These displacement hazards are direct products of the soil behavior phenomena (i.e., high pore water pressure and significant strength reduction) produced by the liquefaction process. Lateral spreads are ground failure phenomena that occur near abrupt topographic features (i.e., free-faces) and on gently sloping ground underlain by liquefied soil. Earthquake ground-shaking affects the stability of sloping ground containing liquefiable materials by causing seismic inertia forces to be added to gravitational forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Lateral spreading movements may be on the order of inches to several feet or more and are typically accompanied by surface fissures and slumping. Flow slides generally occur in liquefied materials found on steeper slopes and may involve ground movements of hundreds of feet. As a result, flowslides can be the most catastrophic of the liquefaction-related ground-failure phenomena. Fortunately, flow slides are much less common occurrences than lateral spreads.

Settlement is a result of the dissipation of excess pore pressure generated by the rearrangement of loosely compacted saturated soils into a denser configuration during shaking. Such dissipation will produce volume decreases (termed consolidation or compaction) within the soil that are manifested at the ground surface as settlement. Volume changes may occur in both liquefied and non-liquefied zones with significantly larger contributions to settlement expected to result from liquefied soil. Densification may also occur in loose unsaturated materials above the ground water table. Spatial variations in material characteristics may cause such settlements to occur differentially. Differential ground settlement may also occur near sand boil manifestations due to liquefied materials being removed from the depths of liquefaction and brought to the ground surface.

These factors have been discussed briefly in preceding sections and incorporated to the extent possible in characterizing relationships of Section 4.2.2.1. The challenge to the user is to translate regional/local data, experience and judgment into defining site-specific relationships. The following paragraphs offer additional comments regarding various aspects of that process.

4.2.3.2.2 Susceptibility

Fundamental soil characteristics and physical processes that affect liquefaction susceptibility have been identified through case histories and laboratory studies. Depositional environments of sediments and their geologic ages control these characteristics and processes, as discussed by Youd and Perkins (1978).

The depositional environments of sediments control grain size distribution and, in part, the relative density and structural arrangement of grains. Grain size characteristics of a soil influence its susceptibility to liquefaction. Fine sands tend to be more susceptible than silts and gravels. All cohesionless soils, however, may be considered potentially liquefiable as the influence of particle size distribution is not thoroughly understood. In general, cohesive soils that contain more than about 20 percent clay may be considered nonliquefiable (Seed and Idriss, 1982, present criteria for classifying a soil as nonliquefiable).

Relative density and structural arrangement of grains (soil structure) greatly influence liquefaction susceptibility of a cohesionless soil. Soils that have higher relative densities and more stable soil structure have a lower susceptibility to liquefaction. These factors may be related to both depositional environment and age. Sediments undisturbed after deposition (e.g., lagoon or bay deposits) tend to have lower densities and less stable structures than sediments subjected to wave or current action. With increasing age of a deposit, relative density may increase as particles gradually work closer together. The soil structure also may become more stable with age through slight particle reorientation or cementation. Also, the thickness of overburden sediments may increase with age, and the increased pressures associated with a thicker overburden will tend to increase the density of the soil deposit.

An increase in the ratio of effective lateral earth pressure to effective vertical or overburden earth pressure in a soil has been shown to reduce its liquefaction susceptibility. Such an increase will occur when overburden is removed by erosion.

In general, it is thought that the soil characteristics and processes that result in a lower liquefaction susceptibility also result in higher penetration resistance when a soil sampler is driven into a soil deposit. Therefore, blow count values, which measure penetration resistance of a soil sampler in a boring, are a useful indicator of liquefaction susceptibility. Similarly, the resistance from pushing a cone penetrometer into the soil is a useful indicator of liquefaction susceptibility. An understanding of the depositional environments and ages of soil units together with penetration resistance data enables assessment of liquefaction susceptibility.

Additional information helpful to enhancing/refining the susceptibility characterization is observation of liquefaction and related phenomena during historical earthquakes, as well as evidence of paleoliquefaction. Although such information does not exist for all locations and its absence does not preclude liquefaction susceptibility, it is available for numerous locations throughout the country; for example, in Northern California (Youd and Hoose, 1978; Tinsley, et. al., 1994); in the New Madrid region (Obermeier, 1989; Wesnousky, et. al., 1989); in the Charleston, South Carolina region (Obermeier, et. al., 1986; Gohn, et. al., 1984), in the northeastern United States (Tuttle and Seeber, 1989); among other locales. Incorporation of such historical information has been shown to significantly enhance liquefaction-related loss estimation predictions (Geomatrix, 1993).

4.2.3.2.3 Liquefaction Probability

As described previously, simplified procedures for evaluating liquefaction potential presented by Seed, et. al. (1985), as well as probabilistic approach presented by Liao, et. al. (1988), are useful tools for helping to characterize the relationships among liquefaction probability, peak ground acceleration, duration of shaking (magnitude), and groundwater depth, etc. A parameter commonly utilized in these procedures is penetration resistance, which was previously discussed relative to susceptibility. Within a given geologic unit, experience indicates that subsurface investigations may obtain a certain scatter in penetration resistance without necessarily any observable trend for variation horizontally or vertically within that unit. In such cases, a single representative penetration resistance value is often selected for evaluating the liquefaction potential at the site. The representative value is very much site-specific and depends on the particular distribution of penetration resistance values measured. For example, if most of the values are very close to each other, with a few much higher or lower values, the representative value might be selected as the value that is close to the mean of the predominant population of values that are close to each other. On the other hand, if the penetration resistance values appear to be widely scattered over a fairly broad range of values, a value near the 33rd percentile might be more appropriate to select (H. B. Seed, personal communication, 1984). A typical distribution of penetration resistance (N_1) for a Holocene alluvial fan deposit (i.e., moderate susceptibility) is shown in Figure 4.19.

The user may elect to eliminate the probabilistic factor that quantifies the proportion of a geologic map unit deemed susceptible to liquefaction (i.e., the likelihood of susceptible conditions existing at any given location within the unit) if regional geotechnical data enables microzonation of susceptibility areas, or define this factor as a probabilistic distribution, or incorporate the susceptibility uncertainty in defining other liquefaction probability relationships.

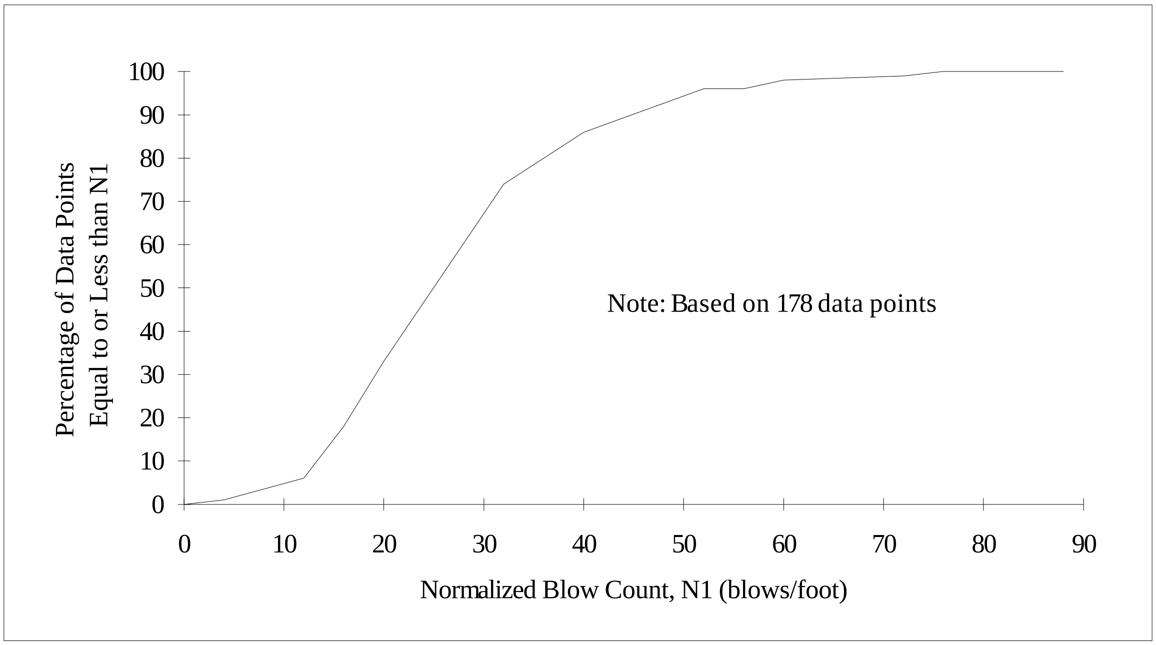


Figure 4.19 Typical Cumulative Distribution Curve of Penetration Resistance for Holocene Alluvial Fan Deposits (after Power, et. al., 1992).

4.2.3.2.4 Permanent Ground Displacement

Lateral Spreading

Various relationships for estimating lateral spreading displacement have been proposed, including the previously utilized Liquefaction Severity Index (LSI) by Youd and Perkins (1978), a relationship incorporating slope and liquefied soil thickness by Hamada, et. al. (1986), a modified LSI approach presented by Baziar, et. al. (1992), and a relationship by

Bartlet and Youd (1992), in which they characterize displacement potential as a function of global earthquake and local site characteristics (e.g., slope, liquefaction thickness, and grain size distribution). Relationships that are more site-specific may be developed based on simple stability and deformation analysis for lateral spreading conditions using undrained residual strengths for liquefied sand (Seed and Harder, 1990) along with Newmark-type (1965) and Makdisi and Seed (1978) displacement approaches. To reasonably represent the lateral spreading hazard by either published relationships or area-specific analyses, generalized information regarding stratigraphic conditions (i.e., depth to and thickness of the liquefied zone) and topographic conditions (i.e., ground slope and free-face situations) are required.

Ground Settlement

Relationships for assessing ground settlement are available (e.g., Tokimatsu and Seed, 1978; Ishihara, 1991) and are suggested to the user for guidance. In addition, test results presented by Lee and Albaisa (1974) suggest that the magnitude of volumetric strain following liquefaction may be dependent on grain-size distribution. Area-specific information required for developing settlement relationships is similar to that for lateral spreading.

4.2.3.3 Landsliding

4.2.3.3.1 Background

The key assessment is the generation of a map denoting areas of equal landslide susceptibility and their corresponding values of critical acceleration. This should be accomplished considering the geographical distribution of facilities at risk in the region and the types of landsliding that could affect the facilities.

4.2.3.3.2 Landslide Susceptibility

Keefer (1984) and Wilson and Keefer (1985) have identified many different types of landsliding, ranging from rock falls to deep-seated coherent soil or rock slumps to soil lateral spreads and flows. For loss estimation purposes, the potential for lateral spreads and flows should be part of the liquefaction potential assessment rather than the landslide potential. The significance of other forms of downslope movement depends on the potential for such movements to damage facilities. The emphasis on characterizing landslide susceptibility should be on failure modes and locations that pose a significant risk to facilities. For example, if the potential for rock falls were high (because of steep terrain and weak rock) but could occur only in undeveloped areas, then it would not be important to characterize the critical acceleration for this mode of failure. As another example, in evaluating the probability of landsliding and the amount of displacements as part of a regional damage assessment for a utility district (Power and others, 1994), it was assessed that two types of landsliding posed the major risk to the facilities and piping: activation of existing deep-seated landslide deposits that had been mapped in hillside areas and that had the potential for disrupting areas in which water lines were located (landslides often covering many square blocks); and local slumping of roadway sidehill fills in which water lines were embedded.

Having identified the modes and geographic areas of potential landsliding of significance, critical acceleration can be evaluated for these modes and areas. It is not necessarily required to estimate a_c as a function of slope angle. In some cases, it may be satisfactory to estimate a_c and corresponding ranges of values for generalized types of landslides and subregions, for example, reactivation of existing landslides within a certain subregion or within the total region. However, it is usually necessary to distinguish between dry and wet conditions because a_c is usually strongly dependent on groundwater conditions.

In general, there are two approaches to estimating a_c: an empirical approach utilizing observations of landsliding in past earthquakes and corresponding records or estimates of ground acceleration; and an analytical approach, in which values of a_c are calculated by pseudo-static slope stability analysis methods. Often, both approaches may be utilized (e.g., Power, et. al., 1994). When using the analytical approach, the sensitivity of results to soil strength parameters must be recognized. In assessing strength parameter values and ranges, it is often useful to back-estimate values, which are operable during static conditions. Thus, for certain types of geology, slope angles, static performance observations during dry and wet seasons, and estimates of static factors of safety, it may be possible to infer reasonable ranges of strength parameters from static slope stability analyses. For earthquake loading conditions, an assessment should also be made as to whether the short-term dynamic, cyclic strength would differ from the static strength. If the soil or rock is not susceptible to strength degradation due to cyclic load applications or large deformations, then it may be appropriate to assign strength values higher than static values due to rate of loading effects. On the other hand, values even lower than static values may be appropriate if significant reduction in strength is expected (such as due to large-deformation-induced remolding of soil).

4.2.3.3.3 Probability of Landsliding

The probability of landsliding at any location is determined by comparing the induced peak ground acceleration (adjusted to the value of the peak acceleration in the landslide mass a_{is}) with the assessed distribution for critical acceleration a_c (Figure 4.20).

4.2.3.3.4 Permanent Ground Displacements

In assessing soil deformations using relationships such as shown in Figure 4.16, it should be kept in mind that the relationships are applicable to slope masses that exhibit essentially constant critical accelerations. For cases where significant reduction in strength may occur during the slope deformation process, these relationships may significantly underestimate deformations if the peak strength values are used. For example, deformations cannot be adequately estimated using these simplified correlations in cases of sudden, brittle failure, such as rock falls or soil or rock avalanches on steep slopes.

4.2.3.4 Surface Fault Rupture

4.2.3.4.1 Permanent Ground Displacements

Refinements or alternatives that an expert may wish to consider in assessing displacements associated with surface fault rupture include: a predictive relationship for maximum fault displacement different from the default relationship (Figure 4.17), specification of regions of the fault rupture (near the ends) where the maximum fault displacement is constrained to lower values, and specification of other than the default relationship for the probability distribution of fault rupture between minimum and maximum values.

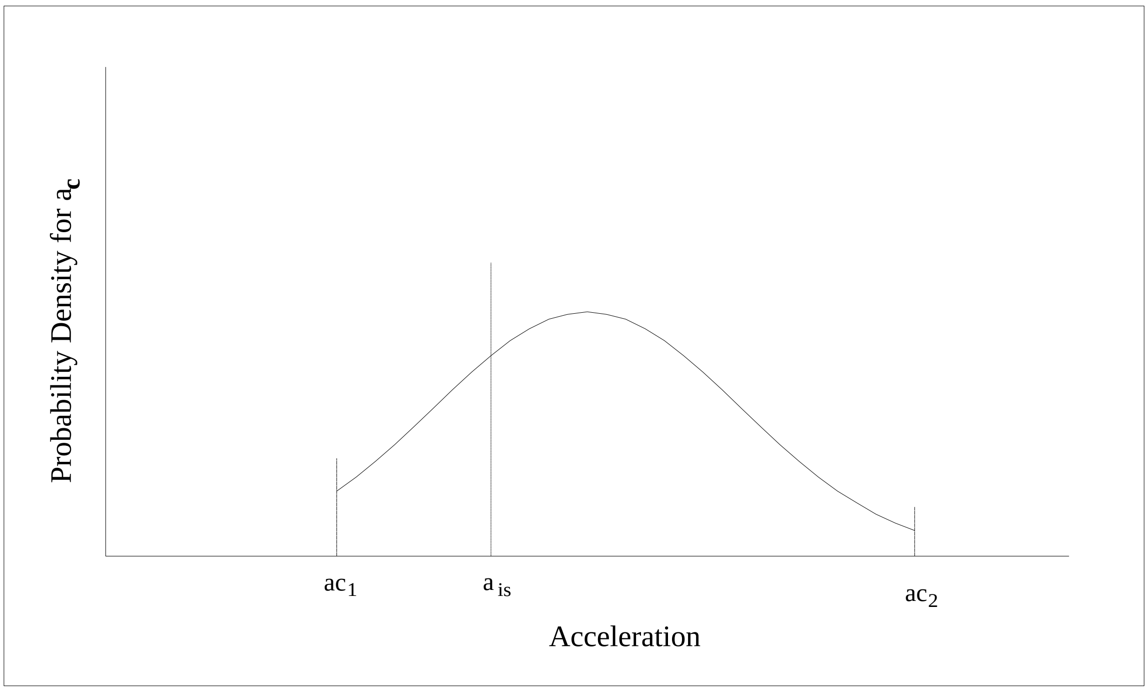


Figure 4.20 Evaluation of Probability of Landsliding.

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Appendix 4A
Tables of Attenuation Values for the Eastern U.S. Attenuation Relationships

This appendix gives tabular results for the default Eastern United States attenuation relationships used in the methodology. Each table gives the peak ground or spectral values in relationship to hypocentral distance (km) and moment magnitude (**M**). The units for the peak ground acceleration and spectral acceleration are fraction of gravity and the peak ground velocity values are in centimeters per second. In these tables, hypocentral distance is used in Frankel et al., and closest horizontal distance is used in Toro et al. The index of the appendix is as follows:

<u>Frankel et al. (1996)</u>

<u>Toro et al. (1997)</u>

Table 4A.1: Peak Ground Acceleration Attenuation Values (in units of g)

Distance	Moment Magnitude						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
10	0.36	0.56	0.85	1.23	1.50*	1.50*	1.50*
20	0.14	0.24	0.37	0.56	0.79	1.15	1.50*
30	0.08	0.14	0.22	0.33	0.49	0.71	1.01
40	0.05	0.09	0.14	0.22	0.33	0.48	0.69
50	0.04	0.06	0.10	0.16	0.24	0.36	0.51
60	0.03	0.05	0.08	0.12	0.19	0.28	0.41
70	0.02	0.04	0.06	0.10	0.16	0.23	0.34
80	0.02	0.03	0.05	0.09	0.14	0.21	0.29
90	0.02	0.03	0.05	0.08	0.13	0.19	0.28
100	0.01	0.03	0.05	0.07	0.12	0.18	0.26
120	0.01	0.02	0.04	0.06	0.10	0.16	0.23
140	0.01	0.02	0.03	0.05	0.09	0.14	0.20
160	0.01	0.02	0.03	0.04	0.07	0.11	0.17
180	0.01	0.01	0.02	0.04	0.06	0.10	0.15
200	0.01	0.01	0.02	0.03	0.05	0.08	0.13
250	0.00	0.01	0.01	0.02	0.04	0.06	0.09
300	0.00	0.00	0.01	0.02	0.03	0.04	0.07
350	0.00	0.00	0.01	0.01	0.02	0.03	0.05

* PGA capped at 1.5g

Table 4A.2: Peak Ground Velocity Attenuation Values (cm/sec)

Distance	Moment Magnitude						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
10	7.4	14.4	26.3	47.1	81.3	145.5	257.0
20	3.2	6.4	12.0	22.0	39.8	71.3	125.9
30	1.9	3.9	7.5	14.1	25.7	46.0	82.6
40	1.3	2.6	5.2	9.9	18.1	33.6	60.0
50	0.9	2.0	3.9	7.6	14.2	26.0	46.9
60	0.7	1.5	3.1	6.1	11.5	21.3	38.8
70	0.6	1.3	2.7	5.2	10.0	18.8	34.3
80	0.5	1.1	2.3	4.7	9.1	17.3	31.6
90	0.5	1.1	2.2	4.6	8.8	16.9	31.3
100	0.4	1.0	2.1	4.4	8.5	16.5	30.9
120	0.4	0.9	2.0	4.1	8.1	15.8	29.8
140	0.3	0.8	1.7	3.7	7.4	14.6	27.9
160	0.3	0.7	1.5	3.2	6.6	13.2	25.5
180	0.3	0.6	1.3	2.9	6.0	12.0	23.4
200	0.2	0.5	1.2	2.6	5.4	10.8	21.3
250	0.2	0.4	0.9	2.0	4.3	9.0	17.9
300	0.1	0.3	0.7	1.6	3.5	7.7	15.5
350	0.1	0.2	0.6	1.4	3.0	6.6	13.6

Table 4A.3: Spectral Acceleration Attenuation Values (T=0.20 sec., units of g)

Distance	**Moment Magnitude**						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
10	0.45	0.79	1.26	1.93	2.88	3.75*	3.75*
20	0.20	0.36	0.59	0.92	1.38	2.00	2.88
30	0.12	0.22	0.37	0.57	0.87	1.28	1.86
40	0.08	0.15	0.25	0.40	0.61	0.91	1.31
50	0.06	0.11	0.19	0.30	0.46	0.69	1.00
60	0.05	0.09	0.15	0.24	0.37	0.56	0.82
70	0.04	0.07	0.12	0.20	0.31	0.48	0.70
80	0.04	0.06	0.11	0.18	0.28	0.43	0.63
90	0.03	0.06	0.10	0.17	0.27	0.41	0.60
100	0.03	0.06	0.10	0.16	0.26	0.39	0.58
120	0.03	0.05	0.09	0.15	0.23	0.36	0.54
140	0.02	0.04	0.07	0.13	0.20	0.32	0.48
160	0.02	0.04	0.06	0.11	0.17	0.27	0.41
180	0.02	0.03	0.05	0.09	0.15	0.24	0.36
200	0.01	0.03	0.05	0.08	0.13	0.20	0.31
250	0.01	0.02	0.03	0.06	0.09	0.15	0.23
300	0.01	0.01	0.02	0.04	0.07	0.11	0.17
350	0.01	0.01	0.02	0.03	0.05	0.09	0.13

* spectral acceleration capped at 3.75g

Table 4A.4: Spectral Acceleration Attenuation Values (T=0.30 sec., units of g)

Distance	**Moment Magnitude**						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
10	0.30	0.55	0.93	1.47	2.24	3.24	3.75*
20	0.14	0.26	0.44	0.69	1.07	1.57	2.29
30	0.09	0.16	0.28	0.44	0.68	1.02	1.48
40	0.06	0.11	0.19	0.31	0.49	0.72	1.04
50	0.04	0.08	0.15	0.24	0.36	0.56	0.82
60	0.04	0.07	0.12	0.19	0.30	0.46	0.66
70	0.03	0.06	0.10	0.16	0.26	0.39	0.58
80	0.03	0.05	0.09	0.14	0.23	0.35	0.52
90	0.02	0.05	0.08	0.14	0.22	0.34	0.51
100	0.02	0.04	0.08	0.13	0.21	0.33	0.49
120	0.02	0.04	0.07	0.12	0.20	0.31	0.46
140	0.02	0.04	0.06	0.11	0.17	0.27	0.41
160	0.02	0.03	0.05	0.09	0.15	0.24	0.36
180	0.01	0.03	0.05	0.08	0.13	0.21	0.32
200	0.01	0.02	0.04	0.07	0.11	0.18	0.28
250	0.01	0.02	0.03	0.05	0.09	0.14	0.22
300	0.01	0.01	0.02	0.04	0.07	0.11	0.17
350	0.00	0.01	0.02	0.03	0.05	0.09	0.14

* spectral acceleration capped at 3.75g

Table 4A.5: Spectral Acceleration Attenuation Values (T=1.00 sec., units of g)

Distance	Moment Magnitude						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
10	0.03	0.09	0.22	0.42	0.71	1.11	1.70
20	0.02	0.05	0.11	0.21	0.35	0.55	0.83
30	0.01	0.03	0.07	0.13	0.22	0.36	0.55
40	0.01	0.02	0.05	0.10	0.17	0.26	0.40
50	0.01	0.02	0.04	0.07	0.13	0.21	0.31
60	0.00	0.01	0.03	0.06	0.10	0.17	0.26
70	0.00	0.01	0.03	0.05	0.09	0.15	0.23
80	0.00	0.01	0.03	0.05	0.09	0.14	0.21
90	0.00	0.01	0.03	0.05	0.08	0.13	0.21
100	0.00	0.01	0.02	0.05	0.08	0.13	0.20
120	0.00	0.01	0.02	0.04	0.08	0.13	0.20
140	0.00	0.01	0.02	0.04	0.07	0.12	0.18
160	0.00	0.01	0.02	0.04	0.06	0.10	0.16
180	0.00	0.01	0.02	0.03	0.06	0.10	0.15
200	0.00	0.01	0.02	0.03	0.05	0.09	0.13
250	0.00	0.01	0.01	0.02	0.04	0.07	0.11
300	0.00	0.00	0.01	0.02	0.03	0.06	0.09
350	0.00	0.00	0.01	0.02	0.03	0.05	0.08

Attenuation Values Based on Toro, Abrahamson and Schneider

Table 4A.6: Peak Ground Acceleration Attenuation Values (in units of g)

Distance	**Moment Magnitude**						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
0	0.28	0.39	0.54	0.72	0.94	1.19	1.47
10	0.18	0.26	0.36	0.50	0.68	0.89	1.13
20	0.10	0.15	0.21	0.30	0.42	0.58	0.77
30	0.07	0.10	0.14	0.20	0.29	0.40	0.55
40	0.05	0.07	0.10	0.15	0.21	0.30	0.41
50	0.04	0.05	0.08	0.11	0.16	0.23	0.32
60	0.03	0.04	0.06	0.09	0.13	0.19	0.26
70	0.02	0.03	0.05	0.07	0.11	0.15	0.22
80	0.02	0.03	0.04	0.06	0.09	0.13	0.19
90	0.02	0.02	0.04	0.05	0.08	0.11	0.16
100	0.01	0.02	0.03	0.05	0.07	0.10	0.14
120	0.01	0.02	0.02	0.04	0.05	0.08	0.11
140	0.01	0.01	0.02	0.03	0.04	0.06	0.09
160	0.01	0.01	0.02	0.02	0.03	0.05	0.07
180	0.01	0.01	0.01	0.02	0.03	0.04	0.06
200	0.01	0.01	0.01	0.02	0.02	0.04	0.05
250	0.00	0.01	0.01	0.01	0.02	0.03	0.04
300	0.00	0.00	0.01	0.01	0.01	0.02	0.03
350	0.00	0.00	0.00	0.01	0.01	0.01	0.02

Table 4A.7: Spectral Acceleration Attenuation Values (T=0.20 sec., units of g)

Distance	**Moment Magnitude**						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
0	0.46	0.66	0.93	1.27	1.70	2.23	2.85
10	0.30	0.43	0.62	0.88	1.22	1.66	2.21
20	0.18	0.26	0.39	0.56	0.80	1.13	1.55
30	0.12	0.18	0.27	0.40	0.57	0.82	1.15
40	0.09	0.14	0.20	0.30	0.44	0.63	0.90
50	0.07	0.11	0.16	0.24	0.35	0.51	0.73
60	0.06	0.09	0.13	0.19	0.28	0.42	0.60
70	0.05	0.07	0.11	0.16	0.24	0.35	0.51
80	0.04	0.06	0.09	0.14	0.20	0.30	0.44
90	0.03	0.05	0.08	0.12	0.17	0.26	0.38
100	0.03	0.05	0.07	0.10	0.16	0.23	0.34
120	0.02	0.04	0.06	0.08	0.13	0.19	0.28
140	0.02	0.03	0.05	0.07	0.11	0.16	0.24
160	0.02	0.03	0.04	0.06	0.09	0.13	0.20
180	0.01	0.02	0.03	0.05	0.08	0.11	0.17
200	0.01	0.02	0.03	0.04	0.07	0.10	0.15
250	0.01	0.01	0.02	0.03	0.05	0.07	0.10
300	0.01	0.01	0.01	0.02	0.03	0.05	0.07
350	0.00	0.01	0.01	0.02	0.02	0.04	0.05

Table 4A.8: Spectral Acceleration Attenuation Values (T=0.30 sec., units of g)

Distance	Moment Magnitude						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
0	0.30	0.47	0.71	1.01	1.38	1.79	2.22
10	0.19	0.31	0.48	0.70	0.99	1.34	1.72
20	0.12	0.19	0.30	0.45	0.66	0.91	1.22
30	0.08	0.13	0.21	0.32	0.47	0.67	0.92
40	0.06	0.10	0.16	0.25	0.36	0.52	0.72
50	0.05	0.08	0.13	0.20	0.29	0.42	0.59
60	0.04	0.06	0.10	0.16	0.24	0.35	0.49
70	0.03	0.05	0.09	0.14	0.20	0.30	0.42
80	0.03	0.05	0.07	0.12	0.17	0.25	0.36
90	0.02	0.04	0.06	0.10	0.15	0.22	0.31
100	0.02	0.04	0.06	0.09	0.14	0.20	0.29
120	0.02	0.03	0.05	0.07	0.11	0.17	0.24
140	0.01	0.02	0.04	0.06	0.10	0.14	0.20
160	0.01	0.02	0.03	0.05	0.08	0.12	0.18
180	0.01	0.02	0.03	0.05	0.07	0.11	0.15
200	0.01	0.02	0.03	0.04	0.06	0.09	0.13
250	0.01	0.01	0.02	0.03	0.04	0.07	0.10
300	0.01	0.01	0.01	0.02	0.03	0.05	0.07
350	0.00	0.01	0.01	0.02	0.03	0.04	0.05

Table 4A.9: Spectral Acceleration Attenuation Values (T=1.00 sec., units of g)

Distance	Moment Magnitude						
(km)	5.0	5.5	6.0	6.5	7.0	7.5	8.0
0	0.04	0.09	0.18	0.31	0.49	0.67	0.82
10	0.03	0.06	0.12	0.22	0.35	0.50	0.64
20	0.02	0.04	0.08	0.14	0.24	0.35	0.46
30	0.01	0.03	0.06	0.11	0.18	0.27	0.36
40	0.01	0.02	0.04	0.08	0.14	0.21	0.29
50	0.01	0.02	0.04	0.07	0.12	0.18	0.24
60	0.01	0.01	0.03	0.06	0.10	0.15	0.21
70	0.01	0.01	0.03	0.05	0.08	0.13	0.18
80	0.00	0.01	0.02	0.04	0.07	0.11	0.16
90	0.00	0.01	0.02	0.04	0.07	0.10	0.14
100	0.00	0.01	0.02	0.03	0.06	0.09	0.13
120	0.00	0.01	0.02	0.03	0.05	0.08	0.12
140	0.00	0.01	0.01	0.03	0.05	0.07	0.10
160	0.00	0.01	0.01	0.02	0.04	0.07	0.09
180	0.00	0.01	0.01	0.02	0.04	0.06	0.08
200	0.00	0.00	0.01	0.02	0.03	0.05	0.08
250	0.00	0.00	0.01	0.02	0.03	0.04	0.06
300	0.00	0.00	0.01	0.01	0.02	0.04	0.05
350	0.00	0.00	0.01	0.01	0.02	0.03	0.04

Chapter 5
Direct Physical Damage - General Building Stock

5.1 Introduction

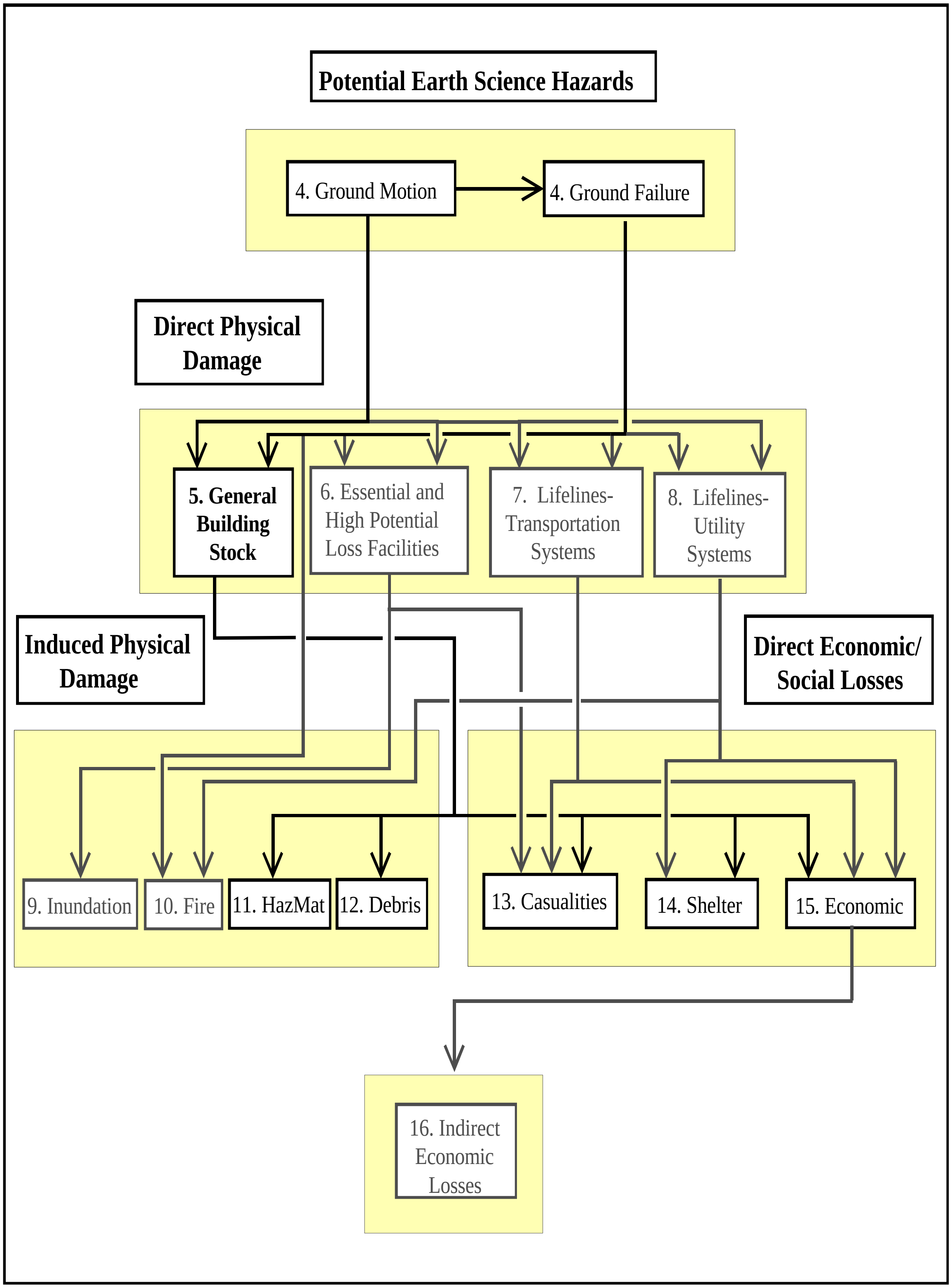
This chapter describes methods for determining the probability of Slight, Moderate, Extensive and Complete damage to general building stock. General building stock represents typical buildings of a given model building type designed to either High-Code, Moderate-Code, or Low-Code seismic standards, or not seismically designed (referred to as Pre-Code buildings). Chapter 6 describes methods for estimating earthquake damage to essential facilities that include Special buildings designed and constructed to standards above normal Code provisions. The flowchart of the overall methodology, highlighting the building damage component and showing its relationship to other components, is shown in Flowchart 5-1.

5.1.1 Scope

The scope of this chapter includes development of methods for estimation of earthquake damage to buildings given knowledge of the model building type and an estimate of the level of ground shaking (or degree of ground failure). Model building types are defined in Section 5.2. The extent and severity of damage to structural and nonstructural components of a building is described by one of five damage states**:** None, Slight, Moderate, Extensive, and Complete. Damage states are defined in Section 5.3 for each model building type by physical descriptions of damage to building elements.

This chapter focuses on the development of functions for estimating building damage due to ground shaking. These building damage functions include: (1) fragility curves that describe the probability of reaching or exceeding different states of damage given peak building response, and (2) building capacity (push-over) curves that are used (with damping-modified demand spectra) to determine peak building response. For use in lifeline damage evaluation, a separate set of building fragility curves expresses the probability of structural damage in terms of peak ground acceleration (PGA). Building damage functions for ground shaking are described in Section 5.4 for each model building type.

While ground shaking typically dominates damage to buildings, ground failure can also be a significant contributor to building damage. Ground failure is characterized by permanent ground deformation (PGD) and fragility curves are used to describe the probability of reaching different states of damage given PGD. These fragility curves are similar to, but less detailed than, those used to estimate damage due to ground shaking. Building damage functions for ground failure are described in Section 5.5.



Flowchart 5.1 Building Damage Relationship to Other Components of the Methodology

Section 5.6 describes implementation of ground shaking damage functions (including development of damping-modified demand spectra) and the calculation of the probability of combined ground shaking and ground failure damage.

The methods described in this chapter may also be used by seismic/structural engineering experts to modify default damage functions (based on improved knowledge of building types, their structural properties and design vintage). Guidance for expert users is provided in Section 5.7

5.1.2 Input Requirements and Output Information

Input required to estimate building damage using fragility and capacity curves includes the following two items:

- model building type (including height) and seismic design level that represents the building (or group of buildings) of interest, and

- response spectrum (or PGA, for lifeline buildings, and PGD for ground failure evaluation) at the building's site or at the centroid of the census tract area where the building (or group of buildings) is located.

Typically, the model building type is not known for each building and must be determined from the inventory of facilities using the relationship of building type and occupancy, described in Chapter 3. The response spectrum, PGA and PGD at the building site (or census tract centroid) are PESH outputs, described in Chapter 4.

The "output" of fragility curves is an estimate of the cumulative probability of being in, or exceeding, each damage state for the given level of ground shaking (or ground failure). Discrete damage state probabilities are created using cumulative damage probabilities, as described in Section 5.6. Discrete damage state probabilities for model building types and occupancy classes are the outputs of the building damage module. These outputs are used directly as inputs to induced physical damage and direct economic and social loss modules, as shown in Flowchart 5.1. While the fragility and capacity curves are applicable, in theory, to a single building as well as to all buildings of given type, they are more reliable as predictors of damage for large, rather than small, population groups. They should not be considered reliable for prediction of damage to a specific facility without confirmation by a seismic/structural engineering expert.

5.1.3 Form of Damage Functions

Building damage functions are in the form of lognormal fragility curves that relate the probability of being in, or exceeding, a building damage state to for a given PESH demand parameter (e.g., response spectrum displacement). Figure 5.1 provides an example of fragility curves for the four damage states used in this methodology.

Each fragility curve is defined by a median value of the PESH demand parameter (i.e., either spectral displacement, spectral acceleration, PGA or PGD) that corresponds to the threshold of the damage state and by the variability associated with that damage state. For example, the spectral displacement, S_d, that defines the threshold of a particular damage state (ds) is assumed to be distributed by:

$$S_d = \overline{S}_{d,ds} \cdot \varepsilon_{ds} \tag{5-1}$$

where: $\overline{S}_{d,ds}$ is the median value of spectral displacement of damage state, ds, and

ε_{ds} is a lognormal random variable with unit median value and logarithmic standard deviation, β_{ds}.

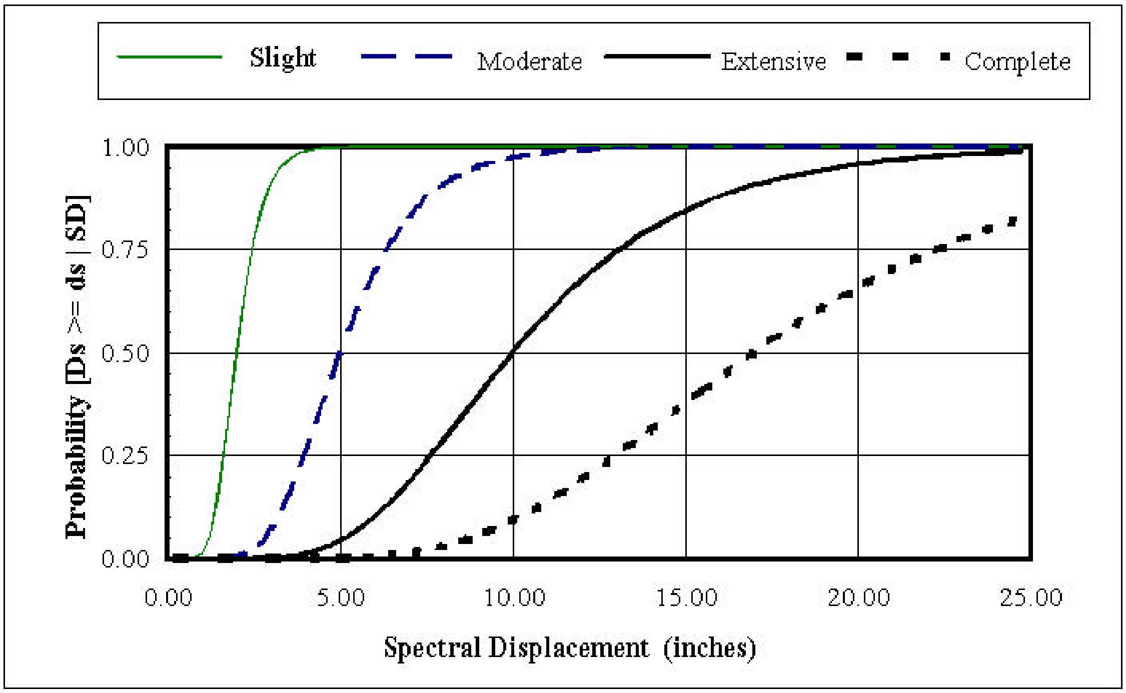


Figure 5.1 Example Fragility Curves for Slight, Moderate, Extensive and Complete Damage.

In a more general formulation of fragility curves, the lognormal standard deviation, β, has been expressed in terms of the randomness and uncertainty components of variability, β_R and β_U, [Kennedy, et. al., 1980]. Since it is not considered practical to separate uncertainty from randomness, the combined random variable term, β, is used to develop a composite "best-estimate" fragility curve. This approach is similar to that used to develop fragility curves for the FEMA-sponsored study of consequences of large earthquakes on six cities of the Mississippi Valley region [Allen & Hoshall, et al., 1985].

The conditional probability of being in, or exceeding, a particular damage state, ds, given the spectral displacement, S_d, (or other PESH parameter) is defined by the function:

$$P\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \tag{5-2}$$

where:

$\bar{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, ds,

β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state, ds, and

Φ is the standard normal cumulative distribution function.

Median spectral displacement (or acceleration) values and the total variability are developed for each of the model building types and damage states of interest by the combination of performance data (from tests of building elements), earthquake experience data, expert opinion and judgment.

In general, the total variability of each damage state, β_{ds}, is modeled by the combination of following three contributors to damage variability:

- uncertainty in the damage state threshold,
- variability in the capacity (response) properties of the model building type of interest, and
- uncertainty in response due to the spatial variability of ground motion demand.

Each of these three contributors to damage state variability is assumed to be lognormally distributed random variables.

The fragility curves are driven by a PESH parameter. For ground failure, the PESH parameter used to drive fragility curves is permanent ground displacement (PGD). For ground shaking, the PESH parameter used to drive building fragility curves is peak spectral response (either displacement or acceleration). Peak ground acceleration (PGA), rather than peak spectral displacement, is used to evaluate ground shaking-induced structural damage to buildings that are components of lifelines (see Section 5.4.4). Peak spectral response varies significantly for buildings that have different response properties (e.g., tall, flexible buildings will displace more than short, stiff buildings). Therefore, determination of peak spectral displacement requires knowledge of the building's response properties.

Building response is characterized by building capacity curves. These curves describe the push-over displacement of each building type and seismic design level as a function of laterally-applied earthquake load. The Methodology uses a technique, similar to the capacity spectrum method [Mahaney, et. al., 1993], to estimate peak building response as the intersection of the building capacity curve and the response spectrum of PESH shaking demand at the building's location (demand spectrum). The capacity spectrum method is one of the two nonlinear static analysis methods described in the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* [FEMA, 1996a] and developed more extensively in *Seismic Evaluation and Retrofit of Concrete Buildings* [SSC, 1996].

The demand spectrum is the 5%-damped PESH input spectrum reduced for higher levels of effective damping (e.g., effective damping includes both elastic damping and hysteretic damping associated with post-yield cyclic response of the building). Figure 5.2 illustrates the intersection of a typical building capacity curve and a typical demand spectrum (reduced for effective damping greater than 5% of critical). Design-, yield- and ultimate-capacity points define the shape of building capacity curves. Peak building response (either spectral displacement or spectral acceleration) at the point of intersection of the capacity curve and demand spectrum is the parameter used with fragility curves to estimate damage state probabilities (see also Section 5.6.2.2).

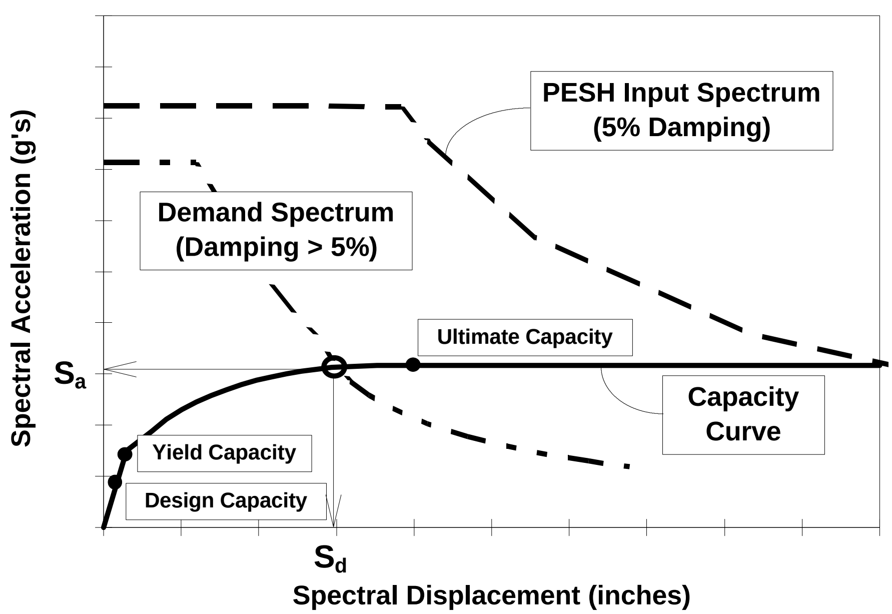


Figure 5.2 Example Building Capacity Curve and Demand Spectrum.

5.2 Description of Model Building Types

Table 5.1 lists the 36 model building types that are used by the Methodology. These model building types are based on the classification system of FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* [FEMA, 1992]. In addition, the methodology breaks down FEMA 178 classes into height ranges, and also includes mobile homes.

Table 5.1 Model Building Types

No.	Label	Description	Height			
			Range		Typical	
			Name	Stories	Stories	Feet
1 2	W1 W2	**Wood, Light Frame (≤ 5,000 sq. ft.)** **Wood, Commercial and Industrial (> 5,000 sq. ft.)**		1 - 2 All	1 2	14 24
3 4 5	S1L S1M S1H	**Steel Moment Frame**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
6 7 8	S2L S2M S2H	**Steel Braced Frame**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
9	S3	**Steel Light Frame**		All	1	15
10 11 12	S4L S4M S4H	**Steel Frame with Cast-in-Place Concrete Shear Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
13 14 15	S5L S5M S5H	**Steel Frame with Unreinforced Masonry Infill Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 13	24 60 156
16 17 18	C1L C1M C1H	**Concrete Moment Frame**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
19 20 21	C2L C2M C2H	**Concrete Shear Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
22 23 24	C3L C3M C3H	**Concrete Frame with Unreinforced Masonry Infill Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
25	PC1	**Precast Concrete Tilt-Up Walls**		All	1	15
26 27 28	PC2L PC2M PC2H	**Precast Concrete Frames with Concrete Shear Walls**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
29 30	RM1L RM1M	**Reinforced Masonry Bearing Walls with Wood or Metal Deck Diaphragms**	Low-Rise Mid-Rise	1-3 4+	2 5	20 50
31 32 33	RM2L RM2M RM2H	**Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms**	Low-Rise Mid-Rise High-Rise	1 - 3 4 - 7 8+	2 5 12	20 50 120
34 35	URML URMM	**Unreinforced Masonry Bearing Walls**	Low-Rise Mid-Rise	1 - 2 3+	1 3	15 35
36	MH	**Mobile Homes**		All	1	10

5.2.1 Structural Systems

A general description of each of the 16 structural systems of model building types is given in the following sections.

Wood, Light Frame (W1):
These are typically single-family or small, multiple-family dwellings of not more than 5,000 square feet of floor area. The essential structural feature of these buildings is repetitive framing by wood rafters or joists on wood stud walls. Loads are light and spans are small. These buildings may have relatively heavy masonry chimneys and may be partially or fully covered with masonry veneer. Most of these buildings, especially the single-family residences, are not engineered but constructed in accordance with "conventional construction" provisions of building codes. Hence, they usually have the components of a lateral-force-resisting system even though it may be incomplete. Lateral loads are transferred by diaphragms to shear walls. The diaphragms are roof panels and floors that may be sheathed with sawn lumber, plywood or fiberboard sheathing. Shear walls are sheathed with boards, stucco, plaster, plywood, gypsum board, particle board, or fiberboard, or interior partition walls sheathed with plaster or gypsum board.

Wood, Greater than 5,000 Sq. Ft. (W2):
These buildings are typically commercial or industrial buildings, or multi-family residential buildings with a floor area greater than 5,000 square feet. These buildings include structural systems framed by beams or major horizontally spanning members over columns. These horizontal members may be glue-laminated (glu-lam) wood, solid-sawn wood beams, or wood trusses, or steel beams or trusses. Lateral loads usually are resisted by wood diaphragms and exterior walls sheathed with plywood, stucco, plaster, or other paneling. The walls may have diagonal rod bracing. Large openings for stores and garages often require post-and-beam framing. Lateral load resistance on those lines may be achieved with steel rigid frames (moment frames) or diagonal bracing.

Steel Moment Frame (S1):
These buildings have a frame of steel columns and beams. In some cases, the beam-column connections have very small moment resisting capacity but, in other cases, some of the beams and columns are fully developed as moment frames to resist lateral forces. Usually the structure is concealed on the outside by exterior nonstructural walls, which can be of almost any material (curtain walls, brick masonry, or precast concrete panels), and on the inside by ceilings and column furring. Diaphragms transfer lateral loads to moment-resisting frames. The diaphragms can be almost any material. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions oriented so that some columns act primarily in one direction while the others act in the other direction. Steel moment frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large interstory drifts that may lead to relatively greater nonstructural damage.

Steel Braced Frame (S2):

These buildings are similar to steel moment frame buildings except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.

Steel Light Frame (S3):

These buildings are pre-engineered and prefabricated with transverse rigid frames. The roof and walls consist of lightweight panels, usually corrugated metal. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light steel plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by diaphragm elements, typically rod-braced steel roof framing bays. Tension rod bracing typically resists loads in the longitudinal direction.

Steel Frame with Cast-In-Place Concrete Shear Walls (S4):

The shear walls in these buildings are cast-in-place concrete and may be bearing walls. The steel frame is designed for vertical loads only. Diaphragms of almost any material transfer lateral loads to the shear walls. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections. In modern "dual" systems, the steel moment frames are designed to work together with the concrete shear walls.

Steel Frame with Unreinforced Masonry Infill Walls (S5):

This is one of the older types of buildings. The infill walls usually are offset from the exterior frame members, wrap around them, and present a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels, when they fully engage the surrounding frame members (i.e. lie in the same plane), may provide stiffness and lateral load resistance to the structure.

Reinforced Concrete Moment Resisting Frames (C1):

These buildings are similar to steel moment frame buildings except that the frames are reinforced concrete. There are a large variety of frame systems. Some older concrete frames may be proportioned and detailed such that brittle failure of the frame members can occur in earthquakes leading to partial or full collapse of the buildings. Modern frames in zones of high seismicity are proportioned and detailed for ductile behavior and are likely to undergo large deformations during an earthquake without brittle failure of frame members and collapse.

Concrete Shear Walls (C2):

The vertical components of the lateral-force-resisting system in these buildings are concrete shear walls that are usually bearing walls. In older buildings, the walls often are quite extensive and the wall stresses are low but reinforcing is light. In newer buildings, the shear walls often are limited in extent, generating concerns about boundary members and overturning forces.

Concrete Frame Buildings with Unreinforced Masonry Infill Walls (C3):
These buildings are similar to steel frame buildings with unreinforced masonry infill walls except that the frame is of reinforced concrete. In these buildings, the shear strength of the columns, after cracking of the infill, may limit the semi-ductile behavior of the system.

Precast Concrete Tilt-Up Walls (PC1):
These buildings have a wood or metal deck roof diaphragm, which often is very large, that distributes lateral forces to precast concrete shear walls. The walls are thin but relatively heavy while the roofs are relatively light. Older or non-seismic-code buildings often have inadequate connections for anchorage of the walls to the roof for out-of-plane forces, and the panel connections often are brittle. Tilt-up buildings usually are one or two stories in height. Walls can have numerous openings for doors and windows of such size that the wall looks more like a frame than a shear wall.

Precast Concrete Frames with Concrete Shear Walls (PC2):
These buildings contain floor and roof diaphragms typically composed of precast concrete elements with or without cast-in-place concrete topping slabs. Precast concrete girders and columns support the diaphragms. The girders often bear on column corbels. Closure strips between precast floor elements and beam-column joints usually are cast-in-place concrete. Welded steel inserts often are used to interconnect precast elements. Precast or cast-in-place concrete shear walls resist lateral loads. For buildings with precast frames and concrete shear walls to perform well, the details used to connect the structural elements must have sufficient strength and displacement capacity; however, in some cases, the connection details between the precast elements have negligible ductility.

Reinforced Masonry Bearing Walls with Wood or Metal Deck Diaphragms (RM1):
These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed with wood joists and beams either with plywood or braced sheathing, the latter either straight or diagonally sheathed, or with steel beams with metal deck with or without concrete fill. Interior wood posts or steel columns support wood floor framing; steel columns support steel beams.

Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms (RM2):
These buildings have bearing walls similar to those of reinforced masonry bearing wall structures with wood or metal deck diaphragms, but the roof and floors are composed of precast concrete elements such as planks or tee-beams and the precast roof and floor elements are supported on interior beams and columns of steel or concrete (cast-in-place or precast). The precast horizontal elements often have a cast-in-place topping.

Unreinforced Masonry Bearing Walls (URM):
These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood

framing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In unreinforced masonry constructed after 1950 (outside California) wood floors usually have plywood rather than board sheathing. In regions of lower seismicity, buildings of this type constructed more recently can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can reduce diaphragm displacements.

Mobile Homes (MH):
These are prefabricated housing units that are trucked to the site and then placed on isolated piers, jack stands, or masonry block foundations (usually without any positive anchorage). Floors and roofs of mobile homes usually are constructed with plywood and outside surfaces are covered with sheet metal.

5.2.2 Nonstructural Components

Nonstructural components include a large variety of different architectural, mechanical and electrical components (e.g., components listed in the NEHRP seismic design provisions for new buildings [FEMA, 1997a]). Contents of the buildings are treated as a separate category. Nonstructural components are grouped as either "drift-sensitive" or "acceleration-sensitive" components, in order to assess their damage due to an earthquake. Damage to drift-sensitive nonstructural components is primarily a function of interstory drift; damage to acceleration-sensitive nonstructural components and building contents is primarily a function of floor acceleration. Table 5.2 lists typical nonstructural components and building contents, and identifies each item as drift-sensitive or acceleration sensitive.

Anchorage/bracing of nonstructural components improves earthquake performance of most components although routine or typical anchorage/bracing provides only limited damage protection. It is assumed that typical nonstructural components and building contents have limited anchorage/bracing. Exceptions, such as special anchorage/bracing requirements for nonstructural components and contents of hospitals are addressed in Chapter 6. Nonstructural damage evaluation is dependent upon the response and performance of structural components, as well as being influenced by characteristics of nonstructural components themselves. Nonstructural damage simplifying assumptions are outlined in the following sections.

Table 5.2 List of Typical Nonstructural Components and Contents of Buildings

Type	Item	Drift-Sensitive*	Acceleration-Sensitive*
Architectural	Nonbearing Walls/Partitions	•	◦
	Cantilever Elements and Parapets		•
	Exterior Wall Panels	•	◦
	Veneer and Finishes	•	◦
	Penthouses	•	
	Racks and Cabinets		•
	Access Floors		•
	Appendages and Ornaments		•
Mechanical and Electrical	General Mechanical (boilers, etc.)		•
	Manufacturing and Process Machinery		•
	Piping Systems	◦	•
	Storage Tanks and Spheres		•
	HVAC Systems (chillers, ductwork, etc.)	◦	•
	Elevators	◦	•
	Trussed Towers		•
	General Electrical (switchgear, ducts, etc.)	◦	•
	Lighting Fixtures		•
Contents	File Cabinets, Bookcases, etc.		•
	Office Equipment and Furnishings		•
	Computer/Communication Equipment		•
	Nonpermanent Manufacturing Equipment		•
	Manufacturing/Storage Inventory		•
	Art and other Valuable Objects		•

* Solid dots indicate primary cause of damage, open dots indicate secondary cause of damage

5.3 Description of Building Damage States

The results of damage estimation methods described in this chapter (i.e., damage predictions for model building types for a given level of ground shaking) are used in other modules of the methodology to estimate: (1) casualties due to structural damage, including fatalities, (2) monetary losses due to building damage (i.e. cost of repairing or replacing damaged buildings and their contents); (3) monetary losses resulting from building damage and closure (e.g., losses due to business interruption); (4) social impacts (e.g., loss of shelter); and, (5) other economic and social impacts.

The building damage predictions may also be used to study expected damage patterns in a given region for different scenario earthquakes (e.g., to identify the most vulnerable building types, or the areas expected to have the most damaged buildings).

In order to meet the needs of such broad purposes, damage predictions must allow the user to glean the nature and extent of the physical damage to a building type from the damage prediction output so that life-safety, societal functional and monetary losses which result from the damage can be estimated. Building damage can best be described in terms of its components (beams, columns, walls, ceilings, piping, HVAC equipment, etc.). For example, such component damage descriptions as “shear walls are cracked”, “ceiling tiles fell”, “diagonal bracing buckled”, “wall panels fell out”, etc. used together with such terms as “some” and “most” would be sufficient to describe the nature and extent of overall building damage.

Damage to nonstructural components of buildings (i.e., architectural components, such as partition walls and ceilings, and building mechanical/electrical systems) primarily affects monetary and societal functional losses and generates numerous casualties of mostly light-to-moderate severity. Damage to structural components (i.e., the gravity and lateral-load-resisting systems) of buildings, Hazard mitigation measures are different for these two categories of building components as well. Hence, it is desirable to separately estimate structural and nonstructural damage.

Building damage varies from “none” to “complete” as a continuous function of building deformations (building response). Wall cracks may vary from invisible or “hairline cracks” to cracks of several inches wide. Generalized “ranges” of damage are used by the Methodology to describe structural and nonstructural damage, since it is not practical to describe building damage as a continuous function.

The Methodology predicts a structural and nonstructural damage state in terms of one of four ranges of damage or “damage states”: Slight, Moderate, Extensive, and Complete. For example, the Slight damage state extends from the threshold of Slight damage up to the threshold of Moderate damage. General descriptions of these damage states are provided for all model building types with reference to observable damage incurred by structural (Section 5.3.1) and nonstructural building components (Section 5.3.2). Damage predictions resulting from this physical damage estimation method are then expressed in terms of the probability of a building being in any of these four damage states.

5.3.1 Structural Damage

Descriptions for Slight, Moderate, Extensive, and Complete structural damage states for the 16 basic model building types are provided below. For estimating casualties, the descriptions of Complete damage include the fraction of the total floor area of each model building type that is likely to collapse. Collapse fractions are based on judgment and limited earthquake data considering the material and construction of different model building types.

It is noted that in some cases the structural damage is not directly observable because the structural elements are inaccessible or not visible due to architectural finishes or fireproofing. Hence, these structural damage states are described, when necessary, with reference to certain effects on nonstructural elements that may be indicative of the structural damage state of concern. Small cracks are assumed, throughout this section, to be visible cracks with a maximum width of less than 1/8". Cracks wider than 1/8" are

Wood, Light Frame (W1):

Slight Structural Damage: Small plaster or gypsum-board cracks at corners of door and window openings and wall-ceiling intersections; small cracks in masonry chimneys and masonry veneer.
Moderate Structural Damage: Large plaster or gypsum-board cracks at corners of door and window openings; small diagonal cracks across shear wall panels exhibited by small cracks in stucco and gypsum wall panels; large cracks in brick chimneys; toppling of tall masonry chimneys.
Extensive Structural Damage: Large diagonal cracks across shear wall panels or large cracks at plywood joints; permanent lateral movement of floors and roof; toppling of most brick chimneys; cracks in foundations; splitting of wood sill plates and/or slippage of structure over foundations; partial collapse of "room-over-garage" or other "soft-story" configurations; small foundations cracks.
Complete Structural Damage: Structure may have large permanent lateral displacement, may collapse, or be in imminent danger of collapse due to cripple wall failure or the failure of the lateral load resisting system; some structures may slip and fall off the foundations; large foundation cracks. Approximately 5% of the total area of W1 buildings with Complete damage is expected to be collapsed.

Wood, Commercial and Industrial (W2):

Slight Structural Damage: Small cracks at corners of door and window openings and wall-ceiling intersections; small cracks on stucco and plaster walls. Some slippage may be observed at bolted connections.
Moderate Structural Damage: Larger cracks at corners of door and window openings; small diagonal cracks across shear wall panels exhibited by cracks in stucco and gypsum wall panels; minor slack (less than 1/8" extension) in diagonal rod bracing requiring re-tightening; minor lateral set at store fronts and other large openings; small cracks or wood splitting may be observed at bolted connections.
Extensive Structural Damage: Large diagonal cracks across shear wall panels; large slack in diagonal rod braces and/or broken braces; permanent lateral movement of floors and roof; cracks in foundations; splitting of wood sill plates and/or slippage of structure over foundations; partial collapse of "soft-story" configurations; bolt slippage and wood splitting at bolted connections.
Complete Structural Damage: Structure may have large permanent lateral displacement, may collapse or be in imminent danger of collapse due to failed shear walls, broken brace rods or failed framing connections; it may fall its foundations; large cracks in the foundations. Approximately 5% of the total area of W2 buildings with Complete damage is expected to be collapsed.

Steel Moment Frame (S1):

Slight Structural Damage: Minor deformations in connections or hairline cracks in few welds.
Moderate Structural Damage: Some steel members have yielded exhibiting observable permanent rotations at connections; few welded connections may exhibit major cracks through welds or few bolted connections may exhibit broken bolts or enlarged bolt holes.
Extensive Structural Damage: Most steel members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some of the structural members or connections may have exceeded their ultimate capacity exhibited by major permanent member rotations at connections, buckled flanges and failed connections. Partial collapse of portions of structure is possible due to failed critical elements and/or connections.
Complete Structural Damage: Significant portion of the structural elements have exceeded their ultimate capacities or some critical structural elements or connections have failed resulting in dangerous permanent lateral displacement, partial collapse or collapse of the building. Approximately 20%(low-rise), 15%(mid-rise) or 10%(high-rise) of the total area of S1 buildings with Complete damage is expected to be collapsed.

Steel Braced Frame (S2):

Slight Structural Damage: Few steel braces have yielded which may be indicated by minor stretching and/or buckling of slender brace members; minor cracks in welded connections; minor deformations in bolted brace connections.
Moderate Structural Damage: Some steel braces have yielded exhibiting observable stretching and/or buckling of braces; few braces, other members or connections have indications of reaching their ultimate capacity exhibited by buckled braces, cracked welds, or failed bolted connections.
Extensive Structural Damage: Most steel brace and other members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some structural members or connections have exceeded their ultimate capacity exhibited by buckled or broken braces, flange buckling, broken welds, or failed bolted connections. Anchor bolts at columns may be stretched. Partial collapse of portions of structure is possible due to failure of critical elements or connections.
Complete Structural Damage: Most the structural elements have reached their ultimate capacities or some critical members or connections have failed resulting in dangerous permanent lateral deflection, partial collapse or collapse of the building. Approximately 20%(low-rise), 15%(mid-rise) or 10%(high-rise) of the total area of S2 buildings with Complete damage is expected to be collapsed.

Steel Light Frame (S3):

These structures are mostly single story structures combining rod-braced frames in one direction and moment frames in the other. Due to repetitive nature of the structural systems, the type of damage to structural members is expected to be rather uniform throughout the structure.

Slight Structural Damage: Few steel rod braces have yielded which may be indicated by minor sagging of rod braces. Minor cracking at welded connections or minor deformations at bolted connections of moment frames may be observed.

Moderate Structural Damage: Most steel braces have yielded exhibiting observable significantly sagging rod braces; few brace connections may be broken. Some weld cracking may be observed in the moment frame connections.

Extensive Structural Damage: Significant permanent lateral deformation of the structure due to broken brace rods, stretched anchor bolts and permanent deformations at moment frame members. Some screw or welded attachments of roof and wall siding to steel framing may be broken. Some purlin and girt connections may be broken.

Complete Structural Damage: Structure is collapsed or in imminent danger of collapse due to broken rod bracing, failed anchor bolts or failed structural members or connections. Approximately 25% of the total area of S3 buildings with Complete damage is expected to be collapsed.

Steel Frame with Cast-In-Place Concrete Shear Walls (S4):

This is a “composite” structural system where primary lateral-force-resisting system is the concrete shear walls. Hence, slight, Moderate and Extensive damage states are likely to be determined by the shear walls while the collapse damage state would be determined by the failure of the structural frame.

Slight Structural Damage: Diagonal hairline cracks on most concrete shear wall surfaces; minor concrete spalling at few locations.

Moderate Structural Damage: Most shear wall surfaces exhibit diagonal cracks; some of the shear walls have exceeded their yield capacities exhibited by larger diagonal cracks and concrete spalling at wall ends.

Extensive Structural Damage: Most concrete shear walls have exceeded their yield capacities; few walls have reached or exceeded their ultimate capacity exhibited by large through-the wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement. Partial collapse may occur due to failed connections of steel framing to concrete walls. Some damage may be observed in steel frame connections.

Complete Structural Damage: Structure may be in danger of collapse or collapse due to total failure of shear walls and loss of stability of the steel frames. Approximately 20%(low-rise), 15%(mid-rise) or 10%(high-rise) of the total area of S4 buildings with Complete damage is expected to be collapsed.

Steel Frame with Unreinforced Masonry Infill Walls (S5):

This is a "composite" structural system where the initial lateral resistance is provided by the infill walls. Upon cracking of the infills, further lateral resistance is provided by the steel frames "braced" by the infill walls acting as diagonal compression struts. Collapse of the structure results when the infill walls disintegrate (due to compression failure of the masonry "struts") and the steel frame loses its stability.

Slight Structural Damage: Diagonal (sometimes horizontal) hairline cracks on most infill walls; cracks at frame-infill interfaces.
Moderate Structural Damage: Most infill wall surfaces exhibit larger diagonal or horizontal cracks; some walls exhibit crushing of brick around beam-column connections.
Extensive Structural Damage: Most infill walls exhibit large cracks; some bricks may be dislodged and fall; some infill walls may bulge out-of-plane; few walls may fall off partially or fully; some steel frame connections may have failed. Structure may exhibit permanent lateral deformation or partial collapse due to failure of some critical members.
Complete Structural Damage: Structure is collapsed or in danger of imminent collapse due to total failure of many infill walls and loss of stability of the steel frames. . Approximately 25%(low-rise), 20%(mid-rise) or 15%(high-rise) of the total area of S5 buildings with Complete damage is expected to be collapsed.

Reinforced Concrete Moment Resisting Frames (C1):

Slight Structural Damage: Flexural or shear type hairline cracks in some beams and columns near joints or within joints.
Moderate Structural Damage: Most beams and columns exhibit hairline cracks. In ductile frames some of the frame elements have reached yield capacity indicated by larger flexural cracks and some concrete spalling. Nonductile frames may exhibit larger shear cracks and spalling.
Extensive Structural Damage: Some of the frame elements have reached their ultimate capacity indicated in ductile frames by large flexural cracks, spalled concrete and buckled main reinforcement; nonductile frame elements may have suffered shear failures or bond failures at reinforcement splices, or broken ties or buckled main reinforcement in columns which may result in partial collapse.
Complete Structural Damage: Structure is collapsed or in imminent danger of collapse due to brittle failure of nonductile frame elements or loss of frame stability. Approximately 20%(low-rise), 15%(mid-rise) or 10%(high-rise) of the total area of C1 buildings with Complete damage is expected to be collapsed.

Concrete Shear Walls (C2):

Slight Structural Damage: Diagonal hairline cracks on most concrete shear wall surfaces; minor concrete spalling at few locations.

Moderate Structural Damage: Most shear wall surfaces exhibit diagonal cracks; some shear walls have exceeded yield capacity indicated by larger diagonal cracks and concrete spalling at wall ends.
Extensive Structural Damage: Most concrete shear walls have exceeded their yield capacities; some walls have exceeded their ultimate capacities indicated by large, through-the-wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement or rotation of narrow walls with inadequate foundations. Partial collapse may occur due to failure of nonductile columns not designed to resist lateral loads.
Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of most of the shear walls and failure of some critical beams or columns. Approximately 20%(low-rise), 15%(mid-rise) or 10%(high-rise) of the total area of C2 buildings with Complete damage is expected to be collapsed.

Concrete Frame Buildings with Unreinforced Masonry Infill Walls (C3):

This is a “composite” structural system where the initial lateral resistance is provided by the infill walls. Upon cracking of the infills, further lateral resistance is provided by the concrete frame “braced” by the infill acting as diagonal compression struts. Collapse of the structure results when the infill walls disintegrate (due to compression failure of the masonry “struts”) and the frame loses stability, or when the concrete columns suffer shear failures due to reduced effective height and the high shear forces imposed on them by the masonry compression struts.

Slight Structural Damage: Diagonal (sometimes horizontal) hairline cracks on most infill walls; cracks at frame-infill interfaces.
Moderate Structural Damage: Most infill wall surfaces exhibit larger diagonal or horizontal cracks; some walls exhibit crushing of brick around beam-column connections. Diagonal shear cracks may be observed in concrete beams or columns.
Extensive Structural Damage: Most infill walls exhibit large cracks; some bricks may dislodge and fall; some infill walls may bulge out-of-plane; few walls may fall partially or fully; few concrete columns or beams may fail in shear resulting in partial collapse. Structure may exhibit permanent lateral deformation.
Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to a combination of total failure of the infill walls and nonductile failure of the concrete beams and columns. Approximately 25%(low-rise), 20%(mid-rise) or 15%(high-rise) of the total area of C3 buildings with Complete damage is expected to be collapsed.

Precast Concrete Tilt-Up Walls (PC1):

Slight Structural Damage: Diagonal hairline cracks on concrete shear wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; minor concrete spalling at few locations; minor separation of walls from the floor and roof diaphragms; hairline cracks around metal connectors between wall panels and at connections of beams to walls.

Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; larger cracks in walls with door or window openings; few shear walls have exceeded their yield capacities indicated by larger diagonal cracks and concrete spalling. Cracks may appear at top of walls near panel intersections indicating "chord" yielding. Some walls may have visibly pulled away from the roof. Some welded panel connections may have been broken, indicated by spalled concrete around connections. Some spalling may be observed at the connections of beams to walls.

Extensive Structural Damage: In buildings with relatively large area of wall openings most concrete shear walls have exceeded their yield capacities and some have exceeded their ultimate capacities indicated by large, through-the-wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement. The plywood diaphragms may exhibit cracking and separation along plywood joints. Partial collapse of the roof may result from the failure of the wall-to-diaphragm anchorages sometimes with falling of wall panels.

Complete Structural Damage: Structure is collapsed or is in imminent danger of collapse due to failure of the wall-to-roof anchorages, splitting of ledgers, or failure of plywood-to-ledger nailing; failure of beams connections at walls; failure of roof or floor diaphragms; or, failure of the wall panels. Approximately 25% of the total area of PC1 buildings with Complete damage is expected to be collapsed.

<u>Precast Concrete Frames with Concrete Shear Walls (PC2):</u>

Slight Structural Damage: Diagonal hairline cracks on most shear wall surfaces; minor concrete spalling at few connections of precast members.

Moderate Structural Damage: Most shear wall surfaces exhibit diagonal cracks; some shear walls have exceeded their yield capacities indicated by larger cracks and concrete spalling at wall ends; observable distress or movement at connections of precast frame connections, some failures at metal inserts and welded connections.

Extensive Structural Damage: Most concrete shear walls have exceeded their yield capacities; some walls may have reached their ultimate capacities indicated by large, through-the wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement. Some critical precast frame connections may have failed resulting partial collapse.

Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of the shear walls and/or failures at precast frame connections. Approximately 25%(low-rise), 20%(mid-rise) or 15%(high-rise) of the total area of PC2 buildings with Complete damage is expected to be collapsed.

<u>Reinforced Masonry Bearing Walls with Wood or Metal Deck Diaphragms (RM1):</u>

Slight Structural Damage: Diagonal hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; minor separation of walls from the floor and roof diaphragms.

Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the shear walls have exceeded their yield capacities indicated by larger diagonal cracks. Some walls may have visibly pulled away from the roof.
Extensive Structural Damage: In buildings with relatively large area of wall openings most shear walls have exceeded their yield capacities and some of the walls have exceeded their ultimate capacities indicated by large, through-the-wall diagonal cracks and visibly buckled wall reinforcement. The plywood diaphragms may exhibit cracking and separation along plywood joints. Partial collapse of the roof may result from failure of the wall-to-diaphragm anchorages or the connections of beams to walls.
Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of the wall anchorages or due to failure of the wall panels. Approximately 20%(low-rise), 15%(mid-rise) or 10%(high-rise) of the total area of RM1 buildings with Complete damage is expected to be collapsed.

Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms (RM2):

Slight Structural Damage: Diagonal hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings.
Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the shear walls have exceeded their yield capacities indicated by larger cracks.
Extensive Structural Damage: In buildings with relatively large area of wall openings most shear walls have exceeded their yield capacities and some of the walls have exceeded their ultimate capacities exhibited by large, through-the wall diagonal cracks and visibly buckled wall reinforcement. The diaphragms may also exhibit cracking
Complete Structural Damage: Structure is collapsed or is in imminent danger of collapse due to failure of the walls. Approximately 20%(low-rise), 15%(mid-rise) or 10%(high-rise) of the total area of RM2 buildings with Complete damage is expected to be collapsed.

Unreinforced Masonry Bearing Walls (URM):

Slight Structural Damage: Diagonal, stair-step hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; movements of lintels; cracks at the base of parapets.
Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the walls exhibit larger diagonal cracks; masonry walls may have visible separation from diaphragms; significant cracking of parapets; some masonry may fall from walls or parapets.
Extensive Structural Damage: In buildings with relatively large area of wall openings most walls have suffered extensive cracking. Some parapets and gable end walls have fallen. Beams or trusses may have moved relative to their supports.
Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to in-plane or out-of-plane failure of the walls. Approximately 25% of the total area of URM buildings with Complete damage is expected to be collapsed.

Mobile Homes (MH):

Slight Structural Damage: Damage to some porches, stairs or other attached components.
Moderate Structural Damage: Major movement of the mobile home over its supports resulting in some damage to metal siding and stairs and requiring resetting of the mobile home on its supports.
Extensive Structural Damage: Mobile home has fallen partially off its supports, often severing utility lines.
Complete Structural Damage: Mobile home has totally fallen off its supports; usually severing utility lines, with steep jack stands penetrating through the floor. Approximately 5% of the total area of MH buildings with Complete damage is expected to be collapsed.

5.3.2 Nonstructural Damage

Four damage states are used to describe nonstructural damage: Slight, Moderate, Extensive and Complete nonstructural damage. Nonstructural damage is considered to be independent of the structural model building type (i.e. partitions, ceilings, cladding, etc. are assumed to incur the same damage when subjected to the same interstory drift or floor acceleration whether they are in a steel frame building or in a concrete shear wall building), consequently, building-specific damage state descriptions are not meaningful. Instead, general descriptions of nonstructural damage states are provided for common nonstructural systems.

Damage to drift-sensitive nonstructural components is primarily a function of interstory drift (e.g. full-height drywall partitions) while for acceleration-sensitive components (e.g. mechanical equipment) damage is a function of the floor acceleration. Developing fragility curves for each possible nonstructural component is not practicable for the purposes of regional loss estimation and there is insufficient data to develop such fragility curves. Hence, in this methodology nonstructural building components are grouped into drift-sensitive and acceleration-sensitive component groups, and the damage functions estimated for each group are assumed to be "typical" of it sub-components. Note, however, that damage depends on the anchorage/bracing provided to the nonstructural components. Damageability characteristics of each group are described by a set of fragility curves (see Subsection 5.4.3.3).

The type of nonstructural components in a given building is a function of the building occupancy-use classification. For example, single-family residences would not have curtain wall panels, suspended ceilings, elevators, etc. while these items would be found in an office building. Hence, the relative values of nonstructural components in relation to the overall building replacement value vary with type of occupancy. In Chapter 15, estimates of replacement cost breakdown between structural building components for different occupancy/use related classifications are provided; further breakdowns are provided by drift- and acceleration-sensitive nonstructural components.

In the following, general descriptions of the four nonstructural damage states are described for common nonstructural building components:

Partitions Walls

Slight Nonstructural Damage: A few cracks are observed at intersections of walls and ceilings and at corners of door openings.
Moderate Nonstructural Damage: Larger and more extensive cracks requiring repair and repainting; some partitions may require replacement of gypsum board or other finishes.
Extensive Nonstructural Damage: Most of the partitions are cracked and a significant portion may require replacement of finishes; some door frames in the partitions are also damaged and require re-setting.
Complete Nonstructural Damage: Most partition finish materials and framing may have to be removed and replaced; damaged studs repaired, and walls be refinished. Most door frames may also have to be repaired and replaced.

Suspended Ceilings

Slight Nonstructural Damage: A few ceiling tiles have moved or fallen down.
Moderate Nonstructural Damage: Falling of tiles is more extensive; in addition the ceiling support framing (T-bars) has disconnected and/or buckled at few locations; lenses have fallen off of some light fixtures and a few fixtures have fallen; localized repairs are necessary.
Extensive Nonstructural Damage: The ceiling system exhibits extensive buckling, disconnected t-bars and falling ceiling tiles; ceiling partially collapses at few locations and some light fixtures fall; repair typically involves removal of most or all ceiling tiles.
Complete Nonstructural Damage: The ceiling system is buckled throughout and/or fallen and requires complete replacement; many light fixtures fall.

Exterior Wall Panels

Slight Nonstructural Damage: Slight movement of the panels, requiring realignment.
Moderate Nonstructural Damage: The movements are more extensive; connections of panels to structural frame are damaged requiring further inspection and repairs; some window frames may need realignment
Extensive Nonstructural Damage: Most of the panels are cracked or otherwise damaged and misaligned, and most panel connections to the structural frame are damaged requiring thorough review and repairs; few panels fall or are in imminent danger of falling; some window panes are broken and some pieces of glass have fallen.
Complete Nonstructural Damage: Most panels are severely damaged, most connections are broken or severely damaged, some panels have fallen and most are in imminent danger of falling; extensive glass breakage and falling.

<u>Electrical-Mechanical Equipment, Piping, Ducts</u>

Slight Nonstructural Damage: The most vulnerable equipment (e.g. unanchored or on spring isolators) moves and damages attached piping or ducts.
Moderate Nonstructural Damage: Movements are larger and damage is more extensive; piping leaks at few locations; elevator machinery and rails may require realignment
Extensive Nonstructural Damage: Equipment on spring isolators topples and falls; other unanchored equipment slides or falls breaking connections to piping and ducts; leaks develop at many locations; anchored equipment indicate stretched bolts or strain at anchorages.
Complete Nonstructural Damage: Equipment is damaged by sliding, overturning or failure of their supports and is not operable; piping is leaking at many locations; some pipe and duct supports have failed causing pipes and ducts to fall or hang down; elevator rails are buckled or have broken supports and/or counterweights have derailed.

5.4 Building Damage Due to Ground Shaking

5.4.1 Overview

This section describes capacity and fragility curves used in the Methodology to estimate the probability of Slight, Moderate, Extensive and Complete damage to general building stocks. General building stock represents a population of a given model building type designed to either High-Code, Moderate-Code, or Low-Code seismic standards, or not seismically designed, referred to as to a Pre-Code buildings. Chapter 6 describes Special building damage functions for estimating damage to hospitals and other essential facilities that are designed and constructed to above average seismic standards.

Capacity curves and fragility curves for High-Code, Moderate-Code, Low-Code and Pre-Code buildings are based on modern code (e.g., 1976 *Uniform Building Code*, 1985 *NEHRP Provisions*, or later editions of these model codes). Design criteria for various seismic design zones, as shown in Table 5.3. Additional description of seismic levels may be found in Section 5.7.

Table 5.3 Approximate Basis for Seismic Design Levels

Seismic Design Level	Seismic Zone (*Uniform Building Code*)	Map Area (*NEHRP Provisions*)
High-Code	4	7
Moderate-Code	2B	5
Low-Code	1	3
Pre-Code	0	1

The capacity and fragility curves represent buildings designed and constructed to modern seismic code provisions. Study areas (e.g., census tracts) of recent construction are appropriately modeled using building damage functions with a seismic design level that corresponds to the seismic zone or map area of the governing provisions. Older areas of construction, not conforming to modern standards, should be modeled using a lower level of seismic design. For example, in areas of high seismicity (e.g., coastal California), buildings of newer construction (e.g., post-1973) are best represented by High-Code damage functions, while buildings of older construction would be best represented by Moderate-Code damage functions, if built after about 1940, or by Pre-Code damage functions, if built before about 1940 (i.e., before seismic codes existed). Pre-Code damage functions are appropriate for modeling older buildings that were not designed for earthquake load, regardless of where they are located in the United States. Guidance is provided to expert users in Section 5.7 for selection of appropriate building damage functions

5.4.2 Capacity Curves

Most buildings are presently designed or evaluated using linear-elastic analysis methods, primarily due to the relative simplicity of these methods in comparison to more complex, nonlinear methods. Typically, building response is based on linear-elastic properties of the structure and forces corresponding to the design-basis earthquake. For design of building elements, linear-elastic (5%-damped) response is reduced by a factor (e.g. the “R-Factor” in 1994 *NEHRP Provisions*) that varies for different types of lateral force resisting systems. The reduction factor is based on empirical data and judgment that account for the inelastic deformation capability (ductility) of the structural system, redundancy, overstrength, increased damping (above 5% of critical) at large deformations, and other factors that influence building capacity. Although this “force-based” approach is difficult to justify by rational engineering analysis, buildings designed using these methods have performed reasonably well in past earthquakes. Aspects of these methods found not to work well in earthquakes have been studied and improved. In most cases, building capacity has been increased by improvements to detailing practices (e.g., better confinement of steel reinforcement in concrete elements).

Except for a few brittle systems and acceleration-sensitive elements, building damage is primarily a function of building displacement, rather than force. In the inelastic range of building response, increasingly larger damage would result from increased building displacement although lateral force would remain constant or decrease. Hence, successful prediction of earthquake damage to buildings requires reasonably accurate estimation of building displacement response in the inelastic range. This, however, can not be accomplished using linear-elastic methods, since the buildings respond inelastically to earthquake ground shaking of magnitudes of interest for damage prediction. Building capacity (push-over) curves, used with capacity spectrum method (CSM) techniques [Mahaney, et. al., 1993, Kircher, 1996], provide simple and reasonably accurate means of predicting inelastic building displacement response for damage estimation purposes.

A building capacity curve (also known as a push-over curve) is a plot of a building's lateral load resistance as a function of a characteristic lateral displacement (i.e., a force-deflection plot). It is derived from a plot of static-equivalent base shear versus building (e.g., roof) displacement. In order to facilitate direct comparison with earthquake demand (i.e. overlaying the capacity curve with a response spectrum), the force (base shear) axis is converted to spectral acceleration and the displacement axis is converted to spectral displacement. Such a plot provides an estimate of the building's "true" deflection (displacement response) for any given earthquake response spectrum.

The building capacity curves developed for the Methodology are based on engineering design parameters and judgment. Three control points that define model building capacity describe each curve:

- Design Capacity
- Yield Capacity
- Ultimate Capacity

Design capacity represents the nominal building strength required by current model seismic code provisions (e.g., 1994 *NEHRP Provisions*) or an estimate of the nominal strength for buildings not designed for earthquake loads. Wind design is not considered in the estimation of design capacity, and certain buildings (e.g., tall buildings located in zones of low or moderate seismicity) may have a lateral design strength considerably greater than that based on seismic code provisions.

Yield capacity represents the true lateral strength of the building considering redundancies in design, conservatism in code requirements and true (rather than nominal) strength of materials. Ultimate capacity represents the maximum strength of the building when the global structural system has reached a fully plastic state. Ultimate capacity implicitly accounts for loss of strength due to shear failure of brittle elements. Typically, buildings are assumed capable of deforming beyond their ultimate point without loss of stability, but their structural system provides no additional resistance to lateral earthquake force.

Up to the yield point, the building capacity curve is assumed to be linear with stiffness based on an estimate of the true period of the building. The true period is typically longer than the code-specified period of the building due to flexing of diaphragms of short, stiff buildings, flexural cracking of elements of concrete and masonry structures, flexibility of foundations and other factors observed to affect building stiffness. From the yield point to the ultimate point, the capacity curve transitions in slope from an essentially elastic state to a fully plastic state. The capacity curve is assumed to remain plastic past the ultimate point. An example building capacity curve is shown in Figure 5.3.

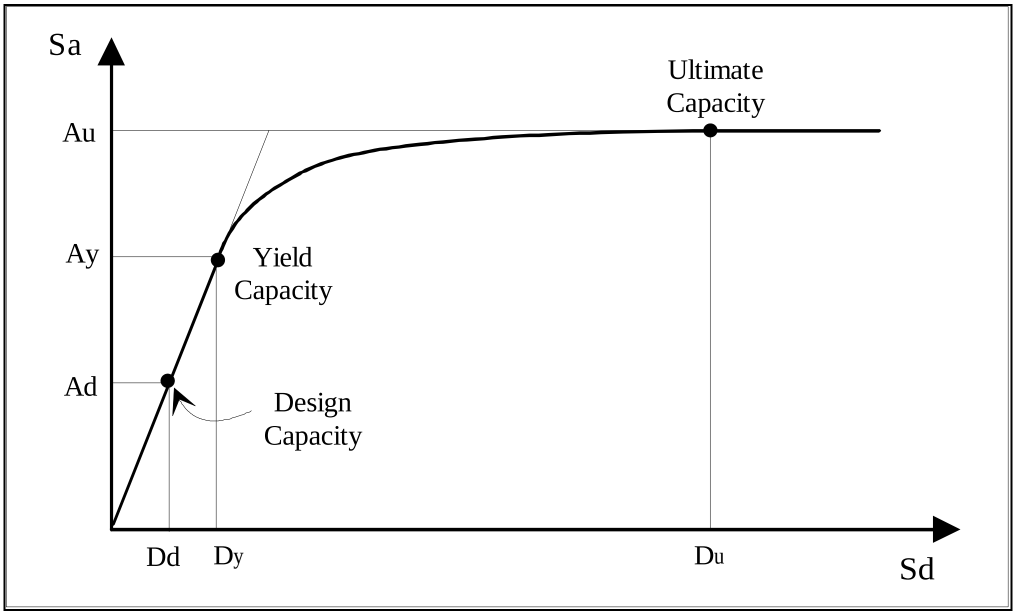


Figure 5.3 Example Building Capacity Curve.

The building capacity curves are constructed based on estimates of engineering properties that affect the design, yield and ultimate capacities of each model building type. These properties are defined by the following parameters:

C_s	design strength coefficient (fraction of building's weight),
T_e	true "elastic" fundamental-mode period of building (seconds),
α_1	fraction of building weight effective in push-over mode,
α_2	fraction of building height at location of push-over mode displacement,
γ	"overstrength" factor relating "true" yield strength to design strength,
λ	"overstrength" factor relating ultimate strength to yield strength, and
μ	"ductility" factor relating ultimate displacement to λ times the yield displacement (i.e., assumed point of significant yielding of the structure)

The design strength, C_s, is approximately based, on the lateral-force design requirements of current seismic codes (e.g., *1994 NEHRP Provisions*). These requirements are a function of the building's seismic zone location and other factors including: site soil condition, type of lateral-force-resisting system and building period. For each of the four design levels (High-Code, Moderate-Code, Low-Code and Pre-Code), design capacity is based on the best estimate of typical design properties. Table 5.4 summarizes design capacity for each building type and design level. Building period, T_e, push-over mode parameters α_1 and α_2, the ratio of yield to design strength, γ, and the ratio of ultimate to yield strength, λ, are assumed to be independent of design level. Values of these parameters are summarized in Table 5.5 for each building type. Values of the "ductility" factor, μ, are given in Table 5.6 for each building type and design level. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 5.4 Code Building Capacity Parameters - Design Strength (C_s)

Building Type	Seismic Design Level (Fraction of Building Weight)			
	High-Code	Moderate-Code	Low-Code	Pre-Code
W1	0.200	0.150	0.100	0.100
W2	0.200	0.100	0.050	0.050
S1L	0.133	0.067	0.033	0.033
S1M	0.100	0.050	0.025	0.025
S1H	0.067	0.033	0.017	0.017
S2L	0.200	0.100	0.050	0.050
S2M	0.200	0.100	0.050	0.050
S2H	0.150	0.075	0.038	0.038
S3	0.200	0.100	0.050	0.050
S4L	0.160	0.080	0.040	0.040
S4M	0.160	0.080	0.040	0.040
S4H	0.120	0.060	0.030	0.030
S5L			0.050	0.050
S5M			0.050	0.050
S5H			0.038	0.038
C1L	0.133	0.067	0.033	0.033
C1M	0.133	0.067	0.033	0.033
C1H	0.067	0.033	0.017	0.017
C2L	0.200	0.100	0.050	0.050
C2M	0.200	0.100	0.050	0.050
C2H	0.150	0.075	0.038	0.038
C3L			0.050	0.050
C3M			0.050	0.050
C3H			0.038	0.038
PC1	0.200	0.100	0.050	0.050
PC2L	0.200	0.100	0.050	0.050
PC2M	0.200	0.100	0.050	0.050
PC2H	0.150	0.075	0.038	0.038
RM1L	0.267	0.133	0.067	0.067
RM1M	0.267	0.133	0.067	0.067
RM2L	0.267	0.133	0.067	0.067
RM2M	0.267	0.133	0.067	0.067
RM2H	0.200	0.100	0.050	0.050
URML			0.067	0.067
URMM			0.067	0.067
MH	0.100	0.100	0.100	0.100

Table 5.5 Code Building Capacity Parameters - Period (T_e), Pushover Mode Response Factors (α_1, α_2) and Overstrength Ratios (γ, λ)

Building Type	Height to Roof (Feet)	Period, T_e (Seconds)	Modal Factors		Overstrength Ratios	
			Weight, α_1	Height, α_2	Yield, γ	Ultimate, λ
W1	14.0	0.35	0.75	0.75	1.50	3.00
W2	24.0	0.40	0.75	0.75	1.50	2.50
S1L	24.0	0.50	0.80	0.75	1.50	3.00
S1M	60.0	1.08	0.80	0.75	1.25	3.00
S1H	156.0	2.21	0.75	0.60	1.10	3.00
S2L	24.0	0.40	0.75	0.75	1.50	2.00
S2M	60.0	0.86	0.75	0.75	1.25	2.00
S2H	156.0	1.77	0.65	0.60	1.10	2.00
S3	15.0	0.40	0.75	0.75	1.50	2.00
S4L	24.0	0.35	0.75	0.75	1.50	2.25
S4M	60.0	0.65	0.75	0.75	1.25	2.25
S4H	156.0	1.32	0.65	0.60	1.10	2.25
S5L	24.0	0.35	0.75	0.75	1.50	2.00
S5M	60.0	0.65	0.75	0.75	1.25	2.00
S5H	156.0	1.32	0.65	0.60	1.10	2.00
C1L	20.0	0.40	0.80	0.75	1.50	3.00
C1M	50.0	0.75	0.80	0.75	1.25	3.00
C1H	120.0	1.45	0.75	0.60	1.10	3.00
C2L	20.0	0.35	0.75	0.75	1.50	2.50
C2M	50.0	0.56	0.75	0.75	1.25	2.50
C2H	120.0	1.09	0.65	0.60	1.10	2.50
C3L	20.0	0.35	0.75	0.75	1.50	2.25
C3M	50.0	0.56	0.75	0.75	1.25	2.25
C3H	120.0	1.09	0.65	0.60	1.10	2.25
PC1	15.0	0.35	0.50	0.75	1.50	2.00
PC2L	20.0	0.35	0.75	0.75	1.50	2.00
PC2M	50.0	0.56	0.75	0.75	1.25	2.00
PC2H	120.0	1.09	0.65	0.60	1.10	2.00
RM1L	20.0	0.35	0.75	0.75	1.50	2.00
RM1M	50.0	0.56	0.75	0.75	1.25	2.00
RM2L	20.0	0.35	0.75	0.75	1.50	2.00
RM2M	50.0	0.56	0.75	0.75	1.25	2.00
RM2H	120.0	1.09	0.65	0.60	1.10	2.00
URML	15.0	0.35	0.50	0.75	1.50	2.00
URMM	35.0	0.50	0.75	0.75	1.25	2.00
MH	10.0	0.35	1.00	1.00	1.50	2.00

Table 5.6 Code Building Capacity Parameter - Ductility (μ)

Building Type	Seismic Design Level			
	High-Code	Moderate-Code	Low-Code	Pre-Code
W1	8.0	6.0	6.0	6.0
W2	8.0	6.0	6.0	6.0
S1L	8.0	6.0	5.0	5.0
S1M	5.3	4.0	3.3	3.3
S1H	4.0	3.0	2.5	2.5
S2L	8.0	6.0	5.0	5.0
S2M	5.3	4.0	3.3	3.3
S2H	4.0	3.0	2.5	2.5
S3	8.0	6.0	5.0	5.0
S4L	8.0	6.0	5.0	5.0
S4M	5.3	4.0	3.3	3.3
S4H	4.0	3.0	2.5	2.5
S5L			5.0	5.0
S5M			3.3	3.3
S5H			2.5	2.5
C1L	8.0	6.0	5.0	5.0
C1M	5.3	4.0	3.3	3.3
C1H	4.0	3.0	2.5	2.5
C2L	8.0	6.0	5.0	5.0
C2M	5.3	4.0	3.3	3.3
C2H	4.0	3.0	2.5	2.5
C3L			5.0	5.0
C3M			3.3	3.3
C3H			2.5	2.5
PC1	8.0	6.0	5.0	5.0
PC2L	8.0	6.0	5.0	5.0
PC2M	5.3	4.0	3.3	3.3
PC2H	4.0	3.0	2.5	2.5
RM1L	8.0	6.0	5.0	5.0
RM1M	5.3	4.0	3.3	3.3
RM2L	8.0	6.0	5.0	5.0
RM2M	5.3	4.0	3.3	3.3
RM2H	4.0	3.0	2.5	2.5
URML			5.0	5.0
URMM			3.3	3.3
MH	6.0	6.0	6.0	6.0

Building capacity curves are assumed to have a range of possible properties that are lognormally distributed as a function of the ultimate strength (A_u) of each capacity curve. Capacity curves described by the values of parameters given in Tables 5.4, 5.5 and 5.6 represent median estimates of building capacity. The variability of the capacity of each building type is assumed to be: $\beta(A_u)$ = 0.25 for code-designed buildings (High-Code, Moderate-Code and Low-Code seismic design levels) and $\beta(A_u)$ = 0.30 for Pre-Code buildings.

Example construction of median, 84th percentile (+1β) and 16th percentile (-1β) building capacity curves for a typical building is illustrated in Figure 5.4. Median capacity curves are intersected with demand spectra to estimate peak building response. The variability of the capacity curves is used, with other sources of variability and uncertainty, to define total fragility curve variability.

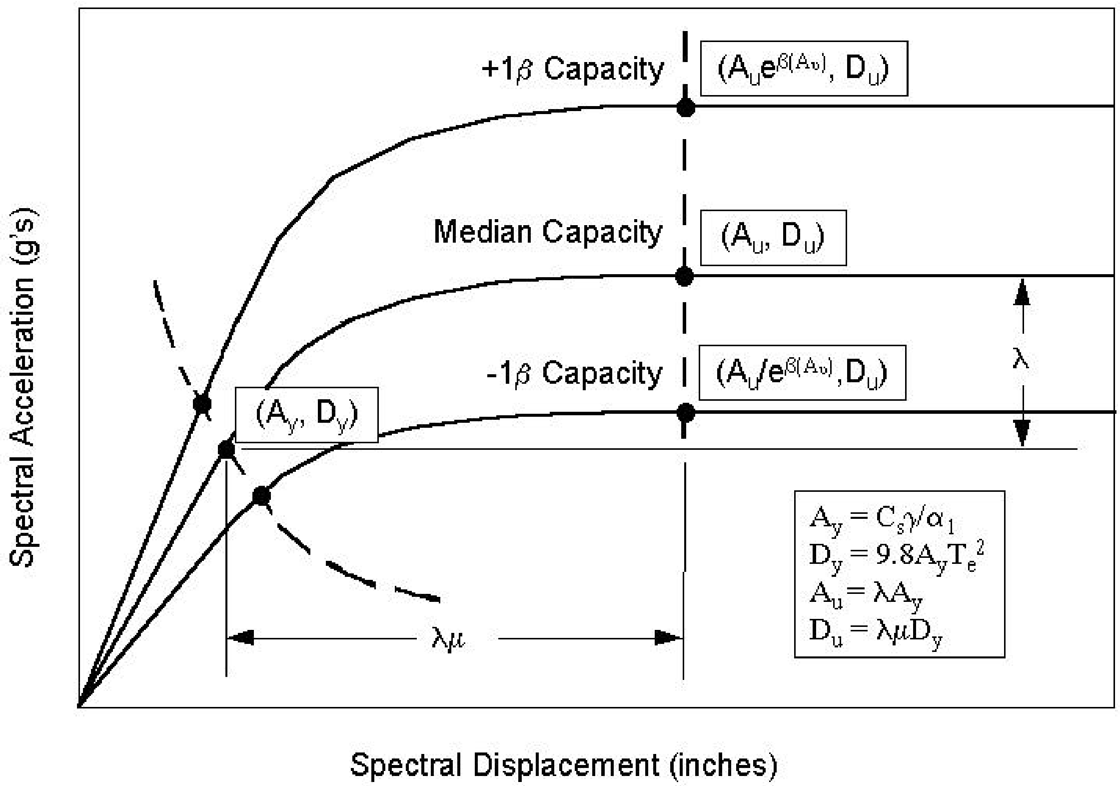


Figure 5.4 Example Construction of Median, +1β and -1β Building Capacity Curves.

Tables 5.7a, 5.7b, 5.7c and 5.7d summarize yield capacity and ultimate capacity control points for High-Code, Moderate-Code, Low-Code and Pre-Code seismic design levels, respectively. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 5.7a Code Building Capacity Curves - High-Code Seismic Design Level

Building Type	Yield Capacity Point		Ultimate Capacity Point	
	D_y (in.)	A_y (g)	D_u (in.)	A_u (g)
W1	0.48	0.400	11.51	1.200
W2	0.63	0.400	12.53	1.000
S1L	0.61	0.250	14.67	0.749
S1M	1.78	0.156	28.40	0.468
S1H	4.66	0.098	55.88	0.293
S2L	0.63	0.400	10.02	0.800
S2M	2.43	0.333	25.88	0.667
S2H	7.75	0.254	61.97	0.508
S3	0.63	0.400	10.02	0.800
S4L	0.38	0.320	6.91	0.720
S4M	1.09	0.267	13.10	0.600
S4H	3.49	0.203	31.37	0.457
S5L				
S5M				
S5H				
C1L	0.39	0.250	9.39	0.749
C1M	1.15	0.208	18.44	0.624
C1H	2.01	0.098	24.13	0.293
C2L	0.48	0.400	9.59	1.000
C2M	1.04	0.333	13.84	0.833
C2H	2.94	0.254	29.39	0.635
C3L				
C3M				
C3H				
PC1	0.72	0.600	11.51	1.200
PC2L	0.48	0.400	7.67	0.800
PC2M	1.04	0.333	11.07	0.667
PC2H	2.94	0.254	23.52	0.508
RM1L	0.64	0.533	10.23	1.066
RM1M	1.38	0.444	14.76	0.889
RM2L	0.64	0.533	10.23	1.066
RM2M	1.38	0.444	14.76	0.889
RM2H	3.92	0.338	31.35	0.677
URML				
URMM				
MH	0.18	0.150	2.16	0.300

Table 5.7b Code Building Capacity Curves - Moderate-Code Seismic Design Level

Building Type	Yield Capacity Point		Ultimate Capacity Point	
	D_y (in.)	A_y (g)	D_u (in.)	A_u (g)
W1	0.36	0.300	6.48	0.900
W2	0.31	0.200	4.70	0.500
S1L	0.31	0.125	5.50	0.375
S1M	0.89	0.078	10.65	0.234
S1H	2.33	0.049	20.96	0.147
S2L	0.31	0.200	3.76	0.400
S2M	1.21	0.167	9.70	0.333
S2H	3.87	0.127	23.24	0.254
S3	0.31	0.200	3.76	0.400
S4L	0.19	0.160	2.59	0.360
S4M	0.55	0.133	4.91	0.300
S4H	1.74	0.102	11.76	0.228
S5L				
S5M				
S5H				
C1L	0.20	0.125	3.52	0.375
C1M	0.58	0.104	6.91	0.312
C1H	1.01	0.049	9.05	0.147
C2L	0.24	0.200	3.60	0.500
C2M	0.52	0.167	5.19	0.417
C2H	1.47	0.127	11.02	0.317
C3L				
C3M				
C3H				
PC1	0.36	0.300	4.32	0.600
PC2L	0.24	0.200	2.88	0.400
PC2M	0.52	0.167	4.15	0.333
PC2H	1.47	0.127	8.82	0.254
RM1L	0.32	0.267	3.84	0.533
RM1M	0.69	0.222	5.54	0.444
RM2L	0.32	0.267	3.84	0.533
RM2M	0.69	0.222	5.54	0.444
RM2H	1.96	0.169	11.76	0.338
URML				
URMM				
MH	0.18	0.150	2.16	0.300

Table 5.7c Code Building Capacity Curves - Low-Code Seismic Design Level

Building Type	Yield Capacity Point		Ultimate Capacity Point	
	D_y (in.)	A_y (g)	D_u (in.)	A_u (g)
W1	0.24	0.200	4.32	0.600
W2	0.16	0.100	2.35	0.250
S1L	0.15	0.062	2.29	0.187
S1M	0.44	0.039	4.44	0.117
S1H	1.16	0.024	8.73	0.073
S2L	0.16	0.100	1.57	0.200
S2M	0.61	0.083	4.04	0.167
S2H	1.94	0.063	9.68	0.127
S3	0.16	0.100	1.57	0.200
S4L	0.10	0.080	1.08	0.180
S4M	0.27	0.067	2.05	0.150
S4H	0.87	0.051	4.90	0.114
S5L	0.12	0.100	1.20	0.200
S5M	0.34	0.083	2.27	0.167
S5H	1.09	0.063	5.45	0.127
C1L	0.10	0.062	1.47	0.187
C1M	0.29	0.052	2.88	0.156
C1H	0.50	0.024	3.77	0.073
C2L	0.12	0.100	1.50	0.250
C2M	0.26	0.083	2.16	0.208
C2H	0.74	0.063	4.59	0.159
C3L	0.12	0.100	1.35	0.225
C3M	0.26	0.083	1.95	0.188
C3H	0.74	0.063	4.13	0.143
PC1	0.18	0.150	1.80	0.300
PC2L	0.12	0.100	1.20	0.200
PC2M	0.26	0.083	1.73	0.167
PC2H	0.74	0.063	3.67	0.127
RM1L	0.16	0.133	1.60	0.267
RM1M	0.35	0.111	2.31	0.222
RM2L	0.16	0.133	1.60	0.267
RM2M	0.35	0.111	2.31	0.222
RM2H	0.98	0.085	4.90	0.169
URML	0.24	0.200	2.40	0.400
URMM	0.27	0.111	1.81	0.222
MH	0.18	0.150	2.16	0.300

Table 5.7d Building Capacity Curves - Pre-Code Seismic Design Level

Building Type	Yield Capacity Point		Ultimate Capacity Point	
	D_y (in.)	A_y (g)	D_u (in.)	A_u (g)
W1	0.24	0.200	4.32	0.600
W2	0.16	0.100	2.35	0.250
S1L	0.15	0.062	2.75	0.187
S1M	0.44	0.039	5.33	0.117
S1H	1.16	0.024	10.48	0.073
S2L	0.16	0.100	1.88	0.200
S2M	0.61	0.083	4.85	0.167
S2H	1.94	0.063	11.62	0.127
S3	0.16	0.100	1.88	0.200
S4L	0.10	0.080	1.30	0.180
S4M	0.27	0.067	2.46	0.150
S4H	0.87	0.051	5.88	0.114
S5L	0.12	0.100	1.20	0.200
S5M	0.34	0.083	2.27	0.167
S5H	1.09	0.063	5.45	0.127
C1L	0.10	0.062	1.76	0.187
C1M	0.29	0.052	3.46	0.156
C1H	0.50	0.024	4.52	0.073
C2L	0.12	0.100	1.80	0.250
C2M	0.26	0.083	2.60	0.208
C2H	0.74	0.063	5.51	0.159
C3L	0.12	0.100	1.35	0.225
C3M	0.26	0.083	1.95	0.188
C3H	0.74	0.063	4.13	0.143
PC1	0.18	0.150	2.16	0.300
PC2L	0.12	0.100	1.44	0.200
PC2M	0.26	0.083	2.08	0.167
PC2H	0.74	0.063	4.41	0.127
RM1L	0.16	0.133	1.92	0.267
RM1M	0.35	0.111	2.77	0.222
RM2L	0.16	0.133	1.92	0.267
RM2M	0.35	0.111	2.77	0.222
RM2H	0.98	0.085	5.88	0.169
URML	0.24	0.200	2.40	0.400
URMM	0.27	0.111	1.81	0.222
MH	0.18	0.150	2.16	0.300

5.4.3 Fragility Curves

This section describes building fragility curves for Slight, Moderate, Extensive and Complete structural damage states and Slight, Moderate, Extensive and Complete nonstructural damage states. Each fragility curve is characterized by median and lognormal standard deviation (β) values of PESH demand. Spectral displacement is the PESH parameter used for structural damage and nonstructural damage to drift-sensitive components. Spectral acceleration is the PESH parameter used for calculating nonstructural damage to acceleration-sensitive components.

5.4.3.1 Background

The probability of being in or exceeding a given damage state is modeled as a cumulative lognormal distribution. For structural damage, given the spectral displacement, S_d, the probability of being in or exceeding a damage state, ds, is modeled as:

$$P\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \tag{5-3}$$

where: $\bar{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of the damage state, ds,

β_{ds} is the standard deviation of the natural logarithm of spectral displacement of damage state, ds, and

Φ is the standard normal cumulative distribution function.

For example, a mid-rise, concrete-frame building (C1M) of High-Code seismic design has Extensive structural damage defined by a median spectral displacement value ($\bar{S}_{d,E}$) of 9.0 inches and a lognormal standard deviation value (β_E) of 0.68. The lognormal fragility curve for Extensive structural damage to this building is shown in Figure 5.5.

In Figure 5.5, the symbol, $\bar{S}$, indicates the median value of 9.0 inches. The symbol, S_+, indicates the +1 lognormal standard deviation level of the fragility curve, which is evaluated as $S_+ = \bar{S}$ x $\exp(\beta)$ = 17.8 inches. Similarly, the symbol, S_-, indicates the -1 lognormal standard deviation level of the fragility curve, which is evaluated as $S_- = \bar{S}/\exp(\beta)$ = 4.6 inches. The corresponding probabilities of being in or exceeding the Extensive damage state for this example are:

$$P\left[\text{Extensive}\cdot\text{Damage}\middle|S_d = S_- = 4.6 \text{ inches}\right] = 0.16$$

$$P\left[\text{Extensive}\cdot\text{Damage}\middle|S_d = \bar{S} = 9.0 \text{ inches}\right] = 0.50$$

$$P\left[\text{Extensive}\cdot\text{Damage}\middle|S_d = S_+ = 17.8 \text{ inches}\right] = 0.84$$

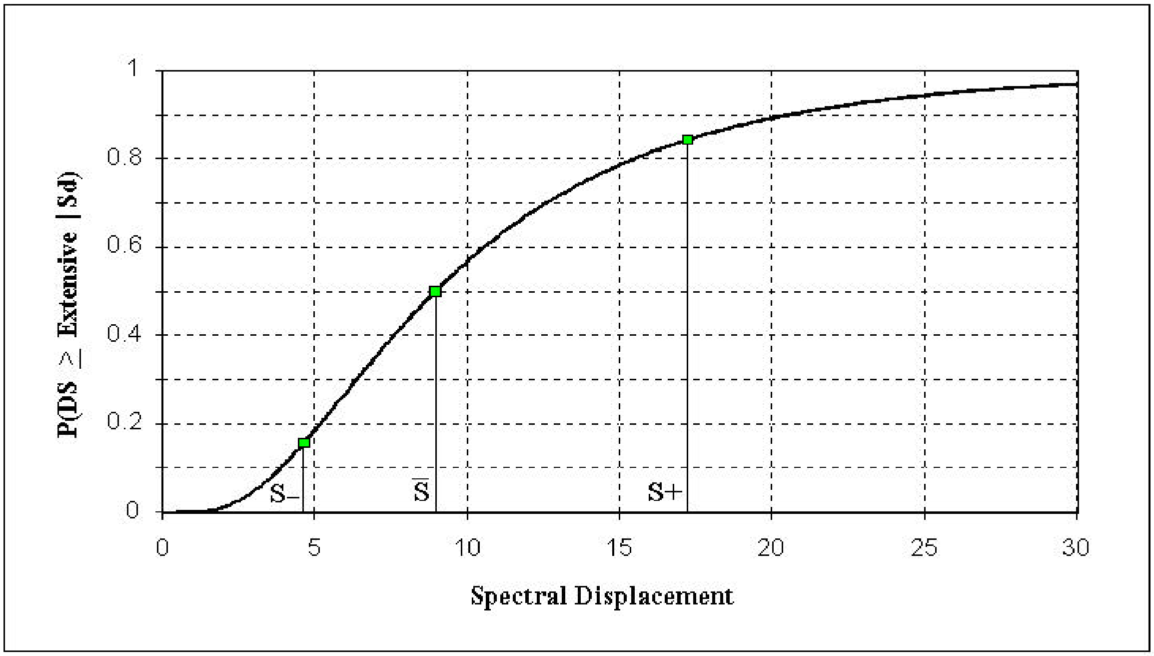


Figure 5.5 Example Fragility Curve - Extensive Structural Damage, C1M Model Building Type, High-Code Seismic Design.

5.4.3.2 Development of Damage State Medians

Median values of fragility curves are developed for each damage states (i.e., Slight, Moderate, Extensive and Complete) and for each of the three types of building components: structural, nonstructural drift-sensitive and nonstructural acceleration-sensitive components. Structural fragility is characterized in terms of spectral displacement and by equivalent-PGA fragility curves (for buildings that are components of lifelines). Section 5.4.4 describes development of median values of equivalent-PGA structural fragility curves based on the structural fragility curves of this section.

Median values of structural component fragility are based on building drift ratios that describe the threshold of damage states. Damage-state drift ratios are converted to spectral displacement using Equation (5-4):

$$\overline{S}_{d,Sds} = \delta_{R,Sds} \cdot \alpha_2 \cdot h \qquad (5\text{-}4)$$

where: $\overline{S}_{d,Sds}$ is the median value of spectral displacement, in inches, of structural components for damage state, ds,

$\delta_{R,Sds}$ is the drift ratio at the threshold of structural damage state, ds,

α_2 is the fraction of the building (roof) height at the location of push-over mode displacement, as specified in Table 5.5, and

h is the typical roof height, in inches, of the model building type of interest (see Table 5.1 for typical building height).

Values of damage-state drift ratios are included in the Methodology based, in part, on a study by OAK Engineering [OAK, 1994] that reviewed and synthesized available drift/damage information from a number of published sources, including Kustu et al. (1982), Ferritto (1982 and 1983), Czarnecki (1973), Hasselman et al. (1980), Whitman et al. (1977) and Wong (1975).

Median values of nonstructural drift-sensitive component fragility are based on building drift ratios that describe the threshold of damage states. Nonstructural drift-sensitive components are identified in Table 5.2. Damage state drift ratios for nonstructural drift-sensitive components are converted to median values of spectral displacement using the same approach as that of Equation (5-4). Values of damage-state drift are based, in part, on the work of Ferrito (1982 and 1983) and on a recent update of this data included in a California Division of the State Architect report [DSA, 1996].

Median values of nonstructural acceleration-sensitive component fragility are based on peak floor (input) acceleration that describes the threshold of damage states. These values of acceleration are used directly as median values of spectral acceleration for nonstructural acceleration-sensitive component fragility curves. Values of damage-state acceleration are based, in part, on the work of Ferrito (1982 and 1983) and on a recent update of this data included in a California Division of the State Architect report [DSA, 1996].

5.4.3.3 Development of Damage State Variability

Lognormal standard deviation (β) values that describe the variability of fragility curves are developed for each damage states (i.e., Slight, Moderate, Extensive and Complete) and for each of the three types of building components: structural, nonstructural drift-sensitive and nonstructural acceleration-sensitive components. Structural fragility is characterized in terms of spectral displacement and by equivalent-PGA fragility curves (for buildings that are components of lifelines). Section 5.4.4 describes development of variability values for equivalent-PGA structural fragility curves.

The total variability of each structural damage state, β_{Sds}, is modeled by the combination of three contributors to structural damage variability, β_C, β_D and $\beta_{M(Sds)}$, as described in Equation (5-5):

$$\beta_{Sds} = \sqrt{\left(\mathrm{CONV}\left[\beta_C, \beta_D, \overline{S}_{d,Sds}\right]\right)^2 + \left(\beta_{M(Sds)}\right)^2} \qquad (5\text{-}5)$$

where: β_{Sds} is the lognormal standard deviation that describes the total variability for structural damage state, ds,

β_C is the lognormal standard deviation parameter that describes the variability of the capacity curve,

β_D	is the lognormal standard deviation parameter that describes the variability of the demand spectrum,
$\beta_{M(Sds)}$	is the lognormal standard deviation parameter that describes the uncertainty in the estimate of the median value of the threshold of structural damage state, ds.

The variability of building response depends jointly on demand and capacity (since capacity curves are nonlinear). The function "CONV" in Equation (5-5) implies a complex process of convolving probability distributions of the demand spectrum and the capacity curve, respectively. Demand spectra and capacity curves are described probabilistically by median properties and variability parameters, β_D and β_C, respectively. Capacity curves are defined for each building type, but the demand spectrum is based on the PESH input spectrum whose shape is a function of source/site conditions. For development of building fragility curves, the demand spectrum shape represented Moderate duration ground shaking of a large-magnitude WUS earthquake at a soil site.

The convolution process produces a surface that describes the probability of each demand/capacity intersection point when the median demand spectrum is scaled to intersect the median capacity curve at a given amplitude of response. Discrete values of the probabilistic surface are summed along a line anchored to the damage state median of interest (e.g., $S_{d,Sds}$) to estimate the probability of reaching or exceeding the median value given building response at the intersection point. This process is repeated for other intersection points to form a cumulative description of the probability of reaching (or exceeding) the damage state of interest. A lognormal function is fit to this cumulative curve yielding an estimate of the lognormal standard deviation of the combined effect of demand and capacity variability on building fragility.

The lognormal standard deviation parameter that describes the uncertainty in the estimate of the median value of the threshold of structural damage state ds, $\beta_{M(Sds)}$, is assumed to be independent of capacity and demand, and is added by the square-root-sum-of-the-squares (SRSS) method to the lognormal standard deviation parameter representing the combined effects of demand and capacity variability.

In the development of the damage state variability for implementation with the USGS probabilistic seismic hazard curves, the procedure was modified. The USGS explicitly incorporated the ground motion uncertainty in their Project 97 seismic hazard curves. (See Chapter 4) These hazard curves were the basis for the **HAZUS** PESH data used in the Methodology s probabilistic analysis procedure. To avoid overestimation of the damage state variability due to this double counting of ground motion uncertainty, the convolution process was modified and reanalyzed. Modified damage state variability parameters were developed for each probabilistic return period (a total of 8 return periods) and used when the probabilistic analysis option is selected. Due to large amount of modified parameters, their values are not reproduced in this chapter. To review the modified parameters, the user can access them via the **HAZUS** software [**Analysis-Damage Functions-Buildings**].

The process, described above for structural components, is the same approach used to estimate the lognormal standard deviation for nonstructural drift-sensitive components. Nonstructural acceleration-sensitive components are treated in a similar manner to nonstructural drift-sensitive components, except that cumulative descriptions of the probability of reaching (or exceeding) the damage state of interest are developed in terms of spectral acceleration (rather than spectra displacement). Also, nonstructural acceleration-sensitive components are divided into two sub-populations: (1) components at or near ground level and (2) components at upper floors or on the roof. PGA, rather than spectral acceleration, is a more appropriate PESH input for components at or near ground level. Fragility curves for nonstructural acceleration-sensitive components assume 50% (low-rise), 33% (mid-rise) or 20% (high-rise) of nonstructural components are located at, or near, the ground floor, and represent a weighted combination of the probability of damage to components located at, or near, ground level and components located at upper-floor levels of the building.

5.4.3.4 Structural Damage

Structural damage fragility curves for buildings are described by median values of drift that define the thresholds of Slight, Moderate, Extensive and Complete damage states. In general, these estimates of drift are different for each model building type (including height) and seismic design level. Table 5.8 summarizes the ranges of drift ratios used to define structural damage for various low-rise building types designed to current High-Code seismic provisions. A complete listing of damage-state drift ratios for all building types and heights are provided for each seismic design level in Tables 5.9a, 5.9b, 5.9c and 5.9d, respectively.

Table 5.8 Typical Drift Ratios Used to Define Median Values of Structural Damage

Seismic Design Level	Building Type (Low-Rise)	Drift Ratio at the Threshold of Structural Damage			
		Slight	Moderate	Extensive	Complete
High-Code	W1/W2	0.004	0.012	0.040	0.100
	C1L, S2L	0.005	0.010	0.030	0.080
	RM1L/RM2L, PC1/PC2L	0.004	0.008	0.024	0.070
Moderate-Code	W1/W2	0.004	0.010	0.031	0.075
	C1L, S2L	0.005	0.009	0.023	0.060
	RM1L/RM2L, PC1/PC2L	0.004	0.007	0.019	0.053
Low-Code	W1/W2	0.004	0.010	0.031	0.075
	C1L, S2L	0.005	0.008	0.020	0.050
	RM1L/RM2L,PC1/PC2L	0.004	0.006	0.016	0.044
	URML, C3L, S5L	0.003	0.006	0.015	0.035
Pre-Code	W1/W2	0.003	0.008	0.025	0.060
	C1L, S2L	0.004	0.006	0.016	0.040
	RM1L/RM2L,PC1/PC2L	0.003	0.005	0.013	0.035
	URML, C3L, S5L	0.002	0.005	0.012	0.028

In general, values of the drift ratio that define Complete damage to Moderate-Code buildings are assumed to be 75% of the drift ratio that define Complete damage to High-Code buildings, and values of the drift ratio that define Complete damage to Low-Code buildings are assumed to be 63% of the drift ratios that define Complete damage to High-

Code buildings. These assumptions are based on the recognition that post-yield capacity is significantly less in buildings designed with limited ductile detailing. Values of the drift ratio that define Slight damage were assumed to be the same for High-Code, Moderate-Code and Low-Code buildings, since this damage state typically does not exceed the building's elastic capacity.

Values of drift ratios that define Moderate and Extensive damage to Moderate-Code and Low-Code buildings are selected such that their distribution between Slight and Complete damage-state drift ratios is in proportion to the distribution of damage-state drift ratios for High-Code buildings.

Values of Pre-Code building drift ratios are based on the drift ratios for Low-Code buildings, reduced slightly to account for inferior performance anticipated for these older buildings. For each damage state, the drift ratio of a Pre-Code building is assumed to be 80% of the drift ratio of the Low-Code building of the same building type.

Drift ratios are reduced for taller buildings assuming that the deflected shape will not affect uniform distribution of drift over the building's height. For all damage states, drift ratios for mid-rise buildings are assumed to be 67% of those of low-rise buildings of the same type, and drift ratios for high-rise buildings are assumed to be 50% of those of low-rise buildings of the same type. Since mid-rise and high-rise buildings are much taller than low-rise buildings, median values of spectral displacement (i.e., drift ratio times height of building at the point of push-over mode displacement) are still much greater for mid-rise and high-rise buildings than for low-rise buildings.

The total variability of each structural damage state, β_{Sds}, is modeled by the combination of following three contributors to damage variability:

- uncertainty in the damage-state threshold of the structural system ($\beta_{M(Sds)}$ = 0.4, for all structural damage states and building types)

- variability in capacity (response) properties of the model building type/seismic design level of interest ($\beta_{C(Au)}$ = 0.25 for Code buildings, $\beta_{C(Au)}$ = 0.30 for Pre-Code buildings) and

- variability in response due to the spatial variability of ground motion

Each of these three contributors to damage state variability is assumed to be lognormally distributed random variables. Capacity and demand are dependent parameters and a convolution process is used to derive combined capacity/demand variability of each structural damage state. Capacity/demand variability is then combined with damage state uncertainty, as described in Section 5.4.3.3.

Tables 5.9a, 5.9b, 5.9c and 5.9d summarize median and lognormal standard deviation (β_{Sds}) values for Slight, Moderate, Extensive and Complete structural damage states

High-Code, Moderate-Code, Low-Code and Pre-Code buildings, respectively. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 5.9a Structural Fragility Curve Parameters - High-Code Seismic Design Level

Building Properties			Interstory Drift at Threshold of Damage State				Spectral Displacement (inches)							
Type	Height (inches)						Slight		Moderate		Extensive		Complete	
	Roof	Modal	Slight	Moderate	Extensive	Complete	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	168	126	0.0040	0.0120	0.0400	0.1000	0.50	0.80	1.51	0.81	5.04	0.85	12.60	0.97
W2	288	216	0.0040	0.0120	0.0400	0.1000	0.86	0.81	2.59	0.88	8.64	0.90	21.60	0.83
S1L	288	216	0.0060	0.0120	0.0300	0.0800	1.30	0.80	2.59	0.76	6.48	0.69	17.28	0.72
S1M	720	540	0.0040	0.0080	0.0200	0.0533	2.16	0.65	4.32	0.66	10.80	0.67	28.80	0.74
S1H	1872	1123	0.0030	0.0060	0.0150	0.0400	3.37	0.64	6.74	0.64	16.85	0.65	44.93	0.67
S2L	288	216	0.0050	0.0100	0.0300	0.0800	1.08	0.81	2.16	0.89	6.48	0.94	17.28	0.83
S2M	720	540	0.0033	0.0067	0.0200	0.0533	1.80	0.67	3.60	0.67	10.80	0.68	28.80	0.79
S2H	1872	1123	0.0025	0.0050	0.0150	0.0400	2.81	0.63	5.62	0.63	16.85	0.64	44.93	0.71
S3	180	135	0.0040	0.0080	0.0240	0.0700	0.54	0.81	1.08	0.82	3.24	0.91	9.45	0.90
S4L	288	216	0.0040	0.0080	0.0240	0.0700	0.86	0.89	1.73	0.89	5.18	0.98	15.12	0.87
S4M	720	540	0.0027	0.0053	0.0160	0.0467	1.44	0.77	2.88	0.72	8.64	0.70	25.20	0.89
S4H	1872	1123	0.0020	0.0040	0.0120	0.0350	2.25	0.64	4.49	0.66	13.48	0.69	39.31	0.77
S5L														
S5M														
S5H														
C1L	240	180	0.0050	0.0100	0.0300	0.0800	0.90	0.81	1.80	0.84	5.40	0.86	14.40	0.81
C1M	600	450	0.0033	0.0067	0.0200	0.0533	1.50	0.68	3.00	0.67	9.00	0.68	24.00	0.81
C1H	1440	864	0.0025	0.0050	0.0150	0.0400	2.16	0.66	4.32	0.64	12.96	0.67	34.56	0.78
C2L	240	180	0.0040	0.0100	0.0300	0.0800	0.72	0.81	1.80	0.84	5.40	0.93	14.40	0.92
C2M	600	450	0.0027	0.0067	0.0200	0.0533	1.20	0.74	3.00	0.77	9.00	0.68	24.00	0.77
C2H	1440	864	0.0020	0.0050	0.0150	0.0400	1.73	0.68	4.32	0.65	12.96	0.66	34.56	0.75
C3L														
C3M														
C3H														
PC1	180	135	0.0040	0.0080	0.0240	0.0700	0.54	0.76	1.08	0.86	3.24	0.88	9.45	0.99
PC2L	240	180	0.0040	0.0080	0.0240	0.0700	0.72	0.84	1.44	0.88	4.32	0.98	12.60	0.94
PC2M	600	450	0.0027	0.0053	0.0160	0.0467	1.20	0.77	2.40	0.81	7.20	0.70	21.00	0.82
PC2H	1440	864	0.0020	0.0040	0.0120	0.0350	1.73	0.64	3.46	0.66	10.37	0.68	30.24	0.81
RM1L	240	180	0.0040	0.0080	0.0240	0.0700	0.72	0.84	1.44	0.86	4.32	0.92	12.60	1.01
RM1M	600	450	0.0027	0.0053	0.0160	0.0467	1.20	0.71	2.40	0.81	7.20	0.76	21.00	0.75
RM2L	240	180	0.0040	0.0080	0.0240	0.0700	0.72	0.80	1.44	0.81	4.32	0.91	12.60	0.98
RM2M	600	450	0.0027	0.0053	0.0160	0.0467	1.20	0.71	2.40	0.79	7.20	0.70	21.00	0.73
RM2H	1440	864	0.0020	0.0040	0.0120	0.0350	1.73	0.66	3.46	0.65	10.37	0.66	30.24	0.72
URML														
URMM														
MH	120	120	0.0040	0.0080	0.0240	0.0700	0.48	0.91	0.96	1.00	2.88	1.03	8.40	0.92

Table 5.9b Structural Fragility Curve Parameters – Moderate Code Seismic Design Level

Building Properties			Interstory Drift at Threshold of Damage State				Spectral Displacement (inches)							
Type	Height (inches)						Slight		Moderate		Extensive		Complete	
	Roof	Modal	Slight	Moderate	Extensive	Complete	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	168	126	0.0040	0.0099	0.0306	0.0750	0.50	0.84	1.25	0.86	3.86	0.89	9.45	1.04
W2	288	216	0.0040	0.0099	0.0306	0.0750	0.86	0.89	2.14	0.95	6.62	0.95	16.20	0.92
S1L	288	216	0.0060	0.0104	0.0235	0.0600	1.30	0.80	2.24	0.75	5.08	0.74	12.96	0.88
S1M	720	540	0.0040	0.0069	0.0157	0.0400	2.16	0.65	3.74	0.68	8.46	0.69	21.60	0.87
S1H	1872	1123	0.0030	0.0052	0.0118	0.0300	3.37	0.64	5.83	0.64	13.21	0.71	33.70	0.83
S2L	288	216	0.0050	0.0087	0.0233	0.0600	1.08	0.93	1.87	0.92	5.04	0.93	12.96	0.93
S2M	720	540	0.0033	0.0058	0.0156	0.0400	1.80	0.70	3.12	0.69	8.40	0.69	21.60	0.89
S2H	1872	1123	0.0025	0.0043	0.0117	0.0300	2.81	0.66	4.87	0.64	13.10	0.69	33.70	0.80
S3	180	135	0.0040	0.0070	0.0187	0.0525	0.54	0.88	0.94	0.92	2.52	0.97	7.09	0.89
S4L	288	216	0.0040	0.0069	0.0187	0.0525	0.86	0.96	1.50	1.00	4.04	1.03	11.34	0.92
S4M	720	540	0.0027	0.0046	0.0125	0.0350	1.44	0.75	2.50	0.72	6.73	0.72	18.90	0.94
S4H	1872	1123	0.0020	0.0035	0.0093	0.0262	2.25	0.66	3.90	0.67	10.50	0.70	29.48	0.90
S5L														
S5M														
S5H														
C1L	240	180	0.0050	0.0087	0.0233	0.0600	0.90	0.89	1.56	0.90	4.20	0.90	10.80	0.89
C1M	600	450	0.0033	0.0058	0.0156	0.0400	1.50	0.70	2.60	0.70	7.00	0.70	18.00	0.89
C1H	1440	864	0.0025	0.0043	0.0117	0.0300	2.16	0.66	3.74	0.66	10.08	0.76	25.92	0.91
C2L	240	180	0.0040	0.0084	0.0232	0.0600	0.72	0.91	1.52	0.97	4.17	1.03	10.80	0.87
C2M	600	450	0.0027	0.0056	0.0154	0.0400	1.20	0.81	2.53	0.77	6.95	0.73	18.00	0.91
C2H	1440	864	0.0020	0.0042	0.0116	0.0300	1.73	0.66	3.64	0.68	10.00	0.70	25.92	0.87
C3L														
C3M														
C3H														
PC1	180	135	0.0040	0.0070	0.0187	0.0525	0.54	0.89	0.94	0.92	2.52	0.97	7.09	1.04
PC2L	240	180	0.0040	0.0069	0.0187	0.0525	0.72	0.96	1.25	1.00	3.37	1.03	9.45	0.88
PC2M	600	450	0.0027	0.0046	0.0125	0.0350	1.20	0.82	2.08	0.79	5.61	0.75	15.75	0.93
PC2H	1440	864	0.0020	0.0035	0.0094	0.0263	1.73	0.68	3.00	0.69	8.08	0.77	22.68	0.89
RM1L	240	180	0.0040	0.0069	0.0187	0.0525	0.72	0.96	1.25	0.99	3.37	1.05	9.45	0.94
RM1M	600	450	0.0027	0.0046	0.0125	0.0350	1.20	0.81	2.08	0.82	5.61	0.80	15.75	0.89
RM2L	240	180	0.0040	0.0069	0.0187	0.0525	0.72	0.91	1.25	0.96	3.37	1.02	9.45	0.93
RM2M	600	450	0.0027	0.0046	0.0125	0.0350	1.20	0.81	2.08	0.80	5.61	0.75	15.75	0.88
RM2H	1440	864	0.0020	0.0035	0.0094	0.0263	1.73	0.67	3.00	0.69	8.08	0.70	22.68	0.86
URML														
URMM														
MH	120	120	0.0040	0.0080	0.0240	0.0700	0.48	0.91	0.96	1.00	2.88	1.03	8.40	0.92

Table 5.9c Structural Fragility Curve Parameters - Low-Code Seismic Design Level

Building Properties			Interstory Drift at Threshold of Damage State				Spectral Displacement (inches)							
Type	Height (inches)						Slight		Moderate		Extensive		Complete	
	Roof	Modal	Slight	Moderate	Extensive	Complete	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	168	126	0.0040	0.0099	0.0306	0.0750	0.50	0.93	1.25	0.98	3.86	1.02	9.45	0.99
W2	288	216	0.0040	0.0099	0.0306	0.0750	0.86	0.97	2.14	0.90	6.62	0.89	16.20	0.99
S1L	288	216	0.0060	0.0096	0.0203	0.0500	1.30	0.77	2.07	0.78	4.38	0.78	10.80	0.96
S1M	720	540	0.0040	0.0064	0.0135	0.0333	2.16	0.68	3.44	0.78	7.30	0.85	18.00	0.98
S1H	1872	1123	0.0030	0.0048	0.0101	0.0250	3.37	0.66	5.37	0.70	11.38	0.76	28.08	0.92
S2L	288	216	0.0050	0.0080	0.0200	0.0500	1.08	0.96	1.73	0.89	4.32	0.86	10.80	0.98
S2M	720	540	0.0033	0.0053	0.0133	0.0333	1.80	0.70	2.88	0.73	7.20	0.85	18.00	0.98
S2H	1872	1123	0.0025	0.0040	0.0100	0.0250	2.81	0.66	4.49	0.67	11.23	0.74	28.08	0.92
S3	180	135	0.0040	0.0064	0.0161	0.0438	0.54	0.98	0.87	0.99	2.17	1.01	5.91	0.90
S4L	288	216	0.0040	0.0064	0.0161	0.0438	0.86	1.05	1.38	0.98	3.47	0.89	9.45	0.98
S4M	720	540	0.0027	0.0043	0.0107	0.0292	1.44	0.76	2.31	0.78	5.78	0.90	15.75	0.99
S4H	1872	1123	0.0020	0.0032	0.0080	0.0219	2.25	0.70	3.60	0.75	9.01	0.90	24.57	0.98
S5L	288	216	0.0030	0.0060	0.0150	0.0350	0.65	1.11	1.30	1.04	3.24	0.99	7.56	0.95
S5M	720	540	0.0020	0.0040	0.0100	0.0233	1.08	0.77	2.16	0.79	5.40	0.87	12.60	0.98
S5H	1872	1123	0.0015	0.0030	0.0075	0.0175	1.68	0.70	3.37	0.73	8.42	0.89	19.66	0.97
C1L	240	180	0.0050	0.0080	0.0200	0.0500	0.90	0.95	1.44	0.91	3.60	0.85	9.00	0.97
C1M	600	450	0.0033	0.0053	0.0133	0.0333	1.50	0.70	2.40	0.74	6.00	0.86	15.00	0.98
C1H	1440	864	0.0025	0.0040	0.0100	0.0250	2.16	0.70	3.46	0.81	8.64	0.89	21.60	0.98
C2L	240	180	0.0040	0.0076	0.0197	0.0500	0.72	1.04	1.37	1.02	3.55	0.99	9.00	0.95
C2M	600	450	0.0027	0.0051	0.0132	0.0333	1.20	0.82	2.29	0.81	5.92	0.81	15.00	0.99
C2H	1440	864	0.0020	0.0038	0.0099	0.0250	1.73	0.68	3.30	0.73	8.53	0.84	21.60	0.95
C3L	240	180	0.0030	0.0060	0.0150	0.0350	0.54	1.09	1.08	1.07	2.70	1.08	6.30	0.91
C3M	600	450	0.0020	0.0040	0.0100	0.0233	0.90	0.85	1.80	0.83	4.50	0.79	10.50	0.98
C3H	1440	864	0.0015	0.0030	0.0075	0.0175	1.30	0.71	2.59	0.74	6.48	0.90	15.12	0.97
PC1	180	135	0.0040	0.0064	0.0161	0.0438	0.54	1.00	0.87	1.05	2.17	1.12	5.91	0.89
PC2L	240	180	0.0040	0.0064	0.0161	0.0438	0.72	1.08	1.15	1.03	2.89	0.98	7.88	0.96
PC2M	600	450	0.0027	0.0043	0.0107	0.0292	1.20	0.81	1.92	0.79	4.81	0.84	13.12	0.99
PC2H	1440	864	0.0020	0.0032	0.0080	0.0219	1.73	0.71	2.77	0.75	6.93	0.89	18.90	0.98
RM1L	240	180	0.0040	0.0064	0.0161	0.0438	0.72	1.11	1.15	1.10	2.89	1.10	7.88	0.92
RM1M	600	450	0.0027	0.0043	0.0107	0.0292	1.20	0.87	1.92	0.84	4.81	0.79	13.12	0.96
RM2L	240	180	0.0040	0.0064	0.0161	0.0438	0.72	1.05	1.15	1.07	2.89	1.09	7.88	0.91
RM2M	600	450	0.0027	0.0043	0.0107	0.0292	1.20	0.84	1.92	0.81	4.81	0.77	13.12	0.96
RM2H	1440	864	0.0020	0.0032	0.0080	0.0219	1.73	0.69	2.77	0.72	6.93	0.87	18.90	0.96
URML	180	135	0.0030	0.0060	0.0150	0.0350	0.41	0.99	0.81	1.05	2.03	1.10	4.73	1.08
URMM	420	315	0.0020	0.0040	0.0100	0.0233	0.63	0.91	1.26	0.92	3.15	0.87	7.35	0.91
MH	120	120	0.0040	0.0080	0.0240	0.0700	0.48	0.91	0.96	1.00	2.88	1.03	8.40	0.92

Table 5.9d Structural Fragility Curve Parameters - Pre-Code Seismic Design Level

Building Properties			Interstory Drift at Threshold of Damage State				Spectral Displacement (inches)							
Type	Height (inches)						Slight		Moderate		Extensive		Complete	
	Roof	Modal	Slight	Moderate	Extensive	Complete	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	168	126	0.0032	0.0079	0.0245	0.0600	0.40	1.01	1.00	1.05	3.09	1.07	7.56	1.06
W2	288	216	0.0032	0.0079	0.0245	0.0600	0.69	1.04	1.71	0.97	5.29	0.90	12.96	0.99
S1L	288	216	0.0048	0.0076	0.0162	0.0400	1.04	0.85	1.65	0.82	3.50	0.80	8.64	0.95
S1M	720	540	0.0032	0.0051	0.0108	0.0267	1.73	0.70	2.76	0.75	5.84	0.81	14.40	0.98
S1H	1872	1123	0.0024	0.0038	0.0081	0.0200	2.70	0.69	4.30	0.71	9.11	0.85	22.46	0.93
S2L	288	216	0.0040	0.0064	0.0160	0.0400	0.86	1.01	1.38	0.96	3.46	0.88	8.64	0.98
S2M	720	540	0.0027	0.0043	0.0107	0.0267	1.44	0.73	2.30	0.75	5.76	0.80	14.40	0.98
S2H	1872	1123	0.0020	0.0032	0.0080	0.0200	2.25	0.70	3.59	0.70	8.99	0.84	22.46	0.91
S3	180	135	0.0032	0.0051	0.0128	0.0350	0.43	1.06	0.69	1.03	1.73	1.07	4.73	0.89
S4L	288	216	0.0032	0.0051	0.0128	0.0350	0.69	1.11	1.11	1.03	2.77	0.99	7.56	0.98
S4M	720	540	0.0021	0.0034	0.0086	0.0233	1.15	0.81	1.85	0.80	4.62	0.94	12.60	1.00
S4H	1872	1123	0.0016	0.0026	0.0064	0.0175	1.80	0.73	2.88	0.75	7.21	0.90	19.66	0.97
S5L	288	216	0.0024	0.0048	0.0120	0.0280	0.52	1.20	1.04	1.11	2.59	1.08	6.05	0.95
S5M	720	540	0.0016	0.0032	0.0080	0.0187	0.86	0.85	1.73	0.83	4.32	0.94	10.08	0.99
S5H	1872	1123	0.0012	0.0024	0.0060	0.0140	1.35	0.72	2.70	0.75	6.74	0.92	15.72	0.96
C1L	240	180	0.0040	0.0064	0.0160	0.0400	0.72	0.98	1.15	0.94	2.88	0.90	7.20	0.97
C1M	600	450	0.0027	0.0043	0.0107	0.0267	1.20	0.73	1.92	0.77	4.80	0.83	12.00	0.98
C1H	1440	864	0.0020	0.0032	0.0080	0.0200	1.73	0.71	2.76	0.80	6.91	0.94	17.28	1.01
C2L	240	180	0.0032	0.0061	0.0158	0.0400	0.58	1.11	1.10	1.09	2.84	1.07	7.20	0.93
C2M	600	450	0.0021	0.0041	0.0105	0.0267	0.96	0.86	1.83	0.83	4.74	0.80	12.00	0.98
C2H	1440	864	0.0016	0.0031	0.0079	0.0200	1.38	0.73	2.64	0.75	6.82	0.92	17.28	0.97
C3L	240	180	0.0024	0.0048	0.0120	0.0280	0.43	1.19	0.86	1.15	2.16	1.15	5.04	0.92
C3M	600	450	0.0016	0.0032	0.0080	0.0187	0.72	0.90	1.44	0.86	3.60	0.90	8.40	0.96
C3H	1440	864	0.0012	0.0024	0.0060	0.0140	1.04	0.73	2.07	0.75	5.18	0.90	12.10	0.95
PC1	180	135	0.0032	0.0051	0.0128	0.0350	0.43	1.14	0.69	1.14	1.73	1.17	4.73	0.98
PC2L	240	180	0.0032	0.0051	0.0128	0.0350	0.58	1.14	0.92	1.10	2.31	1.10	6.30	0.93
PC2M	600	450	0.0021	0.0034	0.0086	0.0233	0.96	0.87	1.54	0.83	3.85	0.91	10.50	1.00
PC2H	1440	864	0.0016	0.0026	0.0064	0.0175	1.38	0.74	2.21	0.75	5.55	0.91	15.12	0.96
RM1L	240	180	0.0032	0.0051	0.0128	0.0350	0.58	1.20	0.92	1.17	2.31	1.17	6.30	0.94
RM1M	600	450	0.0021	0.0034	0.0086	0.0233	0.96	0.91	1.54	0.89	3.85	0.89	10.50	0.96
RM2L	240	180	0.0032	0.0051	0.0128	0.0350	0.58	1.14	0.92	1.10	2.31	1.15	6.30	0.92
RM2M	600	450	0.0021	0.0034	0.0086	0.0233	0.96	0.89	1.54	0.87	3.85	0.87	10.50	0.96
RM2H	1440	864	0.0016	0.0026	0.0064	0.0175	1.38	0.75	2.21	0.75	5.55	0.84	15.12	0.94
URML	180	135	0.0024	0.0048	0.0120	0.0280	0.32	1.15	0.65	1.19	1.62	1.20	3.78	1.18
URMM	420	315	0.0016	0.0032	0.0080	0.0187	0.50	0.99	1.01	0.97	2.52	0.90	5.88	0.88
MH	120	120	0.0032	0.0064	0.0192	0.0560	0.38	1.11	0.77	1.10	2.30	0.95	6.72	0.97

5.4.3.5 Nonstructural Damage - Drift-Sensitive Components

Table 5.10 summarizes drift ratios used by the Methodology to define the median values of damage fragility curves for drift-sensitive nonstructural components of buildings. Nonstructural damage drift ratios are assumed to be the same for each building type and each seismic design level.

Table 5.10 Drift Ratios Used to Define Median Values of Damage for Nonstructural Drift-Sensitive Components

Drift Ratio at the Threshold of Nonstructural Damage			
Slight	Moderate	Extensive	Complete
0.004	0.008	0.025	0.050

Median values of drift-sensitive nonstructural fragility curves are based on global building displacement (in inches), calculated as the product of: (1) drift ratio, (2) building height and (3) the fraction of building height at the location of push-over mode displacement (α_2).

The total variability of each nonstructural drift-sensitive damage state, β_{NSDds}, is modeled by the combination of following three contributors to damage variability:

- uncertainty in the damage-state threshold of nonstructural components ($\beta_{M(NSDds)}$ = 0.5, for all damage states and building types),
- variability in capacity (response) properties of the model building type that contains the nonstructural components of interest ($\beta_{C(Au)}$ = 0.25 for Code buildings, $\beta_{C(Au)}$ = 0.30 for Pre-Code buildings), and
- variability in response of the model building type due to the spatial variability of ground motion demand ($\beta_{D(A)}$ = 0.45 and $\beta_{C(V)}$ = 0.50).

Each of these three contributors to damage state variability is assumed to be lognormally distributed random variables. Capacity and demand are dependent parameters and a convolution process is used to derive combined capacity/demand variability of each nonstructural damage state. Capacity/demand variability is then combined with damage state uncertainty, as described in Section 5.4.3.3.

Table 5.11a, 5.11b, 5.11c and 5.11d summarize median and lognormal standard deviation (β_{NSDds}) values for Slight, Moderate, Extensive and Complete nonstructural drift-sensitive damage states for High-Code, Moderate-Code, Low-Code and Pre-Code buildings, respectively. Median values are the same for all design levels. Lognormal standard deviation values are slightly different for each seismic design level. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 5.11 Nonstructural Drift-Sensitive Fragility Curve Parameters - High-Code Seismic Design Level

Building Type	Median Spectral Displacement (inches) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.50	0.85	1.01	0.88	3.15	0.88	6.30	0.94
W2	0.86	0.87	1.73	0.89	5.40	0.96	10.80	0.94
S1L	0.86	0.81	1.73	0.85	5.40	0.77	10.80	0.77
S1M	2.16	0.71	4.32	0.72	13.50	0.72	27.00	0.80
S1H	4.49	0.72	8.99	0.71	28.08	0.74	56.16	0.77
S2L	0.86	0.84	1.73	0.90	5.40	0.97	10.80	0.92
S2M	2.16	0.71	4.32	0.74	13.50	0.74	27.00	0.84
S2H	4.49	0.71	8.99	0.71	28.08	0.72	56.16	0.78
S3	0.54	0.86	1.08	0.88	3.38	0.98	6.75	0.98
S4L	0.86	0.93	1.73	0.94	5.40	1.01	10.80	0.99
S4M	2.16	0.80	4.32	0.76	13.50	0.76	27.00	0.93
S4H	4.49	0.72	8.99	0.72	28.08	0.79	56.16	0.91
S5L								
S5M								
S5H								
C1L	0.72	0.84	1.44	0.88	4.50	0.90	9.00	0.88
C1M	1.80	0.72	3.60	0.73	11.25	0.74	22.50	0.84
C1H	3.46	0.71	6.91	0.71	21.60	0.78	43.20	0.88
C2L	0.72	0.87	1.44	0.88	4.50	0.97	9.00	0.99
C2M	1.80	0.84	3.60	0.82	11.25	0.74	22.50	0.81
C2H	3.46	0.71	6.91	0.72	21.60	0.74	43.20	0.85
C3L								
C3M								
C3H								
PC1	0.54	0.82	1.08	0.91	3.38	0.95	6.75	1.03
PC2L	0.72	0.89	1.44	0.93	4.50	1.03	9.00	1.04
PC2M	1.80	0.87	3.60	0.83	11.25	0.77	22.50	0.89
PC2H	3.46	0.73	6.91	0.73	21.60	0.77	43.20	0.89
RM1L	0.72	0.89	1.44	0.91	4.50	0.97	9.00	1.06
RM1M	1.80	0.81	3.60	0.86	11.25	0.80	22.50	0.81
RM2L	0.72	0.85	1.44	0.87	4.50	0.95	9.00	1.03
RM2M	1.80	0.82	3.60	0.84	11.25	0.76	22.50	0.80
RM2H	3.46	0.71	6.91	0.73	21.60	0.73	43.20	0.85
URML								
URMM								
MH	0.48	0.96	0.96	1.05	3.00	1.07	6.00	0.93

Table 5.11b Nonstructural Drift-Sensitive Fragility Curve Parameters - Moderate-Code Seismic Design Level

Building Type	Median Spectral Displacement (inches) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.50	0.89	1.01	0.91	3.15	0.90	6.30	1.04
W2	0.86	0.94	1.73	0.99	5.40	1.00	10.80	0.90
S1L	0.86	0.84	1.73	0.83	5.40	0.79	10.80	0.87
S1M	2.16	0.71	4.32	0.74	13.50	0.85	27.00	0.95
S1H	4.49	0.71	8.99	0.74	28.08	0.84	56.16	0.95
S2L	0.86	0.93	1.73	0.99	5.40	0.96	10.80	0.92
S2M	2.16	0.74	4.32	0.74	13.50	0.85	27.00	0.96
S2H	4.49	0.72	8.99	0.73	28.08	0.80	56.16	0.94
S3	0.54	0.93	1.08	0.98	3.38	1.01	6.75	0.94
S4L	0.86	1.00	1.73	1.06	5.40	0.99	10.80	0.96
S4M	2.16	0.77	4.32	0.80	13.50	0.95	27.00	1.04
S4H	4.49	0.73	8.99	0.82	28.08	0.93	56.16	1.01
S5L								
S5M								
S5H								
C1L	0.72	0.93	1.44	0.96	4.50	0.94	9.00	0.88
C1M	1.80	0.77	3.60	0.76	11.25	0.87	22.50	0.98
C1H	3.46	0.74	6.91	0.80	21.60	0.94	43.20	1.03
C2L	0.72	0.96	1.44	1.00	4.50	1.06	9.00	0.95
C2M	1.80	0.84	3.60	0.81	11.25	0.83	22.50	0.98
C2H	3.46	0.73	6.91	0.76	21.60	0.89	43.20	0.99
C3L								
C3M								
C3H								
PC1	0.54	0.94	1.08	0.99	3.38	1.05	6.75	1.08
PC2L	0.72	1.00	1.44	1.06	4.50	1.07	9.00	0.93
PC2M	1.80	0.85	3.60	0.83	11.25	0.92	22.50	1.00
PC2H	3.46	0.74	6.91	0.79	21.60	0.93	43.20	1.02
RM1L	0.72	1.00	1.44	1.06	4.50	1.12	9.00	1.01
RM1M	1.80	0.88	3.60	0.85	11.25	0.84	22.50	0.98
RM2L	0.72	0.96	1.44	1.02	4.50	1.10	9.00	0.99
RM2M	1.80	0.88	3.60	0.83	11.25	0.81	22.50	0.98
RM2H	3.46	0.73	6.91	0.76	21.60	0.88	43.20	0.99
URML								
URMM								
MH	0.48	0.96	0.96	1.05	3.00	1.07	6.00	0.93

Table 5.11c Nonstructural Drift-Sensitive Fragility Curve Parameters - Low-Code Seismic Design Level

Building Type	Median Spectral Displacement (inches) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.50	0.98	1.01	0.99	3.15	1.02	6.30	1.09
W2	0.86	1.01	1.73	0.97	5.40	0.93	10.80	1.03
S1L	0.86	0.86	1.73	0.84	5.40	0.88	10.80	1.00
S1M	2.16	0.74	4.32	0.89	13.50	0.99	27.00	1.05
S1H	4.49	0.75	8.99	0.87	28.08	0.97	56.16	1.04
S2L	0.86	1.01	1.73	0.94	5.40	0.94	10.80	1.03
S2M	2.16	0.77	4.32	0.87	13.50	0.99	27.00	1.05
S2H	4.49	0.74	8.99	0.86	28.08	0.97	56.16	1.05
S3	0.54	1.03	1.08	1.02	3.38	0.96	6.75	0.99
S4L	0.86	1.09	1.73	0.99	5.40	0.96	10.80	1.03
S4M	2.16	0.83	4.32	0.95	13.50	1.04	27.00	1.07
S4H	4.49	0.84	8.99	0.95	28.08	1.05	56.16	1.07
S5L	0.86	1.14	1.73	1.04	5.40	0.98	10.80	1.01
S5M	2.16	0.84	4.32	0.95	13.50	1.03	27.00	1.07
S5H	4.49	0.84	8.99	0.95	28.08	1.03	56.16	1.06
C1L	0.72	0.99	1.44	0.96	4.50	0.90	9.00	1.01
C1M	1.80	0.79	3.60	0.88	11.25	0.99	22.50	1.06
C1H	3.46	0.87	6.91	0.96	21.60	1.02	43.20	1.06
C2L	0.72	1.08	1.44	1.05	4.50	0.95	9.00	0.99
C2M	1.80	0.84	3.60	0.87	11.25	1.00	22.50	1.06
C2H	3.46	0.79	6.91	0.93	21.60	0.99	43.20	1.07
C3L	0.72	1.13	1.44	1.08	4.50	0.95	9.00	1.00
C3M	1.80	0.88	3.60	0.92	11.25	1.00	22.50	1.06
C3H	3.46	0.83	6.91	0.96	21.60	1.02	43.20	1.06
PC1	0.54	1.05	1.08	1.10	3.38	1.10	6.75	0.93
PC2L	0.72	1.12	1.44	1.04	4.50	0.93	9.00	1.02
PC2M	1.80	0.86	3.60	0.93	11.25	1.02	22.50	1.07
PC2H	3.46	0.83	6.91	0.94	21.60	1.04	43.20	1.07
RM1L	0.72	1.15	1.44	1.12	4.50	1.03	9.00	0.99
RM1M	1.80	0.89	3.60	0.89	11.25	1.00	22.50	1.05
RM2L	0.72	1.09	1.44	1.08	4.50	1.01	9.00	0.99
RM2M	1.80	0.85	3.60	0.86	11.25	0.99	22.50	1.06
RM2H	3.46	0.79	6.91	0.92	21.60	0.99	43.20	1.06
URML	0.54	1.07	1.08	1.13	3.38	1.16	6.75	1.01
URMM	1.26	0.97	2.52	0.91	7.88	0.98	15.75	1.04
MH	0.48	0.96	0.96	1.05	3.00	1.07	6.00	0.93

Table 5.11d Nonstructural Drift-Sensitive Fragility Curve Parameters - Pre-Code Seismic Design Level

Building Type	Median Spectral Displacement (inches) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.50	1.07	1.01	1.11	3.15	1.11	6.30	1.14
W2	0.86	1.06	1.73	1.00	5.40	0.93	10.80	1.01
S1L	0.86	0.90	1.73	0.87	5.40	0.91	10.80	1.02
S1M	2.16	0.80	4.32	0.92	13.50	0.99	27.00	1.06
S1H	4.49	0.79	8.99	0.89	28.08	1.00	56.16	1.07
S2L	0.86	1.06	1.73	0.97	5.40	0.96	10.80	1.04
S2M	2.16	0.80	4.32	0.90	13.50	1.02	27.00	1.06
S2H	4.49	0.79	8.99	0.89	28.08	0.99	56.16	1.06
S3	0.54	1.11	1.08	1.05	3.38	0.96	6.75	1.00
S4L	0.86	1.12	1.73	1.00	5.40	0.99	10.80	1.05
S4M	2.16	0.86	4.32	0.99	13.50	1.06	27.00	1.10
S4H	4.49	0.88	8.99	0.99	28.08	1.07	56.16	1.09
S5L	0.86	1.18	1.73	1.06	5.40	0.98	10.80	1.03
S5M	2.16	0.86	4.32	0.99	13.50	1.05	27.00	1.09
S5H	4.49	0.88	8.99	0.91	28.08	1.05	56.16	1.09
C1L	0.72	1.02	1.44	0.98	4.50	0.93	9.00	1.03
C1M	1.80	0.81	3.60	0.91	11.25	1.02	22.50	1.06
C1H	3.46	0.90	6.91	0.99	21.60	1.05	43.20	1.10
C2L	0.72	1.14	1.44	1.08	4.50	0.97	9.00	1.00
C2M	1.80	0.88	3.60	0.90	11.25	1.03	22.50	1.07
C2H	3.46	0.83	6.91	0.97	21.60	1.05	43.20	1.07
C3L	0.72	1.19	1.44	1.11	4.50	0.99	9.00	1.02
C3M	1.80	0.92	3.60	0.95	11.25	1.03	22.50	1.09
C3H	3.46	0.86	6.91	0.90	21.60	1.04	43.20	1.09
PC1	0.54	1.18	1.08	1.16	3.38	1.12	6.75	0.95
PC2L	0.72	1.16	1.44	1.06	4.50	0.96	9.00	1.02
PC2M	1.80	0.87	3.60	0.95	11.25	1.04	22.50	1.07
PC2H	3.46	0.87	6.91	0.99	21.60	1.06	43.20	1.08
RM1L	0.72	1.22	1.44	1.14	4.50	1.03	9.00	0.99
RM1M	1.80	0.93	3.60	0.92	11.25	1.02	22.50	1.07
RM2L	0.72	1.17	1.44	1.12	4.50	1.01	9.00	0.99
RM2M	1.80	0.89	3.60	0.90	11.25	1.01	22.50	1.07
RM2H	3.46	0.82	6.91	0.96	21.60	1.04	43.20	1.07
URML	0.54	1.21	1.08	1.23	3.38	1.23	6.75	1.03
URMM	1.26	0.99	2.52	0.95	7.88	0.99	15.75	1.06
MH	0.48	1.15	0.96	1.09	3.00	0.93	6.00	0.99

5.4.3.6 Nonstructural Damage - Acceleration-Sensitive Components

Table 5.12 summarizes the peak floor acceleration values used by the Methodology to define the median values of fragility curves for acceleration-sensitive nonstructural components of buildings. Nonstructural damage acceleration values are assumed to be the same for each model building type, but to vary by seismic design level.

Table 5.12 Peak Floor Accelerations Used to Define Median Values of Damage to Nonstructural Acceleration-Sensitive Components

Seismic Design Level	Floor Acceleration at the Threshold of Nonstructural Damage (g)			
	Slight	Moderate	Extensive	Complete
High-Code	0.30	0.60	1.20	2.40
Moderate-Code	0.25	0.50	1.00	2.00
Low-Code	0.20	0.40	0.80	1.60
Pre-Code	0.20	0.40	0.80	1.60

The floor acceleration values are used directly as median values, assuming average upper-floor demand is represented by response at the point of the push-over mode displacement.

The total variability of each damage state, β_{NSAds}, is modeled by the combination of following three contributors to nonstructural acceleration-sensitive damage variability:

- uncertainty in the damage-state threshold of nonstructural components ($\beta_{M(NSAds)}$ = 0.6, for all damage states and building types),

- variability in capacity (response) properties of the model building type that contains the nonstructural components of interest ($\beta_{C(Au)}$ = 0.25 for Code buildings, $\beta_{C(Au)}$ = 0.30 for Pre-Code buildings), and

- variability in response of the model building type due to the spatial variability of ground motion demand ($\beta_{D(A)}$ = 0.45 and $\beta_{C(V)}$ = 0.50).

Each of these three contributors to damage state variability is assumed to be lognormally distributed random variables. Capacity and demand are dependent parameters and a convolution process is used to derive combined capacity/demand variability of each nonstructural damage state. Capacity/demand variability is then combined with damage state uncertainty, as described in Section 5.4.3.3.

Tables 5.13a, 5.13b, 5.13c and 5.13d summarize median and lognormal standard deviation (β_{NSAds}) values for Slight, Moderate, Extensive and Complete nonstructural acceleration-sensitive damage states for High-Code. Moderate-Code, Low-Code and Pre-Code buildings, respectively. Median values are the same for all building types. Lognormal standard deviation values are slightly different for each building type. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 5.13a Nonstructural Acceleration-Sensitive Fragility Curve Parameters - High-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.30	0.73	0.60	0.68	1.20	0.68	2.40	0.68
W2	0.30	0.70	0.60	0.67	1.20	0.67	2.40	0.68
S1L	0.30	0.67	0.60	0.67	1.20	0.68	2.40	0.67
S1M	0.30	0.67	0.60	0.68	1.20	0.67	2.40	0.67
S1H	0.30	0.68	0.60	0.67	1.20	0.67	2.40	0.67
S2L	0.30	0.67	0.60	0.67	1.20	0.67	2.40	0.67
S2M	0.30	0.69	0.60	0.66	1.20	0.66	2.40	0.66
S2H	0.30	0.68	0.60	0.66	1.20	0.65	2.40	0.65
S3	0.30	0.68	0.60	0.67	1.20	0.67	2.40	0.67
S4L	0.30	0.68	0.60	0.68	1.20	0.67	2.40	0.67
S4M	0.30	0.67	0.60	0.65	1.20	0.66	2.40	0.66
S4H	0.30	0.67	0.60	0.66	1.20	0.65	2.40	0.65
S5L								
S5M								
S5H								
C1L	0.30	0.68	0.60	0.68	1.20	0.67	2.40	0.67
C1M	0.30	0.68	0.60	0.68	1.20	0.66	2.40	0.66
C1H	0.30	0.66	0.60	0.66	1.20	0.66	2.40	0.66
C2L	0.30	0.69	0.60	0.67	1.20	0.66	2.40	0.64
C2M	0.30	0.70	0.60	0.65	1.20	0.65	2.40	0.65
C2H	0.30	0.68	0.60	0.66	1.20	0.65	2.40	0.65
C3L								
C3M								
C3H								
PC1	0.30	0.74	0.60	0.67	1.20	0.67	2.40	0.64
PC2L	0.30	0.68	0.60	0.67	1.20	0.67	2.40	0.67
PC2M	0.30	0.68	0.60	0.65	1.20	0.66	2.40	0.66
PC2H	0.30	0.67	0.60	0.65	1.20	0.65	2.40	0.65
RM1L	0.30	0.70	0.60	0.67	1.20	0.67	2.40	0.63
RM1M	0.30	0.72	0.60	0.66	1.20	0.65	2.40	0.65
RM2L	0.30	0.70	0.60	0.66	1.20	0.67	2.40	0.64
RM2M	0.30	0.72	0.60	0.65	1.20	0.65	2.40	0.65
RM2H	0.30	0.70	0.60	0.65	1.20	0.65	2.40	0.65
URML								
URMM								
MH	0.30	0.65	0.60	0.67	1.20	0.67	2.40	0.67

Table 5.13b Nonstructural Acceleration-Sensitive Fragility Curve Parameters - Moderate-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.25	0.73	0.50	0.68	1.00	0.67	2.00	0.64
W2	0.25	0.68	0.50	0.67	1.00	0.68	2.00	0.68
S1L	0.25	0.67	0.50	0.66	1.00	0.67	2.00	0.67
S1M	0.25	0.66	0.50	0.67	1.00	0.67	2.00	0.67
S1H	0.25	0.66	0.50	0.68	1.00	0.68	2.00	0.68
S2L	0.25	0.66	0.50	0.66	1.00	0.68	2.00	0.68
S2M	0.25	0.66	0.50	0.65	1.00	0.65	2.00	0.65
S2H	0.25	0.65	0.50	0.65	1.00	0.65	2.00	0.65
S3	0.25	0.67	0.50	0.66	1.00	0.65	2.00	0.65
S4L	0.25	0.66	0.50	0.66	1.00	0.66	2.00	0.66
S4M	0.25	0.65	0.50	0.65	1.00	0.65	2.00	0.65
S4H	0.25	0.65	0.50	0.66	1.00	0.66	2.00	0.66
S5L								
S5M								
S5H								
C1L	0.25	0.67	0.50	0.66	1.00	0.66	2.00	0.66
C1M	0.25	0.66	0.50	0.65	1.00	0.63	2.00	0.63
C1H	0.25	0.65	0.50	0.67	1.00	0.67	2.00	0.67
C2L	0.25	0.68	0.50	0.66	1.00	0.68	2.00	0.68
C2M	0.25	0.67	0.50	0.64	1.00	0.67	2.00	0.67
C2H	0.25	0.66	0.50	0.65	1.00	0.65	2.00	0.65
C3L								
C3M								
C3H								
PC1	0.25	0.68	0.50	0.67	1.00	0.66	2.00	0.66
PC2L	0.25	0.66	0.50	0.66	1.00	0.65	2.00	0.65
PC2M	0.25	0.65	0.50	0.65	1.00	0.65	2.00	0.65
PC2H	0.25	0.64	0.50	0.65	1.00	0.65	2.00	0.65
RM1L	0.25	0.68	0.50	0.67	1.00	0.67	2.00	0.67
RM1M	0.25	0.67	0.50	0.64	1.00	0.67	2.00	0.67
RM2L	0.25	0.68	0.50	0.66	1.00	0.67	2.00	0.67
RM2M	0.25	0.67	0.50	0.64	1.00	0.67	2.00	0.67
RM2H	0.25	0.66	0.50	0.64	1.00	0.64	2.00	0.64
URML								
URMM								
MH	0.25	0.65	0.50	0.67	1.00	0.67	2.00	0.67

Table 5.13c Nonstructural Acceleration-Sensitive Fragility Curve Parameters - Low-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.20	0.71	0.40	0.68	0.80	0.66	1.60	0.66
W2	0.20	0.67	0.40	0.67	0.80	0.70	1.60	0.70
S1L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S1M	0.20	0.66	0.40	0.68	0.80	0.68	1.60	0.68
S1H	0.20	0.67	0.40	0.65	0.80	0.65	1.60	0.65
S2L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S2M	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S2H	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S3	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S4L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S4M	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S4H	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S5L	0.20	0.65	0.40	0.68	0.80	0.67	1.60	0.67
S5M	0.20	0.64	0.40	0.68	0.80	0.67	1.60	0.67
S5H	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
C1L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
C1M	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
C1H	0.20	0.67	0.40	0.67	0.80	0.67	1.60	0.67
C2L	0.20	0.66	0.40	0.67	0.80	0.66	1.60	0.66
C2M	0.20	0.64	0.40	0.66	0.80	0.65	1.60	0.65
C2H	0.20	0.64	0.40	0.66	0.80	0.66	1.60	0.66
C3L	0.20	0.65	0.40	0.67	0.80	0.66	1.60	0.66
C3M	0.20	0.64	0.40	0.67	0.80	0.66	1.60	0.66
C3H	0.20	0.64	0.40	0.67	0.80	0.67	1.60	0.67
PC1	0.20	0.66	0.40	0.66	0.80	0.66	1.60	0.66
PC2L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
PC2M	0.20	0.64	0.40	0.68	0.80	0.68	1.60	0.68
PC2H	0.20	0.64	0.40	0.67	0.80	0.67	1.60	0.67
RM1L	0.20	0.66	0.40	0.67	0.80	0.64	1.60	0.64
RM1M	0.20	0.64	0.40	0.66	0.80	0.64	1.60	0.64
RM2L	0.20	0.66	0.40	0.67	0.80	0.64	1.60	0.64
RM2M	0.20	0.64	0.40	0.66	0.80	0.65	1.60	0.65
RM2H	0.20	0.64	0.40	0.66	0.80	0.66	1.60	0.66
URML	0.20	0.68	0.40	0.65	0.80	0.65	1.60	0.65
URMM	0.20	0.64	0.40	0.66	0.80	0.66	1.60	0.66
MH	0.20	0.65	0.40	0.67	0.80	0.67	1.60	0.67

Table 5.13d Nonstructural Acceleration-Sensitive Fragility Curve Parameters - Pre-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.20	0.72	0.40	0.70	0.80	0.67	1.60	0.67
W2	0.20	0.66	0.40	0.67	0.80	0.65	1.60	0.65
S1L	0.20	0.66	0.40	0.68	0.80	0.68	1.60	0.68
S1M	0.20	0.66	0.40	0.68	0.80	0.68	1.60	0.68
S1H	0.20	0.68	0.40	0.68	0.80	0.68	1.60	0.68
S2L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S2M	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S2H	0.20	0.65	0.40	0.67	0.80	0.67	1.60	0.67
S3	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S4L	0.20	0.66	0.40	0.68	0.80	0.68	1.60	0.68
S4M	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S4H	0.20	0.66	0.40	0.68	0.80	0.68	1.60	0.68
S5L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S5M	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
S5H	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
C1L	0.20	0.66	0.40	0.68	0.80	0.68	1.60	0.68
C1M	0.20	0.66	0.40	0.68	0.80	0.68	1.60	0.68
C1H	0.20	0.68	0.40	0.68	0.80	0.68	1.60	0.68
C2L	0.20	0.65	0.40	0.67	0.80	0.67	1.60	0.67
C2M	0.20	0.64	0.40	0.67	0.80	0.67	1.60	0.67
C2H	0.20	0.65	0.40	0.67	0.80	0.67	1.60	0.67
C3L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
C3M	0.20	0.64	0.40	0.67	0.80	0.67	1.60	0.67
C3H	0.20	0.65	0.40	0.67	0.80	0.67	1.60	0.67
PC1	0.20	0.67	0.40	0.66	0.80	0.66	1.60	0.66
PC2L	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
PC2M	0.20	0.65	0.40	0.68	0.80	0.68	1.60	0.68
PC2H	0.20	0.66	0.40	0.67	0.80	0.67	1.60	0.67
RM1L	0.20	0.66	0.40	0.67	0.80	0.66	1.60	0.66
RM1M	0.20	0.64	0.40	0.66	0.80	0.65	1.60	0.65
RM2L	0.20	0.66	0.40	0.67	0.80	0.67	1.60	0.67
RM2M	0.20	0.64	0.40	0.66	0.80	0.66	1.60	0.66
RM2H	0.20	0.65	0.40	0.67	0.80	0.67	1.60	0.67
URML	0.20	0.69	0.40	0.65	0.80	0.65	1.60	0.65
URMM	0.20	0.64	0.40	0.66	0.80	0.66	1.60	0.66
MH	0.20	0.67	0.40	0.65	0.80	0.65	1.60	0.65

5.4.4 Structural Fragility Curves - Equivalent Peak Ground Acceleration

Structural damage functions are expressed in terms of an equivalent value of PGA (rather than spectral displacement) for evaluation of buildings that are components of lifelines. Only structural damage functions are developed based on PGA, since structural damage is considered the most appropriate measure of damage for lifeline facilities. Similar methods could be used to develop nonstructural damage functions based on PGA. In this case, capacity curves are not necessary to estimate building response and PGA is used directly as the PESH input to building fragility curves. This section develops equivalent-PGA fragility curves based on the structural damage functions of Tables 5.9a - 5.9d and standard spectrum shape properties of Chapter 4.

Median values of equivalent-PGA fragility curves are based on median values of spectral displacement of the damage state of interest and an assumed demand spectrum shape that relates spectral response to PGA. As such, median values of equivalent PGA are very sensitive to the shape assumed for the demand spectrum (i.e., PESH-input spectrum reduced for damping greater than 5% of critical as described in Section 5.6.2.1). Spectrum shape is influenced by earthquake source (i.e., WUS vs. CEUS attenuation functions), earthquake magnitude (e.g., large vs. small magnitude events), distance from source to site, site conditions (e.g., soil vs. rock) and effective damping which varies based on building properties and earthquake duration (e.g., Short, Moderate or Long duration).

It is not practical to create equivalent-PGA fragility curves for all possible factors that influence demand spectrum shape. Rather, equivalent-PGA fragility curves are developed for a single set of spectrum shape factors (reference spectrum), and a formula is provided for modifying damage state medians to approximate other spectrum shapes. The reference spectrum represents ground shaking of a large-magnitude (i.e., **M** $\cong$ 7.0) western United States (WUS) earthquake for soil sites (e.g., Site Class D) at site-to-source distances of 15 km, or greater. The demand spectrum based on these assumptions is scaled uniformly at each period such that the spectrum intersects the building capacity curve at the spectral displacement of the median value of the damage state of interest. The PGA of the scaled demand spectrum defines the median value of equivalent-PGA fragility. Figure 5.6 illustrates this scaling and intersection process for a typical building capacity curve and Slight, Moderate, Extensive and Complete structural damage states.

The total variability of each equivalent-PGA structural damage state, β_{SPGA}, is modeled by the combination of following two contributors to damage variability:

- uncertainty in the damage-state threshold of the structural system ($\beta_{M(SPGA)}$ = 0.4 for all building types and damage states),
- variability in response due to the spatial variability of ground motion demand ($\beta_{D(V)}$ = 0.5 for long-period spectral response).

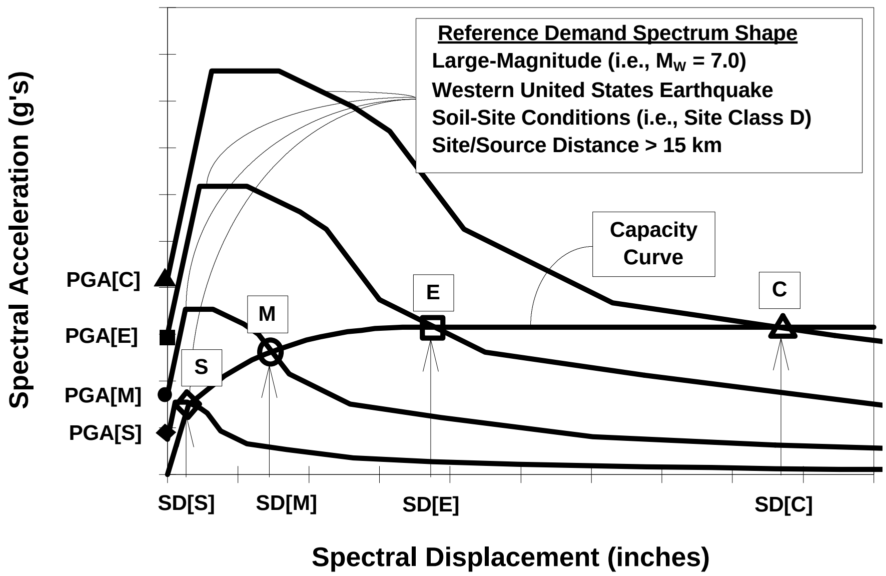


Figure 5.6 Development of Equivalent-PGA Median Damage Values.

The two contributors to damage state variability are assumed to be lognormally distributed, independent random variables and the total variability is simply the square-root-sum-of-the-squares combination of individual variability terms (i.e., β_{SPGA} = 0.64). Tables 5.16a, 5.16b, 5.16c and 5.16d summarize median and lognormal standard deviation (β_{SPGA}) values for Slight, Moderate, Extensive and Complete PGA-based structural damage states for High-Code, Moderate-Code, Low-Code and Pre-Code buildings, respectively.

The values given in Tables 5.16a through 5.16d are appropriate for use in the evaluation of scenario earthquakes whose demand spectrum shape is based on, or similar to, large-magnitude, WUS ground shaking at soil sites (reference spectrum shape). For evaluation of building damage due to scenario earthquakes whose spectra are not similar to the reference spectrum shape, damage-state median parameters may be adjusted to better represent equivalent-PGA structural fragility for the spectrum shape of interest. This adjustment is based on: (1) site condition (if different from Site Class D) and (2) the ratio of long-period spectral response (i.e., S_{A1}) to PGA (if different from a value of 1.5, the ratio of S_{A1} to PGA of the reference spectrum shape). Damage-state variability is not adjusted assuming that the variability associated with ground shaking (although different for different source/site conditions) when combined with the uncertainty in damage-state threshold, is approximately the same for all demand spectrum shapes.

Tables 4.2 and 4.3 provide spectral acceleration response factors for WUS rock (Site Class B) and CEUS rock (Site Class B) locations, respectively. These tables are based on the default WUS and CEUS attenuation functions and describe response ratios, S_{AS}/PGA and S_{AS}/S_{A1}, as a function of distance and earthquake magnitude. Although both short-period response (S_{AS}) and long-period response (S_{A1}) can influence building fragility, long-period response typically dominates building fragility and is the parameter used to relate spectral demand to PGA. Spectral response factors given in Tables 4.2 and 4.3 are combined to form ratios of PGA/S_{A1} as given in Table 5.14 and Table 5.15, respectively, for different earthquake magnitudes and source/site distances.

Table 5.14 Spectrum Shape Ratio, $R_{PGA/SA1}$ - WUS Rock (Site Class B)

Closest Distance to Fault Rupture	PGA/S_{A1} given Magnitude, M:			
	≤ 5	6	7	≥ 8
≤ 10 km	3.8	2.1	1.5	0.85
20 km	3.3	1.8	1.2	0.85
40 km	2.9	1.6	1.05	0.80
≥ 80 km	3.2	1.7	1.0	0.75

Table 5.15 Spectrum Shape Ratio, $R_{PGA/SA1}$ - CEUS Rock (Site Class B)

Hypocentral Distance	PGA/S_{A1} given Magnitude, M:			
	≤ 5	6	7	≥ 8
≤ 10 km	7.8	3.5	2.1	1.1
20 km	8.1	3.1	2.1	1.7
40 km	6.1	2.6	1.8	1.6
≥ 80 km	4.3	1.9	1.4	1.3

Equivalent-PGA medians specified in Tables 5.16a through 5.16d for the reference spectrum shape are converted to medians representing other spectrum shapes using the ratios of Tables 5.14 and 5.15, the soil amplification factor, F_V, and Equation (5-6):

$$\overline{PGA}_{ds} = \overline{PGA}_{R,ds} \cdot R_{PGA/SA1} \cdot \left(\frac{1.5}{F_V}\right) \tag{5-6}$$

where:

$\overline{PGA}_{ds}$	is the median PGA of structural damage state, ds,
$\overline{PGA}_{R,ds}$	is the median PGA of structural damage state, ds, as given in Tables 5-13a through 5-13d for the reference spectrum shape
$R_{PGA/SA1}$	is the spectrum shape ratio, given in Tables 5.14 - 5.15, and
F_V	is the soil amplification factor, given in Table 4.10

In general, implementation of Equation (5-6) requires information on earthquake magnitude and source-to-site distance to estimate the spectrum shape ratio for rock sites, and 1-second period spectral acceleration at the site (to estimate the soil amplification factor). Note that for Tables 5.16a through 5.16d, shaded boxes indicate types that are not permitted by current seismic codes.

Table 5.16a Equivalent-PGA Structural Fragility - High-Code Seismic Design Level

Building Type	Median Equivalent-PGA (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.26	0.64	0.55	0.64	1.28	0.64	2.01	0.64
W2	0.26	0.64	0.56	0.64	1.15	0.64	2.08	0.64
S1L	0.19	0.64	0.31	0.64	0.64	0.64	1.49	0.64
S1M	0.14	0.64	0.26	0.64	0.62	0.64	1.43	0.64
S1H	0.10	0.64	0.21	0.64	0.52	0.64	1.31	0.64
S2L	0.24	0.64	0.41	0.64	0.76	0.64	1.46	0.64
S2M	0.14	0.64	0.27	0.64	0.73	0.64	1.62	0.64
S2H	0.11	0.64	0.22	0.64	0.65	0.64	1.60	0.64
S3	0.15	0.64	0.26	0.64	0.54	0.64	1.00	0.64
S4L	0.24	0.64	0.39	0.64	0.71	0.64	1.33	0.64
S4M	0.16	0.64	0.28	0.64	0.73	0.64	1.56	0.64
S4H	0.13	0.64	0.25	0.64	0.69	0.64	1.63	0.64
S5L								
S5M								
S5H								
C1L	0.21	0.64	0.35	0.64	0.70	0.64	1.37	0.64
C1M	0.15	0.64	0.27	0.64	0.73	0.64	1.61	0.64
C1H	0.11	0.64	0.22	0.64	0.62	0.64	1.35	0.64
C2L	0.24	0.64	0.45	0.64	0.90	0.64	1.55	0.64
C2M	0.17	0.64	0.36	0.64	0.87	0.64	1.95	0.64
C2H	0.12	0.64	0.29	0.64	0.82	0.64	1.87	0.64
C3L								
C3M								
C3H								
PC1	0.20	0.64	0.35	0.64	0.72	0.64	1.25	0.64
PC2L	0.24	0.64	0.36	0.64	0.69	0.64	1.23	0.64
PC2M	0.17	0.64	0.29	0.64	0.67	0.64	1.51	0.64
PC2H	0.12	0.64	0.23	0.64	0.63	0.64	1.49	0.64
RM1L	0.30	0.64	0.46	0.64	0.93	0.64	1.57	0.64
RM1M	0.20	0.64	0.37	0.64	0.81	0.64	1.90	0.64
RM2L	0.26	0.64	0.42	0.64	0.87	0.64	1.49	0.64
RM2M	0.17	0.64	0.33	0.64	0.75	0.64	1.83	0.64
RM2H	0.12	0.64	0.24	0.64	0.67	0.64	1.78	0.64
URML								
URMM								
MH	0.11	0.64	0.18	0.64	0.31	0.64	0.60	0.64

Table 5.16b Equivalent-PGA Structural Fragility - Moderate-Code Seismic Design Level

Building Type	Median Equivalent-PGA (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.24	0.64	0.43	0.64	0.91	0.64	1.34	0.64
W2	0.20	0.64	0.35	0.64	0.64	0.64	1.13	0.64
S1L	0.15	0.64	0.22	0.64	0.42	0.64	0.80	0.64
S1M	0.13	0.64	0.21	0.64	0.44	0.64	0.82	0.64
S1H	0.10	0.64	0.18	0.64	0.39	0.64	0.78	0.64
S2L	0.20	0.64	0.26	0.64	0.46	0.64	0.84	0.64
S2M	0.14	0.64	0.22	0.64	0.53	0.64	0.97	0.64
S2H	0.11	0.64	0.19	0.64	0.49	0.64	1.02	0.64
S3	0.13	0.64	0.19	0.64	0.33	0.64	0.60	0.64
S4L	0.19	0.64	0.26	0.64	0.41	0.64	0.78	0.64
S4M	0.14	0.64	0.22	0.64	0.51	0.64	0.92	0.64
S4H	0.12	0.64	0.21	0.64	0.51	0.64	0.97	0.64
S5L								
S5M								
S5H								
C1L	0.16	0.64	0.23	0.64	0.41	0.64	0.77	0.64
C1M	0.13	0.64	0.21	0.64	0.49	0.64	0.89	0.64
C1H	0.11	0.64	0.18	0.64	0.41	0.64	0.74	0.64
C2L	0.18	0.64	0.30	0.64	0.49	0.64	0.87	0.64
C2M	0.15	0.64	0.26	0.64	0.55	0.64	1.02	0.64
C2H	0.12	0.64	0.23	0.64	0.57	0.64	1.07	0.64
C3L								
C3M								
C3H								
PC1	0.18	0.64	0.24	0.64	0.44	0.64	0.71	0.64
PC2L	0.18	0.64	0.25	0.64	0.40	0.64	0.74	0.64
PC2M	0.15	0.64	0.21	0.64	0.45	0.64	0.86	0.64
PC2H	0.12	0.64	0.19	0.64	0.46	0.64	0.90	0.64
RM1L	0.22	0.64	0.30	0.64	0.50	0.64	0.85	0.64
RM1M	0.18	0.64	0.26	0.64	0.51	0.64	1.03	0.64
RM2L	0.20	0.64	0.28	0.64	0.47	0.64	0.81	0.64
RM2M	0.16	0.64	0.23	0.64	0.48	0.64	0.99	0.64
RM2H	0.12	0.64	0.20	0.64	0.48	0.64	1.01	0.64
URML								
URMM								
MH	0.11	0.64	0.18	0.64	0.31	0.64	0.60	0.64

Table 5.16c Equivalent-PGA Structural Fragility - Low-Code Seismic Design Level

Building Type	Median Equivalent-PGA (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.20	0.64	0.34	0.64	0.61	0.64	0.95	0.64
W2	0.14	0.64	0.23	0.64	0.48	0.64	0.75	0.64
S1L	0.12	0.64	0.17	0.64	0.30	0.64	0.48	0.64
S1M	0.12	0.64	0.18	0.64	0.29	0.64	0.49	0.64
S1H	0.10	0.64	0.15	0.64	0.28	0.64	0.48	0.64
S2L	0.13	0.64	0.17	0.64	0.30	0.64	0.50	0.64
S2M	0.12	0.64	0.18	0.64	0.35	0.64	0.58	0.64
S2H	0.11	0.64	0.17	0.64	0.36	0.64	0.63	0.64
S3	0.10	0.64	0.13	0.64	0.20	0.64	0.38	0.64
S4L	0.13	0.64	0.16	0.64	0.26	0.64	0.46	0.64
S4M	0.12	0.64	0.17	0.64	0.31	0.64	0.54	0.64
S4H	0.12	0.64	0.17	0.64	0.33	0.64	0.59	0.64
S5L	0.13	0.64	0.17	0.64	0.28	0.64	0.45	0.64
S5M	0.11	0.64	0.18	0.64	0.34	0.64	0.53	0.64
S5H	0.10	0.64	0.18	0.64	0.35	0.64	0.58	0.64
C1L	0.12	0.64	0.15	0.64	0.27	0.64	0.45	0.64
C1M	0.12	0.64	0.17	0.64	0.32	0.64	0.54	0.64
C1H	0.10	0.64	0.15	0.64	0.27	0.64	0.44	0.64
C2L	0.14	0.64	0.19	0.64	0.30	0.64	0.52	0.64
C2M	0.12	0.64	0.19	0.64	0.38	0.64	0.63	0.64
C2H	0.11	0.64	0.19	0.64	0.38	0.64	0.65	0.64
C3L	0.12	0.64	0.17	0.64	0.26	0.64	0.44	0.64
C3M	0.11	0.64	0.17	0.64	0.32	0.64	0.51	0.64
C3H	0.09	0.64	0.16	0.64	0.33	0.64	0.53	0.64
PC1	0.13	0.64	0.17	0.64	0.25	0.64	0.45	0.64
PC2L	0.13	0.64	0.15	0.64	0.24	0.64	0.44	0.64
PC2M	0.11	0.64	0.16	0.64	0.31	0.64	0.52	0.64
PC2H	0.11	0.64	0.16	0.64	0.31	0.64	0.55	0.64
RM1L	0.16	0.64	0.20	0.64	0.29	0.64	0.54	0.64
RM1M	0.14	0.64	0.19	0.64	0.35	0.64	0.63	0.64
RM2L	0.14	0.64	0.18	0.64	0.28	0.64	0.51	0.64
RM2M	0.12	0.64	0.17	0.64	0.34	0.64	0.60	0.64
RM2H	0.11	0.64	0.17	0.64	0.35	0.64	0.62	0.64
URML	0.14	0.64	0.20	0.64	0.32	0.64	0.46	0.64
URMM	0.10	0.64	0.16	0.64	0.27	0.64	0.46	0.64
MH	0.11	0.64	0.18	0.64	0.31	0.64	0.60	0.64

Table 5.16d Equivalent-PGA Structural Fragility - Pre-Code Seismic Design Level

Building Type	Median Equivalent-PGA (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.18	0.64	0.29	0.64	0.51	0.64	0.77	0.64
W2	0.12	0.64	0.19	0.64	0.37	0.64	0.60	0.64
S1L	0.09	0.64	0.13	0.64	0.22	0.64	0.38	0.64
S1M	0.09	0.64	0.14	0.64	0.23	0.64	0.39	0.64
S1H	0.08	0.64	0.12	0.64	0.22	0.64	0.38	0.64
S2L	0.11	0.64	0.14	0.64	0.23	0.64	0.39	0.64
S2M	0.10	0.64	0.14	0.64	0.28	0.64	0.47	0.64
S2H	0.09	0.64	0.13	0.64	0.29	0.64	0.50	0.64
S3	0.08	0.64	0.10	0.64	0.16	0.64	0.30	0.64
S4L	0.10	0.64	0.13	0.64	0.20	0.64	0.36	0.64
S4M	0.09	0.64	0.13	0.64	0.25	0.64	0.43	0.64
S4H	0.09	0.64	0.14	0.64	0.27	0.64	0.47	0.64
S5L	0.11	0.64	0.14	0.64	0.22	0.64	0.37	0.64
S5M	0.09	0.64	0.14	0.64	0.28	0.64	0.43	0.64
S5H	0.08	0.64	0.14	0.64	0.29	0.64	0.46	0.64
C1L	0.10	0.64	0.12	0.64	0.21	0.64	0.36	0.64
C1M	0.09	0.64	0.13	0.64	0.26	0.64	0.43	0.64
C1H	0.08	0.64	0.12	0.64	0.21	0.64	0.35	0.64
C2L	0.11	0.64	0.15	0.64	0.24	0.64	0.42	0.64
C2M	0.10	0.64	0.15	0.64	0.30	0.64	0.50	0.64
C2H	0.09	0.64	0.15	0.64	0.31	0.64	0.52	0.64
C3L	0.10	0.64	0.14	0.64	0.21	0.64	0.35	0.64
C3M	0.09	0.64	0.14	0.64	0.25	0.64	0.41	0.64
C3H	0.08	0.64	0.13	0.64	0.27	0.64	0.43	0.64
PC1	0.11	0.64	0.14	0.64	0.21	0.64	0.35	0.64
PC2L	0.10	0.64	0.13	0.64	0.19	0.64	0.35	0.64
PC2M	0.09	0.64	0.13	0.64	0.24	0.64	0.42	0.64
PC2H	0.09	0.64	0.13	0.64	0.25	0.64	0.43	0.64
RM1L	0.13	0.64	0.16	0.64	0.24	0.64	0.43	0.64
RM1M	0.11	0.64	0.15	0.64	0.28	0.64	0.50	0.64
RM2L	0.12	0.64	0.15	0.64	0.22	0.64	0.41	0.64
RM2M	0.10	0.64	0.14	0.64	0.26	0.64	0.47	0.64
RM2H	0.09	0.64	0.13	0.64	0.27	0.64	0.50	0.64
URML	0.13	0.64	0.17	0.64	0.26	0.64	0.37	0.64
URMM	0.09	0.64	0.13	0.64	0.21	0.64	0.38	0.64
MH	0.08	0.64	0.11	0.64	0.18	0.64	0.34	0.64

5.5 Building Damage Due to Ground Failure

5.5.1 Overview

Building damage is characterized by four damage states (i.e., Slight, Moderate, Extensive and Complete). These four states are simplified for ground failure to include only one combined Extensive/Complete damage state. In essence, buildings are assumed to be either undamaged or severely damaged due to ground failure. In fact, Slight or Moderate damage can occur due to ground failure, but the likelihood of this damage is considered to be small (relative to ground shaking damage) and tacitly included in predictions of Slight or Moderate damage due to ground shaking.

Given the earthquake demand in terms of permanent ground deformation (PGD), the probability of being in the Extensive/Complete damage state is estimated using fragility curves of a form similar to those used to estimate shaking damage. Separate fragility curves distinguish between ground failure due to lateral spreading and ground failure due to ground settlement, and between shallow and deep foundations.

5.5.2 Fragility Curves - Peak Ground Displacement

There is no available relationship between the likelihood of Extensive/Complete damage of buildings and PGD. Engineering judgment is used to develop a set of assumptions, which define building fragility. These assumptions are shown in Table 5.17 for buildings with shallow foundations (e.g., spread footings).

Table 5.17 Building Damage Relationship to PGD - Shallow Foundations

$P[E \text{ or } C \mid PGD]$	Settlement PGD (inches)	Lateral Spread PGD (inches)
0.1	2	12
0.5 (median)	10	60

The above assumptions are based on the expectation that about 10 (i.e., 8 Extensive damage, 2 Complete damage) out of 100 buildings on spread footings would be severely damaged for 2 inches of settlement PGD or 12 inches of lateral spread PGD, and that about 50 (i.e., 40 Extensive damage, 10 Complete damage) out of 100 buildings on spread footings would be severely damaged for 10 inches of settlement PGD or 60 inches of lateral spread PGD. Lateral spread is judged to require significantly more PGD to effect severe damage than ground settlement. Many buildings in lateral spread areas are expected to move with the spread, but not to be severely damaged until the spread becomes quite significant.

Median PGD values given in the Table 5.17 are used with a lognormal standard deviation value of $\beta_{PGD} = 1.2$ to estimate P[E or C|PGD] for buildings on shallow foundations or buildings of unknown foundation type. The value of $\beta_{PGD} = 1.2$ is based on the factor of 5 between the PGD values at the 10 and 50 percentile levels.

No attempt is made to distinguish damage based on building type, since model building descriptions do not include foundation type. Foundation type is critical to PGD performance and buildings on deep foundations (e.g., piles) perform much better than buildings on spread footings, if the ground settles. When the building is known to be supported by a deep foundation, the probability of Extensive or Complete damage is reduced by a factor of 10 from that predicted for settlement-induced damage of the same building on a shallow foundation. Deep foundations will improve building performance by only a limited amount, if ground spreads laterally. When the building is known to be supported by a deep foundation, the probability of Extensive or Complete damage is reduced by a factor of 2 from that predicted for spread-induced damage of the same building on a shallow foundation.

5.6 Evaluation of Building Damage

5.6.1 Overview

During an earthquake, the building may be damaged either by ground shaking, ground failure, or both. Buildings are evaluated separately for the two modes of failure the resulting damage-state probabilities are combined for evaluation of loss.

5.6.2 Damage Due to Ground Shaking

This section describes the process of developing damage state probabilities based on structural and nonstructural fragility curves, model building capacity curves and a demand spectrum. Building response (e.g., peak displacement) is determined by the intersection of the demand spectrum and the building capacity curve. The demand spectrum is based on the PESH input spectrum reduced for effective damping (when effective damping exceeds the 5% damping level of the PESH input spectrum).

5.6.2.1 Demand Spectrum Reduction for Effective Damping

The elastic response spectra provided as a PESH input apply only to buildings that remain elastic during the entire ground shaking time history and have elastic damping values equal to 5% of critical. This is generally not true on both accounts. Therefore, two modifications are made to elastic response spectra: (a) demand spectra are modified for buildings with elastic damping not equal to 5%, and (b) demand spectra are modified for the hysteretic energy dissipated by buildings “pushed” beyond their elastic limits. Modifications are represented by reduction factors by which the spectral ordinates are divided to obtain the damped demand spectra.

Extensive work has been published in the past two decades on the effect of damping and/or energy dissipation on spectral demand. The Methodology reduces demand spectra for effective damping greater than 5% based on statistically-based formulas of Newmark and Hall (1982). Other methods are available for estimating spectral reduction factors based on statistics relating reduction to ductility demand. It is believed that both methods yield the same results for most practical purposes (FEMA 273). Newmark and Hall provide formulas for construction of elastic response spectra at different damping ratios, B (expressed as a percentage). These formulas represent all site classes (soil types) distinguishing between domains of constant acceleration and constant velocity. Ratios of these formulas are used to develop an acceleration-domain (short-period) reduction factor, R_A, and a velocity-domain (1-second spectral acceleration) reduction factor, R_V, for modification of 5%-damped, elastic response spectra (PESH input). These reduction factors are based on effective damping, B_{eff}, as given in Equations (5-7) and (5-8) below:

$$R_A = 2.12 \big/ \left(3.21 - 0.68 \ln(B_{eff})\right) \tag{5-7}$$

$$R_V = 1.65 \big/ \left(2.31 - 0.41 \ln(B_{eff})\right) \tag{5-8}$$

for which effective damping is defined as the sum of elastic damping, B_E, and hysteretic damping, B_H:

$$B_{eff} = B_E + B_H \tag{5-9}$$

Elastic damping, B_E, is dependent on structure type and is based on the recommendations of Newmark & Hall for materials at or just below their yield point. Hysteretic damping, B_H, is dependent on the amplitude of response and is based on the area enclosed by the hysteresis loop, considering potential degradation of energy-absorption capacity of the structure during cyclic earthquake load. Effective damping, B_{eff}, is also a function of the amplitude of response (e.g., peak displacement), as expressed in Equation (5-10):

$$B_{eff} = B_E + \kappa \cdot \left(\frac{\text{Area}}{2\pi \cdot D \cdot A} \right) \tag{5-10}$$

where:

- B_E is the elastic (pre-yield) damping of the model building type
- Area is the area enclosed by the hysteresis loop, as defined by a symmetrical push-pull of the building capacity curve up to peak positive and negative displacements, $\pm D$
- D is the peak displacement response of the push-over curve,
- A is the peak acceleration response at peak displacement, D
- κ is a degradation factor that defines the effective amount of hysteretic damping as a function of earthquake duration, as specified in Table 5.18.

Table 5.18 Degradation Factor (κ) as a Function of Short, Moderate and Long Earthquake Duration

Building Type		High-Code Design			Moderate-Code Design			Low-Code Design			Pre-Code Design		
No.	Label	Short	Moderate	Long	Short	Moderate	Long	Short	Moderate	Long	Short	Moderate	Long
1	W1	1.00	0.80	0.50	0.90	0.60	0.30	0.70	0.40	0.20	0.50	0.30	0.10
2	W2	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
3	S1L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
4	S1M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
5	S1H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
6	S2L	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
7	S2M	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
8	S2H	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
9	S3	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
10	S4L	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
11	S4M	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
12	S4H	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
13	S5L	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
14	S5M	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
15	S5H	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
16	C1L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
17	C1M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
18	C1H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
19	C2L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
20	C2M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
21	C2H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
22	C3L	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
23	C3M	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
24	C3H	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
25	PC1	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
26	PC2L	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
27	PC2M	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
28	PC2H	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
29	RM1L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
30	RM1M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
31	RM2L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
32	RM2M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
33	RM2H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
34	URML	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
35	URMM	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
36	MH	0.80	0.40	0.20	0.80	0.40	0.20	0.80	0.40	0.20	0.60	0.30	0.10

The Methodology recognizes the importance of the duration of ground shaking on building response by reducing effective damping (i.e., κ factors) as a function of shaking duration. Shaking duration is described qualitatively as either Short, Moderate or Long, and is assumed to be a function of earthquake magnitude (although proximity to fault rupture also influences the duration of ground shaking). For scenario earthquakes of magnitude $\mathbf{M} \leq 5.5$, effective damping is based on the assumption of ground shaking of Short duration. For scenario earthquakes of magnitude $\mathbf{M} \geq 7.5$, effective damping is based on the assumption of ground shaking of Long duration. Effective damping is based on the assumption of Moderate duration for all other earthquake magnitudes (including probabilistic, or other, analyses of unknown magnitude).

Construction of Demand Spectra

Demand spectral acceleration, $S_A[T]$, in units of acceleration (g) is defined by Equation (5-11a) at short periods (acceleration domain), Equation (5-11b) at long periods (velocity domain) and Equation (5-11c) at very long periods (displacement domain).

At short periods, $0 < T \leq T_{AV\beta}$:

$$S_A[T] = S_{ASi} \big/ R_A[B_{eff}] = S_{ASi} \big/ \left(2.12 \big/ \left(3.21 - 0.68 \ln\left(B_{eff}\right)\right)\right) \tag{5-11a}$$

At long periods, $T_{AV\beta} < T \leq T_{VD}$:

$$S_A[T] = \left(\frac{S_{A1i}}{T}\right) \Big/ R_V[B_{eff}] = \left(\frac{S_{A1i}}{T}\right) \Big/ \left(1.65 \big/ \left(2.31 - 0.41 \ln\left(B_{eff}\right)\right)\right) \tag{5-11b}$$

At very long periods, $T > T_{VD}$:

$$S_A[T] = \left(\frac{S_{A1i} \cdot T_{VD}}{T^2}\right) \Big/ R_V[B_{TVD}] = \left(\frac{S_{A1i} \cdot T_{VD}}{T^2}\right) \Big/ \left(1.65 \big/ \left(2.31 - 0.41 \ln\left(B_{TVD}\right)\right)\right) \tag{5-11c}$$

where:

- S_{ASi} is the 5%-damped, short-period spectral acceleration for Site Class i (in units of g), as defined by Equation (4-5),
- S_{A1i} is the 5%-damped, 1-second-period spectral acceleration for Site Class i (units of g), as defined by Equation (4-6), times 1 second,
- T_{AVi} is the transition period between 5%-damped constant spectral acceleration and 5%-damped constant spectral velocity for Site Class i (sec.), as defined by Equation (4-7),
- B_{TVD} is the value of effective damping at the transition period, T_{VD}, and
- B_{TAVB} is the value of effective damping at the transition period, T_{AVB}.

The transition period, T_{AVB}, between acceleration and velocity domains is a function of the effective damping at this period, as defined by Equation (5-12). The transition period, T_{VD}, between velocity and displacement domains is independent of effective damping, as defined by Equation (4-4).

$$T_{AVB} = T_{AVi}\left(\frac{R_A\left[B_{TAVB}\right]}{R_V\left[B_{TAVB}\right]}\right) = T_{AVi}\left(\frac{2.12 \big/ \left(3.21 - 0.68 \ln\left(B_{TAVB}\right)\right)}{1.65 \big/ \left(2.31 - 0.41 \ln\left(B_{TAVB}\right)\right)}\right) \tag{5-12}$$

Demand spectral displacement, $S_D[T]$, in inches, is based on $S_A[T]$, in units of g, as given on Equation (5-13):

$$S_D[T] = 9.8 \cdot S_A[T] \cdot T^2 \tag{5-13}$$

Figure 5.7 shows typical demand spectra (spectral acceleration plotted as a function of spectral displacement) for three demand levels. These three demand levels represent Short (κ = 0.80), Moderate (κ = 0.40) and Long (κ = 0.20) duration ground shaking, respectively. Also shown in the figure is the building capacity curve of a low-rise building of Moderate-Code seismic design that was used to estimate effective damping.

The intersection of the capacity curve with each of the three demand spectra illustrates the significance of duration (damping) on building response.

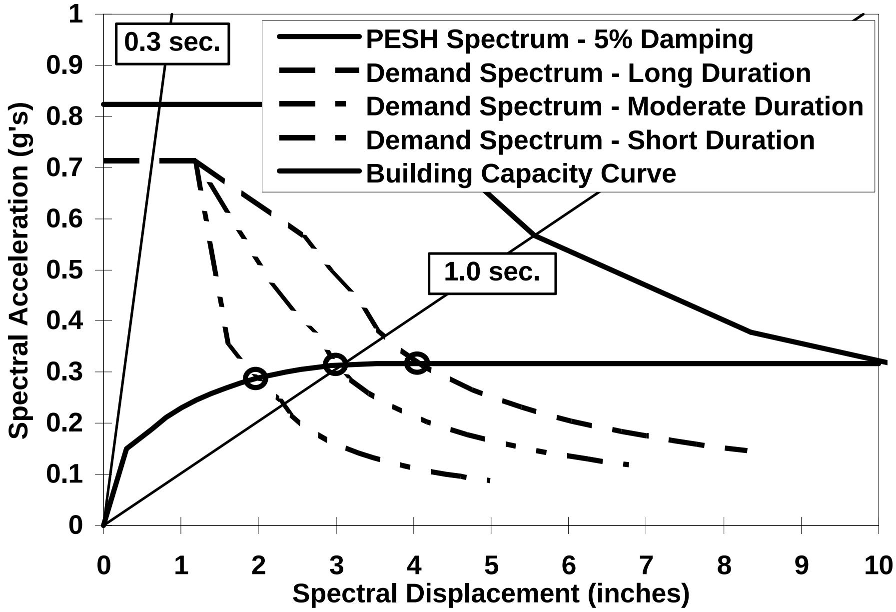


Figure 5.7 Example Demand Spectra - Moderate-Code Building (M = 7.0 at 20 km, WUS, Site Class E).

5.6.2.2 Damage State Probability

Structural and nonstructural fragility curves are evaluated for spectral displacement and spectral acceleration defined by the intersection of the capacity and demand curves. Each of these curves describes the cumulative probability of being in, or exceeding, a particular damage state. Nonstructural components (both drift- and acceleration-sensitive components) may, in some cases, be dependent on the structural damage state (e.g., Complete structural damage may cause Complete nonstructural damage). The Methodology assumes nonstructural damage states to be independent of structural damage states. Cumulative probabilities are differenced to obtain discrete probabilities of being in each of the five damage states. This process is shown schematically in Figure 5.8.

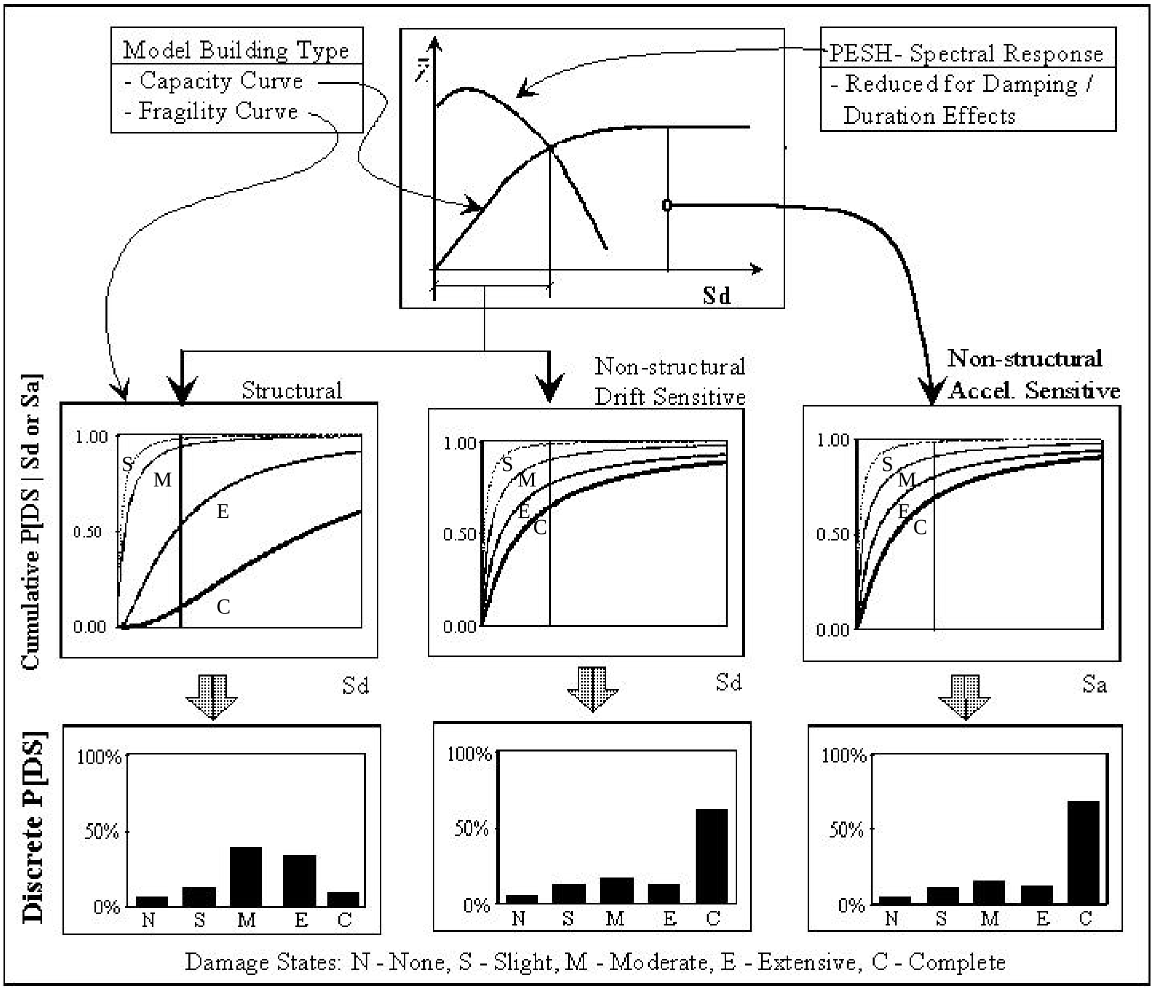


Figure 5.8 Example Building Damage Estimation Process.

It is also meaningful to interpret damage probabilities as the fraction of all buildings (of the same type) that would be in the particular damage state of interest. For example, a 30% probability of Moderate damage may also be thought of as 30 out of 100 buildings (of the same type) being in the Moderate damage state.

5.6.3 Combined Damage Due to Ground Failure and Ground Shaking

This section describes the combination of damage state probabilities due to ground failure (Section 5.5.2) and ground shaking (Section 5.6.2.2). It is assumed that damage due to ground shaking (GS) is independent of damage due to ground failure (GF). Ground failure tends to cause severe damage to buildings and is assumed to contribute only to Extensive and Complete damage states (refer to Section 5.5.1). These assumptions are described by the following formulas:

$$P_{GF}[DS \geq S] = P_{GF}[DS \geq E] \tag{5-14}$$

$$P_{GF}[DS \geq M] = P_{GF}[DS \geq E] \tag{5-15}$$

$$P_{GF}[DS \geq C] = 0.2 \times P_{GF}[DS \geq E] \tag{5-16}$$

The damage state probability (probability of being in or exceeding a given damage state) for GF is assumed to be the maximum of the three types of ground failure (liquefaction, landsliding, and spread). Thus, the combined (due to occurrence of GF or GS) probabilities of being in or exceeding given damage states are:

$$P_{COMB}[DS \geq S] = P_{GF}[DS \geq S] + P_{GS}[DS \geq S] - P_{GF}[DS \geq S] \times P_{GS}[DS \geq S] \tag{5-17}$$

$$P_{COMB}[DS \geq M] = P_{GF}[DS \geq M] + P_{GS}[DS \geq M] - P_{GF}[DS \geq M] \times P_{GS}[DS \geq M] \tag{5-18}$$

$$P_{COMB}[DS \geq E] = P_{GF}[DS \geq E] + P_{GS}[DS \geq E] - P_{GF}[DS \geq E] \times P_{GS}[DS \geq E] \tag{5-19}$$

$$P_{COMB}[DS \geq C] = P_{GF}[DS \geq C] + P_{GS}[DS \geq C] - P_{GF}[DS \geq C] \times P_{GS}[DS \geq C] \tag{5-20}$$

where DS is damage state, and the symbols: S, M, E, and C stand for Slight, Moderate, Extensive, and Complete damage, respectively. COMB indicates the combined probability for the damage state due to occurrence of ground failure or ground shaking. Note that the following condition must always be true:

$$1 \geq P_{COMB}[DS \geq S] \geq P_{COMB}[DS \geq M] \geq P_{COMB}[DS \geq E] \geq P_{COMB}[DS \geq C] \tag{5-21}$$

The discrete probabilities (probabilities of being in a given damage state) are given as:

$$P_{COMB}[DS = C] = P_{COMB}[DS \geq C] \tag{5-22}$$

$$P_{COMB}[DS = E] = P_{COMB}[DS \geq E] - P_{COMB}[DS \geq C] \tag{5-23}$$

$$P_{COMB}[DS = M] = P_{COMB}[DS \geq M] - P_{COMB}[DS \geq E] \tag{5-24}$$

$$P_{COMB}[DS = S] = P_{COMB}[DS \geq S] - P_{COMB}[DS \geq M] \tag{5-25}$$

$$P_{COMB}[DS = \text{None}] = 1 - P_{COMB}[DS \geq S] \tag{5-26}$$

5.6.4 Combined Damage to Occupancy Classes

The damage state probabilities for model building types (as estimated from Section 5.6.3) are combined to yield the damage state probabilities of the occupancy classes to which they belong. For each damage state, the probability of damage to each model building type is weighted according to the fraction of the total floor area of that model building type and summed over all building types. This is expressed in equation form:

$$POSTR_{ds,i} = \sum_{j=1}^{36}\left[PMBTSTR_{ds,j} \times \frac{FA_{i,j}}{FA_i}\right] \tag{5-27}$$

where $PMBTSTR_{ds,j}$ is the probability of the model building type j being in damage state ds. $POSTR_{ds,i}$ is the probability of occupancy class i being in damage state ds. $FA_{i,j}$ indicates the floor area of model building type j in occupancy class i, and FA_i denotes the total floor area of the occupancy class i (refer to Chapter 3 for floor area distributions of model building types by occupancy class). Similarly, the damage-state probabilities for nonstructural components can be estimated.

$$PONSD_{ds,i} = \sum_{j=1}^{36}\left[PMBTNSD_{ds,j} \times \frac{FA_{i,j}}{FA_i}\right] \tag{5-28}$$

$$PONSA_{ds,i} = \sum_{j=1}^{36}\left[PMBTNSA_{ds,j} \times \frac{FA_{i,j}}{FA_i}\right] \tag{5-29}$$

where $PMBTNSD_{ds,j}$ and $PMBTNSA_{ds,j}$ refer to the probabilities of model building type j being in nonstructural drift- and acceleration-sensitive damage state ds, respectively; and $PONSD_{ds,i}$ and $PONSA_{ds,i}$ refer to the probabilities of the occupancy class i being the nonstructural drift-sensitive and acceleration-sensitive damage state, ds, respectively. These occupancy class probabilities are used in Chapter 15 to estimate direct economic loss.

5.7 Guidance for Expert Users

This section provides guidance for users who are seismic/structural experts interested in modifying the building damage functions supplied with the methodology. This section also provides the expert user with guidance regarding the selection of the appropriate mix of design levels for the region of interest.

5.7.1 Selection of Representative Seismic Design Level

The methodology permits the user to select the seismic design level considered appropriate for the study region and to define a mix of seismic design levels for each model building type. The building damage functions provided are based on current-Code provisions and represent buildings of modern design and construction. Most buildings in a study region will likely not be of modern design and construction (i.e., do not conform to 1976 *UBC*, *1985 NEHRP Provisions*, or later editions of these model Codes). For many study regions, particularly those in the Central and Eastern United States, seismic provisions may not be enforced (or only adopted very recently). Building damage functions for new buildings designed and constructed to meet modern-Code provisions should not be used for older, non-complying buildings.

The building damage functions represent specific cells of a three by three matrix that defines three seismic design levels (High, Moderate and Low) and, for each of these design levels, three seismic performance levels (Inferior, Ordinary and Superior), as shown in Table 5.19. For completeness, cells representing Special buildings of Chapter 6 (Essential Facilities) are also included in the matrix.

Table 5.19 Seismic Design and Performance Levels of Default Building Damage Functions (and Approximate Structural Strength and Ductility)

Seismic Design Level	**Seismic Performance Level**		
	Superior[1]	**Ordinary**	**Inferior**
High **(*UBC* Zone 4)**	<u>Special High-Code</u> *Maximum Strength* *Maximum Ductility*	<u>High-Code</u> *High Strength* *High Ductility*	*Moderate Strength* *Mod./Low Ductility*
Moderate **(*UBC* Zone 2B)**	<u>Special Moderate-Code</u> *High/Mod. Strength* *High Ductility*	<u>Moderate-Code</u> *Moderate Strength* *Moderate Ductility*	*Low Strength* *Low Ductility*
Low **(*UBC* Zone 1)**	<u>Special Low-Code</u> *Mod./Low Strength* *Moderate Ductility*	<u>Low-Code</u> *Low Strength* *Low Ductility*	<u>Pre-Code</u> *Minimal Strength* *Minimal Ductility*

1. See Chapter 6 for Special High-Code, Moderate-Code and Low-Code building damage functions.

Table 5.19 also defines the approximate structural strength and ductility attributes of buildings occupying each of the nine cells. The design level is defined by Seismic Zones of the *Uniform Building Code* (*UBC*), since most buildings in the United States that have been designed for earthquakes used some version of the *UBC.* Table 5.20 relates *UBC* seismic zones to seismic design regions of the *NEHRP Provisions*.

Expert users may tailor the damage functions to their study area of interest by determining the appropriate fraction of each building type that conforms essentially to modern-Code provisions (based on age of construction). Buildings deemed not to conform to modern-Code provisions should be assigned a lower seismic design level, or defined as Pre-Code buildings if not seismically designed. For instance, older buildings located in High-Code seismic design areas should be evaluated using damage functions for either Moderate-Code buildings or Pre-Code buildings, for buildings that pre-date seismic codes. Table 5.20 provides guidance for selecting appropriate building damage functions based on building location (i.e., seismic region) and building age. The years shown as break-off points should be considered very approximate and may not be appropriate for many seismic regions, particularly regions of low and moderate seismicity where seismic codes have not been routinely enforced.

Table 5.20 Guidelines for Selection of Damage Functions for Typical Buildings Based on *UBC* Seismic Zone and Building Age

UBC **Seismic Zone (NEHRP Map Area)**	**Post-1975**	**1941 - 1975**	**Pre-1941**
Zone 4 (Map Area 7)	High-Code	Moderate-Code	Pre-Code (W1 = Moderate-Code)
Zone 3 (Map Area 6)	Moderate-Code	Moderate-Code	Pre-Code (W1 = Moderate-Code)
Zone 2B (Map Area 5)	Moderate-Code	Low-Code	Pre-Code (W1 = Low-Code)
Zone 2A (Map Area 4)	Low-Code	Low-Code	Pre-Code (W1 = Low-Code)
Zone 1 (Map Area 2/3)	Low-Code	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)
Zone 0 (Map Area 1)	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)

The guidelines given in Table 5.20 assume that buildings in the study region are not designed for wind. The user should consider the possibility that mid-rise and high-rise buildings could be designed for wind and may have considerable lateral strength (though not ductility), even if not designed for earthquake. Users must be knowledgeable about the type and history of construction in the study region of interest and apply engineering judgment in assigning the fraction of each building type to a seismic design group.

5.7.2 Development of Damage Functions for Other Buildings

For a building type other than one of the 36 described in Table 5.1, expert users should select a set of building damage functions that best represents the type of construction, strength and ductility of the building type of interest. Such buildings include rehabilitated structures that have improved seismic capacity. For example, URM (Pre-Code) buildings retrofitted in accordance with Division 88, the Los Angeles City Ordinance to "reduce the risk of life loss," demonstrated significantly improved seismic performance during the 1994 Northridge earthquake [SSC, 1995]. Structural damage to these buildings would be better estimated using either essential facility damage functions of either Low-Code or Moderate-Code RM1 buildings.

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Chapter 6
Direct Physical Damage - Essential and High Potential Loss Facilities

6.1 Introduction

This chapter describes methods for determining the probability of Slight, Moderate, Extensive and Complete damage to essential facilities. These methods are identical to those of Chapter 5 that describe damage to "Code" buildings, except that certain essential facilities are represented by "Special" building damage functions. Special building damage functions are appropriate for evaluation of essential facilities when the user anticipates above-Code seismic performance for these facilities. The flowchart of the methodology highlighting the essential and high potential loss facility damage components and showing its relationship to other components is shown in Flowchart 6.1.

This chapter also provides guidance for high potential loss (HPL) facilities. The methodology highlighting the Direct Physical Damage is shown in Flowchart 6.1.

6.1.1 Scope

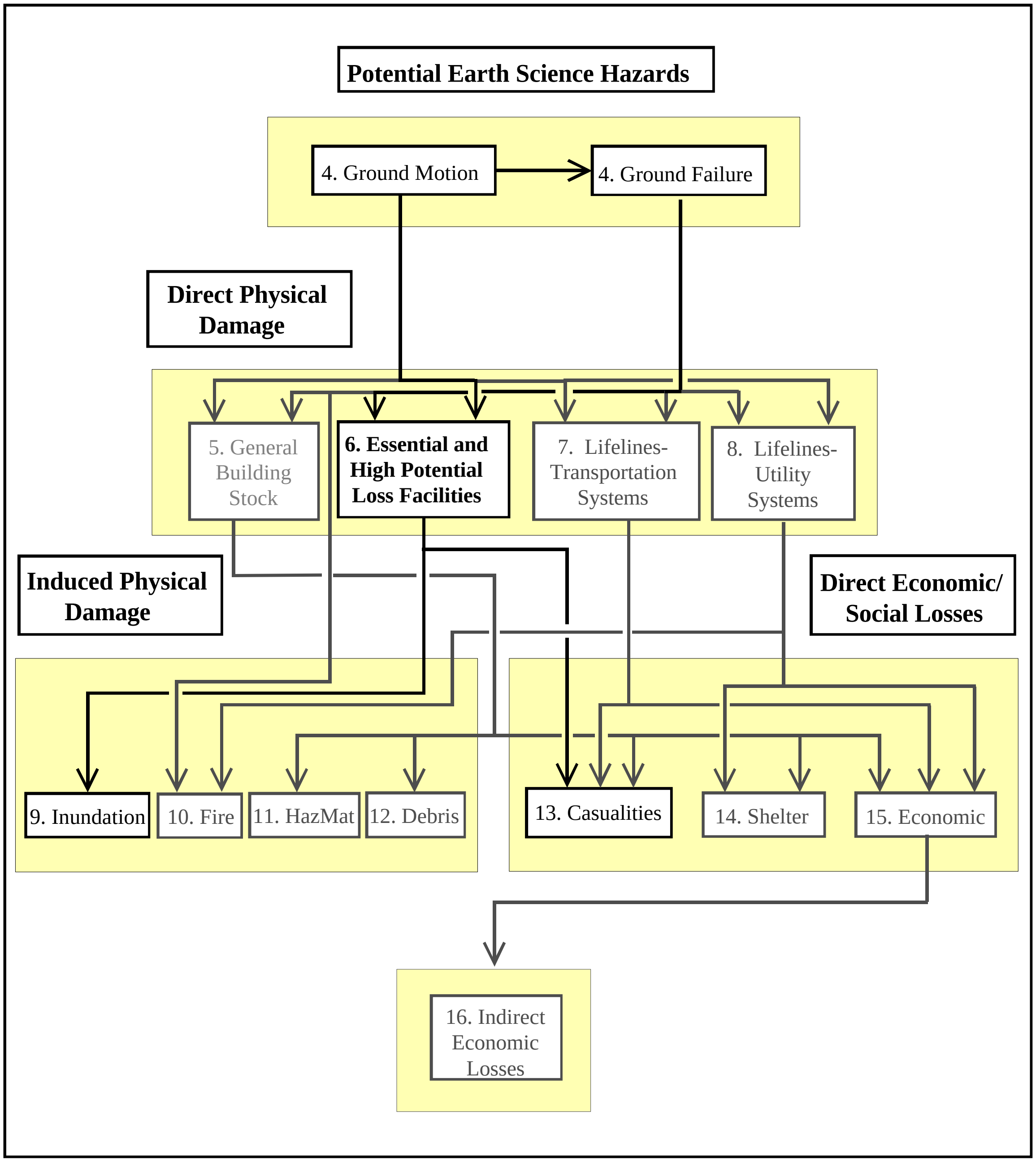
The scope of this chapter includes: (1) classification of essential facilities, (2) building damage functions for Special buildings, (3) methods for estimation of earthquake damage to essential facilities, given knowledge of the model building type and seismic design level, and an estimate of earthquake demand, and (3) guidance for expert users, including estimation of damage to high potential loss (HPL) facilities.

Special buildings and their damage functions are described in Sections 6.2 through 6.5. Evaluation of damage to essential facilities is given in Section 6.6 and guidance for expert users is given in Section 6.7. Typically, sections of Chapter 6 reference (rather than repeat) material of the corresponding section of Chapter 5.

6.1.2 Essential Facility Classification

Facilities that provide services to the community and those that should be functional following an earthquake are considered to be essential facilities. Examples of essential facilities include hospitals, police stations, fire stations, emergency operations centers (EOC's) and schools. The methodology adopted for damage assessment of such facilities is explained in this section.

Essential facilities are classified on the basis of facility function and, in the case of hospitals, size. Table 6.1 lists the classes of essential facilities used in the Methodology. Hospitals are classified on the basis of number of beds, since the structural and nonstructural systems of a hospital are related to the size of the hospital (i.e., to the number of beds it contains).



Flowchart 6.1: Essential and High Potential Loss Facility Component Relationship to other Components in the Methodology

Table 6.1 Classification of Essential Facilities

No.	Label	Occupancy Class	Description
		Medical Care Facilities	
1	EFHS	Small Hospital	Hospital with less than 50 Beds
2	EFHM	Medium Hospital	Hospital with beds between 50 & 150
3	EFHL	Large Hospital	Hospital with greater than 150 Beds
4	EFMC	Medical Clinics	Clinics, Labs, Blood Banks
		Emergency Response	
5	EFFS	Fire Station	
6	EFPS	Police Station	
7	EFEO	Emergency Operation Centers	
		Schools	
8	EFS1	Schools	Primary/ Secondary Schools (K-12)
9	EFS2	Colleges/Universities	Community and State Colleges, State and Private Universities

It is the responsibility of the user to identify each essential facility as either a Code building or a Special building of a particular model building type and seismic design level. This chapter provides building damage functions for Special buildings that have significantly better than average seismic capacity. Chapter 5 provides building damage functions for Code-buildings. If the user is not able to determine that the essential facility is significantly better than average, then the facility should be modeled using Code building damage functions (i.e., the same methods as those developed in Chapter 5 for general building stock).

6.1.3 Input Requirements and Output Information

Input required to estimate essential facility damage using fragility and capacity curves includes the following two items:

- model building type (including height) and seismic design level that represents the essential facility (or type of essential facilities) of interest, and

- response spectrum (or PGA, for lifeline buildings, and PGD for ground failure evaluation) at the essential facility's site.

The response spectrum, PGA and PGD at the essential facility site are PESH outputs, described in Chapter 4.

The "output" of fragility curves is an estimate of the cumulative probability of being in or exceeding, each damage state for the given level of ground shaking (or ground failure). Cumulative damage probabilities are differenced to create discrete damage state probabilities, as described in Chapter 5 (Section 5.6). Discrete probabilities of damage are used directly as inputs to induced physical damage and direct economic and social loss modules, as shown in Flowchart 6.1.

Typically, the model building type (including height) is not known for each essential facility and must be inferred from the inventory of essential facilities using the occupancy/building type relationships described in Chapter 3. In general, performance of essential facilities is not expected to be better than the typical building of the representative model building type. Exceptions to this generalization include California hospitals of recent (post-1973) construction.

6.1.4 Form of Damage Functions

Building damage functions for essential facilities are of the same form as those described in Chapter 5 for general building stock. For each damage state, a lognormal fragility curve relates the probability of damage to PGA, PGD or spectral demand determined by the intersection of the model building type's capacity curve and the demand spectrum. Figure 6.1 provides an example of fragility curves for four damage states: Slight, Moderate, Extensive and Complete.

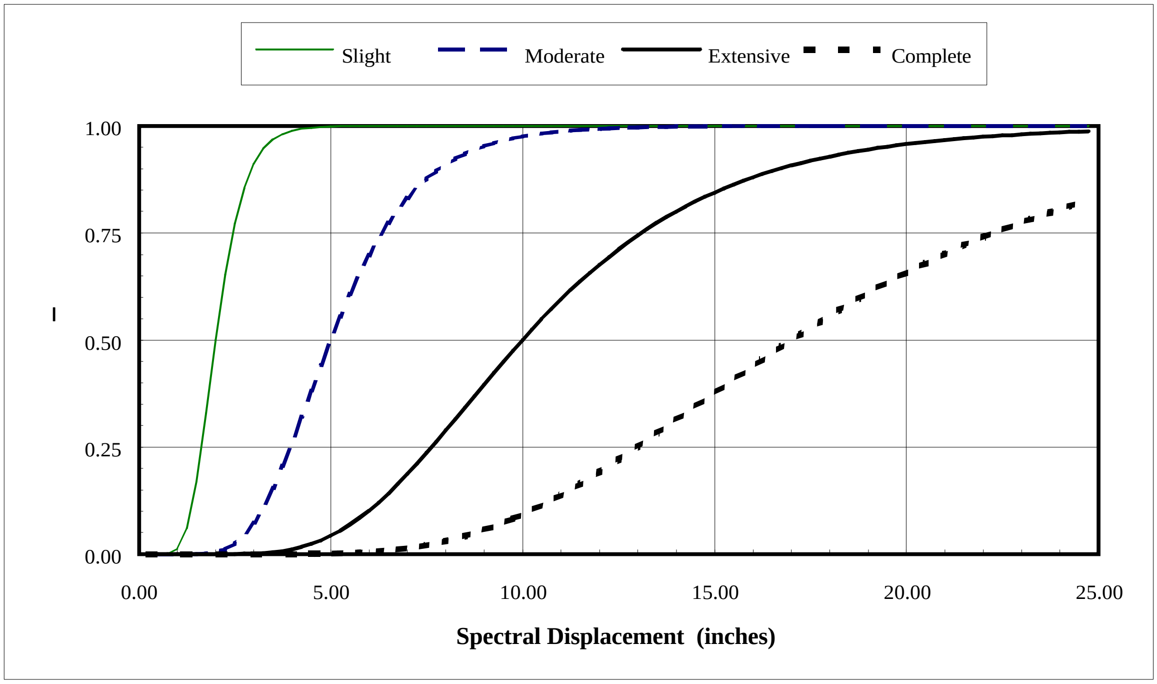


Figure 6.1 Example Fragility Curves for Slight, Moderate, Extensive and Complete Damage.

The fragility curves are driven by a PESH parameter. For ground failure, the PESH parameter used to drive fragility curves is permanent ground displacement (PGD). For ground shaking, the PESH parameter used to drive building fragility curves is peak spectral response (either displacement or acceleration), or peak ground acceleration (PGA) for essential lifeline facilities. Peak spectral response varies significantly for buildings that have different response properties and, therefore, requires knowledge of these properties.

Building response is characterized by building capacity curves. These curves describe the push-over displacement of each building type and seismic design level as a function of laterally-applied earthquake load. Design-, yield- and ultimate-capacity points define the shape of each building capacity curve. The Methodology estimates peak building response as the intersection of the building capacity curve and the demand spectrum at the building's location.

The demand spectrum is the 5%-damped PESH input spectrum reduced for higher levels of effective damping (e.g., effective damping includes both elastic damping and hysteretic damping associated with post-yield cyclic response of the building). Figure 6.2 illustrates the intersection of a typical building capacity curve and a typical demand spectrum (reduced for effective damping greater than 5% of critical).

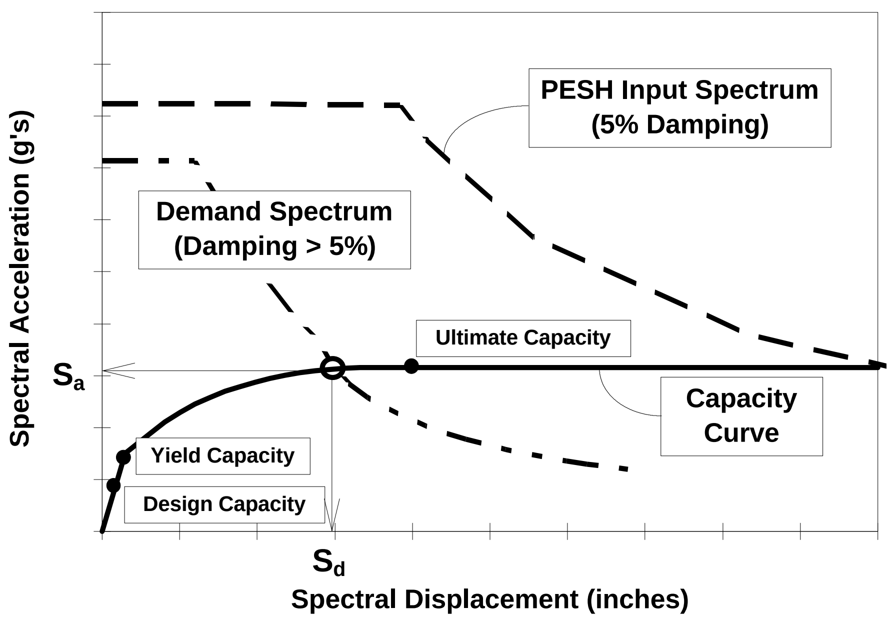


Figure 6.2 Example Building Capacity Curve and Demand Spectrum.

6.2 Description of Model Building Types

The model building types used for essential facilities are identical to those used for general building stock. These building types are described in Section 5.2 and listed in

Table 5.1. Typical nonstructural components of essential facilities include those architectural, mechanical and electrical, and contents listed in Table 5.2 for general building stock.

Essential facilities also include certain special equipment, such as emergency generators, and certain special contents, such as those used to operate a hospital. Special equipment and contents of essential facilities are considered to be acceleration-sensitive nonstructural components of these facilities.

6.3 Description of Building Damage States

Building damage states for structural and nonstructural components of essential facilities are the same as those described in Section 5.3 for general building stock.

6.4 Building Damage Due to Ground Shaking - Special Buildings

6.4.1 Overview

This section describes capacity and fragility curves used in the Methodology to estimate the probability of Slight, Moderate, Extensive and Complete damage to Special buildings of a given model building type designed to High-, Moderate-, or Low-Code seismic standards. Special building damage functions are appropriate for evaluation of essential facilities when the user anticipates above-Code seismic performance for these facilities.

Capacity curves and fragility curves for Special buildings of High-Code, Moderate-Code, or Low-Code seismic design are based on modern code (e.g., 1976 *Uniform Building Code*, 1996 *NEHRP Provisions*, or later editions of these model codes) design criteria for various seismic design zones, as shown in Table 6.2. Additional description of seismic design levels may be found in Section 6.7.

Table 6.2 Approximate Basis for Seismic Design Levels

Seismic Design Level (I = 1.5)	Seismic Zone (1994 *Uniform Building Code*)	Map Area (1994 *NEHRP Provisions*)
High-Code	4	7
Moderate-Code	2B	5
Low-Code	1	3

The capacity and fragility curves represent buildings designed and constructed to modern seismic code provisions (e.g.,1994 *UBC*) using an importance factor of I = 1.5. Moderate-Code and Low-Code seismic design levels are included for completeness. Most essential facilities located in Seismic Zones O, T, 2A or 2B have not been designed for Special building code criteria.

6.4.2 Capacity Curves - Special Buildings

The building capacity curves for Special buildings are similar to those for the general building stock (Chapter 5), but with increased strength. Each curve is described by three control points that define model building capacity:

- Design Capacity
- Yield Capacity
- Ultimate Capacity

Design capacity represents the nominal building strength required by current model seismic code provisions (e.g., 1994 *UBC*) including an importance factor of I = 1.5. Wind design is not considered in the estimation of design capacity and certain buildings (e.g., taller buildings located in zones of low or moderate seismicity) may have a lateral design strength considerably greater than based on seismic code provisions.

Yield capacity represents the true lateral strength of the building considering redundancies in design, conservatism in code requirements and true (rather than nominal) strength of materials. Ultimate capacity represents the maximum strength of the building when the global structural system has reached a fully plastic state. An example building capacity curve is shown in Figure 6-3.

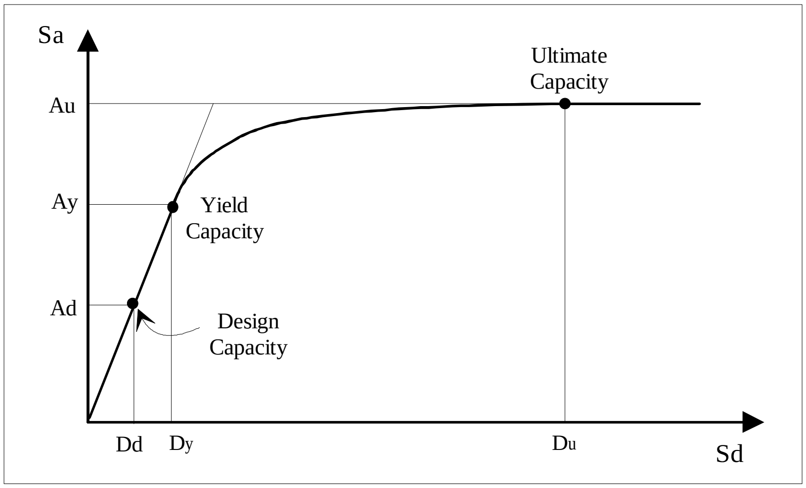


Figure 6.3 Example Building Capacity Curve.

The building capacity curves for Special buildings are constructed based on the same engineering properties (i.e., C_s, T_e, α_1, α_2, γ, λ, μ) as those used to describe capacity curves of Code buildings (i.e., Tables 5.4, 5.5 and 5.6), except for design strength, C_s, and ductility (μ). The design strength, C_s, is approximately based on the lateral-force design requirements of current seismic codes (e.g., 1994 *NEHRP* or 1994 *UBC*) using an importance factor of I = 1.5. Values of the "ductility" factor, μ, for Special buildings are based on Code-building ductility increased by 1.33 for Moderate-Code buildings and by 1.2 for Low-Code buildings. The ductility parameter defines the displacement value of capacity curve at the point where the curve reaches a fully plastic state.

Building capacity curves are assumed to have a range of possible properties that are lognormally distributed as a function of the ultimate strength (A_u) of each capacity curve. Special building capacity curves represent median estimates of building capacity. The variability of the capacity of each building type is assumed to be: $\beta(A_u) = 0.15$ for Special buildings. An example construction of median, 84th percentile (+1β) and 16th percentile (−1β) building capacity curves for a typical building is illustrated in Figure 6.4. Median capacity curves are intersected with demand spectra to estimate peak building response. The variability of the capacity curves is used, with other sources of variability and uncertainty, to define total fragility curve variability.

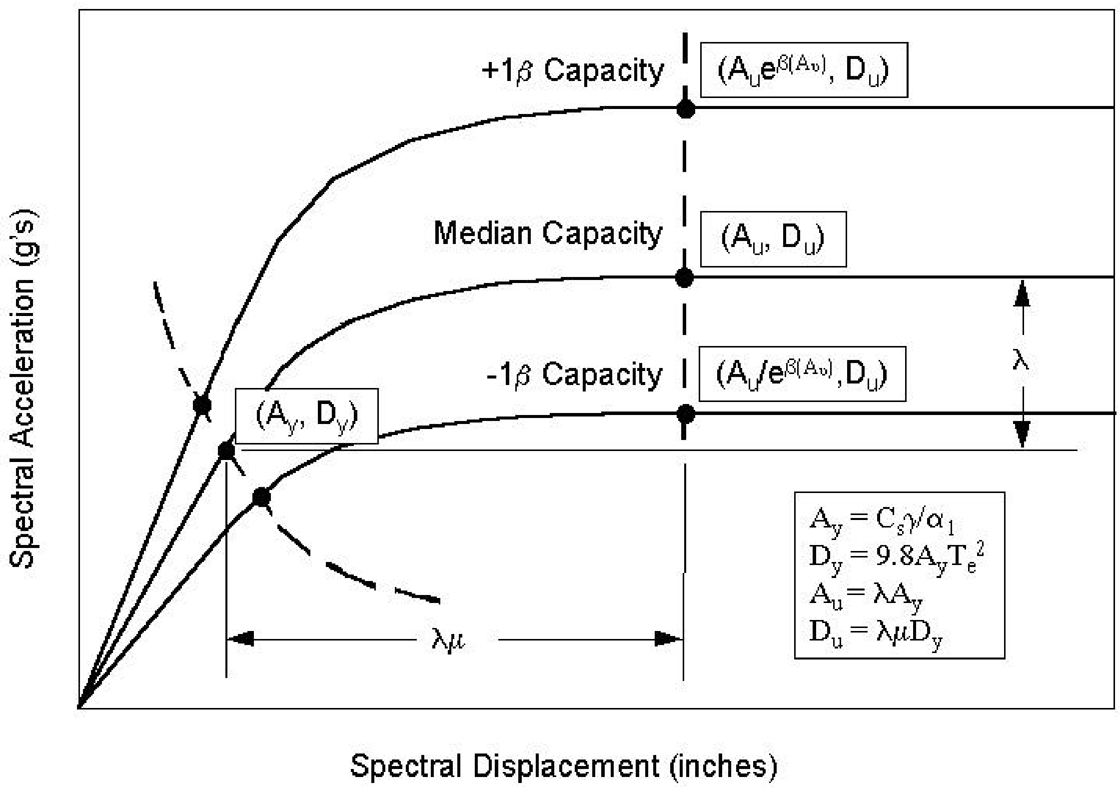


Figure 6.4 Example Construction of Median, +1β and -1β Building Capacity Curves.

Tables 6.3a, 6.3b and 6.3c summarize yield capacity and ultimate capacity control points for Special buildings of High-Code, Moderate-Code and Low-Code seismic design levels, respectively. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 6.3a Special Building Capacity Curves - High-Code Seismic Design Level

Building Type	Yield Capacity Point		Ultimate Capacity Point	
	D_y (in.)	A_y (g)	D_u (in.)	A_u (g)
W1	0.72	0.600	17.27	1.800
W2	0.94	0.600	18.79	1.500
S1L	0.92	0.375	22.00	1.124
S1M	2.66	0.234	42.60	0.702
S1H	6.99	0.147	83.83	0.440
S2L	0.94	0.600	15.03	1.200
S2M	3.64	0.500	38.82	1.000
S2H	11.62	0.381	92.95	0.762
S3	0.94	0.600	15.03	1.200
S4L	0.58	0.480	10.36	1.080
S4M	1.64	0.400	19.65	0.900
S4H	5.23	0.305	47.05	0.685
S5L				
S5M				
S5H				
C1L	0.59	0.375	14.08	1.124
C1M	1.73	0.312	27.65	0.937
C1H	3.02	0.147	36.20	0.440
C2L	0.72	0.600	14.39	1.500
C2M	1.56	0.500	20.76	1.250
C2H	4.41	0.381	44.09	0.952
C3L				
C3M				
C3H				
PC1	1.08	0.900	17.27	1.800
PC2L	0.72	0.600	11.51	1.200
PC2M	1.56	0.500	16.61	1.000
PC2H	4.41	0.381	35.27	0.762
RM1L	0.96	0.800	15.34	1.600
RM1M	2.08	0.667	22.14	1.333
RM2L	0.96	0.800	15.34	1.600
RM2M	2.08	0.667	22.14	1.333
RM2H	5.88	0.508	47.02	1.015
URML				
URMM				
MH	0.27	0.225	4.32	0.450

Table 6.3b Special Building Capacity Curves - Moderate-Code Seismic Design Level

Building Type	Yield Capacity Point		Ultimate Capacity Point	
	D_y (in.)	A_y (g)	D_u (in.)	A_u (g)
W1	0.54	0.450	12.95	1.350
W2	0.47	0.300	9.40	0.750
S1L	0.46	0.187	11.00	0.562
S1M	1.33	0.117	21.30	0.351
S1H	3.49	0.073	41.91	0.220
S2L	0.47	0.300	7.52	0.600
S2M	1.82	0.250	19.41	0.500
S2H	5.81	0.190	46.47	0.381
S3	0.47	0.300	7.52	0.600
S4L	0.29	0.240	5.18	0.540
S4M	0.82	0.200	9.83	0.450
S4H	2.61	0.152	23.53	0.343
S5L				
S5M				
S5H				
C1L	0.29	0.187	7.04	0.562
C1M	0.86	0.156	13.83	0.468
C1H	1.51	0.073	18.10	0.220
C2L	0.36	0.300	7.19	0.750
C2M	0.78	0.250	10.38	0.625
C2H	2.21	0.190	22.05	0.476
C3L				
C3M				
C3H				
PC1	0.54	0.450	8.63	0.900
PC2L	0.36	0.300	5.76	0.600
PC2M	0.78	0.250	8.31	0.500
PC2H	2.21	0.190	17.64	0.381
RM1L	0.48	0.400	7.67	0.800
RM1M	1.04	0.333	11.07	0.667
RM2L	0.48	0.400	7.67	0.800
RM2M	1.04	0.333	11.07	0.667
RM2H	2.94	0.254	23.51	0.508
URML				
URMM				
MH	0.27	0.225	4.32	0.450

Table 6.3c Special Building Capacity Curves - Low-Code Seismic Design Level

Building Type	Yield Capacity Point		Ultimate Capacity Point	
	D_y (in.)	A_y (g)	D_u (in.)	A_u (g)
W1	0.36	0.300	6.48	0.900
W2	0.24	0.150	3.52	0.375
S1L	0.23	0.094	4.13	0.281
S1M	0.67	0.059	7.99	0.176
S1H	1.75	0.037	15.72	0.110
S2L	0.24	0.150	2.82	0.300
S2M	0.91	0.125	7.28	0.250
S2H	2.91	0.095	17.43	0.190
S3	0.24	0.150	2.82	0.300
S4L	0.14	0.120	1.94	0.270
S4M	0.41	0.100	3.69	0.225
S4H	1.31	0.076	8.82	0.171
S5L	0.18	0.150	2.16	0.300
S5M	0.51	0.125	4.09	0.250
S5H	1.63	0.095	9.80	0.190
C1L	0.15	0.094	2.64	0.281
C1M	0.43	0.078	5.19	0.234
C1H	0.75	0.037	6.79	0.110
C2L	0.18	0.150	2.70	0.375
C2M	0.39	0.125	3.89	0.313
C2H	1.10	0.095	8.27	0.238
C3L	0.18	0.150	2.43	0.338
C3M	0.39	0.125	3.50	0.281
C3H	1.10	0.095	7.44	0.214
PC1	0.27	0.225	3.24	0.450
PC2L	0.18	0.150	2.16	0.300
PC2M	0.39	0.125	3.11	0.250
PC2H	1.10	0.095	6.61	0.190
RM1L	0.24	0.200	2.88	0.400
RM1M	0.52	0.167	4.15	0.333
RM2L	0.24	0.200	2.88	0.400
RM2M	0.52	0.167	4.15	0.333
RM2H	1.47	0.127	8.82	0.254
URML	0.36	0.300	4.32	0.600
URMM	0.41	0.167	3.26	0.333
MH	0.27	0.225	4.32	0.450

6.4.3 Fragility Curves - Special Buildings

This section describes Special building fragility curves for Slight, Moderate, Extensive and Complete structural damage states and Slight, Moderate, Extensive and Complete nonstructural damage states. Each fragility curve is characterized by a median and a lognormal standard deviation (β) value of PESH demand. Spectral displacement is the PESH parameter used for structural damage and nonstructural damage to drift-sensitive components. Spectral acceleration is the PESH parameter used for nonstructural damage to acceleration-sensitive components.

Special building fragility curves for ground failure are the same as those of Code buildings (Section 5.5).

6.4.3.1 Background

The form of the fragility curves for Special buildings is the same as that used for Code buildings. The probability of being in, or exceeding, a given damage state is modeled as a cumulative lognormal distribution. Given the appropriate PESH parameter (e.g., spectral displacement, S_d, for structural damage), the probability of being in or exceeding a damage state, ds, is modeled as follows:

$$P\left[ds \mid S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \tag{6-1}$$

where:

	$\bar{S}_{d,ds}$	is the median value of spectral displacement at which the building reaches the threshold of the damage state, ds,
	β_{ds}	is the standard deviation of the natural logarithm of spectral displacement of damage state, ds, and
	Φ	is the standard normal cumulative distribution function.

6.4.3.2 Structural Damage - Special Buildings

Structural damage states for Special buildings are based on drift ratios that are assumed to be slightly higher than those of Code buildings of the same model building type and seismic design level. It is difficult to quantify this improvement in displacement capacity since it is a function not just of building type and design parameters, but also design review and construction inspection. It is assumed that the improvement in displacement capacity results in a 1.25 increase in drift capacity of each damage state for all Special building types and seismic design levels. Special buildings perform better than Code buildings due to increased structure strength (of the capacity curves) and increased displacement capacity (of the fragility curves). In general, increased strength tends to most improve building performance near yield and improved displacement capacity tends to most improve the ultimate capacity of the building.

Median values of Special building structural fragility are based on drift ratios (that describe the threshold of damage states and the height of the building to point of push-over mode displacement using the same approach as that of Code buildings (Section 5.4.3.2).

The variability of Special building structural damage is based on the same approach as that of Code buildings (Section 5.4.3.3). The total variability of each structural damage state, β_{Sds}, is modeled by the combination of following three contributors to damage variability:

- uncertainty in the damage state threshold of the structural system ($\beta_{M(Sds)}$ = 0.4, for all structural damage states and building types)

- variability in capacity (response) properties of the model building type/seismic design level of interest ($\beta_{C(Au)}$ = 0.15 for Special buildings), and

- variability in response due to the spatial variability of ground motion demand ($\beta_{D(A)}$ = 0.45 and $\beta_{C(V)}$ = 0.50).is based on the dispersion factor typical of the attenuation of large-magnitude earthquakes as in the WUS (Chapter 4).

Each of these three contributors to damage state variability are assumed to be lognormally distributed random variables. Capacity and demand are dependent parameters and a convolution process is used to derive combined capacity/demand variability of each structural damage state. Capacity/demand variability is then combined with damage state uncertainty, as described in Section 5.4.3.3.

Tables 6.4a, 6.4b and 6.4c summarize median and lognormal standard deviation (β_{Sds}) values for Slight, Moderate, Extensive and Complete structural damage states of Special buildings for High-Code, Moderate-Code and Low-Code seismic design levels, respectively. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 6.4a Building Structural Fragility - High-Code Seismic Design Level

Building Properties			Interstory Drift at Threshold of Damage State				Spectral Displacement (inches)							
Type	Height (inches)						Slight		Moderate		Extensive		Complete	
	Roof	Modal	Slight	Moderate	Extensive	Complete	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	168	126	0.0050	0.0150	0.0500	0.1250	0.63	0.66	1.89	0.72	6.30	0.72	15.75	0.91
W2	288	216	0.0050	0.0150	0.0500	0.1250	1.08	0.69	3.24	0.77	10.80	0.89	27.00	0.85
S1L	288	216	0.0075	0.0150	0.0375	0.1000	1.62	0.67	3.24	0.70	8.10	0.71	21.60	0.68
S1M	720	540	0.0050	0.0100	0.0250	0.0667	2.70	0.62	5.40	0.62	13.50	0.63	36.00	0.71
S1H	1872	1123	0.0037	0.0075	0.0188	0.0500	4.21	0.63	8.42	0.62	21.06	0.62	56.16	0.63
S2L	288	216	0.0063	0.0125	0.0375	0.1000	1.35	0.69	2.70	0.80	8.10	0.89	21.60	0.84
S2M	720	540	0.0042	0.0083	0.0250	0.0667	2.25	0.62	4.50	0.66	13.50	0.66	36.00	0.71
S2H	1872	1123	0.0031	0.0063	0.0188	0.0500	3.51	0.62	7.02	0.63	21.06	0.63	56.16	0.66
S3	180	135	0.0050	0.0100	0.0300	0.0875	0.68	0.66	1.35	0.71	4.05	0.80	11.81	0.90
S4L	288	216	0.0050	0.0100	0.0300	0.0875	1.08	0.77	2.16	0.82	6.48	0.92	18.90	0.91
S4M	720	540	0.0033	0.0067	0.0200	0.0583	1.80	0.69	3.60	0.67	10.80	0.68	31.50	0.82
S4H	1872	1123	0.0025	0.0050	0.0150	0.0438	2.81	0.62	5.62	0.63	16.85	0.65	49.14	0.73
S5L														
S5M														
S5H														
C1L	240	180	0.0063	0.0125	0.0375	0.1000	1.13	0.69	2.25	0.74	6.75	0.82	18.00	0.81
C1M	600	450	0.0042	0.0083	0.0250	0.0667	1.87	0.63	3.75	0.65	11.25	0.66	30.00	0.71
C1H	1440	864	0.0031	0.0063	0.0188	0.0500	2.70	0.63	5.40	0.63	16.20	0.63	43.20	0.69
C2L	240	180	0.0050	0.0125	0.0375	0.1000	0.90	0.69	2.25	0.72	6.75	0.82	18.00	0.95
C2M	600	450	0.0033	0.0083	0.0250	0.0667	1.50	0.65	3.75	0.69	11.25	0.66	30.00	0.70
C2H	1440	864	0.0025	0.0063	0.0188	0.0500	2.16	0.62	5.40	0.63	16.20	0.64	43.20	0.69
C3L														
C3M														
C3H														
PC1	180	135	0.0050	0.0100	0.0300	0.0875	0.68	0.63	1.35	0.74	4.05	0.79	11.81	0.96
PC2L	240	180	0.0050	0.0100	0.0300	0.0875	0.90	0.76	1.80	0.80	5.40	0.87	15.75	0.97
PC2M	600	450	0.0033	0.0067	0.0200	0.0583	1.50	0.66	3.00	0.73	9.00	0.72	26.25	0.73
PC2H	1440	864	0.0025	0.0050	0.0150	0.0438	2.16	0.62	4.32	0.64	12.96	0.65	37.80	0.74
RM1L	240	180	0.0050	0.0100	0.0300	0.0875	0.90	0.70	1.80	0.74	5.40	0.76	15.75	0.98
RM1M	600	450	0.0033	0.0067	0.0200	0.0583	1.50	0.63	3.00	0.68	9.00	0.70	26.25	0.70
RM2L	240	180	0.0050	0.0100	0.0300	0.0875	0.90	0.66	1.80	0.70	5.40	0.76	15.75	0.97
RM2M	600	450	0.0033	0.0067	0.0200	0.0583	1.50	0.63	3.00	0.70	9.00	0.69	26.25	0.68
RM2H	1440	864	0.0025	0.0050	0.0150	0.0438	2.16	0.63	4.32	0.63	12.96	0.63	37.80	0.65
URML														
URMM														
MH	120	120	0.0050	0.0100	0.0300	0.0875	0.60	0.81	1.20	0.89	3.60	0.97	10.50	0.86

Table 6.4b Building Structural Fragility - Moderate-Code Seismic Design Level

Building Properties			Interstory Drift at Threshold of Damage State				Spectral Displacement (inches)							
Type	Height (inches)						Slight		Moderate		Extensive		Complete	
	Roof	Modal	Slight	Moderate	Extensive	Complete	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	168	126	0.0050	0.0124	0.0383	0.0937	0.63	0.76	1.56	0.77	4.82	0.78	11.81	0.96
W2	288	216	0.0050	0.0124	0.0383	0.0938	1.08	0.79	2.68	0.86	8.27	0.88	20.25	0.84
S1L	288	216	0.0075	0.0130	0.0294	0.0750	1.62	0.73	2.80	0.71	6.35	0.70	16.20	0.77
S1M	720	540	0.0050	0.0086	0.0196	0.0500	2.70	0.64	4.67	0.65	10.58	0.66	27.00	0.75
S1H	1872	1123	0.0037	0.0065	0.0147	0.0375	4.21	0.62	7.29	0.62	16.51	0.66	42.12	0.70
S2L	288	216	0.0063	0.0108	0.0292	0.0750	1.35	0.82	2.34	0.85	6.30	0.89	16.20	0.85
S2M	720	540	0.0042	0.0072	0.0194	0.0500	2.25	0.66	3.90	0.66	10.50	0.68	27.00	0.81
S2H	1872	1123	0.0031	0.0054	0.0146	0.0375	3.51	0.62	6.08	0.63	16.38	0.65	42.12	0.71
S3	180	135	0.0050	0.0087	0.0234	0.0656	0.68	0.77	1.17	0.81	3.16	0.89	8.86	0.89
S4L	288	216	0.0050	0.0087	0.0234	0.0656	1.08	0.88	1.87	0.92	5.05	0.98	14.18	0.87
S4M	720	540	0.0033	0.0058	0.0156	0.0437	1.80	0.70	3.12	0.67	8.41	0.70	23.62	0.90
S4H	1872	1123	0.0025	0.0043	0.0117	0.0328	2.81	0.66	4.87	0.66	13.13	0.70	36.86	0.81
S5L														
S5M														
S5H														
C1L	240	180	0.0063	0.0108	0.0292	0.0750	1.13	0.80	1.95	0.82	5.25	0.84	13.50	0.81
C1M	600	450	0.0042	0.0072	0.0194	0.0500	1.87	0.66	3.25	0.67	8.75	0.66	22.50	0.84
C1H	1440	864	0.0031	0.0054	0.0146	0.0375	2.70	0.64	4.68	0.64	12.60	0.68	32.40	0.81
C2L	240	180	0.0050	0.0105	0.0289	0.0750	0.90	0.77	1.89	0.86	5.21	0.91	13.50	0.89
C2M	600	450	0.0033	0.0070	0.0193	0.0500	1.50	0.71	3.16	0.70	8.68	0.69	22.50	0.83
C2H	1440	864	0.0025	0.0053	0.0145	0.0375	2.16	0.64	4.55	0.65	12.51	0.66	32.40	0.79
C3L														
C3M														
C3H														
PC1	180	135	0.0050	0.0087	0.0234	0.0656	0.68	0.79	1.17	0.81	3.16	0.86	8.86	1.00
PC2L	240	180	0.0050	0.0087	0.0234	0.0656	0.90	0.83	1.56	0.89	4.21	0.97	11.81	0.89
PC2M	600	450	0.0033	0.0058	0.0156	0.0438	1.50	0.76	2.60	0.74	7.01	0.73	19.69	0.88
PC2H	1440	864	0.0025	0.0043	0.0117	0.0328	2.16	0.65	3.75	0.66	10.10	0.70	28.35	0.81
RM1L	240	180	0.0050	0.0087	0.0234	0.0656	0.90	0.80	1.56	0.85	4.21	0.92	11.81	0.97
RM1M	600	450	0.0033	0.0058	0.0156	0.0438	1.50	0.73	2.60	0.75	7.01	0.75	19.69	0.80
RM2L	240	180	0.0050	0.0087	0.0234	0.0656	0.90	0.77	1.56	0.81	4.21	0.92	11.81	0.96
RM2M	600	450	0.0033	0.0058	0.0156	0.0438	1.50	0.72	2.60	0.72	7.01	0.72	19.69	0.77
RM2H	1440	864	0.0025	0.0043	0.0117	0.0328	2.16	0.63	3.75	0.65	10.10	0.66	28.35	0.76
URML														
URMM														
MH	120	120	0.0050	0.0100	0.0300	0.0875	0.60	0.81	1.20	0.89	3.60	0.97	10.50	0.86

Table 6.4c Special Building Structural Fragility - Low-Code Seismic Design Level

Building Properties			Interstory Drift at Threshold of Damage State				Spectral Displacement (inches)							
Type	Height (inches)						Slight		Moderate		Extensive		Complete	
	Roof	Modal	Slight	Moderate	Extensive	Complete	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	168	126	0.0050	0.0124	0.0383	0.0937	0.63	0.80	1.56	0.81	4.82	0.88	11.81	1.01
W2	288	216	0.0050	0.0124	0.0383	0.0938	1.08	0.89	2.68	0.89	8.27	0.86	20.25	0.97
S1L	288	216	0.0075	0.0119	0.0253	0.0625	1.62	0.73	2.58	0.73	5.47	0.75	13.50	0.93
S1M	720	540	0.0050	0.0080	0.0169	0.0417	2.70	0.66	4.30	0.70	9.12	0.78	22.50	0.91
S1H	1872	1123	0.0037	0.0060	0.0127	0.0313	4.21	0.64	6.72	0.66	14.23	0.68	35.10	0.86
S2L	288	216	0.0063	0.0100	0.0250	0.0625	1.35	0.89	2.16	0.89	5.40	0.88	13.50	0.97
S2M	720	540	0.0042	0.0067	0.0167	0.0417	2.25	0.67	3.60	0.68	9.00	0.74	22.50	0.92
S2H	1872	1123	0.0031	0.0050	0.0125	0.0313	3.51	0.62	5.62	0.63	14.04	0.68	35.10	0.84
S3	180	135	0.0050	0.0080	0.0201	0.0547	0.68	0.89	1.08	0.90	2.71	0.98	7.38	0.85
S4L	288	216	0.0050	0.0080	0.0200	0.0547	1.08	0.98	1.73	0.95	4.33	0.97	11.81	0.98
S4M	720	540	0.0033	0.0053	0.0134	0.0364	1.80	0.69	2.88	0.72	7.22	0.81	19.68	0.98
S4H	1872	1123	0.0025	0.0040	0.0100	0.0273	2.81	0.66	4.50	0.67	11.26	0.78	30.71	0.93
S5L	288	216	0.0038	0.0075	0.0188	0.0438	0.81	1.00	1.62	1.00	4.05	1.03	9.45	0.91
S5M	720	540	0.0025	0.0050	0.0125	0.0292	1.35	0.74	2.70	0.72	6.75	0.78	15.75	0.94
S5H	1872	1123	0.0019	0.0037	0.0094	0.0219	2.11	0.67	4.21	0.69	10.53	0.74	24.57	0.90
C1L	240	180	0.0063	0.0100	0.0250	0.0625	1.13	0.85	1.80	0.85	4.50	0.88	11.25	0.95
C1M	600	450	0.0042	0.0067	0.0167	0.0417	1.87	0.70	3.00	0.69	7.50	0.75	18.75	0.95
C1H	1440	864	0.0031	0.0050	0.0125	0.0313	2.70	0.66	4.32	0.71	10.80	0.79	27.00	0.95
C2L	240	180	0.0050	0.0096	0.0247	0.0625	0.90	0.91	1.72	0.94	4.44	1.01	11.25	0.90
C2M	600	450	0.0033	0.0064	0.0164	0.0417	1.50	0.76	2.86	0.74	7.40	0.74	18.75	0.94
C2H	1440	864	0.0025	0.0048	0.0123	0.0313	2.16	0.66	4.12	0.67	10.66	0.74	27.00	0.91
C3L	240	180	0.0038	0.0075	0.0188	0.0438	0.68	0.92	1.35	0.99	3.38	1.04	7.88	0.88
C3M	600	450	0.0025	0.0050	0.0125	0.0292	1.12	0.77	2.25	0.79	5.62	0.78	13.12	0.93
C3H	1440	864	0.0019	0.0038	0.0094	0.0219	1.62	0.68	3.24	0.69	8.10	0.70	18.90	0.88
PC1	180	135	0.0050	0.0080	0.0201	0.0547	0.68	0.89	1.08	0.95	2.71	1.00	7.38	0.96
PC2L	240	180	0.0050	0.0080	0.0201	0.0547	0.90	0.98	1.44	0.98	3.61	1.02	9.84	0.91
PC2M	600	450	0.0033	0.0053	0.0134	0.0364	1.50	0.76	2.40	0.75	6.02	0.75	16.40	0.94
PC2H	1440	864	0.0025	0.0040	0.0100	0.0273	2.16	0.66	3.46	0.68	8.66	0.73	23.63	0.92
RM1L	240	180	0.0050	0.0080	0.0201	0.0547	0.90	0.97	1.44	1.01	3.61	1.07	9.84	0.88
RM1M	600	450	0.0033	0.0053	0.0134	0.0364	1.50	0.78	2.40	0.78	6.02	0.78	16.40	0.94
RM2L	240	180	0.0050	0.0080	0.0201	0.0547	0.90	0.94	1.44	0.98	3.61	1.05	9.84	0.89
RM2M	600	450	0.0033	0.0053	0.0134	0.0364	1.50	0.76	2.40	0.75	6.02	0.75	16.40	0.92
RM2H	1440	864	0.0025	0.0040	0.0100	0.0273	2.16	0.66	3.46	0.67	8.66	0.80	23.63	0.89
URML	180	135	0.0038	0.0075	0.0187	0.0438	0.51	0.89	1.01	0.91	2.53	0.96	5.91	1.09
URMM	420	315	0.0025	0.0050	0.0125	0.0292	0.79	0.81	1.57	0.84	3.94	0.87	9.19	0.82
MH	120	120	0.0050	0.0100	0.0300	0.0875	0.60	0.81	1.20	0.89	3.60	0.97	10.50	0.86

6.4.3.3 Nonstructural Damage - Drift-Sensitive

Damage states of nonstructural drift-sensitive components of Special buildings are based on the same drift ratios as those of Code buildings (Table 5.10). Even for essential facilities, nonstructural components are typically not designed or detailed for special earthquake displacements. Improvement in the performance of drift-sensitive components of Special buildings is assumed to be entirely a function of drift reduction due to the increased stiffness and strength of the structures of these buildings.

Median values of drift-sensitive nonstructural fragility curves are based on global building displacement (in inches), calculated as the product of: (1) drift ratio, (2) building height and (3) the fraction of building height at the location of push-over mode displacement (α_2).

The total variability of each nonstructural drift-sensitive damage state, β_{NSDds}, is modeled by the combination of following three contributors to damage variability:

- uncertainty in the damage state threshold of nonstructural components ($\beta_{M(NSDds)} = 0.5$, for all structural damage states and building types,

- variability in capacity (response) properties of the model building type that contains the nonstructural components of interest ($\beta_{C(Au)} = 0.15$ for Special buildings, and

- variability in response of the model building type due to the spatial variability of ground motion demand ($\beta_{D(A)} = 0.45$ and $\beta_{C(V)} = 0.50$).

Each of these three contributors to damage state variability are assumed to be lognormally distributed random variables. Capacity and demand are dependent parameters and a convolution process is used to derive combined capacity/demand variability of each nonstructural damage state. Capacity/demand variability is then combined with damage state uncertainty, as described in Section 5.4.3.3.

Tables 6.5a, 6.5b and 6.5c summarize median and lognormal standard deviation (β_{NSDds}) values for Slight, Moderate, Extensive and Complete damage states of nonstructural drift-sensitive components of Special buildings for High-Code, Moderate-Code and Low-Code seismic design levels, respectively. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 6.5a Special Building Nonstructural Drift-Sensitive Fragility - High-Code Seismic Design Level

Building Type	Median Spectral Displacement (inches) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.50	0.74	1.01	0.77	3.15	0.79	6.30	0.78
W2	0.86	0.76	1.73	0.77	5.40	0.88	10.80	0.93
S1L	0.86	0.72	1.73	0.76	5.40	0.75	10.80	0.74
S1M	2.16	0.68	4.32	0.68	13.50	0.70	27.00	0.73
S1H	4.49	0.70	8.99	0.69	28.08	0.69	56.16	0.70
S2L	0.86	0.74	1.73	0.77	5.40	0.90	10.80	0.95
S2M	2.16	0.70	4.32	0.72	13.50	0.73	27.00	0.72
S2H	4.49	0.71	8.99	0.69	28.08	0.70	56.16	0.73
S3	0.54	0.70	1.08	0.76	3.38	0.83	6.75	0.93
S4L	0.86	0.81	1.73	0.84	5.40	0.93	10.80	1.00
S4M	2.16	0.76	4.32	0.74	13.50	0.75	27.00	0.82
S4H	4.49	0.70	8.99	0.71	28.08	0.72	56.16	0.80
S5L								
S5M								
S5H								
C1L	0.72	0.77	1.44	0.76	4.50	0.84	9.00	0.88
C1M	1.80	0.71	3.60	0.71	11.25	0.72	22.50	0.71
C1H	3.46	0.70	6.91	0.69	21.60	0.71	43.20	0.75
C2L	0.72	0.76	1.44	0.76	4.50	0.80	9.00	0.94
C2M	1.80	0.74	3.60	0.76	11.25	0.73	22.50	0.74
C2H	3.46	0.69	6.91	0.69	21.60	0.71	43.20	0.75
C3L								
C3M								
C3H								
PC1	0.54	0.69	1.08	0.78	3.38	0.85	6.75	0.88
PC2L	0.72	0.80	1.44	0.83	4.50	0.90	9.00	1.03
PC2M	1.80	0.75	3.60	0.80	11.25	0.77	22.50	0.77
PC2H	3.46	0.70	6.91	0.71	21.60	0.73	43.20	0.82
RM1L	0.72	0.74	1.44	0.80	4.50	0.80	9.00	0.94
RM1M	1.80	0.70	3.60	0.77	11.25	0.77	22.50	0.77
RM2L	0.72	0.74	1.44	0.76	4.50	0.78	9.00	0.96
RM2M	1.80	0.71	3.60	0.78	11.25	0.74	22.50	0.74
RM2H	3.46	0.69	6.91	0.69	21.60	0.71	43.20	0.74
URML								
URMM								
MH	0.48	0.85	0.96	0.92	3.00	0.98	6.00	0.99

Table 6.5b Special Building Nonstructural Drift-Sensitive Fragility - Moderate-Code Seismic Design Level

Building Type	Median Spectral Displacement (inches) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.50	0.77	1.01	0.82	3.15	0.84	6.30	0.87
W2	0.86	0.84	1.73	0.88	5.40	0.93	10.80	0.93
S1L	0.86	0.78	1.73	0.78	5.40	0.78	10.80	0.76
S1M	2.16	0.71	4.32	0.71	13.50	0.73	27.00	0.81
S1H	4.49	0.69	8.99	0.69	28.08	0.72	56.16	0.82
S2L	0.86	0.81	1.73	0.91	5.40	0.96	10.80	0.89
S2M	2.16	0.73	4.32	0.74	13.50	0.73	27.00	0.87
S2H	4.49	0.69	8.99	0.70	28.08	0.74	56.16	0.84
S3	0.54	0.82	1.08	0.86	3.38	0.97	6.75	0.95
S4L	0.86	0.89	1.73	0.97	5.40	1.02	10.80	0.94
S4M	2.16	0.76	4.32	0.74	13.50	0.84	27.00	0.97
S4H	4.49	0.71	8.99	0.73	28.08	0.83	56.16	0.94
S5L								
S5M								
S5H								
C1L	0.72	0.80	1.44	0.86	4.50	0.88	9.00	0.88
C1M	1.80	0.73	3.60	0.72	11.25	0.74	22.50	0.89
C1H	3.46	0.71	6.91	0.71	21.60	0.79	43.20	0.93
C2L	0.72	0.84	1.44	0.87	4.50	0.95	9.00	1.00
C2M	1.80	0.79	3.60	0.76	11.25	0.76	22.50	0.88
C2H	3.46	0.70	6.91	0.71	21.60	0.77	43.20	0.87
C3L								
C3M								
C3H								
PC1	0.54	0.82	1.08	0.87	3.38	0.93	6.75	1.02
PC2L	0.72	0.88	1.44	0.95	4.50	1.03	9.00	0.99
PC2M	1.80	0.84	3.60	0.77	11.25	0.79	22.50	0.95
PC2H	3.46	0.72	6.91	0.74	21.60	0.84	43.20	0.94
RM1L	0.72	0.86	1.44	0.88	4.50	0.99	9.00	1.04
RM1M	1.80	0.80	3.60	0.79	11.25	0.79	22.50	0.88
RM2L	0.72	0.81	1.44	0.86	4.50	0.97	9.00	1.03
RM2M	1.80	0.78	3.60	0.77	11.25	0.77	22.50	0.88
RM2H	3.46	0.71	6.91	0.71	21.60	0.74	43.20	0.87
URML								
URMM								
MH	0.48	0.85	0.96	0.92	3.00	0.98	6.00	0.99

Table 6.5c Special Building Nonstructural Drift-Sensitive Fragility - Low-Code Seismic Design Level

Building Type	Median Spectral Displacement (inches) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.50	0.83	1.01	0.86	3.15	0.88	6.30	1.00
W2	0.86	0.93	1.73	0.94	5.40	0.99	10.80	0.93
S1L	0.86	0.81	1.73	0.80	5.40	0.80	10.80	0.94
S1M	2.16	0.73	4.32	0.76	13.50	0.86	27.00	0.98
S1H	4.49	0.71	8.99	0.74	28.08	0.87	56.16	0.98
S2L	0.86	0.94	1.73	0.93	5.40	0.93	10.80	0.98
S2M	2.16	0.73	4.32	0.76	13.50	0.91	27.00	0.99
S2H	4.49	0.71	8.99	0.74	28.08	0.85	56.16	0.96
S3	0.54	0.89	1.08	0.96	3.38	1.01	6.75	0.90
S4L	0.86	1.02	1.73	0.99	5.40	0.95	10.80	1.01
S4M	2.16	0.76	4.32	0.84	13.50	0.95	27.00	1.04
S4H	4.49	0.74	8.99	0.87	28.08	0.96	56.16	1.03
S5L	0.86	1.04	1.73	1.04	5.40	1.00	10.80	0.99
S5M	2.16	0.78	4.32	0.84	13.50	0.97	27.00	1.04
S5H	4.49	0.76	8.99	0.87	28.08	0.96	56.16	1.03
C1L	0.72	0.90	1.44	0.92	4.50	0.93	9.00	0.93
C1M	1.80	0.74	3.60	0.77	11.25	0.94	22.50	1.00
C1H	3.46	0.75	6.91	0.86	21.60	0.97	43.20	1.03
C2L	0.72	0.93	1.44	0.99	4.50	1.06	9.00	0.92
C2M	1.80	0.80	3.60	0.80	11.25	0.91	22.50	1.00
C2H	3.46	0.73	6.91	0.80	21.60	0.93	43.20	1.01
C3L	0.72	0.99	1.44	1.05	4.50	1.06	9.00	0.93
C3M	1.80	0.84	3.60	0.83	11.25	0.95	22.50	1.01
C3H	3.46	0.76	6.91	0.84	21.60	0.96	43.20	1.03
PC1	0.54	0.92	1.08	0.99	3.38	1.07	6.75	1.02
PC2L	0.72	0.99	1.44	1.02	4.50	1.02	9.00	0.95
PC2M	1.80	0.81	3.60	0.82	11.25	0.95	22.50	1.02
PC2H	3.46	0.74	6.91	0.86	21.60	0.96	43.20	1.02
RM1L	0.72	0.98	1.44	1.06	4.50	1.08	9.00	0.94
RM1M	1.80	0.83	3.60	0.84	11.25	0.91	22.50	0.99
RM2L	0.72	0.94	1.44	1.03	4.50	1.07	9.00	0.92
RM2M	1.80	0.81	3.60	0.80	11.25	0.91	22.50	0.99
RM2H	3.46	0.74	6.91	0.79	21.60	0.92	43.20	1.01
URML	0.54	0.93	1.08	0.98	3.38	1.05	6.75	1.11
URMM	1.26	0.89	2.52	0.88	7.88	0.87	15.75	0.99
MH	0.48	0.85	0.96	0.92	3.00	0.98	6.00	0.99

6.4.3.4 Nonstructural Damage - Acceleration-Sensitive Components

Damage states of nonstructural acceleration-sensitive components of Special buildings are based on the peak floor accelerations of Code buildings of seismic design level (Table 5.12) increased by a factor of 1.5. A factor of 1.5 on damage-state acceleration reflects increased anchorage strength of nonstructural acceleration-sensitive components of Special buildings.

The floor acceleration values are used directly as median values, assuming average upper-floor demand is represented by response at the point of the push-over mode displacement.

The total variability of each damage state, β_{NSAds}, is modeled by the combination of following three contributors to nonstructural acceleration-sensitive damage variability:

- uncertainty in the damage state threshold of nonstructural components ($\beta_{M(NSAds)}$ = 0.6, for all structural damage states and building types,

- variability in capacity (response) properties of the model building type that contains the nonstructural components of interest ($\beta_{C(Au)}$ = 0.15 for Special buildings, and

- variability in response of the model building type due to the spatial variability of ground motion demand ($\beta_{D(A)}$ = 0.45 and $\beta_{C(V)}$ = 0.50).

Each of these three contributors to damage state variability are assumed to be lognormally distributed random variables. Capacity and demand are dependent parameters and a convolution process is used to derive combined capacity/demand variability of each nonstructural damage state. Capacity/demand variability is then combined with damage state uncertainty, as described in Section 5.4.3.3.

Tables 6.6a, 6.6b and 6.6c summarize median and lognormal standard deviation (β_{NSAds}) values for Slight, Moderate, Extensive and Complete damage states of nonstructural acceleration-sensitive components of Special buildings for High-Code, Moderate-Code and Low-Code seismic design levels, respectively.

Table 6.6a Special Building Nonstructural Acceleration-Sensitive Fragility - High-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.45	0.72	0.90	0.68	1.80	0.68	3.60	0.68
W2	0.45	0.69	0.90	0.67	1.80	0.68	3.60	0.68
S1L	0.45	0.66	0.90	0.67	1.80	0.67	3.60	0.67
S1M	0.45	0.66	0.90	0.67	1.80	0.68	3.60	0.68
S1H	0.45	0.67	0.90	0.66	1.80	0.66	3.60	0.66
S2L	0.45	0.66	0.90	0.67	1.80	0.66	3.60	0.66
S2M	0.45	0.68	0.90	0.65	1.80	0.65	3.60	0.65
S2H	0.45	0.67	0.90	0.65	1.80	0.65	3.60	0.65
S3	0.45	0.68	0.90	0.67	1.80	0.66	3.60	0.66
S4L	0.45	0.67	0.90	0.67	1.80	0.67	3.60	0.67
S4M	0.45	0.66	0.90	0.65	1.80	0.66	3.60	0.66
S4H	0.45	0.66	0.90	0.65	1.80	0.63	3.60	0.63
S5L								
S5M								
S5H								
C1L	0.45	0.67	0.90	0.68	1.80	0.67	3.60	0.67
C1M	0.45	0.66	0.90	0.66	1.80	0.66	3.60	0.66
C1H	0.45	0.67	0.90	0.65	1.80	0.65	3.60	0.65
C2L	0.45	0.68	0.90	0.67	1.80	0.67	3.60	0.63
C2M	0.45	0.68	0.90	0.65	1.80	0.64	3.60	0.64
C2H	0.45	0.68	0.90	0.65	1.80	0.64	3.60	0.64
C3L								
C3M								
C3H								
PC1	0.45	0.72	0.90	0.66	1.80	0.67	3.60	0.63
PC2L	0.45	0.68	0.90	0.67	1.80	0.66	3.60	0.66
PC2M	0.45	0.67	0.90	0.64	1.80	0.65	3.60	0.65
PC2H	0.45	0.66	0.90	0.64	1.80	0.63	3.60	0.63
RM1L	0.45	0.73	0.90	0.66	1.80	0.68	3.60	0.64
RM1M	0.45	0.69	0.90	0.65	1.80	0.64	3.60	0.64
RM2L	0.45	0.71	0.90	0.66	1.80	0.67	3.60	0.63
RM2M	0.45	0.70	0.90	0.65	1.80	0.64	3.60	0.64
RM2H	0.45	0.69	0.90	0.65	1.80	0.64	3.60	0.64
URML								
URMM								
MH	0.38	0.66	0.75	0.67	1.50	0.67	3.00	0.67

Table 6.6b Special Building Nonstructural Acceleration-Sensitive Fragility - Moderate-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.38	0.71	0.75	0.68	1.50	0.68	3.00	0.65
W2	0.38	0.67	0.75	0.68	1.50	0.68	3.00	0.68
S1L	0.38	0.67	0.75	0.67	1.50	0.68	3.00	0.68
S1M	0.38	0.67	0.75	0.67	1.50	0.67	3.00	0.67
S1H	0.38	0.67	0.75	0.66	1.50	0.66	3.00	0.66
S2L	0.38	0.66	0.75	0.66	1.50	0.68	3.00	0.68
S2M	0.38	0.65	0.75	0.65	1.50	0.64	3.00	0.64
S2H	0.38	0.65	0.75	0.65	1.50	0.65	3.00	0.65
S3	0.38	0.66	0.75	0.66	1.50	0.66	3.00	0.66
S4L	0.38	0.67	0.75	0.66	1.50	0.65	3.00	0.65
S4M	0.38	0.65	0.75	0.65	1.50	0.65	3.00	0.65
S4H	0.38	0.65	0.75	0.65	1.50	0.65	3.00	0.65
S5L								
S5M								
S5H								
C1L	0.38	0.68	0.75	0.66	1.50	0.68	3.00	0.68
C1M	0.38	0.66	0.75	0.65	1.50	0.65	3.00	0.65
C1H	0.38	0.65	0.75	0.65	1.50	0.65	3.00	0.65
C2L	0.38	0.67	0.75	0.67	1.50	0.67	3.00	0.67
C2M	0.38	0.65	0.75	0.64	1.50	0.66	3.00	0.66
C2H	0.38	0.65	0.75	0.64	1.50	0.64	3.00	0.64
C3L								
C3M								
C3H								
PC1	0.38	0.67	0.75	0.67	1.50	0.65	3.00	0.65
PC2L	0.38	0.66	0.75	0.66	1.50	0.64	3.00	0.64
PC2M	0.38	0.64	0.75	0.64	1.50	0.64	3.00	0.64
PC2H	0.38	0.64	0.75	0.65	1.50	0.65	3.00	0.65
RM1L	0.38	0.67	0.75	0.67	1.50	0.67	3.00	0.67
RM1M	0.38	0.65	0.75	0.64	1.50	0.66	3.00	0.66
RM2L	0.38	0.67	0.75	0.67	1.50	0.67	3.00	0.67
RM2M	0.38	0.65	0.75	0.64	1.50	0.66	3.00	0.66
RM2H	0.38	0.65	0.75	0.64	1.50	0.64	3.00	0.64
URML								
URMM								
MH	0.38	0.66	0.75	0.67	1.50	0.67	3.00	0.67

Table 6.6c Special Building Nonstructural Acceleration-Sensitive Fragility - Low-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.30	0.71	0.60	0.68	1.20	0.66	2.40	0.65
W2	0.30	0.66	0.60	0.66	1.20	0.69	2.40	0.69
S1L	0.30	0.66	0.60	0.68	1.20	0.68	2.40	0.68
S1M	0.30	0.66	0.60	0.68	1.20	0.68	2.40	0.68
S1H	0.30	0.67	0.60	0.67	1.20	0.67	2.40	0.67
S2L	0.30	0.65	0.60	0.68	1.20	0.68	2.40	0.68
S2M	0.30	0.65	0.60	0.67	1.20	0.67	2.40	0.67
S2H	0.30	0.64	0.60	0.67	1.20	0.67	2.40	0.67
S3	0.30	0.65	0.60	0.67	1.20	0.67	2.40	0.67
S4L	0.30	0.65	0.60	0.68	1.20	0.68	2.40	0.68
S4M	0.30	0.64	0.60	0.68	1.20	0.68	2.40	0.68
S4H	0.30	0.64	0.60	0.67	1.20	0.67	2.40	0.67
S5L	0.30	0.65	0.60	0.68	1.20	0.68	2.40	0.68
S5M	0.30	0.64	0.60	0.67	1.20	0.67	2.40	0.67
S5H	0.30	0.64	0.60	0.67	1.20	0.67	2.40	0.67
C1L	0.30	0.65	0.60	0.68	1.20	0.68	2.40	0.68
C1M	0.30	0.64	0.60	0.67	1.20	0.67	2.40	0.67
C1H	0.30	0.67	0.60	0.67	1.20	0.67	2.40	0.67
C2L	0.30	0.66	0.60	0.66	1.20	0.65	2.40	0.65
C2M	0.30	0.63	0.60	0.65	1.20	0.65	2.40	0.65
C2H	0.30	0.63	0.60	0.66	1.20	0.66	2.40	0.66
C3L	0.30	0.65	0.60	0.67	1.20	0.67	2.40	0.67
C3M	0.30	0.63	0.60	0.66	1.20	0.66	2.40	0.66
C3H	0.30	0.63	0.60	0.67	1.20	0.67	2.40	0.67
PC1	0.30	0.66	0.60	0.65	1.20	0.65	2.40	0.65
PC2L	0.30	0.65	0.60	0.68	1.20	0.68	2.40	0.68
PC2M	0.30	0.63	0.60	0.67	1.20	0.67	2.40	0.67
PC2H	0.30	0.64	0.60	0.66	1.20	0.66	2.40	0.66
RM1L	0.30	0.66	0.60	0.66	1.20	0.65	2.40	0.65
RM1M	0.30	0.64	0.60	0.65	1.20	0.65	2.40	0.65
RM2L	0.30	0.66	0.60	0.66	1.20	0.66	2.40	0.66
RM2M	0.30	0.64	0.60	0.65	1.20	0.65	2.40	0.65
RM2H	0.30	0.63	0.60	0.65	1.20	0.65	2.40	0.65
URML	0.30	0.68	0.60	0.66	1.20	0.64	2.40	0.64
URMM	0.30	0.64	0.60	0.65	1.20	0.65	2.40	0.65
MH	0.38	0.66	0.75	0.67	1.50	0.67	3.00	0.67

6.4.4 Structural Fragility Curves - Equivalent Peak Ground Acceleration

Structural damage fragility curves are expressed in terms of an equivalent value of PGA (rather than spectral displacement) for evaluation of Special buildings that are components of lifelines. Only structural damage functions are developed based on PGA, since structural damage is considered the most appropriate measure of damage for lifeline facilities. Similar methods could be used to develop nonstructural damage functions based on PGA. In this case, capacity curves are not necessary to estimate building response and PGA is used directly as the PESH input to building fragility curves.

This section provides equivalent-PGA fragility curves for Special buildings based on the structural damage functions of Tables 6.4a - 6.4c and standard spectrum shape properties of Chapter 4. These functions have the same format and are based on the same approach and assumptions as those described in Section 5.4.4 for development equivalent-PGA fragility curves for Code buildings.

The values given in Tables 6.7a through 6.7c are appropriate for use in the evaluation of scenario earthquakes whose demand spectrum shape is based on, or similar to, large-magnitude, WUS ground shaking at soil sites (reference spectrum shape). For evaluation of building damage due to scenario earthquakes whose spectra are not similar to the reference spectrum shape, damage-state median parameters may be adjusted to better represent equivalent-PGA structural fragility for the spectrum shape of interest.

Median values of equivalent-PGA are adjusted for: (1) the site condition (if different from Site Class D) and (2) the ratio of long-period spectral response (i.e., S_{A1}) to PGA (if different from a value of 1.5, the ratio of S_{A1} to PGA of the reference spectrum shape). Damage-state variability is not adjusted assuming that the variability associated with ground shaking (although different for different source/site conditions) when combined with the uncertainty in damage-state threshold, is approximately the same for all demand spectrum shapes.

Equivalent-PGA medians, given in Tables 6.7a through 6.7c for the reference spectrum shape, are adjusted to represent other spectrum shapes using the spectrum shape ratios of Tables 5.14 and 5.15, the soil amplification factor, F_V, and Equation (5-6). In general, implementation of Equation (5-6) requires information on earthquake magnitude and source-to-site distance to estimate the spectrum shape ratio for rock sites, and 1-second period spectral acceleration at the site (to estimate the soil amplification factor). Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

Table 6.7a Equivalent-PGA Structural Fragility - Special High-Code Seismic Design Level

Building Type	Median Equivalent-PGA (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.32	0.64	0.78	0.64	2.00	0.64	3.22	0.64
W2	0.35	0.64	0.82	0.64	1.76	0.64	3.13	0.64
S1L	0.25	0.64	0.44	0.64	0.92	0.64	2.17	0.64
S1M	0.17	0.64	0.34	0.64	0.85	0.64	2.10	0.64
S1H	0.13	0.64	0.26	0.64	0.65	0.64	1.73	0.64
S2L	0.33	0.64	0.58	0.64	1.10	0.64	2.07	0.64
S2M	0.18	0.64	0.35	0.64	0.97	0.64	2.34	0.64
S2H	0.14	0.64	0.27	0.64	0.81	0.64	2.13	0.64
S3	0.19	0.64	0.36	0.64	0.79	0.64	1.44	0.64
S4L	0.34	0.64	0.54	0.64	1.04	0.64	1.91	0.64
S4M	0.21	0.64	0.37	0.64	0.98	0.64	2.27	0.64
S4H	0.16	0.64	0.32	0.64	0.90	0.64	2.29	0.64
S5L								
S5M								
S5H								
C1L	0.29	0.64	0.51	0.64	1.07	0.64	2.06	0.64
C1M	0.19	0.64	0.36	0.64	1.02	0.64	2.48	0.64
C1H	0.14	0.64	0.28	0.64	0.83	0.64	2.03	0.64
C2L	0.33	0.64	0.66	0.64	1.42	0.64	2.40	0.64
C2M	0.22	0.64	0.49	0.64	1.24	0.64	2.97	0.64
C2H	0.15	0.64	0.37	0.64	1.11	0.64	2.80	0.64
C3L								
C3M								
C3H								
PC1	0.25	0.64	0.48	0.64	1.02	0.64	1.86	0.64
PC2L	0.32	0.64	0.51	0.64	1.03	0.64	1.78	0.64
PC2M	0.22	0.64	0.40	0.64	0.92	0.64	2.25	0.64
PC2H	0.15	0.64	0.30	0.64	0.83	0.64	2.13	0.64
RM1L	0.39	0.64	0.65	0.64	1.52	0.64	2.53	0.64
RM1M	0.25	0.64	0.50	0.64	1.15	0.64	2.76	0.64
RM2L	0.34	0.64	0.59	0.64	1.41	0.64	2.36	0.64
RM2M	0.22	0.64	0.43	0.64	1.05	0.64	2.65	0.64
RM2H	0.15	0.64	0.30	0.64	0.89	0.64	2.58	0.64
URML								
URMM								
MH	0.16	0.64	0.26	0.64	0.45	0.64	0.88	0.64

Table 6.7b Equivalent-PGA Structural Fragility - Special Moderate-Code Seismic Design Level

Building Type	Median Equivalent-PGA (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.32	0.64	0.59	0.64	1.32	0.64	2.08	0.64
W2	0.28	0.64	0.51	0.64	1.00	0.64	1.83	0.64
S1L	0.20	0.64	0.31	0.64	0.60	0.64	1.29	0.64
S1M	0.16	0.64	0.28	0.64	0.60	0.64	1.27	0.64
S1H	0.13	0.64	0.22	0.64	0.51	0.64	1.17	0.64
S2L	0.27	0.64	0.37	0.64	0.67	0.64	1.27	0.64
S2M	0.17	0.64	0.28	0.64	0.69	0.64	1.40	0.64
S2H	0.14	0.64	0.23	0.64	0.63	0.64	1.44	0.64
S3	0.18	0.64	0.26	0.64	0.46	0.64	0.86	0.64
S4L	0.26	0.64	0.36	0.64	0.61	0.64	1.17	0.64
S4M	0.18	0.64	0.29	0.64	0.69	0.64	1.33	0.64
S4H	0.16	0.64	0.26	0.64	0.66	0.64	1.42	0.64
S5L								
S5M								
S5H								
C1L	0.23	0.64	0.33	0.64	0.63	0.64	1.22	0.64
C1M	0.17	0.64	0.28	0.64	0.70	0.64	1.38	0.64
C1H	0.14	0.64	0.23	0.64	0.59	0.64	1.15	0.64
C2L	0.26	0.64	0.44	0.64	0.77	0.64	1.34	0.64
C2M	0.20	0.64	0.35	0.64	0.81	0.64	1.63	0.64
C2H	0.15	0.64	0.30	0.64	0.78	0.64	1.63	0.64
C3L								
C3M								
C3H								
PC1	0.24	0.64	0.33	0.64	0.63	0.64	1.05	0.64
PC2L	0.24	0.64	0.35	0.64	0.59	0.64	1.06	0.64
PC2M	0.19	0.64	0.29	0.64	0.62	0.64	1.27	0.64
PC2H	0.15	0.64	0.25	0.64	0.60	0.64	1.30	0.64
RM1L	0.31	0.64	0.44	0.64	0.79	0.64	1.33	0.64
RM1M	0.24	0.64	0.36	0.64	0.74	0.64	1.65	0.64
RM2L	0.28	0.64	0.41	0.64	0.74	0.64	1.27	0.64
RM2M	0.21	0.64	0.32	0.64	0.69	0.64	1.58	0.64
RM2H	0.15	0.64	0.25	0.64	0.64	0.64	1.53	0.64
URML								
URMM								
MH	0.16	0.64	0.26	0.64	0.45	0.64	0.88	0.64

Table 6.7c Equivalent-PGA Structural Fragility - Special Low-Code Seismic Design Level

Building Type	Median Equivalent-PGA (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.28	0.64	0.50	0.64	1.00	0.64	1.51	0.64
W2	0.21	0.64	0.34	0.64	0.68	0.64	1.10	0.64
S1L	0.16	0.64	0.23	0.64	0.42	0.64	0.71	0.64
S1M	0.15	0.64	0.23	0.64	0.42	0.64	0.73	0.64
S1H	0.13	0.64	0.20	0.64	0.40	0.64	0.71	0.64
S2L	0.19	0.64	0.25	0.64	0.44	0.64	0.74	0.64
S2M	0.16	0.64	0.24	0.64	0.52	0.64	0.88	0.64
S2H	0.14	0.64	0.21	0.64	0.50	0.64	0.93	0.64
S3	0.14	0.64	0.18	0.64	0.30	0.64	0.57	0.64
S4L	0.19	0.64	0.23	0.64	0.38	0.64	0.68	0.64
S4M	0.16	0.64	0.23	0.64	0.47	0.64	0.81	0.64
S4H	0.15	0.64	0.23	0.64	0.48	0.64	0.87	0.64
S5L	0.18	0.64	0.26	0.64	0.41	0.64	0.68	0.64
S5M	0.14	0.64	0.24	0.64	0.50	0.64	0.80	0.64
S5H	0.13	0.64	0.24	0.64	0.51	0.64	0.84	0.64
C1L	0.17	0.64	0.22	0.64	0.39	0.64	0.67	0.64
C1M	0.15	0.64	0.23	0.64	0.48	0.64	0.80	0.64
C1H	0.13	0.64	0.20	0.64	0.39	0.64	0.66	0.64
C2L	0.19	0.64	0.27	0.64	0.44	0.64	0.79	0.64
C2M	0.16	0.64	0.26	0.64	0.56	0.64	0.93	0.64
C2H	0.14	0.64	0.25	0.64	0.56	0.64	0.96	0.64
C3L	0.17	0.64	0.25	0.64	0.39	0.64	0.65	0.64
C3M	0.14	0.64	0.23	0.64	0.46	0.64	0.75	0.64
C3H	0.12	0.64	0.22	0.64	0.48	0.64	0.79	0.64
PC1	0.18	0.64	0.24	0.64	0.38	0.64	0.65	0.64
PC2L	0.18	0.64	0.23	0.64	0.36	0.64	0.66	0.64
PC2M	0.16	0.64	0.22	0.64	0.45	0.64	0.79	0.64
PC2H	0.14	0.64	0.21	0.64	0.45	0.64	0.81	0.64
RM1L	0.22	0.64	0.29	0.64	0.44	0.64	0.80	0.64
RM1M	0.19	0.64	0.26	0.64	0.50	0.64	0.92	0.64
RM2L	0.20	0.64	0.27	0.64	0.41	0.64	0.77	0.64
RM2M	0.17	0.64	0.24	0.64	0.47	0.64	0.88	0.64
RM2H	0.14	0.64	0.22	0.64	0.49	0.64	0.92	0.64
URML	0.19	0.64	0.28	0.64	0.47	0.64	0.68	0.64
URMM	0.14	0.64	0.22	0.64	0.38	0.64	0.70	0.64
MH	0.16	0.64	0.26	0.64	0.45	0.64	0.88	0.64

6.5 Damage Due to Ground Failure - Special Buildings

Damage to Special buildings due to ground failure is assumed to be the same as the damage to Code buildings for the same amount of permanent ground deformation (PGD). Fragility curves developed in Section 5.5 for Code buildings are also appropriate for prediction of damage to Special buildings due to ground failure.

6.6 Evaluation of Building Damage - Essential Facilities

6.6.1 Overview

Special building capacity and fragility curves for structural and nonstructural systems are used to predict essential facility damage when the user is able to determine that the essential facility is superior to a typical building of the model building type and design level of interest. If such a determination cannot be made by the user, then the Code building functions of Chapter 5 are used to evaluate essential building damage. These criteria are summarized in Table 6.8.

Table 6.8 Criteria for Evaluating Essential Facility Damage

Evaluate Essential Facility Using:	User Deems Essential Facility to be:
Code building damage functions (High-Code, Moderate-Code, Low-Code and Pre-Code functions of Chapter 5)	Typical of the model building type and seismic design level of interest (i.e., no special seismic protection of components)
Special building damage functions (High-Code, Moderate-Code and Low-Code functions of Chapter 6)	Superior to the model building type and seismic design level of interest (e.g., 50 percent stronger lateral-force-resisting structural system, and special anchorage and bracing of nonstructural components)

During an earthquake, the essential facilities may be damaged either by ground shaking, ground failure, or both. Essential facilities are evaluated separately for the two modes of, ground shaking and ground failure, and the resulting damage-state probabilities combined for evaluation of loss.

6.6.2 Damage Due to Ground Shaking

Damage to essential facilities due to ground shaking uses the same methods as those described in Section 5.6.2 for Code buildings, with the exception that Special buildings are assumed to have less degradation and greater effective damping than Code buildings.

6.6.2.1 Demand Spectrum Reduction for Effective Damping - Special Buildings

Demand spectra for evaluation of damage to Special buildings are constructed using the same approach, assumptions and formulas as those described in Section 5.6.2.1 for Code buildings, except values of the degradation factor, κ, that defines the effective amount of hysteretic damping as a function of duration are different for Special buildings. Degradation factors for Special buildings are given in Table 6.9.

Figure 6.5 shows typical demand spectra (spectral acceleration plotted as a function of spectral displacement) for three demand levels. These three demand levels represent Short (κ = 0.90), Moderate (κ = 0.60) and Long (κ = 0.40) duration ground shaking, respectively. Also shown in the figure is the building capacity curve of a low-rise Special building (Moderate-Code seismic design) that was used to estimate effective damping. The intersection of the capacity curve with each of the three demand spectra illustrates the significance of duration (damping) on building response.

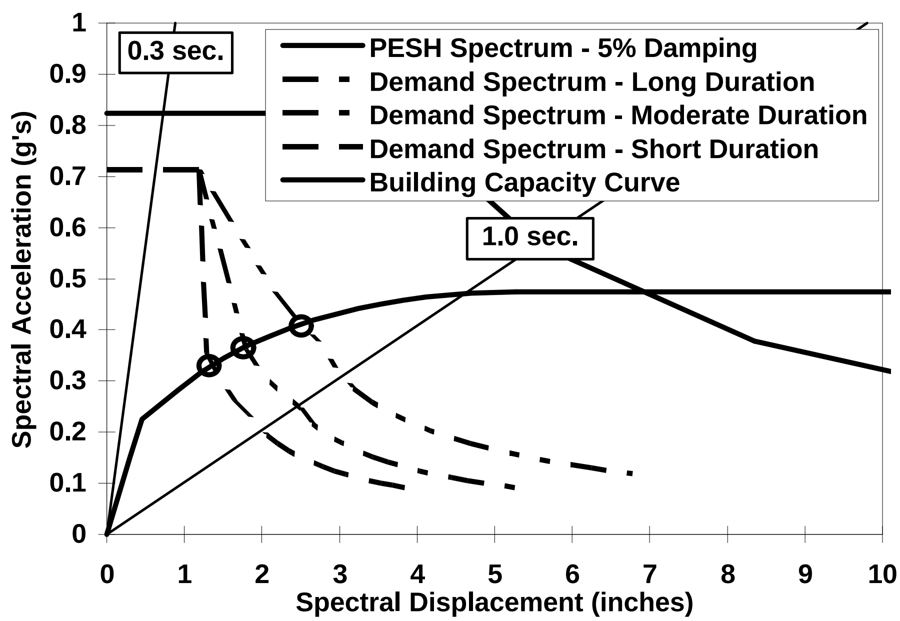


Figure 6.5 Example Demand Spectra - Special Building (M = 7.0 at 20 km, WUS, Site Class E).

Comparison of Figure 6.5 with Figure 5.7 (same example building and PESH demand, except capacity curve and damping represents Code building properties) illustrates the significance of increased strength and damping (reduced degradation) of Special buildings on the reduction of building displacement. In this case, the Special building displaces only about one-half as much as a comparable Code building for the same level of PESH demand. Forces on nonstructural acceleration-sensitive components are not reduced, but are slightly increased due to the higher strength of the Special building.

Table 6.9 Special Building Degradation Factor (κ) as a Function of Short, Moderate and Long Earthquake Duration

Building Type		High-Code Design			Moderate-Code Design			Low-Code Design		
No.	Label	Short	Moderate	Long	Short	Moderate	Long	Short	Moderate	Long
1	W1	1.0	1.0	0.7	1.0	0.8	0.5	0.9	0.6	0.3
2	W2	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
3	S1L	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
4	S1M	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
5	S1H	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
6	S2L	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
7	S2M	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
8	S2H	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
9	S3	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
10	S4L	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
11	S4M	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
12	S4H	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
13	S5L	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
14	S5M	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
15	S5H	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
16	C1L	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
17	C1M	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
18	C1H	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
19	C2L	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
20	C2M	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
21	C2H	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
22	C3L	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
23	C3M	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
24	C3H	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
25	PC1	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
26	PC2L	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
27	PC2M	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
28	PC2H	0.8	0.6	0.4	0.7	0.5	0.3	0.6	0.4	0.2
29	RM1L	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
30	RM1M	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
31	RM2L	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
32	RM2M	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
33	RM2H	1.0	0.8	0.6	0.9	0.6	0.4	0.8	0.4	0.2
34	URML	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
35	URMM	0.6	0.4	0.2	0.6	0.4	0.2	0.6	0.4	0.2
36	MH	0.9	0.6	0.4	0.9	0.6	0.4	0.9	0.6	0.4

6.6.2.2 Damage State Probability

Structural and nonstructural fragility curves of essential facilities are evaluated for spectral displacement and spectral acceleration defined by the intersection of the capacity and demand curves. Each of these curves describe the cumulative probability of being in or exceeding, a particular damage state. Nonstructural components (both drift- and acceleration-sensitive components) may, in some cases, be dependent on the structural damage state (e.g., Complete structural damage may cause Complete nonstructural damage). The Methodology assumes nonstructural damage states to be independent of structural damage states. Cumulative probabilities are differenced to obtain discrete probabilities of being in each of the five damage states. This process is shown schematically in Figure 6.6.

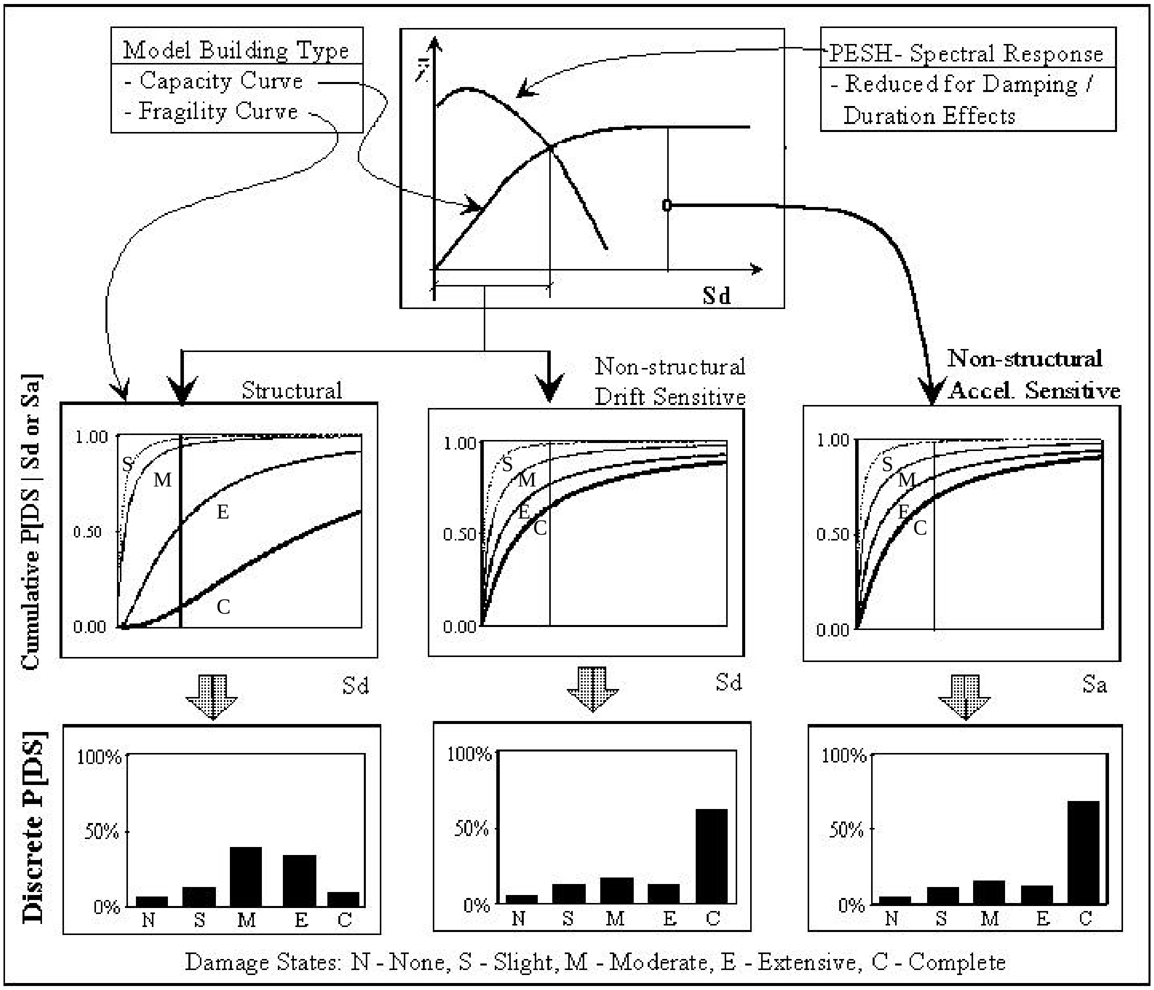


Figure 6.6 Example Essential Facility Damage Estimation Process.

6.6.3 Combined Damage Due to Ground Failure and Ground Shaking

Damage to essential facilities is based either on Code building damage functions or Special building damage functions. Code building damage due to ground shaking is combined with damage due to ground shaking as specified in Section 5.6.3. Special building damage due to ground failure (Section 6.5.2) is combined with damage due to ground shaking (Section 6.6.2.2) using the same approach, assumptions and formulas as those given in Section 5.6.3 for Code buildings.

6.6.4 Combined Damage to Essential Facilities

Combined ground shaking/ground failure damage to the model building type and design level of interest (either a Special or a Code building) represents combined damage to the essential facility.

6.7 Guidance for Expert Users

This section provides guidance for users who are seismic/structural experts interested in modifying essential facility damage functions supplied with the methodology. This section also provides the expert user with guidance regarding the selection of the appropriate mix of design levels for the region of interest.

6.7.1 Selection of Representative Seismic Design Level

The methodology permits the user to select the seismic design level considered appropriate for each essential facility and to designate the facility as a Special building, when designed and constructed to above-Code standards. In general, performance of essential facilities is <u>not</u> expected to be better than the typical (Code) building of the representative model building type. Exceptions to this generalization include California hospitals of recent (post-1973) construction. If the user is not able to determine that the essential facility is significantly better than average, then the facility should be modeled using Code building damage functions (i.e., same methods as those developed in Chapter 5 for general building stock).

Table 6.10 provides guidance for selecting appropriate building damage functions for essential facilities based on design vintage. These guidelines are applicable to the following facilities:

1. hospitals and other medical facilities having surgery or emergency treatment areas,
2. fire and police stations, and
3. municipal government disaster operation and communication centers deemed (for design) to be vital in emergencies,

provided that seismic codes (e.g., *Uniform Building Code*) were adopted and enforced in the study area of interest. Such adoption and enforcement is generally true for jurisdictions of California, but may not be true other areas.

Table 6.10 Guidelines for Selection of Damage Functions for Essential Facilities Based on *UBC* Seismic Zone and Building Age

***UBC* Seismic Zone (NEHRP Map Area)**	**Post-1973**	**1941 - 1973**	**Pre-1941**
Zone 4 (Map Area 7)	Special High-Code	Moderate-Code	Pre-Code (W1 = Moderate-Code)
Zone 3 (Map Area 6)	Special Moderate-Code	Moderate-Code	Pre-Code (W1 = Moderate-Code)
Zone 2B (Map Area 5)	Moderate-Code	Low-Code	Pre-Code (W1 = Low-Code)
Zone 2A (Map Area 4)	Low-Code	Low-Code	Pre-Code (W1 = Low-Code)
Zone 1 Map Area 2/3)	Low-Code	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)
Zone 0 (Map Area 1)	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)

The guidelines given in Table 6.1 assume that essential buildings in the study region are not designed for wind. The user should consider the possibility that mid-rise and high-rise facilities could be designed for wind and may have considerable lateral strength, even if not designed for earthquake. Users must be knowledgeable about the type and history of construction in the study region of interest and apply engineering judgment in assigning essential facilities to a building type and seismic design level.

6.7.2 High Potential Loss Facilities

6.7.2.1 Introduction

This section describes damage evaluation of high potential loss (HPL) facilities. HPL facilities are likely to cause heavy earthquake losses, if significantly damaged. Examples of such facilities include nuclear power plants, certain military and industrial facilities, dams, etc.

6.7.2.2 Input Requirements and Output Information

The importance of these facilities (in terms of potential earthquake losses) suggests that damage assessment be done in a special way as compared to ordinary buildings. Each HPL facility should be treated on an individual basis by users who have sufficient expertise to evaluate damage to such facilities. Required input to the damage evaluation module includes the following items:

- capacity curves that represents median (typical) properties of the HPL facility structure, or a related set of engineering parameters, such as period, yield strength, and ultimate capacity, that may be used by seismic/structural engineering experts with the methods of Chapter 5 to select representative damage functions,

- fragility curves for the HPL facility under consideration, or related set engineering parameters, that can be used by seismic/structural engineering experts with the methods of Chapter 5 to select appropriate damage functions.

The direct output (damage estimate) from implementation of the fragility curves is an estimate of the probability of being in, or exceeding, each damage state for the given level of ground shaking. This output is used directly as an input to other damage or loss estimation methods or combined with inventory information to predict the distribution of damage as a function of facility type, and geographical location. In the latter case, the number and geographical location of facilities of interest would be a required input to the damage estimation method.

6.7.2.3 Form of Damage Functions and Damage Evaluation

The form of user-supplied HPL facility damage functions should be the same as that of buildings (Chapter 5) and their use in the methodology would be similar to that of essential facilities.

6.8 Essential Facility and HPL Damage References

Refer to Section 5.8 for building damage references.