# Guidelines for Seismic Performance Assessment of Buildings

## ATC-58 35% Draft

Prepared by

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## Notice

This document has been prepared by the ATC-58 Project Team to assist interested parties in obtaining an understanding of the methodology as it is being developed, and to facilitate comment and feedback to the project team on its further development. The guidelines presented in this document are incomplete at this time. The data and procedures are not necessarily appropriate for use in actual projects at this time, and should not be used for that purpose. Reader notes have been provided to describe the present status of development, and to identify portions of the methodology that are not yet ready for implementation. The information contained herein will be subject to further revision and enhancement as the methodology is completed in future years.

# Glossary

**Annualized loss** – the average annual magnitude of loss ((i.e. casualties, cost to repair damage to the building and contents, downtime) that will be incurred by a building over a long period of time, considering all earthquakes that can affect the building in that period of time,

**Basic Assessment –** a performance assessment prepared using minimal definition of the building's **configuration**, structural characteristics and nonstructural components and systems, and using simplified methods of structural analysis.

**Casualties –** loss of life, or serious injury to persons

**Component –** a small part that is one of many small parts that comprise a building.  Components can be either structural or nonstructural

**Consequence function**  - the relationship that indicates the probability of losses (i.e. casualties, cost to repair damage to the building and contents, downtime) expressed as a function of building damage

**Correlation**  - the degree to which two random variables will tend to exhibit the same outcome.

**Damage function -** the detailed description for each Damage State of the significant effects of the damage (for direct damage to the building and its contents, the list of repair measures that would be required to return the materials and components to the their pre-realization condition, for casualties, the behaviors that could result in casualties, e.g. collapse, or quantity of falling debris))

**Damage State –** an extent of damage sustained by one or more building elements having common characteristics that has specific identifiable consequences with regard to occupant safety, repair actions and post-earthquake occupancy or function**.**

**Demand Parameter –**  a response quantity, such as story drift or floor acceleration that can be obtained from structural analysis and can be used to assess the probability that one or more building elements will experience damage

**Discount rate –** a factor, typically expressed as a percentage representing the time value of money net of inflation

**Direct Economic Loss** – the economic costs of repairing earthquake-induced damage or replacing buildings that are damaged beyond repair

**Downtime** – the amount of time, following an earthquake, that a building cannot be used for its normal intended function, either because it is unsafe to occupy, damage has rendered it unfit for functions normally carried on in the building, or repair activities make it impractical to conduct normal building functions.

**Earthquake Scenario -** a specific earthquake event, defined by a magnitude and geographic location.  The location may consist of identification of the fault or seismic source zone on which the earthquake occurs or the geographic coordinates of the epicenter or hypocenter.

**Enhanced Assessment** – a performance assessment prepared using more detailed definition of the building's configuration, structural characteristics and nonstructural components and systems than is required for basic assessment, or using more rigorous methods of analysis, or both.

**Element –** an assembly of structural components (e.g., an assembly of beams, columns and joints to form a moment frame)

**Fragility –** a probability distribution relating the probability of damage for a specific damage state associated with building components and *performance groups* of components to a single demand parameter, expressed in the form of a median demand and related dispersion..

**Fragility Specification** – a detailed description of damage states, related fragilities, and loss consequences associated with one or more *performance groups*

**Intensity** – the severity of ground shaking as represented by a 5%-damped, elastic acceleration response spectrum

**Intensity-based Assessment –** an assessment of probable building performance given that the building is subjected to a specific intensity of ground shaking**.**

**Net present value –** the value today of an income or expense stream over time at a specified *discount rate*

**Non-structural Component –** a part of the building such as a ceiling assembly, façade panel, piping run, etc. that does not provide significant structural resistance to vertical or lateral loads or displacements

**Performance –** the probable consequences of a building's response to earthquake shaking expressed in terms of the expected repair costs, occupancy interruption time and casualties.

**Performance group –** an assembly of building components that share a common demand parameter (e.g. story drift, floor acceleration) and related fragility specification.

**Realization -** a unique set of floor accelerations, story drifts and other response parameters of interest that could occur as one possible peak response state for the structure for a particular intensity of motion

**Return on investment** – the annual income or loss from an investment divided by the value of the investment

**Scenario-based Assessment** – an assessment of a building's probable performance given that the building is subjected to a specific earthquake scenario (an earthquake with a specified moment magnitude at a specified distance from the building site)

**Structural Component –** a beam, column, wall, brace, foundation, etc., that provides significant resistance to vertical or lateral loading or displacement

**Time-based Assessment** – an assessment of probable building performance in a specified period of time, considering all earthquake scenarios that could occur during that period of time, and the probability of occurrence of each.

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# Chapter 1
# Introduction

## 1.1 Purpose

This *Guidelines for Seismic Performance Assessment of Buildings* provides a methodology, procedures and criteria to predict the probable earthquake performance of individual buildings based on their unique structural, nonstructural and occupancy characteristics, and the seismic hazard exposure at a given site. These *Guidelines* are intended to be used as part of a performance-based seismic design process, either for design of new buildings, or evaluation and upgrade of existing buildings. They can also be applied to the seismic performance assessment of new or existing buildings, undertaken independent of a design process. Such assessments may be of value to prospective building owners, tenants, lenders, insurers and others who may experience adverse impacts as result of earthquake-induced damage to buildings.

These *Guidelines* have been prepared by the Applied Technology Council, under its ATC-58 Project to develop Next-generation Performance-based Seismic Design Criteria. Funding was provided by the Federal Emergency Management Agency of the Department of Homeland Security.

## 1.2 Use of the Guidelines in Design

These *Guidelines* have been developed as the first in a series of documents presenting next-generation performance-based design criteria. Performance-based design is a process that explicitly considers the way a building is likely to perform, as its design features are determined. In the performance-based design process, the performance capability of a building is identified and evaluated as an inherent part of the design process, and guides the many design decisions that must be made. Figure 1-1 is a flowchart that presents the key steps in the performance-based design process.

The process initiates with design criteria selection. Design criteria are stated in the form of one or more performance objectives. Each performance objective is a statement of the acceptable risk of incurring damage and the consequential losses that occur as a result of this damage. Generally, a team of decision makers, including building owners, design professionals, and building officials, will participate in selecting the performance objectives for a building. This team may consider the needs and desires of a wider group of stakeholders including prospective tenants, lenders, insurers and others who have impact on the value of a building, but who generally do not directly participate in the design process.

Once performance objectives for the project have been selected, the design professional must develop a preliminary design to a sufficient level to allow the performance characteristics to be determined. For a new building, this will include, as a minimum, identification of the location and characteristics of the site, the size and configuration of the building, occupancy, quality and character of finishes and nonstructural systems, structural system, and estimates of strength, stiffness and ductility. For an existing building, these characteristics are already be defined. It is necessary to determine what they are, and then include any concepts for retrofit measures that might be taken in the building definition.

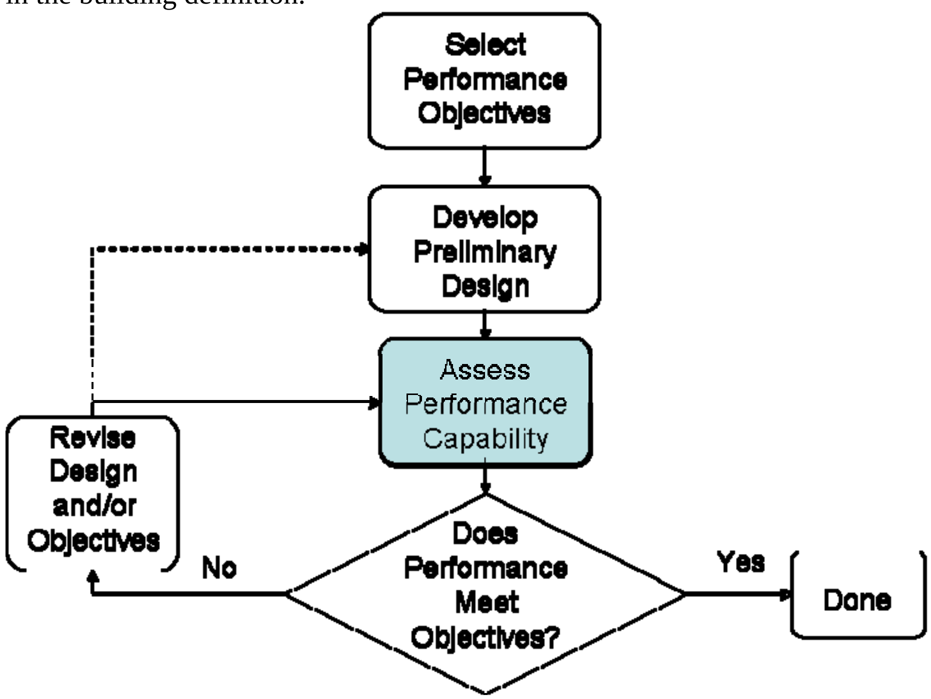


Figure 1-1 Performance-based design flow diagram

Once the building has been defined, and the preliminary design developed, it is necessary to assess the capability of the design to achieve the desired performance. To do this, a series of simulations are performed to estimate probable building performance under various design scenario events. Following a performance assessment, the building's predicted performance is compared with that identified in the performance objectives. If the predicted performance matches or exceeds the stated performance objectives, the design is completed and the project constructed. If the predicted performance does not meet the performance objectives, the design is revised, in an iterative process, until the performance capabilities adequately match the desired objectives.

These *Guidelines* address the performance assessment process (the shaded box in the flowchart of Figure 1-1). This includes determining the characteristics of the building, evaluating its response to earthquake shaking, and based on this response, projecting the amount of damage that might occur and the consequences of this damage. These *Guidelines* do not address selection of appropriate performance objectives, retrofit of buildings, or procedures to develop preliminary designs that are likely to meet desired performance objectives. These topics are the subject of companion documents that are planned for future development.

## 1.3 Other Guideline Uses

In addition to use in a performance-based design process, it is anticipated that these *Guidelines* can be used for other purposes, including:

- Use by engineers to determine the probable performance of buildings (e.g., probable maximum loss) in support of real estate investment transactions.
- Use by building product suppliers to determine the seismic performance of building components and the effect of these components on overall building performance.
- Use by building code developers to determine the performance capability of typical buildings designed using prescriptive code procedures, as a means of evaluating the adequacy of these procedures.
- Use by educators as instructional materials in engineering curricula.
- Use by researchers in identifying areas where additional building performance research is needed.
- Use by software developers to develop applications that implement the performance assessment methodology either coupled with, or independent of, structural analyses

## 1.4 Measures of Performance

In these *Guidelines*, performance is measured in terms of the risk of incurring earthquake-induced losses. The types of losses that are considered include casualties, direct economic losses, and downtime. Casualties include loss of life and serious injuries. Direct economic losses include the cost of repair and replacement of damaged systems and components. Downtime includes the time of occupancy interruption while a building is inspected for damage, repaired, cleaned up and restored to a state that permits normal occupancy and use. These measures of earthquake performance have been selected as being most relevant to the broad group of decision makers, including building officials, developers, owners, lenders, insurers, tenants and others, who must make decisions as to the acceptable performance for an individual building or broad classes of buildings.

## 1.5 Application of the Guidelines

These *Guidelines* present a general methodology for seismic performance assessment of individual buildings and one possible set of procedures that can be used to implement this methodology. Nothing contained in these *Guidelines* is intended to prevent or discourage the use of alternative procedures that appropriately implement the general methodology and consider the uncertainties inherently associated with building performance assessment.

The methodology and procedures presented herein can be applied to the performance assessment of any building type, regardless of age, construction type or occupancy. However, in order to effectively implement this methodology and procedures, basic data is needed on the damageability of components that comprise the building, and the consequences of this damage in terms of potential casualties, direct economic loss and downtime. The appropriate data to use for a given building is dependent on the type of structural system, the specific details of its construction, the type, location and means of installation of the nonstructural components and systems, and the occupancy and use of the building. Sources of such data can include laboratory testing of individual building components, analytical evaluation, statistical information on the actual performance of buildings in past earthquakes, and expert judgment. At the present time, the availability of such data is quite limited. An attempt has been made to collect data presently available, and to incorporate such data into the procedures contained in these *Guidelines*. This data is generally sufficient to allow performance assessment of buildings having structural systems of light wood frame, moment-resisting steel or concrete frame, braced steel frame, and concrete or masonry wall construction; and conforming to one of the following occupancies: commercial office, education, healthcare, hospitality, multi-family residential, research, retail, and warehouse. The *Guidelines* also include procedures that can be used to develop and incorporate additional data into the methodology, as it becomes available from future research, so that the procedures can be improved and extended to buildings having other structural systems or occupancies.

An electronic performance assessment calculation tool (PACT) is distributed with these procedures. This PACT contains electronic databases of default contents and fragility data for buildings of typical construction and occupancy, and automates the repetitive calculations necessary to assess the probable performance of a building. All default information contained within PACT can be replaced with building-specific values. In addition, it is possible to use other procedures to perform the calculations programmed into PACT.

## 1.6 Guideline Organization

Chapter 2 presents basic information on the use of potential earthquake losses as a means of quantifying the performance capability of a building. Chapter 3 introduces the general performance assessment methodology. Chapter 4 introduces one set of acceptable procedures for implementing this methodology. Chapter 5 presents detailed information on representation of seismic hazards, and Chapter 6 presents information on structural analysis as part of the procedures introduced in Chapter 4. Chapter 7 presents example applications of these procedures to representative buildings. A separate user's manual is provided that contains instructions on running the electronic performance assessment calculation tool (PACT) to perform assessment calculations.

The basic procedures presented in Chapters 4, 5 6 and 7 rely on nonlinear dynamic analysis to characterize building response to earthquake shaking. Data on building contents and construction, and the fragility of individual components, can be defined by the engineer or assembled using default templates specific to common structural systems and occupancies. An alternative, simplified analysis procedure that does not require nonlinear dynamic analysis is also available.

These *Guidelines* also include a series of appendices containing background information. Appendix A provides a basic tutorial on probability and statistics and the types of probabilistic distributions used to represent uncertainty in performance assessment; Appendix B provides detailed information on seismic hazard evaluation and attenuation relationships; Appendix C provides procedures that can be used to derive and characterize the damageability of structural and nonstructural components; Appendix D summarizes the categorization of structural systems, and the default damage and loss data for these systems embedded within PACT; and, Appendix E summarizes the categorization of nonstructural components and systems, and default damage and loss data embedded within PACT for these systems. Appendix F describes the mathematical procedure used to derive a large number of earthquake response realizations from a limited set of response data statistics.

## 1.7 Limitations

These *Guidelines* provide a general methodology and specific procedures that can be used to assess the probable performance of buildings when subjected to earthquake shaking. Specifically, the methodology assesses the likelihood that building structural and nonstructural components and systems will be damaged by earthquake shaking, and estimates the potential casualties, direct economic losses, and interruption of beneficial building occupancy that could occur as a result of such damage.

Earthquake shaking can cause other significant effects including loss of offsite power, water and sewage, initiation of fires, and release of hazardous materials. Similarly, earthquake effects other than ground shaking, including ground fault rupture, landslide, liquefaction, and lateral spreading can significantly affect building performance.

While these effects can have significant impact on the losses associated with an earthquake, and the methodology that underlies these *Guidelines* could be used to assess these effects, assessment of these losses is presently beyond the scope of these *Guidelines*. When conducting seismic performance assessments of buildings using the procedures contained within these *Guidelines*, qualitative evaluation of these other effects should be conducted, and, if found to be significant, should be noted and reported.

# Chapter 2
# Measures of Performance

## 2.1 Introduction

For the past 70 years, the metrics of seismic performance have been quantitative measures of force and deformation—metrics that structural engineers compute by structural analysis for a specified intensity of earthquake shaking. These traditional measures are interpreted by structural engineers with respect to limits set forth in building codes and standards, but this exercise has no direct relationship to loss as understood by stakeholders, namely, building tenants, building owners, insurers and banking institutions. Recognizing this in recent years, engineers began applying these traditional metrics to a series of standard performance levels, termed Immediate Occupancy, Life Safety, and Collapse Prevention. These performance levels are an improvement in that they are related to loss, but they lack the quantitative information needed by many decision-makers to select appropriate performance goals for buildings.

In these *Guidelines*, new measures of performance are introduced that are more closely aligned with the needs of decision-makers and stakeholders. Seismic performance is defined as the likelihood or probability of loss. Loss is measured in terms of 1) direct economic loss, 2) indirect economic loss or downtime, and 3) earthquake-induced casualties, which include injuries and deaths.

Section 2.2 introduces seismic risk and identifies the key factors affecting risk: intensity of earthquake shaking and vulnerability of the building to damage. Section 2.3 describes how earthquake performance, expressed in terms of risk, can be used to provide important information for decision makers as part of the design process. Section 2.4 presents summary information on the theoretical framework for the performance assessment procedures presented in these *Guidelines*.

## 2.2 Factors Affecting Seismic Risk

The occurrence of extreme natural events such as earthquakes and hurricanes are unavoidable, but do not alone cause loss. It is possible to construct a built environment to be more or less vulnerable to loss caused by natural hazards. Effective planning and design can mitigate loss.

Earthquakes can seriously damage buildings, even those buildings constructed in accordance with modern codes and standards. Damage to a building can lead to direct economic loss, building downtime and casualties. Importantly, buildings that suffer no structural damage might sustain losses from damage to nonstructural components and contents, and deaths or injuries due to heavy falling objects.

Damage to a building in an earthquake depends on 1) the intensity of the ground shaking and other seismic hazards at the building site, and 2) the vulnerabiltiy of the building to damage. Intensity is a quantitative measure of the effects of an earthquake at a building site and depends on a number of parameters including the earthquake magnitude, the rupture characteristics, the distance from the site to the plane of rupture, the regional geology and the local soil conditions. The vulnerability of a building to damage is dependant on many factors, including the type of structural system and its strength, stiffness and ductility; the type of foundation system; the age of construction; the physical condition of the structure at the time of the earthquake; and the type, location and quality of architectural, mechanical, electrical and plumbing systems, and building contents.

Direct economic losses are a measure of the financial costs associated with repair of building damage and contents, or replacement of an entire building if damage is severe enough. Indirect economic loss, or downtime, occurs when damage prevents a building from being used for its intended purpose for a period of time. Downtime can be the result of the building being rendered unsafe or appearing unsafe, or because the damage or repair activities result in too much noise, dust, or disruption to permit the building to be used in a normal manner.

Indirect losses can often be much larger than the direct economic loss. If a building is unusable, the tenants must find other space to occupy. Efforts associated with locating suitable alternative space, preparing it for occupancy, and relocating to it can be both expensive and time-consuming, particularly if the earthquake results in a significant reduction in available space in the local market. These types of losses have a ripple effect that can extend beyond the building owners and occupants to other individuals and businesses. Tenants who cannot occupy rented space in a building might stop paying rent, resulting in further loss to the building owner, who may then be unable to make loan payments on a mortgage, resulting in loss to the bank or the financial institution that holds the mortgage. A business that is unable to operate in a damaged building will suffer financial loss, but so will others that depend on the goods and services that are no longer being produced by that business.

Losses that occur when earthquakes damage buildings are dependent on the extent of damage, the nature and intensity of the building occupancy, and the decisions that people make after the damage has occurred. Economic impacts, in terms of direct and indirect economic losses, are important potential consequences of earthquakes, even for low intensity motions. The risk of injury or death at an electrical substation, for example, might be small, even if the building is quite vulnerable to earthquake damage. However, the loss of equipment and services as a result of that damage can be important and costly both to the operator of the facility and to everyone in the region that relies on the substation for electrical power.

## 2.3 Measuring Performance Using Risk

### *2.3.1 Uncertainty*

Each of the factors that affect seismic risk is difficult, if not impossible to predict precisely. For example, it is not presently possible to determine exactly where the next earthquake will occur and what magnitude it will be, let alone the direction in which the fault rupture will propagate or the exact character of the ground shaking that results. Similarly, it is not possible to predict the time of day at which the earthquake will occur, which tenants and people will be in the building, what contents and furnishings they may have within the building, what condition the building is in, or the economic conditions that prevail at the time of the earthquake. The result of these uncertainties is that it is not possible to predict precisely the losses, whether casualties, direct economic loss or downtime that will be incurred for a particular building.

Although these uncertainties make it impossible to make a precise prediction of the losses that will occur in a future earthquake, it is possible to bound potential losses by identifying the expected values of each of the factors that affect risk, and to characterize the variability in each of these factors around their estimated values. The methodology and procedures presented in these *Guidelines* account for these uncertainties.

### *2.3.2 The Use of Loss Distributions in Decision-making*

Decisions regarding the degree of initial investment in seismic protection for a new building and the appropriate level of retrofit for existing buildings can be made with the aid of cumulative loss distributions or loss functions. Figure 2-1 presents a sample loss function for a hypothetical building. It plots the probability (y axis) that the direct loss is less than a dollar amount as function of the dollar loss (*x* axis). This particular loss curve presents the probability of loss for a specific intensity of shaking with a return period of

475 years. Scenario loss curves present the probability of loss for a particular earthquake magnitude and distance. Annualized loss curves present the probability of loss considering all earthquakes that might occur in the period of a year and the probability of each such earthquake.

The shape of the loss function will vary as a function of earthquake intensity and the vulnerability of the building to damage. Reducing the vulnerability of a new or retrofitted building will shift the curve in Figure 2-1 to the left. Loss curves are often described by a cumulative lognormal probability distribution, which is fully characterized by a median value and a dispersion.

**Sidebar: Probability, Statistics and Distributions**. These *Guidelines* express losses, and therefore performance, statistically in the form of probability distributions. Appendix A provides a brief tutorial on probability and statistics for readers who are unfamiliar with them. Many textbooks also provide information on the use of probability and statistics in engineering applications.

Once a loss function is established for a given building, intensity of earthquake shaking, scenario or annual period, and performance measure (direct economic loss, downtime, casualties), it can be used to make quantitative statements regarding loss and to make decisions regarding acceptable levels of loss. Figure 2-1 identifies a median loss for the hypothetical building of $1.1M USD, where the median or 50th percentile loss has a 50% probability of being conservative (high) and a 50% probability of being unconservative (low).

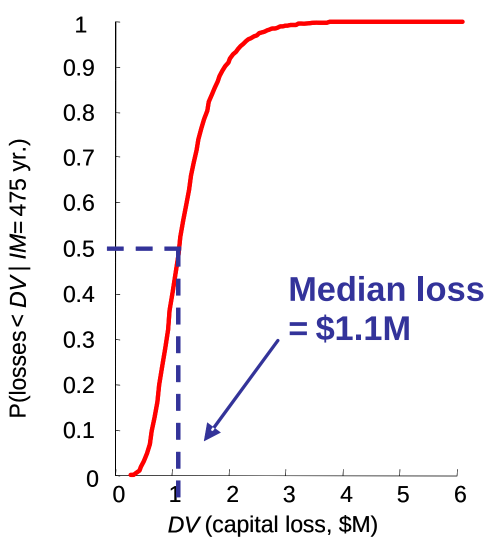


Figure 2-1 Cumulative loss function identifying a median (50th percentile) loss

Another measure of loss is the average, expected or mean loss. If the loss function is represented by a cumulative lognormal distribution, the mean loss will exceed the median loss by an amount that is dependent on the dispersion in the distribution. (Appendix A provides the equations needed to relate median and mean values in a lognormal distribution.)

A measure of loss that is commonly used by lenders and insurers is the loss that has a 90% chance of nonexceedance, commonly termed either a probable maximum loss (PML) or upper bound loss. Figure 2-2 illustrates that the PML can be determined from the loss curve of Figure 2-1.

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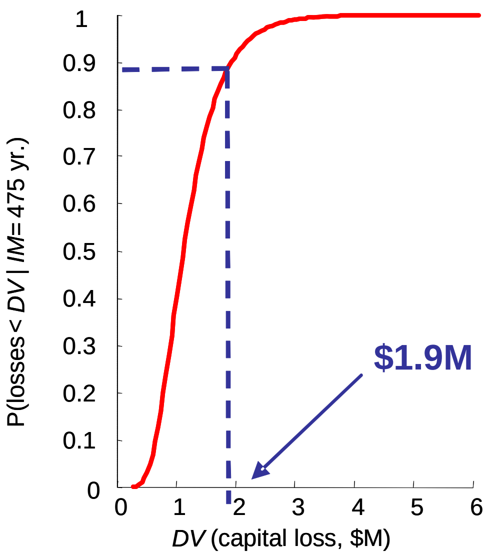



Figure 2-2 Illustration of Probable Maximum Loss (PML)

Some decision-makers prefer to bound loss. Figure 2-3 illustrates how loss can be bounded on a loss curve. The figure illustrates an 80% probability that the loss will fall between $0.7M and $1.9M USD for the hypothetical building and the given intensity of shaking of Figure 2-1.

Another presentation of loss is annualized loss, calculated as the area under an annualized loss distribution. Annualized loss is a potentially powerful metric for those tasked with defining standards of performance for a building. Consider Figure 2-4 that presents loss curves from two assessments of an existing building, one for the building's present condition, and the other for the building after retrofit to a specified standard. Before retrofit, the average annualized loss is $156,000 per year; after retrofit the annualized average loss is $96,000 per year. If retrofitted to the assumed standard, the average annualized loss is reduced by $60,000 per year, which is a net economic benefit to the owner. An owner could use this information in conjunction with simple and conventional economic analysis techniques to compare the net present value of the upgrade to the upgrade cost and decide if the investment was economically beneficial.

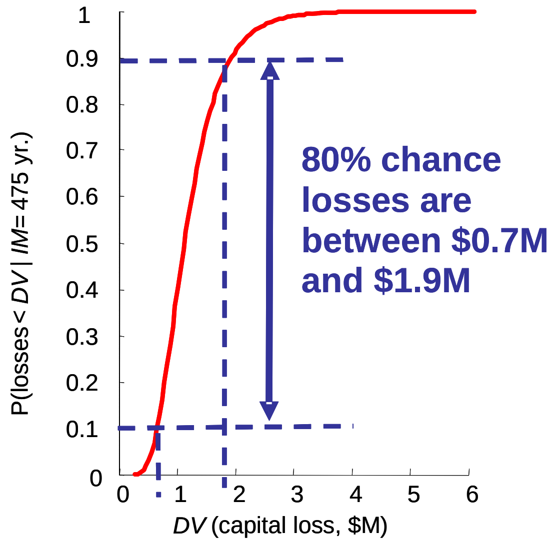



Figure 2-3 Illustration of a bounded loss

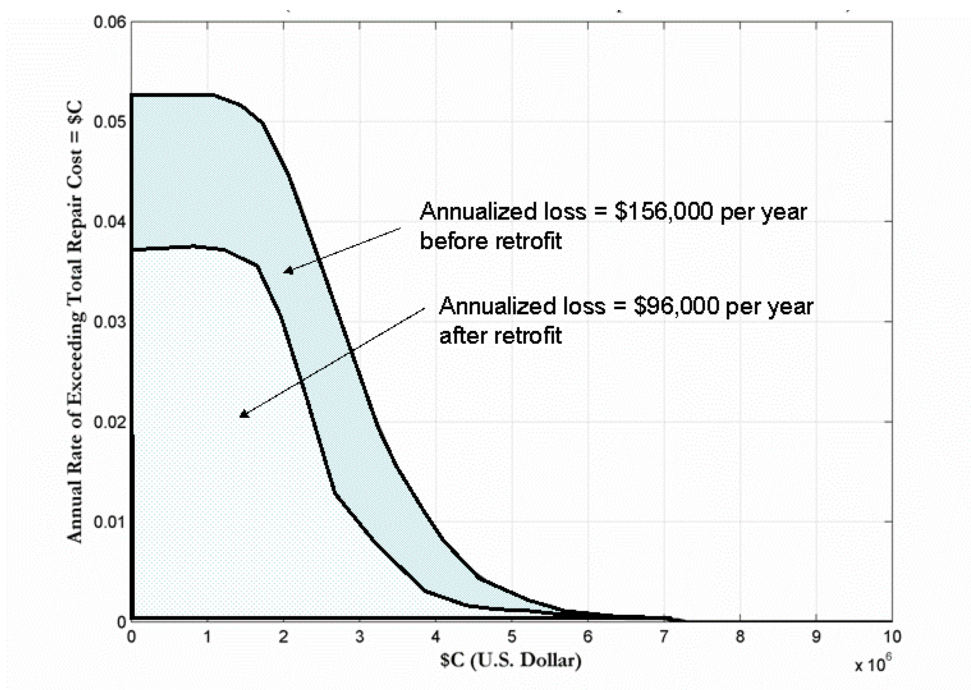



Figure 2-4 Annualized loss before and after proposed retrofit

For this example, if the discount rate is 7%, this benefit stream of $60,000 per year would have a net present value (equivalent lump sum today) of $840,000 over a fifty-year period. As long as this net present value is greater than the cost of retrofit, there is a net economic benefit to performing the retrofit. Note that, this illustration neglects the losses associated with downtime that could be avoided, or the protection of occupant safety that may be obtained with the retrofit.

Another way of using this information is in the form of the internal rate of return on the investment. Many corporations and investors will not make an investment unless they can achieve a target rate of return on their money, and will use this metric as a means of deciding between alternatives. Rate of return is a simple expression of the equivalent interest rate that would be obtained from a particular investment. For a long duration investment, such as 50 years in the case of this illustration, the return on investment can be calculated as the annualized benefit of the investment divided by the cost of the investment. If the cost of retrofit is $470,000, the return on investment would be 60/470 or about 13%.

Note that the procedures presented above can be applied to both existing and new buildings. For new building construction, the benefits and costs of various design alternatives could be compared on a sound economic basis.

# Chapter 3

# Performance Assessment Methodology

## 3.1 Introduction

This chapter describes the methodology for seismic performance assessment of buildings contained within these *Guidelines*, and identifies sources of uncertainty that are inherent in the process. Figure 3-1 identifies a procedure consisting of five basic steps, some of which might be familiar to design professionals involved in performance-based engineering, while others might not.

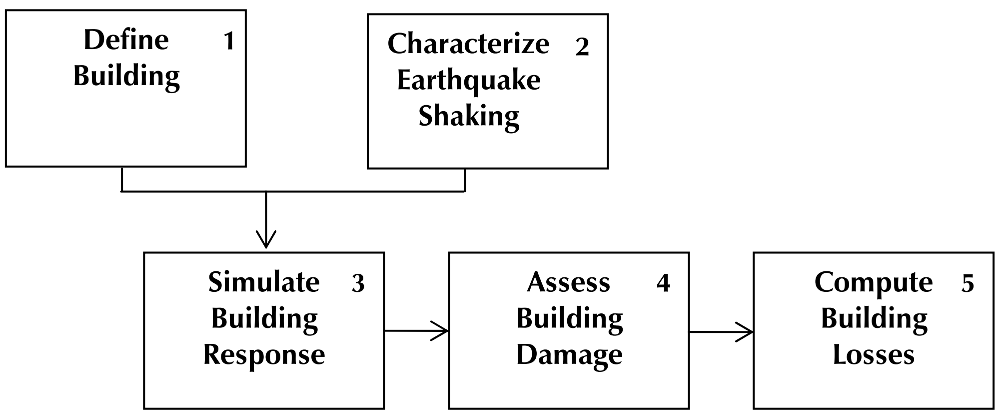



Figure 3-1 Procedure for Seismic Performance Assessment

Section 3.2 introduces the basic principles of seismic performance assessment, and three types of assessment that can be performed using this methodology. Section 3.3 describes the five steps for seismic performance assessment identified in Figure 3-1. One set of acceptable procedures for implementing each of these steps is presented in the following chapters: Implementation (Chapter 4), Hazard Analysis (Chapter 5), and Response Analysis (Chapter 6). Other procedures that adequately predict the probable distributions of earthquake demands, building response, damage, and loss can also be used.

**Sidebar: Uncertainty and randomness**. The methodology and procedures described in these *Guidelines* include explicit treatment of the large inherent uncertainties in the prediction of losses due to earthquakes. This formal treatment of uncertainty and randomness represents a substantial advance in performance based engineering.

## 3.2 Types and Products of Performance Assessment

This section introduces the probabilistic framework for performance assessment (loss computations), and identifies three types of assessment (intensity-, scenario- and time-based assessments) enabled by this framework.

### *3.2.1 Introduction*

Three measures of seismic performance are considered in these *Guidelines*: exposure to direct economic loss, indirect economic loss (downtime) and casualties (including injuries and death). Each of these performance measures is treated as a potential loss.

The probabilistic framework that serves as the technical basis for the procedures described in these *Guidelines* is based on a methodology developed by the Pacific Earthquake Engineering Research (PEER) Center, which is described in Moehle and Deierlein (2004), and Yang et al. (2006). The framework enables the calculation of the probability of loss, *L*, exceeding a value, *l*, using either:

$$P(L > l) = P(L > l \mid E = e) \quad \text{(3-1a)}$$

or

$$P(L > l) = \int_{\lambda} P(L > l \big| E = e)\, d\lambda \quad \text{(3-1b)}$$

where *E* is an earthquake intensity variable (e.g., spectral acceleration at the first mode period), *e* is a value of the earthquake intensity (e.g., 0.37*g*), $P(L > l \mid E = e)$ is the probability of loss exceeding *l* for an earthquake intensity of *e*, $\lambda(e)$ is the mean annual frequency of exceeding *e*, and the integration is performed over a range of $\lambda$.

The calculation of the probability that the loss exceeds *l* for earthquake shaking of intensity *e* involves a number of steps that are described in Sections 3.3 and Chapters 4, 5 and 6. In brief, the PEER framework involves: (1) the calculation of building response, including both structural and nonstructural components) for a given value of *e*; (2) the assessment of damage to components in the building for the calculated building response; and (3) the transformation of the building damage state into loss.

Intensity-based and scenario-based loss computations are performed using (3-1a). Equation (3-1b) is used for time-based assessments and the integration is performed over a range of mean annual frequency of exceedance, though, as described later, the integration is replaced by a discrete summation over intervals of earthquake intensity. More information on each type of assessment is presented in the following three subsections.

**Sidebar**: Scenario-based assessments could be performed using (3-1b) but $\lambda$ in this instance would represent the distribution of earthquake intensity given a user-selected combination of earthquake magnitude and distance.

### *3.2.2 Intensity-Based Assessments*

An intensity-based performance assessment provides a distribution of the probable losses, given that the building experiences a specified intensity of shaking. In these *Guidelines*, ground shaking intensity is represented by a 5% damped, elastic acceleration response spectrum. Intensity could also include representation of permanent ground displacements produced by fault rupture, landslide, liquefaction, and compaction/settlement, although procedures for doing this are not included herein. This type of assessment could be used to answer questions such as:

- What is the probability of loss in a given range, if the building experiences a ground motion of a specific intensity? For example, what are the probabilities of direct economic loss greater than $1 M, if the building experiences a ground motion represented by a smoothed spectrum with a peak ground acceleration of 0.5 g?
- What is the probability that the building will be closed to occupancy for more than 15 days if it experiences ground shaking matching the design spectrum contained in the building code?
- What is the probability of incurring one or more casualties, if the building experiences a ground motion with intensity corresponding to the maximum considered earthquake spectrum as described by the building code?

For intensity based assessments, the value of the earthquake intensity, *e*, is deterministic, that is, it takes on a single value of spectral acceleration. Equation (3-1a) is used for intensity-based loss computations.

As an example of the results of intensity-based assessments, Figure 3-2 presents cumulative probability distributions for direct economic loss in a hypothetical building for four independent intensity levels, I1 through I4, where I1 represents the smallest intensity and I4 the largest. In the figure, the probability that total repair costs exceed a specified value of total repair cost (*trc*) is plotted versus *trc*. For shaking intensity I4, there is a 50% probability that the total repair cost will exceed $1.8 M and a 90% probability that the total repair cost will exceed $0.9 M. The complementary curves of Figure 3-

3 plot the probability that the total repair cost will be less than or equal to a specified value. The complementary probability is computed as $P(TRC \le trc) = 1 - P(TRC \ge trc)$.

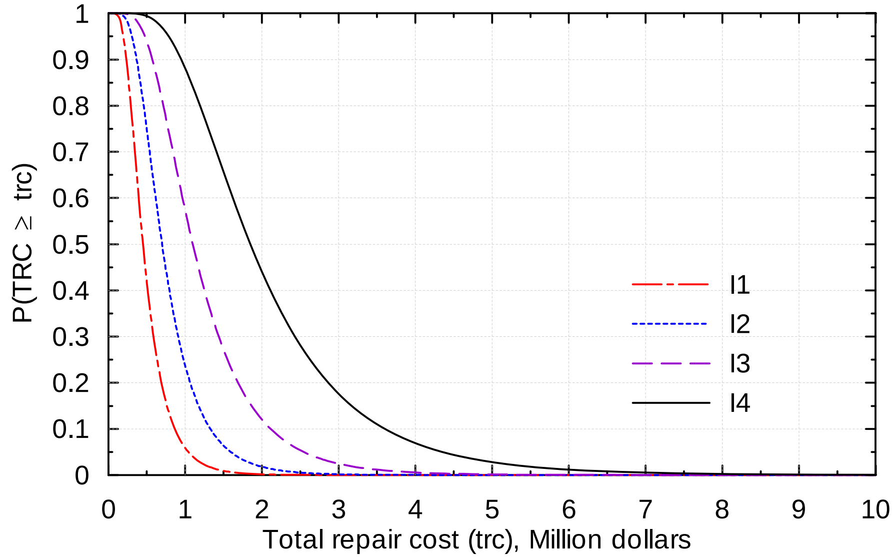



Figure 3-2 Example cumulative probability distributions for loss exceeding a specified value for a hypothetical building at four ground motion intensities

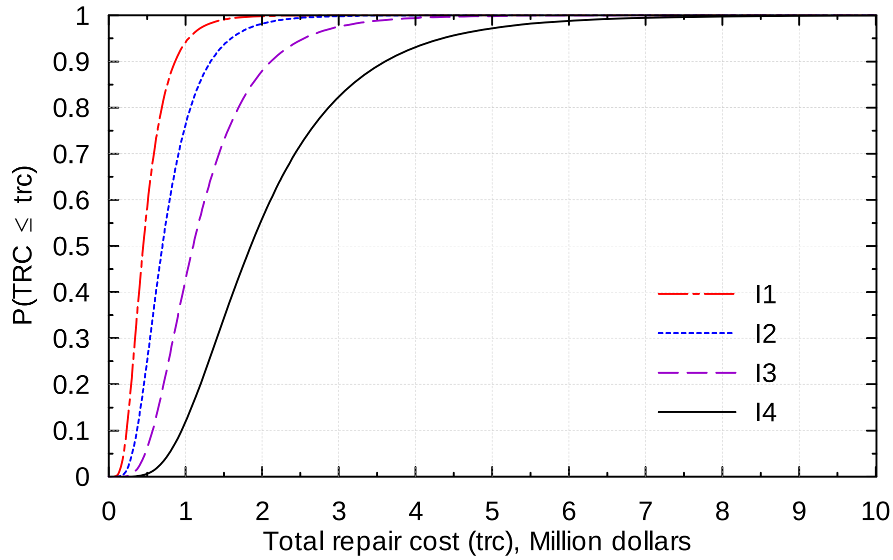



Figure 3-3 Example cumulative probability distributions for loss less than a specified value for a hypothetical building at four ground motion intensities

### *3.2.3 Scenario-Based Assessments*

A scenario-based performance assessment is similar in many regards to an intensity-based assessment, and enables an estimate of loss, given that a building experiences a specific earthquake, defined as a combination of earthquake magnitude and distance, as measured between the fault on which the earthquake occurs and the site on which the building resides. This type of assessment can be used to answer the following types of questions:

- What is the probability of more than ten casualties from an M 6 earthquake on the fault ten kilometers from the building site?
- What is the probability of repair costs exceeding $5 M if my building is subjected to a repeat of the 1906 San Francisco earthquake?

Scenario assessments may be useful for decision makers with buildings located close to one or more known active faults. For scenario-based assessments, the earthquake intensity variable, *E*, is a random variable that is described by a probability distribution (say $\hat{e}$). Loss can be computed using either (3-1a) or (3-1b), depending on how the uncertainty in the earthquake shaking intensity is addressed. The product of a scenario-based assessment is a *single* loss curve, such as one of the loss curves in either Figure 3-2 or Figure 3-3.

### 3.2.4 Time-Based Assessments

A time-based performance assessment is an estimate of the probable earthquake loss, considering all potential earthquakes that may occur in a given time period, and the mean probability of the occurrence of each. A time-based assessment could be used to answer the following types of questions:

- What is the mean annual frequency of earthquake-induced direct economic loss resulting from damage to my building and contents exceeding $300,000?
- What is the mean frequency of losing the use of my building for more than 30 days from an earthquake over its fifty-year life?
- What is the mean frequency of having at least one earthquake-caused casualty in my building over a fifty-year period?

For a time-based assessment, the earthquake-intensity variable is described by a seismic hazard curve (see Section 3.3.6, Chapter 5 and Appendix B), which plots the relationship between earthquake intensity, *e*, and the mean annual frequency of exceedance of *e*, $\lambda(e)$. Loss curves are developed for intensities of earthquake shaking that span the intensity range of interest and are then integrated (summed) over the hazard curve to construct an annualized loss curve of the type shown in Figure 3-4 The mean annual total loss is computed by integrating the area under the loss curve, which is equal to approximately $37,900 in this example. The accuracy of the annualized loss curve is a function of the number of intervals of earthquake intensity used in the computation.

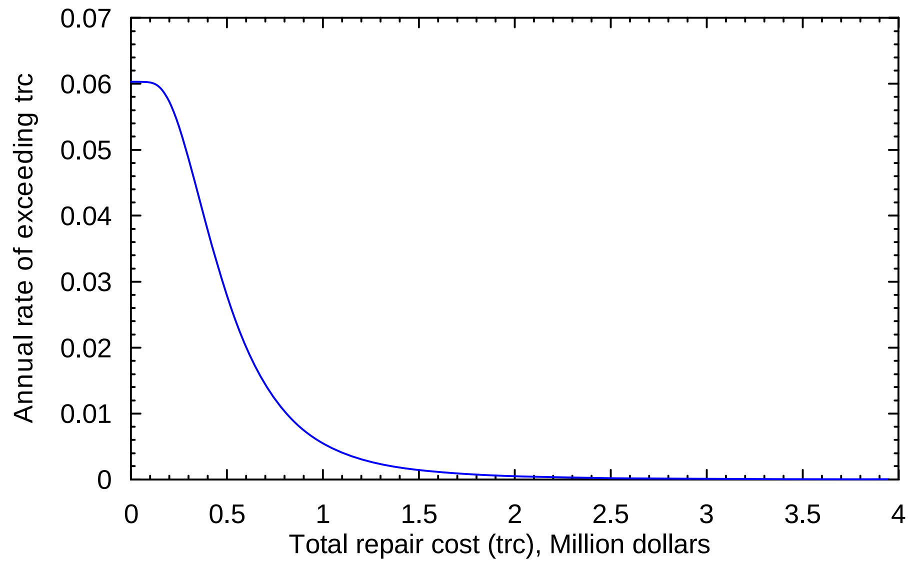



Figure 3-4 Distribution of mean annual total repair cost

## 3.3 Methodology for Performance Assessment

### *3.3.1 Introduction*

The five basic steps in a seismic performance assessment are described below:

Step 1 requires the user to define the building in sufficient detail to compute losses.

Step 2 involves the appropriate characterization of the seismic hazard, which depends on the type of assessment.

Step 3 involves analysis of the building described in Step 1, subjected to the hazard of Step 2, to predict its response in the form of accelerations, forces, displacements, and deformations induced into the building components.

Step 4 involves assessment of damage to structural and nonstructural components using the demands computed in Step 3 and fragility functions (see Sidebar) that are based on the user-specified definition of the building components (Step 1).

Step 5 involves the computation of loss using consequence functions (and a hazard curve for time-based assessment).

**Sidebar: Fragility functions.** Fragility functions or fragility curves are cumulative probability distributions that indicate the probability that a building component or system will be damaged to or in excess of a specified damage state, given that the component or system experiences a particular demand. These functions are described in more detail in Section 3.3.5.2.

### *3.3.2 Define Building, Step 1*

The first step in a performance assessment involves the definition of building location, configuration and characteristics pertinent to response to earthquake ground shaking, including

- Site location - identifying the seismic hazard and ground motion intensity
- Site conditions - identifying how local soil conditions will affect earthquake ground shaking intensities and characteristics
- Construction - providing information on structural (seismic and gravity) and nonstructural components and systems
- Occupancy - providing information regarding tenants and contents in the building.

It is not possible to define these four characteristics precisely. For example, at the time of a future earthquake it is not possible to define exactly: (1) the total number of persons that will be present in the building; (2) the locations and value of all furnishings; (3) the age and condition of the mechanical equipment; (4) the subsurface conditions; and (4) the strength, stiffness, ductility and damping of the framing system. However, it is possible to make reasonable estimates of the likely value of the key characteristics that affect performance together with estimates of their possible variations (see sidebar).

**Sidebar: Bounding the characteristics of a building**. On the basis of a geotechnical investigation, subsurface materials may be assessed as having shear wave velocities that range from 500 meters per second to 700 meters per second; structural concrete may have a minimum specified strength of 3,000 psi, a median strength of 3,750 psi and a maximum strength of 4,500 psi in a few isolated members; the weight of partitions may have a median value of 10 pounds per square foot, with possible variation between 5 and 15 pounds per square foot, depending on the office design used by particular tenants; and the typical occupancy during normal building hours may be 100 persons, with as many as 150 persons present in peak periods and no one present during the evenings, on holidays and weekends. Each of the above characteristics can be described by a statistical distribution, typically defined by a median value and a measure of the dispersion about that median. Appendix A provides a tutorial on statistical distributions.

Information on the site location and site conditions are required to establish the seismic hazard for scenario- and time-based assessments, and will likely be used to develop a response spectrum for an intensity-based assessment. Information on site conditions is also important for the selection of ground motions for response-history analysis. Construction information, either as proposed, as existing, or as a combination of both (for retrofits of existing buildings), is required to: (1) establish the seismic and gravity load-resisting systems to enable the development of a numerical model of the building that is suitable for analysis; and (2) select appropriate structural-component fragility curves to compute damage and losses once the demands are known. Occupancy information is required so that the user can: (1) identify likely inventories and quantities of nonstructural components and contents in the building; (2) assign fragility curves to the components and contents to enable calculations of damage and associated losses; and (3) evaluate casualty and downtime losses associated with occupants and the building function.

### *3.3.3 Characterize Earthquake Shaking, Step 2*

A primary input into the performance assessment process is the definition of the earthquake effects that cause building damage and loss. In the most general case, earthquake hazards can include ground shaking, ground fault rupture, liquefaction, lateral spreading and land sliding. Each of these can have different levels of severity, or intensity. Generally, as the intensity of these hazards increases, so also does the potential for damage and loss. In these *Guidelines*, only the effects of earthquake shaking are considered for loss computations, although the framework is easily adapted to accommodate other earthquake hazards.

There are a number of different ways to characterize hazards for intensity-, scenario-, and time-based assessments. An intensity-based assessment utilizes a response spectrum; a scenario-based assessment uses a median spectrum and its period-dependant dispersion; and a time-based assessment uses a mean seismic hazard curve (or a median hazard curve and its dispersion). One set of acceptable procedures for characterizing seismic hazard, and scaling earthquake ground motions to represent the hazard, is presented in Chapter 5, although alternate procedures can be used.

### *3.3.4 Building Response Simulation, Step 3*

The third step in a performance assessment is to perform an analysis of the building defined in Step 1 for ground shaking consistent with the seismic hazard of Step 2. For analysis, the building defined in Step 1 must be transformed into a numerical model of a complexity that is dictated by: (1)

the availability of information; (2) the degree of accuracy required from the loss computation; and (3) the time and effort available to the user. The least accurate estimates of structural demand (smallest confidence in the answer) will result from the use of approximate *linear* models of the framing system and the simplest characterizations of seismic demand. The most accurate estimates of demand will be computed using detailed *nonlinear* models of the vertical and horizontal framing systems, foundations and subsurface materials, and rigorous characterizations of building responses.

Either simplified or robust nonlinear methods of analysis will be used to compute peak component and system demands. Since the mechanical characteristics of a building and earthquake shaking are both highly uncertain, it is not possible to calculate precise (deterministic) values of these demands. Instead, it is necessary to predict a statistical distribution of the likely values of demands, considering the possible variation in earthquake intensity, ground motion characteristics, and structural modeling uncertainty (associated with variations in the building's properties and the extent to which these are accurately captured by an idealized analysis model). The distributions in each demand parameter are used to assess damage and estimate loss. One consistent set of acceptable procedures for capturing distribution in the seismic hazard and performing response simulations are described in Chapters 5 and 6, respectively, although other procedures can be used.

### *3.3.5 Assess Building Damage, Step 4*

The fourth step in a performance assessment is to calculate the possible distribution of damage to structural and nonstructural building components using the response data from the structural analysis together with data on the building configuration. Each analysis will produce a vector of response quantities that can be applied as demands to one or more structural and nonstructural components in the building. Component-specific fragility functions can then be used to characterize damage at the component level for the demands computed by the analysis. The prediction of damage and identification of damage states for a component are also uncertain, even for a specific value of the demand.

Assessment of damage given demand is performed using fragility curves that relate the probability of damage to structural demand parameters (e.g., story drifts, floor accelerations, or other response quantities). Since fragility curves are not widely used in the practice of structural and earthquake engineering at this time, they are introduced here. Quantitative procedures for developing fragility curves, based in the large part on studies performed for the U.S. Nuclear Regulatory Commission in the 1980s, are provided in Appendix C.

**Seismic Fragility Curves**

Each structural and nonstructural component in a building will have a unique probability of sustaining damage in an earthquake, based on its construction characteristics, location in the building and the response of the building to earthquake shaking. The loss computation methodology described in this Chapter utilizes fragility curves to relate the probability of damage to demand, where demand can be measured using any useful response quantity, including story drift, floor acceleration, component force, and component deformation.

To enable computations of loss, a series of discrete damage states must be defined for each component in the building. These damage states must be meaningful in terms of the considered performance measure (e.g., repair costs, downtime and casualties). Damage states that are meaningful for one performance measure (e.g., direct economic loss or repair cost) may not be useful for another performance measure (e.g. casualties), and alternate damage states must then be identified.

In these *Guidelines*, fragility curves are required for all measures of performance. They are introduced here using the performance measure of direct economic loss (repair cost). Damage states for direct economic loss are defined in terms the degree or scope of repair. Real damage generally occurs as a continuum and not as a series of discrete states. For example, consider damage to a steel beam measured using the amplitude of flange local buckling. The amplitude of buckling is a continuous function of beam deformation.

The cost for repair of this damage, however, is not a continuous function of flange buckling amplitude. It may not matter if the buckling amplitude is ¼-inch or 3/8-inch, since the repairs in either case will be very similar, and the costs nearly identical. Conversely, modest increases in the level of damage can trigger large increments in construction activity and cost. For example, a buckling amplitude of 1/16-inch may not require repairs, but 1/8-inch may require heat straightening of the beam flange, which is a repair activity requiring substantial work and cost.

Figure 3-5 presents a sample family of fragility curves for a special steel moment frame connection. Herein, three damage states, $DS_1$, $DS_2$ and $DS_3$ are used, where the damage states are defined using discrete and well separated (in terms of cost) states of repair:

- $DS_1$ : Flange and web local buckling in the beam requiring heat straightening of the buckled region.
- $DS_2$ : $DS_1$ damage and lateral-torsional distortion of the beam in the hinge region requiring heat straightening, partial replacement of the beam flange and web in the hinge region, and corresponding construction work to other structural and nonstructural components.
- $DS_3$ : Low-cycle fatigue fracture of the beam flanges in the hinge region requiring replacement of a large length of beam in the distorted/fractured region, and corresponding construction work to other structural and nonstructural components.

Fragility curves like those of Figure 3-5 plot the probability that a component or system will be damaged to a given damage state, or more severe damage state, as a function of demand, expressed here using story drift ratio. Each curve is represented by a lognormal distribution (see Appendix A for details) with a median (50th percentile) demand $\theta_{DS_i}$ and a dispersion $\beta_{DS_i}$ . The dispersion is associated solely with the onset of the associated damage as a function of building response (i.e., demand) and is independent of the uncertainty associated with the intensity of shaking or the prediction of demand. The greater the value of $\beta_{DS_i}$ , the *flatter* the curve. The dispersion reflects variability in construction and material quality, the extent that the occurrence of damage is totally dependent on a single demand parameter, and the relative amount of knowledge or data on the response of the component.

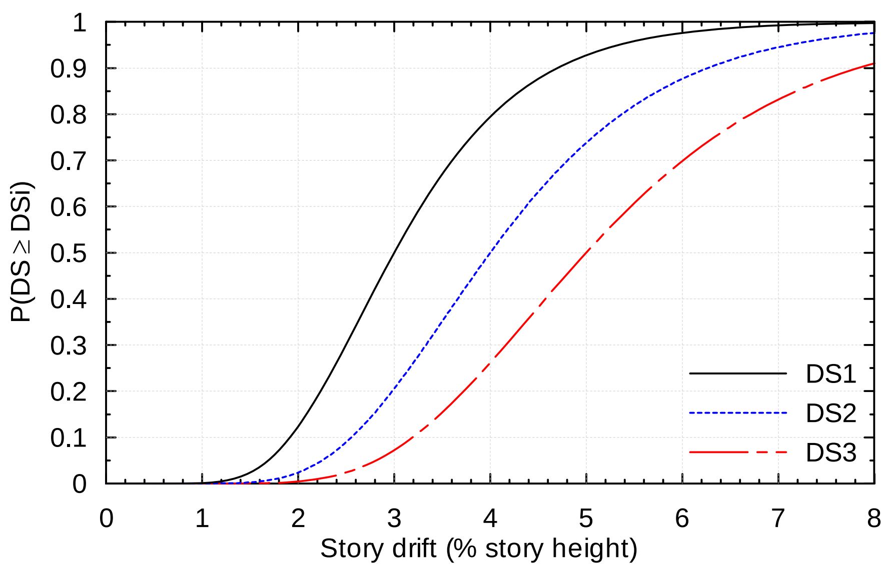



Figure 3-5 Example family of fragility curves for special steel moment frames

For the family of fragility curves presented above, for which medians and dispersion for the three damage states are (3%, 0.35) for $DS_1$, (4%, 0.35) for $DS_2$ and (5%,0.35) for $DS_3$, the following interpretations are possible:

- At a story drift ratio of 4% (or 0.04 if expressed as a decimal fraction), the probability of no damage is 20% (=1.00-0.80), the probability of damage in state $DS_1$ (assumed to be representative of damage between curves $DS_1$ and $DS_2$) is 30% (=0.80-0.50), the probability of damage in state $DS_2$ is 24% (=0.50-0.26) and the probability of damage in state $DS_3$ is 26% (=0.26-0).
- At a story drift ratio of 4%, the probability of being in damage state $DS_1$ or greater is 80% (equal to the sum of the probabilities of being in states $DS_1$, $DS_2$ and $DS_3$, 30%+24%+26%).

Fragility curves are required for each type of loss and for each component in a building that might contribute to the loss in order to perform an assessment using the procedures set forth in these *Guidelines*.

**Building Damage States and Consequence Functions**

A building damage state is developed for each earthquake analysis or simulation (e.g., response- spectrum or response-history analysis). The building damage state is a complete description of the repair actions required to return a building to its pre-earthquake condition, the potential restrictions to occupancy, and the risks to occupant safety. It is assembled from the component damage states identified above, using the component-level fragility functions, the vector of demands from the simulation, and the likelihood of total building collapse.

Consequence functions, which are distributions of the likely consequences of a building being damaged to a given state, are then used for the purpose of assembling single estimates of repair cost, casualties and downtime. Families of consequence functions are developed for each performance measure, and these families will generally differ across types of buildings. The general functions are complex and uncertain and must be simplified using heuristic procedures and approximations for practical implementation. A sample consequence function for cost of repair is presented in Figure 3-6.

Most structural engineers will not be familiar with building damage states and consequence functions. Structural and nonstructural damage, and the cost of returning damaged components to pre-earthquake conditions, are considered below to illustrate the use of such states and functions.

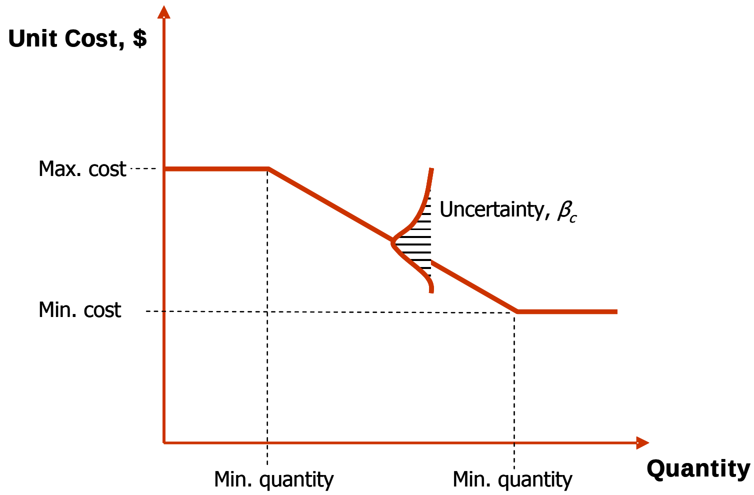



Figure 3-6 Sample consequence function for cost of repair

A building damage state, for purposes of direct economic loss calculations, includes a detailed description of the condition of the building in terms of the required repairs. This description could be given to a contractor to form the basis for an estimate of the costs to repair the building and replace its contents that may have been damaged. When a contractor makes such an estimate, the unit costs applied to the various repair quantities depend on the total quantities of basic repair measures. In some instances (e.g. scaffolding, protection of finishes, clean-up), costs are distributed to more than a single repair measure.

Contractors' overhead and profit depend on the total amount of work and the type of tradesmen and subcontractors required. In effect, the contractor applies a *direct economic loss consequence function* to the damage to calculate the loss. The consequence functions for direct economic losses use the building damage state to determine the need for shoring, staging, finish protection, cleaning, and other general condition costs; the costs associated with contractor overhead and profit and indirect project costs including design services, fees and permits as well as the costs of the actual labor and materials associated with the individual repairs required.

Consequence functions for direct economic loss should account for the effect of quantities on unit price. These are of the general form illustrated in Figure 3-6 above. For small quantities the unit cost is constant at a maximum value. Beyond a certain quantity the cost diminishes as the contractor can take advantage of economies of scale until a minimum unit cost for large quantity repairs is reached. Since costs are subject to uncertainty from market conditions, contractor bidding strategy, and other factors, unit costs are assigned a median value (solid line in the figure) and dispersion, $\beta_c$.

**Sidebar: Consequence functions**. Different contractors will typically charge different amounts to repair damage, depending on their own individual cost structures, how busy each one is, the skill of their individual workers, the amount of profit they wish to make, and how careful they are in estimating the project. For the performance measure of downtime, the amount of time that a damaged structure will be unoccupied will depend not only on the extent of damage but also on the efficiency of the owner in retaining a contractor to make repairs, the efficiency of the contractor, the availability of materials and labor, and other factors that are impossible to predict with certainty. As with other steps in the performance assessment process, consequence functions cannot be determined precisely and must be represented by probability distributions.

### *3.3.6 Compute Building Losses, Step 5*

**Monte Carlo Procedures for Loss Computation**

Monte Carlo type procedures are used to develop mean estimates of casualties, direct economic losses and downtime as well as information on the possible variation in these losses. In Monte Carlo analysis, each of the factors that affect performance, namely, earthquake intensity; structural response as measured by demand parameters; damage, as measured by damage states; and consequences (losses); are assumed to be random variables, each with a specific probability distribution defined by a median value and a dispersion.

A large set (hundreds) of simulations is required per intensity level to generate a loss curve using Monte Carlo procedures. Each simulation represents one possible outcome of the building experiencing the given intensity of motion – from definition of the character of the specific ground motion to an assumed set of building properties, to a derived set of demands. The large set of simulations can be: (1) generated directly by a large number of analyses; or (2) generated indirectly by statistical manipulation of the results of a smaller number of analyses. Later sections of these *Guidelines* and Appendix F present one set of acceptable procedures for generating a large number of simulations through statistical manipulation of a relatively small number of structural analyses.

Each simulation of response enables the development of a building damage state and the calculation of a single value of the performance measure (loss). By repeating the simulations and calculations many times, a distribution of loss (repair cost, downtime or casualties) is constructed for the chosen intensity of earthquake shaking. Sorting the losses in ascending or descending order enables the calculation of the probability that the total loss will be less than a specific value for a given intensity of shaking, producing a loss curve (see the sample curves in Figure 3-2 and 3-3). A loss curve can be used to determine:

- Median performance. The number of casualties, direct economic loss and downtime loss exceeded by half of the realizations. There is a 50% chance that actual earthquake losses will be less than or greater than the median.
- Mean performance. The average number of casualties, direct economic loss and downtime values obtained from all of the realizations. This mean value is sometimes termed the "expected" value.
- Dispersion. A measure of the amount that the building performance, as measured in casualties, direct economic loss and downtime, can be greater or less than the median values.

It is possible to use the median and dispersion to calculate values of performance measures at any probability of exceedance.

**Types of Assessment**

The product of an intensity-based assessment is a loss curve of the type shown in Figure 3-2. The curve is constructed using the Monte Carlo procedures outlined above.

The product of a scenario-based assessment is also a loss curve, also constructed using Monte Carlo procedures. The key difference between the intensity- and scenario-based assessments is that a distribution of earthquake shaking conditioned on a given magnitude and site-to-source distance is used for a scenario assessment.

The product of a time-based assessment is a curve of the type shown in Figure 3-4, which plots the total repair cost versus the annual rate of exceeding the total repair cost. (Similar curves are developed for downtime and casualties.) The curve shown in Figure 3-4 can be constructed using the results of a series of intensity-based assessments and the appropriate seismic hazard curve. A sample seismic hazard curve is shown in Figure 3-7, where the annual frequency of exceeding an earthquake intensity, $\lambda(e)$, is plotted versus the earthquake intensity, $e$, where the typical earthquake intensity is spectral acceleration (at the first mode period of the building).

Equation (3-2) is used to perform a time-based loss calculation, namely, to calculate the annual frequency that the loss $L$ will exceed a value $l$:

$$P(L > l) = \int_{\lambda} P(L > l \,|\, E = e)\, d\lambda(e) = \sum_{i=1}^{n} P(L > l \mid E = e_{Ii}) \Delta\lambda_i \qquad (3\text{-}2)$$

where most terms are as defined in equation (3-1). For the summation, the spectral range of interest is split into $n$ equal intervals, $\Delta e_i$. The midpoint intensity in each interval is $e_{Ii}$, and the annual frequency of earthquake intensity in the range $\Delta e_i$ is $\Delta\lambda_j$. Figure 3-7 defines $\Delta e_i$, $e_{Ii}$ and $\Delta\lambda_j$ for the sample hazard curve using $n$ = 4 (This low value on $n$ is chosen for clarity of the figure).

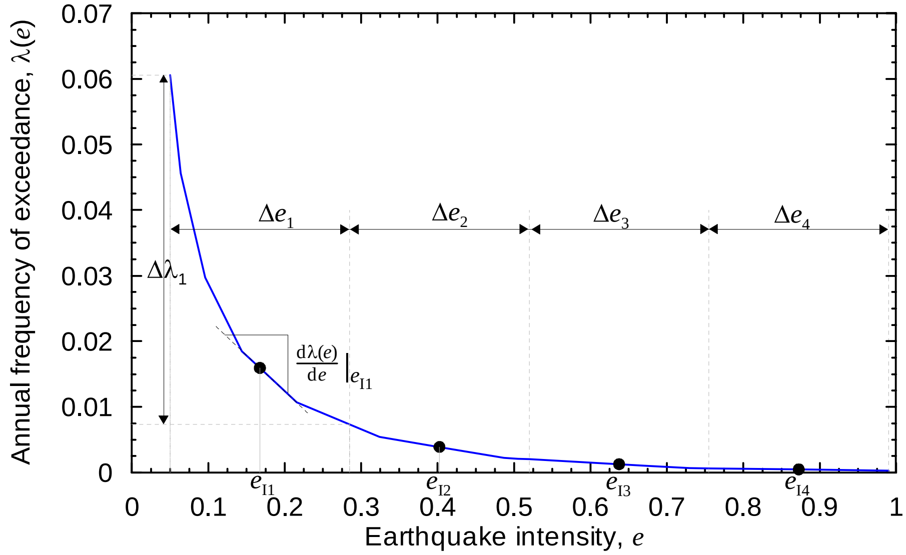



Figure 3-7 Seismic hazard curve and time-based loss calculations

For a time-based assessment, a series of $n$ intensity-based assessments are performed at discrete, $e_{I1}$ through $e_{In}$, where the user-selected range of earthquake intensity is from zero damage (small $e$) through collapse (large $e$). The number $n$ is selected by the user. Earthquake intensity at intensity $e_{I1}$ is assumed to represent all shaking in the interval $\Delta e_1$. The product of the $n$ intensity-based assessments is $n$ loss curves of the type shown in Figure 3-2. The annual frequency of shaking of intensity $e_{Ij}$, $\Delta\lambda_j$, is calculated directly from the seismic hazard curve. A sample calculation is shown in Figure 3-7 for interval $\Delta e_1$ for which $\Delta\lambda_1 = 0.054$. Figure 3-4 is constructed by: (1) multiplying each loss curve by the annual frequency of shaking in the interval of earthquake intensity used to construct the loss curve; and (2) summing the annual frequencies for a given value of the loss.

# Chapter 4
# Implementation

This chapter presents one set of acceptable and practical procedures that can be used to implement the general methodology described in Chapter 3. Other approaches are possible. A Performance Assessment Calculation Tool (PACT) has been developed that utilizes the procedures presented in this chapter to perform the calculations Screen captures from PACT are used to guide the reader through the loss computations. A user's guide for PACT is published separately. The source code and algorithms implanted in PACT are open source, and it is possible to develop alternative software to perform these calculations.

> **Reader Note**: The source code for PACT is presently under development. An executable version is available for download from the ATC website. The source code will be included as an appendix in later versions of these Guidelines.

This chapter follows the basic steps of the general methodology described in Chapter 3. First, a general description of the building is developed. This includes a description of the structural and nonstructural components and the contents. Next the earthquake ground motion hazard at the building site is determined and structural analyses of the building are performed. To translate the structural response into damage and loss, the component vulnerability to accelerations and deformations is specified. The losses in terms of direct economic loss, downtime and casualties are then calculated.

## 4.1 Building Definition (Step 1)

To estimate potential losses due to earthquake ground motion, it is necessary to develop a complete description of the building in the form of a loss assessment model. Engineers are familiar with structural analysis models that include building geometry, mass, structural strength, stiffness, and damping. The loss assessment model constitutes a more extensive description of the building than is contained in traditional models of structural framing systems. In addition to the basic geometry of the building, the loss assessment model includes information on the type and quantity of all of the building components (structural and nonstructural) and contents for which losses are to be considered, as well as information on the number and distribution of occupants and the vulnerability of building operations to damage. Building components are assembled into groups based on common characteristics (e.g., location, construction trade, sensitivity to story drift or floor acceleration).

### *4.1.1 Building Geometry*

Figure 4-1 is a screen capture from PACT illustrating the information used to define the building geometry, including:

- Number of stories
- Floor area at each story
- Length of perimeter walls at each story in each direction
- Height in each story
- Orientation of the framing systems (direction 1, direction 2)

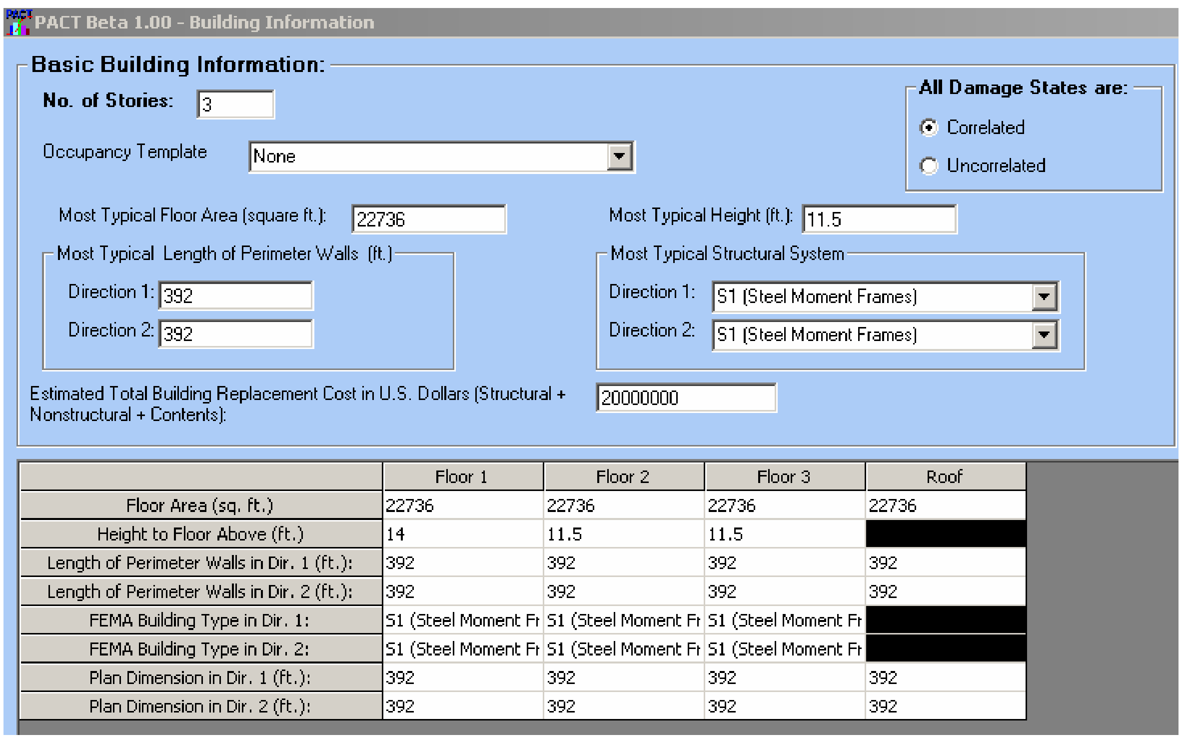



Figure 4-1 Representative building information

The data input process is simplified using the *Most Typical* quantities designation. In the figure, 11.5 has been entered as the *Most Typical Story Height*. The datasheet in the lower panel is interactive, and will initially populate the data for each story based on the *Most Typical* quantities provided. The *Most Typical* quantities can then be overridden at locations where the building configuration is not typical. In this example, the height for the first story has been changed from the *Most Typical* value of 11.5 feet to 14 feet.

### *4.1.2 Performance Groups*

Each structural and nonstructural component and system in a building will have a unique probability of damage for a given seismic input, based on its construction characteristics and location in the building. However, given the very large number of components and systems that comprise a building, it is both impractical and inappropriate to compute a loss for each individual component and each realization. To make the loss-computation tractable, components are assembled into sets called performance groups, which are then evaluated for probable performance or loss. Each performance group is composed of components that have the same damage states and fragilities, have the same consequences for casualties, downtime and repair cost (when damaged), and experience the same demands when the building undergoes shaking. As an example, all of the desk-mounted equipment on the second floor of a building may comprise one performance group because they are all subject to sliding based on the value of floor acceleration (assuming constant acceleration across the second floor of the building). Similarly, interior partitions in the east-west and north-south directions will comprise two separate performance groups because the damage to these walls will depend on east-west story drift or north-south story drift, respectively.

Performance groups are defined by the quantity, type and location of components, and by definition of component damage states, fragilities and effects on building losses.

A general strategy for grouping components is as follows:

1. **Identify the components that must be assigned to performance groups.** These are components that both can be both directly damaged and will have significant impact on building performance if they are damaged. Components that are inherently rugged and not subject to significant damage for credible levels of demand need not be considered. For example, in a steel moment-frame building the foundations may not be subject to damage since the steel moment frames might not be strong enough to force inelastic behavior into the footings. Also, some nonstructural components (e.g. toilet fixtures) can often be neglected as they are inherently rugged and generally not damaged by earthquake shaking.

   Some components may not be damaged directly, but can be affected by the required repairs of other components. For example, a conduit embedded in an interior partition, by itself is inherently rugged. However, if the partition is sufficiently damaged to require complete replacement, it will be necessary to replace the conduit, even though the conduit was not damaged. In this case, the wall, and not the conduit, would be assigned to the performance group.

2. **Group components in logical sets that are generally grouped in the normal design and construction process and specification sections**. A logical performance group is exterior cladding at a story level, including precast panels, glazing, storefront windows, and door systems. These items have both a design and construction relationship that leads to a logical association for monitoring damage. If the design or construction of one is changed, the change will likely affect all. Similarly, repair of damage for one component is likely to involve others in the group

3. **Group components into performance groups such that all components in the group experience approximately the same demands.** Damage for each component within a performance group is related to a single demand parameter for the entire group. For example, ceilings, fire-sprinklers, and light fixtures could be grouped together because damage to each of these components is related to floor acceleration. In some cases, the demand parameter used for a particular component in a group might not be the best parameter for that individual component.

4. **Group components that are located in a particular story.** Accelerations and drifts in each story are generally different. Whether components are acceleration-sensitive or drift-sensitive, they should be separated into different performance groups so that losses can be differentiated in each story.

5. **Group components that are sensitive to motion in a particular direction.** Interior partitions in a story that are aligned in the east-west direction are sensitive to east-west drift while partitions aligned in the north-south direction are sensitive to north-south drift. Note that more than one performance group can utilize the same demand parameter.

6. **Group components such that damage states and fragilities are logical for monitoring and repair**. Damage states are discussed below in Section 4.4. Interior partitions and exterior cladding at each floor level might be sensitive to drift, yet they are likely to be placed in different performance groups because the damage states could not be logically combined while maintaining a reasonable fragility relationship.

More than one performance group may utilize the same fragility relationships.  Groups might differ primarily because of the location of the components within the building.  For example, the fragility for drywall partitions at each floor level are identical, even though the demands at each floor level will be different, and therefore, the partitions at each floor level will be assigned to a different performance group.

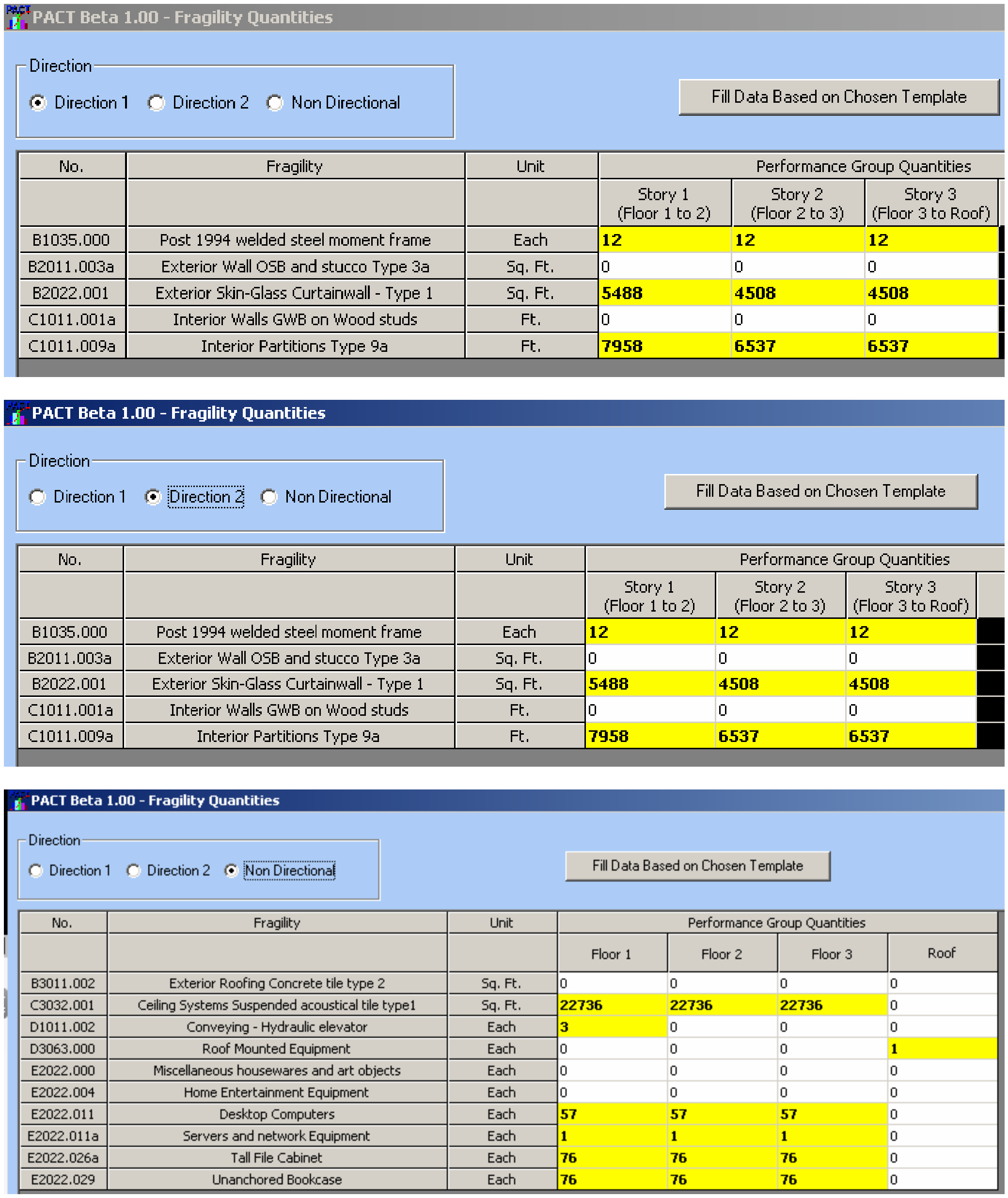
The description of damage repair actions associated with the damage states for one performance group will generally differ between the other performance groups having the same types of components (e.g., partitions at the $2^{nd}$ floor vs those at the $3^{rd}$ floor) because of the variable quantity of components and potential repairs in each group.  This means that the repair measure quantities for the detailed damage states will differ.  However, if these floor level quantities are the same, it is possible that the damage states for more than one group could be the same.

The repair measures used to define damage states are generally not unique to a single performance group nor to a single damage state.  In the example, the same repair measures occur in a number of different groups and damage states.

Figure 4-2 illustrates the minimum set of performance groups used to assess the performance of a 3-story, steel frame office structure.  A total of 32 different performance groups are used. These include:

- Three groups representing moment-resisting beam-column connections in the east-west direction (direction 1) in the first, second and third stories.  As shown in the figure, each of these three performance groups has a total of 12 beam-column connections.
- Three performance groups for the exterior cladding on the north and south faces of the building, at each of the three stories, with 4,508 square feet of cladding in each group.
- Three performance groups representing interior partitions aligned in the east-west direction of the building at the three stories, with a total of 6,537 square feet of wall in each group.  Thus there are a total of 9 performance groups that use east-west story drift as the parameter determining the extent of damage.
- In parallel with this, there are 9 additional performance groups that are used to account for moment connections, exterior walls and interior partitions at each of the three levels, which would be damaged by north-south story drift.

- Finally, there a series of 14 performance groups consisting of components that are sensitive to floor acceleration including mechanical equipment, desktop equipment, ceiling systems, unanchored book cases and filing cabinets. These groups are distributed at the 1st floor, 2nd floor, 3rd floor and roof as appropriate, with the quantity of each type of component in each performance group shown in the tables in the figure.



PACT Beta 1.00 - Fragility Quantities

Direction: (•) Direction 1 ( ) Direction 2 ( ) Non Directional

Fill Data Based on Chosen Template

| No. | Fragility | Unit | Performance Group Quantities | | |
|---|---|---|---|---|---|
| | | | Story 1 (Floor 1 to 2) | Story 2 (Floor 2 to 3) | Story 3 (Floor 3 to Roof) |
| B1035.000 | Post 1994 welded steel moment frame | Each | 12 | 12 | 12 |
| B2011.003a | Exterior Wall OSB and stucco Type 3a | Sq. Ft. | 0 | 0 | 0 |
| B2022.001 | Exterior Skin-Glass Curtainwall - Type 1 | Sq. Ft. | 5488 | 4508 | 4508 |
| C1011.001a | Interior Walls GWB on Wood studs | Ft. | 0 | 0 | 0 |
| C1011.009a | Interior Partitions Type 9a | Ft. | 7958 | 6537 | 6537 |

PACT Beta 1.00 - Fragility Quantities

Direction: ( ) Direction 1 (•) Direction 2 ( ) Non Directional

Fill Data Based on Chosen Template

| No. | Fragility | Unit | Performance Group Quantities | | |
|---|---|---|---|---|---|
| | | | Story 1 (Floor 1 to 2) | Story 2 (Floor 2 to 3) | Story 3 (Floor 3 to Roof) |
| B1035.000 | Post 1994 welded steel moment frame | Each | 12 | 12 | 12 |
| B2011.003a | Exterior Wall OSB and stucco Type 3a | Sq. Ft. | 0 | 0 | 0 |
| B2022.001 | Exterior Skin-Glass Curtainwall - Type 1 | Sq. Ft. | 5488 | 4508 | 4508 |
| C1011.001a | Interior Walls GWB on Wood studs | Ft. | 0 | 0 | 0 |
| C1011.009a | Interior Partitions Type 9a | Ft. | 7958 | 6537 | 6537 |

PACT Beta 1.00 - Fragility Quantities

Direction: ( ) Direction 1 ( ) Direction 2 (•) Non Directional

Fill Data Based on Chosen Template

| No. | Fragility | Unit | Performance Group Quantities | | | |
|---|---|---|---|---|---|---|
| | | | Floor 1 | Floor 2 | Floor 3 | Roof |
| B3011.002 | Exterior Roofing Concrete tile type 2 | Sq. Ft. | 0 | 0 | 0 | 0 |
| C3032.001 | Ceiling Systems Suspended acoustical tile type1 | Sq. Ft. | 22736 | 22736 | 22736 | 0 |
| D1011.002 | Conveying - Hydraulic elevator | Each | 3 | 0 | 0 | 0 |
| D3063.000 | Roof Mounted Equipment | Each | 0 | 0 | 0 | 1 |
| E2022.000 | Miscellaneous housewares and art objects | Each | 0 | 0 | 0 | 0 |
| E2022.004 | Home Entertainment Equipment | Each | 0 | 0 | 0 | 0 |
| E2022.011 | Desktop Computers | Each | 57 | 57 | 57 | 0 |
| E2022.011a | Servers and network Equipment | Each | 1 | 1 | 1 | 0 |
| E2022.026a | Tall File Cabinet | Each | 76 | 76 | 76 | 0 |
| E2022.029 | Unanchored Bookcase | Each | 76 | 76 | 76 | 0 |

Figure 4-2 Performance group quantities from PACT

As noted in the figure, each performance group has been assigned an identification number, of the form C3702.001. This categorization number is based on the NISTR system classification system for building components.

To simplify the assessment process, PACT allows the user a broad range of data entry options for defining a building and the relevant performance groups. The user can start from scratch and simply enter the performance group information directly using the datasheet in the lower panel of Figure 4-2. Alternatively, the user may specify the use of an occupancy template that is included in PACT. When this alternative is selected, PACT "seeds" the performance groups and quantities of performance groups for the building using typical quantities based on architectural standards.

For the specified occupancy, the template assigns default performance groups and related fragility specifications that likely would comprise such a building. For example, most buildings will have interior partitions and exterior walls. If the building is a hospital, it might also have specialized equipment among other components.

The occupancy templates also contain normative quantities for each performance group. These are used to determine quantities based on the specified building geometry. For example, the area of interior partitions in one orthogonal direction for a typical office building might be equal to 0.065 (normative quantity) times the floor height times the floor area. Examples of two occupancy templates are shown in Figures 4-3 and 4-4. PACT makes these calculations and fills out the quantity data shown in Figure 4-2 when a template is specified. A user can start with quantities based on a template, and then modify or add to the performance groups or quantities in each group.

Once the performance groups are specified and populated with quantities, the basic building is defined in sufficient terms to permit assessment of performance.

## 4.2 Characterization of Earthquake Ground Motions (Step 2)

Chapter 5 presents procedures for characterizing earthquake ground motion hazards for intensity-based, scenario-based and time-based assessments, respectively. For intensity-based and scenario-based assessments, ground motion intensity is represented either in the form of spectral accelerations, response spectra or suites of scaled ground motion pairs that are then used in analyses to generated vectors of demands that are used to calculate damage and loss.

| **Components** | | SMRF connections | Exterior Enclosure | Interior Partitions | Elevator | Ceilings | HVAC Equip. | Desk Top Computers | File Cabinets | Book Cases | Servers and network |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Units** | | ea | sq ft | sq ft | ea | sq ft | ea | ea | ea | ea | ea |
| **Fragility Specification** | | B1035.000 | B2022.001 | C1011.009a | D1011.002 | C3032.001 | D3063.000 | E2022.011 | E2022.026a | E2022.029 | E2022.011a |
| **Performance groups** | **Normative Quantities** | | | | | | | | | | |
| | **Direction 1 (drift demand)** | | | | | | | | | | |
| | Roof | | | | | | | | | | |
| | | Y | Direction 1 perimeter times story height | 0.025 times the floor area times story height | | | | | | | |
| | Third | | | | | | | | | | |
| | | Y | Direction 1 perimeter times story height | 0.025 times the floor area times story height | | | | | | | |
| | Second | | | | | | | | | | |
| | | user specified | Direction 1 perimeter times story height | 0.025 times the floor area times story height | | | | | | | |
| | First | | | | | | | | | | |
| | **Direction 2 (drift demand)** | | | | **Unidirectional (acceleration demand)** | | | | | | |
| | Roof | | | | | Floor area below | One set per 70,000sf total area | | | | |
| | | user specified | Direction 2 perimeter times story height | 0.025 times the floor area times story height | | | | | | | |
| | Third | | | | | Floor area below | | Floor area divided by 400 | Floor area divided by 300 | Floor area divided by 300 | One unit per 20,000sf flr area |
| | | user specified | Direction 2 perimeter times story height | 0.025 times the floor area times story height | | | | | | | |
| | Second | | | | | Floor area below | | Floor area divided by 400 | Floor area divided by 300 | Floor area divided by 300 | One unit per 20,000sf flr area |
| | | user specified | Direction 2 perimeter times story height | 0.025 times the floor area times story height | | | | | | | |
| | First | | | | one per 30,000 total floor area (except for single floor) | | | Floor area divided by 400 | Floor area divided by 300 | Floor area divided by 300 | One unit per 20,000sf flr area |

Figure 4-3 Performance group template for commercial office occupancy.

| Components | | Exterior Skin | Interior Walls | Roofing | Desk Top Computers | Home Entertainment | China/Art Objects |
|---|---|---|---|---|---|---|---|
| Units | | sq ft | sq ft | sq ft | unit | unit | unit |
| Fragility Specification | | B2011.003a | C1011.001a | B3011.002 | E2022.000 | E2022.004 | E2022.011 |
| Performance groups | Normative Quantities | | | | | | |
| | Direction 1 (drift demand) | | | | | | |
| | Roof | | | | | | |
| | | Direction 1 perimeter times story height | 0.065 times the floor area times story height | | | | |
| | Second | | | | | | |
| | | Direction 1 perimeter times story height | 0.065 times the floor area times story height | | | | |
| | First | | | | | | |
| | Direction 2 (drift demand) | | | Unidirectional (acceleration demand) | | | |
| | | | | | | | |
| | Roof | | | 1.15 times the upper floor area | | | |
| | | Direction 2 perimeter times story height | 0.065 times the floor area times story height | | | | |
| | Second | | | | 1.0 unit per 600sf | 1.0 unit per 600sf | 1.0 units per 1200sf |
| | | Direction 2 perimeter times story height | 0.065 times the floor area times story height | | | | |
| | First | | | | 1.0 set per 600sf | 1.0 set per 600sf | 1.0 units per 1200sf |

Figure 4-4 Performance group template for residential occupancy

For time-based assessments, it is necessary to calculate probable loss for a series of intensities, each of which has a defined mean annual frequency of exceedance, as defined by a seismic hazard curve for the building site. A minimum of 8 intensities, selected as described in Chapter 5 are used. Response analyses are performed at each of these intensity levels to develop the distribution of loss at each intensity of motion. A simple numerical integration is then performed over the range of mean annual frequency of exceedance to establish the annualized loss curve.

Figure 4-5 below is a screen capture from PACT that shows a sample seismic hazard curve for a site. The relationship between the mean annual frequency of exceedance ($y$ axis) and spectral acceleration at a selected period ($x$ axis) is established by the user outside of PACT (see Section 5.4 for details) and is input to PACT using the interactive panel shown in the upper left hand portion of the figure. PACT fits a differentiable curve to the hazard-curve data points. The user inputs the values of the spectral intensities used for scaling the pairs of ground motions (see Section 5.6.6) and PACT computes the absolute value of the tangent of the slope to the hazard curve at each value of intensity for use in loss computations.

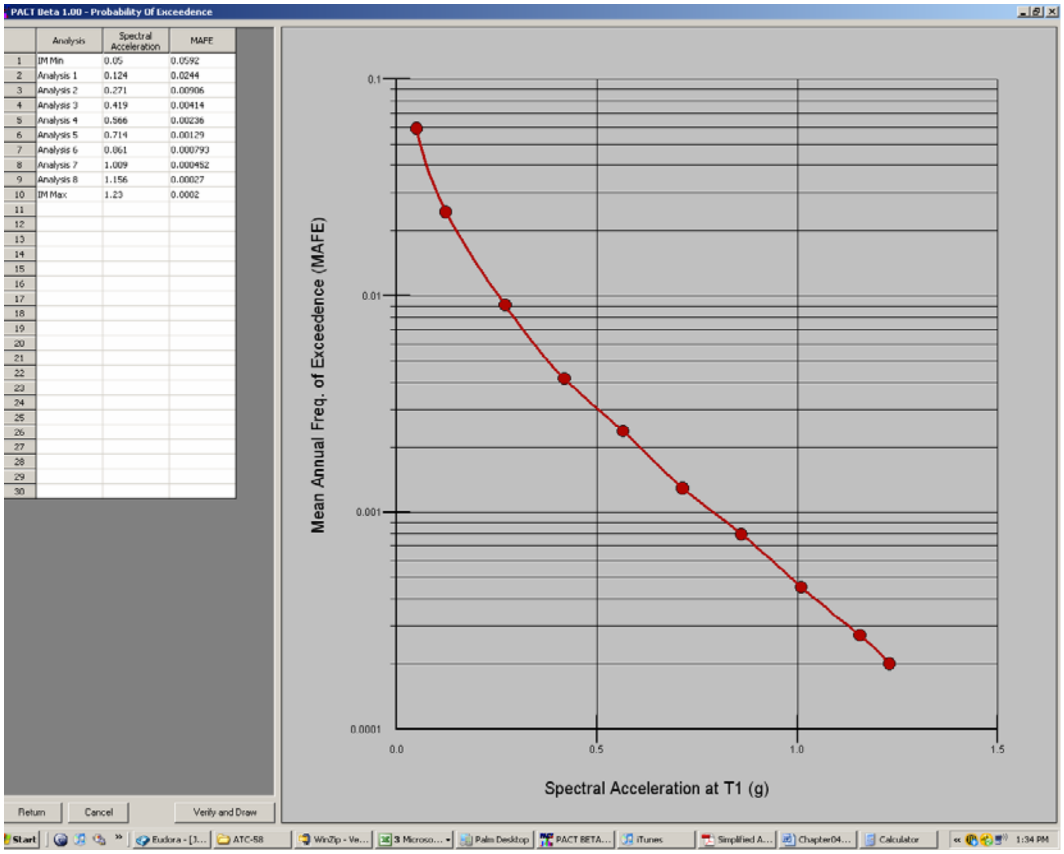


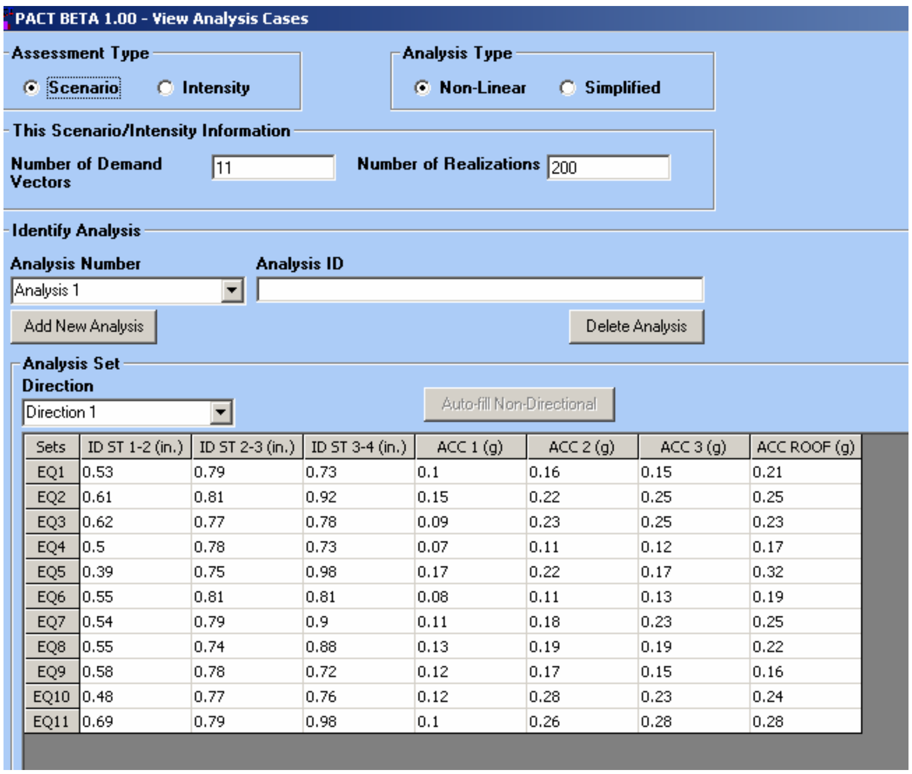

Figure 4-5 Sample seismic hazard curve in PACT

## 4.3 Building Response Simulation (Step 3)

Chapter 6 describes two methods of structural analysis that can be used to determine the probable distribution of structural response (e.g., floor accelerations and story drifts) for intensity-based, scenario-based or time-based assessments. Either nonlinear response-history analysis or simplified analysis can be used for this purpose. If nonlinear response-history analysis is used for intensity- and scenario-based assessments, multiple analyses are performed to define the statistical distribution of response quantities given the particular intensity of motion, or the occurrence of the particular scenario. A minimum of eleven analyses is recommended for each intensity. If nonlinear response-history analysis is used for a time-based assessment, a minimum of 8 intensities of ground motion spanning the hazard curve are recommended.

If simplified analyses are used, a single analysis is performed for intensity- and scenario-based assessments and default dispersions are applied to the response quantities obtained from these analyses. For a time-based assessment, a single analysis is performed at each of a minimum of 8 spectral intensities and default dispersions are applied to the resulting response quantities.

Figure 4-6 illustrates the input of analysis data to PACT for an intensity-based assessment. In this example, nonlinear response-history analysis was used, and a suite of 11 analyses were performed. For each analysis, labeled EQ1, EQ2, …, EQ11, the maximum absolute value of the story drift and floor acceleration at each level of the structure from a given analysis are assembled into a row vector as shown. The analysis data must be input for each of the two principal directions of response (direction 1 and direction 2).




| Sets | ID ST 1-2 (in.) | ID ST 2-3 (in.) | ID ST 3-4 (in.) | ACC 1 (g) | ACC 2 (g) | ACC 3 (g) | ACC ROOF (g) |
|---|---|---|---|---|---|---|---|
| EQ1 | 0.53 | 0.79 | 0.73 | 0.1 | 0.16 | 0.15 | 0.21 |
| EQ2 | 0.61 | 0.81 | 0.92 | 0.15 | 0.22 | 0.25 | 0.25 |
| EQ3 | 0.62 | 0.77 | 0.78 | 0.09 | 0.23 | 0.25 | 0.23 |
| EQ4 | 0.5 | 0.78 | 0.73 | 0.07 | 0.11 | 0.12 | 0.17 |
| EQ5 | 0.39 | 0.75 | 0.98 | 0.17 | 0.22 | 0.17 | 0.32 |
| EQ6 | 0.55 | 0.81 | 0.81 | 0.08 | 0.11 | 0.13 | 0.19 |
| EQ7 | 0.54 | 0.79 | 0.9 | 0.11 | 0.18 | 0.23 | 0.25 |
| EQ8 | 0.55 | 0.74 | 0.88 | 0.13 | 0.19 | 0.19 | 0.22 |
| EQ9 | 0.58 | 0.78 | 0.72 | 0.12 | 0.17 | 0.15 | 0.16 |
| EQ10 | 0.48 | 0.77 | 0.76 | 0.12 | 0.28 | 0.23 | 0.24 |
| EQ11 | 0.69 | 0.79 | 0.98 | 0.1 | 0.26 | 0.28 | 0.28 |

Figure 4-6 Typical response data input to PACT

A statistical distribution of losses is developed using a series of realizations that are statistically consistent with the distribution of structural response data obtained from analysis. Each realization represents one possible state of response given an intensity of ground motion. Appendix F describes how these realizations are generated. It is necessary to specify how many realizations should be performed. At present, 200 realizations are recommended.

**Reader Note:** Each realization is generated and rapidly evaluated within PACT. There is no penalty to specifying a large number of realizations. At the present time, 200 has been selected as a number that seems to provide reasonable balance between smoothness in the generated loss distribution and computing economy. This number will be evaluated and refined as the project progresses.

## 4.4 Assessment of Damage (Step 4)

The basic building model includes the definition of the various types of structural and nonstructural components that make up the building, which could be damaged by earthquake shaking, and could result in potential repair costs, downtime and casualties. In addition to identifying the important components, it is also necessary to identify their likely damage states, the fragilities associated with these damage states, and the consequences of each damage state.

### *4.4.1 Fragility Specifications*

In these procedures, damageability data for each performance group are assembled in a format termed a fragility specification. Figure 4-7 presents a sample fragility specification for reinforced concrete shear walls. As illustrated in the figure, key data associated with each fragility specification include:

1. **Fragility Number**: this is a unique identifier that follows the Uniformat-II system of classification.
2. **Name**: a short descriptive name for the fragility specification.
3. **Basic composition**: a description of the component groups to which the fragility specification applies.
4. **Units for basic quantities**: the units that are used to characterize how many components of the given type are present in the building. These may include linear feet, square feet, each, etc.
5. **Damage State Descriptions**: a description of each Damage State that indicates the types of damage represented by the damage state and includes:
    a. **Illustrations**: a drawing or photograph of typical damage to a component for each Damage State.
    b. **Demand Parameter**: identification of the demand parameter, for example story drift, or floor acceleration, used to predict damage
    c. **Median demands**: for each damage state, the median value of the demand parameter at which the damage state is likely to initiate.

**Fragility Specification**

**B1044.000 Reinforced Concrete Shearwalls**

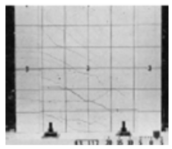
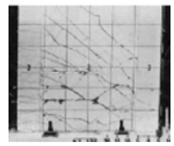
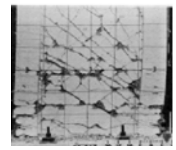
| | | | |
|---|---|---|---|
| BASIC COMPOSITION | Reinforced concrete and finishes both sides | | |
| Units for basic quantities | Square feet of wall area | | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | | |
| | DS1 | DS2 | DS3 |
| DESCRIPTION | Flexural cracks < 3/16"<br>Shear (diagonal) cracks < 1/16" | Flexural cracks > 1/4"<br>Shear (diagonal) cracks > 1/8" | Max. crack widths >3/8"<br>Significant spalling/ loose cover |
| ILLUSTRATION<br>(example photo or drawing) |  |  |  |
| MEDIAN DEMAND | 1.5% | 3.0% | 5.0% |
| BETA | 0.2 | 0.3 | 0.4 |
| CORRELATION (%) | 70% | | |
| DAMAGE FUNCTIONS | Patch cracks each side with caulk<br>Paint each side | Remove loose concrete<br>Patch spalls with NS grout<br>Patch cracks each side with caulk<br>Paint each side | Shore<br>Demo existing wall<br>Replace<br>Patch and paint |
| CONSEQUENCE FUNCTION | | | |
| Max. consequence up to lower quantity | $4.00 per sq ft up to 800 sq ft | $10.00 per sq ft up to 800 sq ft | $50.00 per sq ft up to 200 sq ft |
| Min consequence over upper quantity | $2.00 per sq ft over 4000 sq ft | $5.00 per sq ft over to 4000 sq ft | $30.00 per sq ft over 2000 sq ft |
| Beta (consequence) | 0.2 | 0.3 | 0.3 |
| TIMEFRAME TO ADDRESS CONSEQUENCES | days | weeks | months |

Figure 4-7 Sample fragility specification

d. **Damage state dispersion**: the dispersion for each damage state.

e. **Damage consequence**: a description of the consequences that are associated with each damage state; for direct loss and downtime calculations, these functions are in the form of itemized repairs that would be required to return the materials and components to the their pre-earthquake condition. For casualty losses, these are in the form of a description of the building area in which casualties can occur and the probability of incurring casualties per exposed person.

f. **Consequence functions**: specify the consequences of the damage in terms of losses that apply to each damage state. These are applied to a global damage state generated for each realization of losses and are discussed in more detail in the following subsection. For direct loss, these functions are the unit costs of repairs as follows:

   i. Lower quantity: the minimum quantity below which there is no cost discount

   ii. Max. cost: maximum cost per unit without discount

iii. Upper quantity: maximum quantity above which there is no further cost discount

iv. Min. cost: minimum cost per unit with maximum discount

v. Dispersion: the dispersion associated with the consequence function

Figures 4-8 and 4-9 illustrate how typical fragility specifications are input and displayed in PACT. For each fragility specification there is a series of pages that display data. The first page, shown in Figure 4-8, describes the fragility specification and illustrates the fragility functions for each damage state. A second page for each damage state, reproduced in Figure 4-9, presents detailed information for the damage state.

If the user has specified an occupancy template in the building definition, the default fragilities are automatically assigned to the performance groups. Users can specify the use of alternative fragility specifications if appropriate, or can define their own fragility specifications.

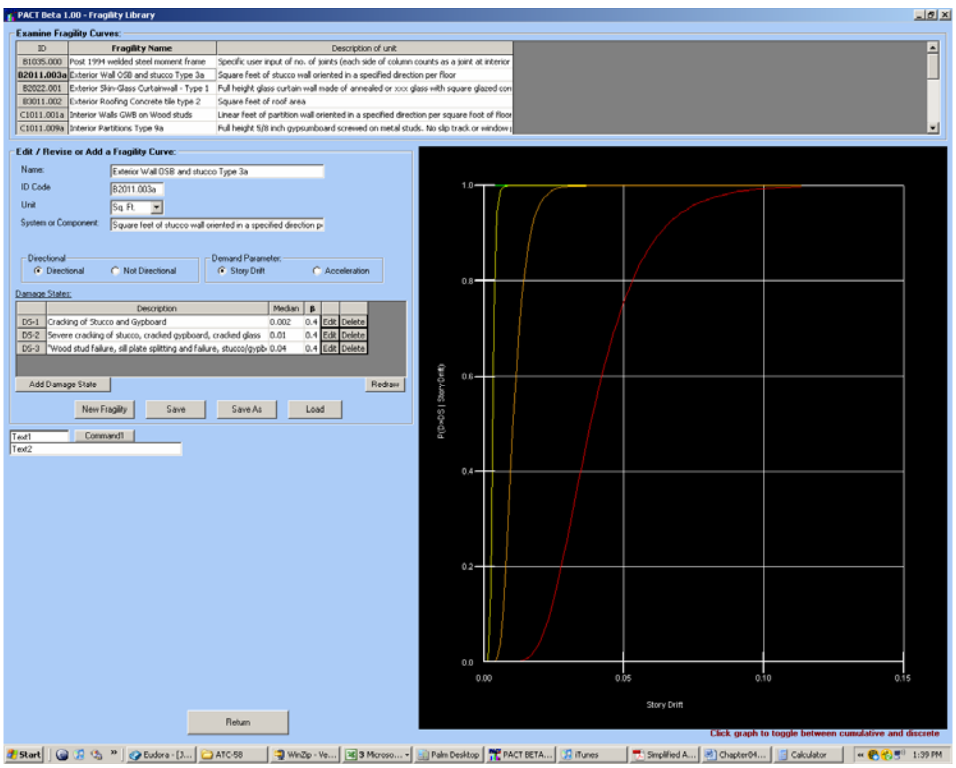



Figure 4-8 Fragility specification page from PACT

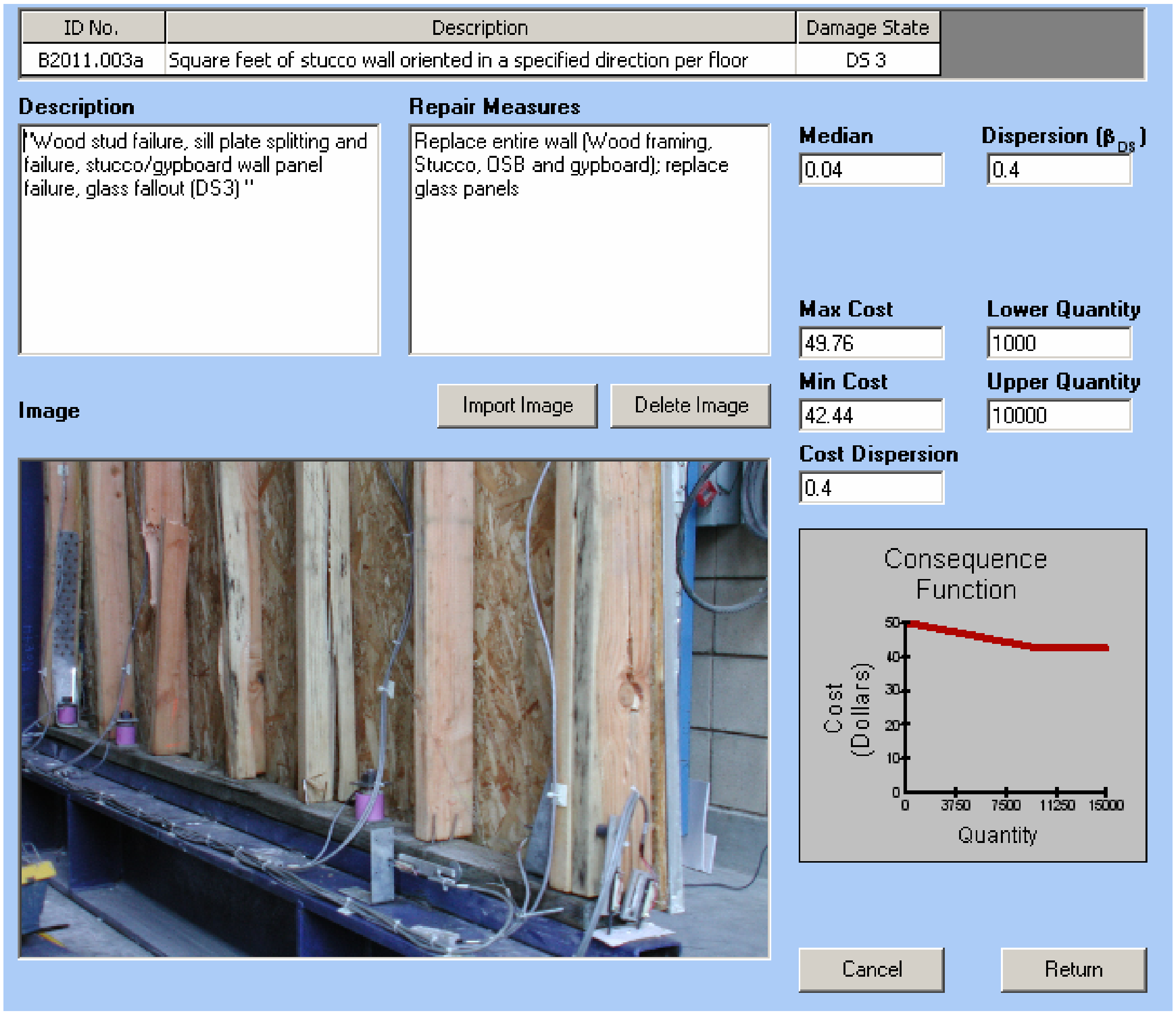



Figure 4-9 Typical damage state description from PACT

**Reader Note**. The current version of PACT includes only a limited set of fragility functions as indicated in the table below. The user may add or modify fragility specifications as illustrated in Figures 4-8 and 4-9.

| Number | Description |
|---|---|
| B1035.000 | Steel Connections, moment resisting, post 1994 |
| B2011.003a | Exterior shearwall, 7/16 OSB, 2x4, 16" OC, 7/8" stucco ext, GWB interior side |
| B2022.001 | Highrise curtain-wall systems with annealed glass |
| B3011.002 | Concrete, clay, and slate roofing tiles that are individually fastened to the roof sheathing |
| C1011.001a | GWB partition, no structural sheathing, 1/2" GWB two sides, 2x4, 16" OC |
| C1011.009a | Drywall finish, 5/8-in., 2 sides, on 3-5/8-in metal stud, screws |
| C3032.001 | Lightweight acoustical ceiling 4'-x-2' aluminum tee-bar grid |
| C3033.001 | GWB on wood joists |
| D1011.002 | Hydraulic passenger elevators |
| D3063.000 | Heating/Cooling Air Handling Units |
| E2022.000 | Furniture & Accessories |
| E2022.004 | Household entertainment equipment |
| E2022.011 | Desktop computer system unit and CRT monitor |
| E2022.011a | Computer system servers and network equipment |
| E2022.026a | Tall file cabinets |
| E2022.029 | Unanchored bookcases |

### *4.4.2 Consequence Functions*

Although each individual performance group is assigned a fragility specification including consequence functions, the losses in a building for any realization depend on the overall total damage to the building. This means that the consequence functions are applied to the total damage state for the building as opposed to the damage determined for an individual performance group. The characteristics of consequence functions depend on the type of loss and are summarized in the following subsections.

**Direct Loss**

Each building damage state comprises a detailed description of the building's condition for a given ground motion or, realization in terms of repairs required to return the building to pre-earthquake condition. This description could be given to a contractor to form the basis for an estimate of the costs to repair the building and replace its contents that may have been damaged. When a contractor makes such an estimate, the unit costs applied to the various repair quantities depend on the total quantities of basic repair measures. In some instances (e.g. scaffolding, protection of finishes, clean-up), costs are distributed to more than a single repair measure. Contractors' overhead and profit depend on the total amount of work and the type of tradesmen and subcontractors required. In effect, the contractor applies a "direct economic loss consequence function" to the damage to calculate the loss. Consequence functions for direct economic losses use the building damage state to determine the need for shoring, staging, finish protection, cleaning, and other general condition costs; the costs associated with contractor overhead and profit and indirect project costs including design services, fees and permits as well as the costs of the actual labor and materials associated with the individual repairs required.

Consequence functions for direct economic loss should account for the effect of quantities on unit price. These are of the general form illustrated in Figure 3-6. For small quantities the unit cost is constant at a maximum. Beyond a certain quantity the cost diminishes as the contractor can take advantage of economies of scale, until ultimately, a minimum unit cost for large quantity repairs is reached. Since costs are subject to uncertainty from market conditions, contractor bidding strategy, and other factors, unit costs are assigned a median value and a dispersion, $\beta_c$.

**Casualties**

Casualty losses, measured in deaths and serious injuries in buildings can be caused by several conditions. One of these is collapse. Local collapse occurs when a portion of a building (e.g., a single interior column) loses ability to support gravity loads, and the floor supported by the column collapses on to the next level. This may trigger additional collapses, depending on the type of construction. General collapse typically occurs as a result of excessive lateral deformation in a story or stories, resulting in P-delta effects that exceed the lateral resistance of the structure. General collapse can occur in a single story, or the entire structure.

In addition to collapse, casualties can be caused by falling objects. Each building damage state, generated for a specific ground motion or realization, includes a complete description of the damage that has occurred that can cause casualties, be it structural collapse or toppling of a bookshelf or heavy piece of equipment.

The casualty consequence function for each damage state with the potential for resulting in casualties consists of a median number of casualties per person present in the affected area, and the associated dispersion. To obtain the casualty potential of a given building damage state, it is necessary to randomly select a number of casualties per building occupant consistent with the median and dispersion values contained in the casualty consequence function for the damage state, then to modify this value by the number of occupants present in the affected area.

Determining the number of occupants present is complex. Many buildings will be occupied during some days (e.g. Monday through Friday) and not occupied during other days, such as weekends and holidays. Most buildings will have high occupancies during certain hours of the day (e.g., 8:00 am to 5:00 pm for commercial buildings, 7:00 pm to 7:00 am for residences), and could be unoccupied, or close to unoccupied, during other hours.

For each ground motion or realization, a random selection is made as to the day of the week and hour of the day at which the realization or ground motion occurs. Based on the building occupancy, functions of the average number of persons present per square foot, on a given day and time, together with dispersion on this value, are used to determine the number of persons at risk at the time of the ground motion or realization. This is then applied to the randomly selected casualties per occupant for the realization to compute the total number of casualties for the realization.

**Downtime**

Downtime losses can be of two basic types. The first of these is initial loss of building use that occurs because damage has rendered a building unsuitable for occupancy. The second is time associated with conducting repairs to the building. Downtime for initial loss of building use is difficult to predict, and depends on the ability of the building owner to cope with the damage and mobilize the necessary economic resources, design professionals and contractors to effect repairs. If a building owner has marginal economic mean, or lacks insurance, this type of downtime can last from months to years. Even in cases where adequate insurance exists, negotiations on appropriate costs of repair and coverage can increase the initial phase of downtime to many months.

Downtime associated with designing and constructing repairs is closely related to direct economic losses, and the "contractor" could determine the amount of time required to complete the repairs. There are additional considerations including whether the building will be occupied during construction that can affect downtime associated with repairs.

## 4.5 Computation of Losses (Step 5)

After all of the data relating to the building definition, seismic hazard, structural response, and damageability are assembled, PACT generates a number of realizations for Monte Carlo analysis to determine losses.

### *4.5.1 Intensity- and Scenario-Based Assessments*

Sample results from an intensity- or scenario-based loss assessment are shown in Figure 4-10. The analysis on which the losses are based is specified in the dialogue box in the upper left-hand corner. The performance groups to be included in the loss calculation are specified in the dialogue boxes below. The first box lists the various performance groups for which losses can be individually viewed, organized by fragility specification. The second box allows specification of the direction of shaking. The lower box designates the story within the levels of the building for which the losses are to be displayed. In Figure 4-10 all performance groups at all floors in each direction are included. The lower panel represents the total losses for the building for the given intensity or scenario.

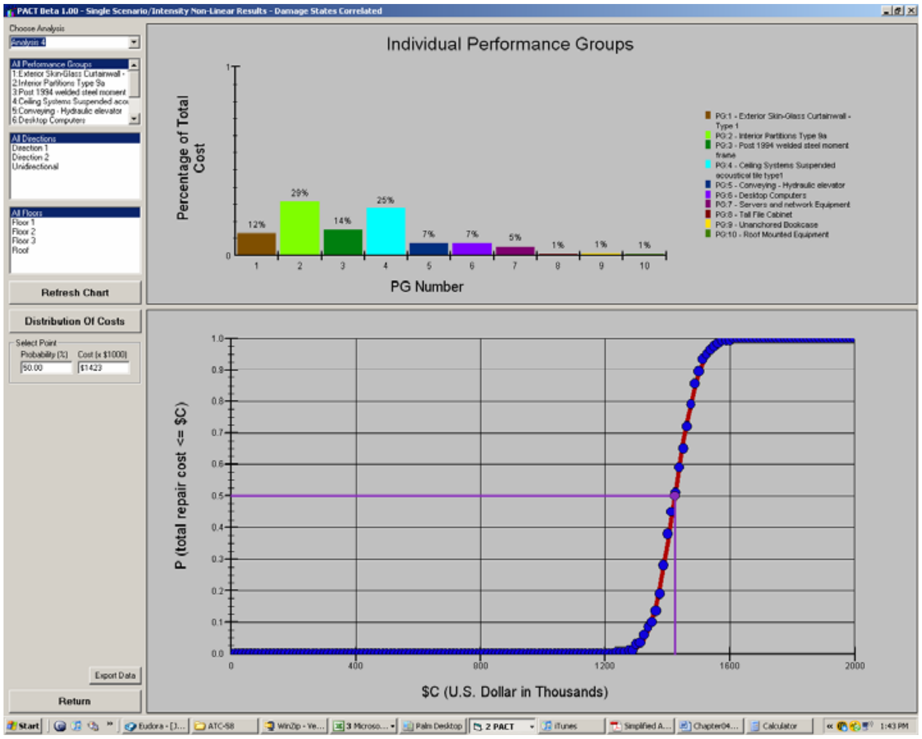



Figure 4-10 Sample loss calculations from PACT for a scenario or intensity-based assessment.

In the lower left corner of the display screen are controls that facilitate the reading of losses from the curve. The cursor tool indicates the loss and related probability as a coordinate point. If a loss value is typed into the dialog box, the corresponding probability will appear in the adjacent box and the cursor will move to the designated point on the plot. Alternatively, the probability can be entered and the loss will be provided automatically in the adjacent box. Also, it is possible to simply move the cursor to the desired point along the graph to obtain readouts in the output boxes. The vertical bar chart above the loss curve shows the relative contributions of the individual performance groups to the loss specified by the cursor. The bars change as the cursor is moved along the curve.

### *4.5.2 Time-Based Assessments*

A time-based assessment generates the probability of losses from all intensities of shaking. The results of this procedure are illustrated in Figure 4-11. The vertical axis for a time-based assessment is the annual probability that the loss on the horizontal axis is exceeded. The area under the loss curve is the average annualized loss, which is also output by PACT.

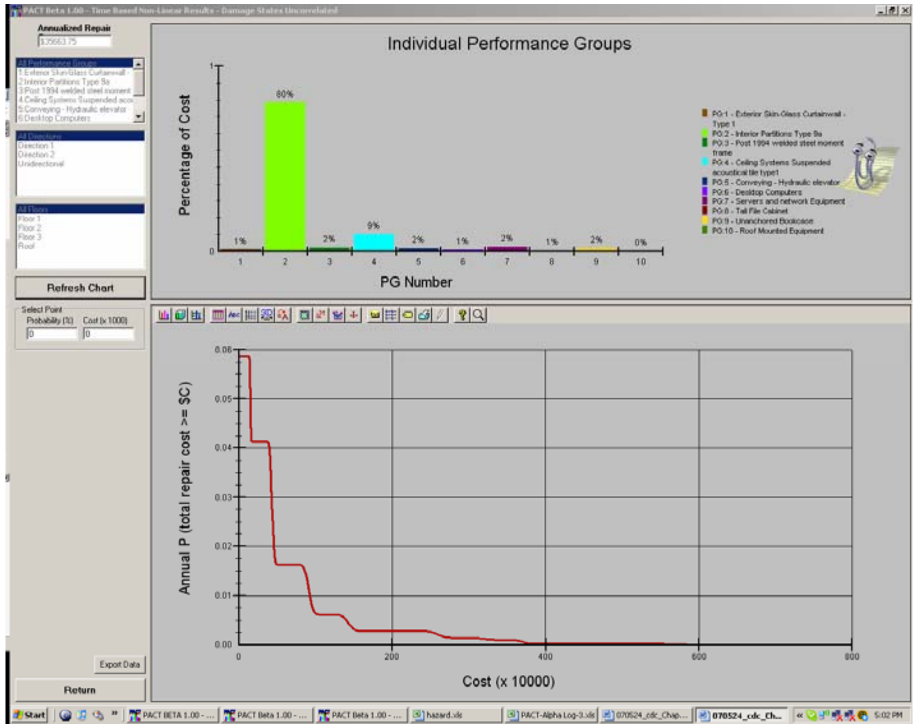



Figure 4-11 Sample loss calculations from PACT for a time-based assessment.

# Chapter 5

# Hazards

## 5.1 Introduction

This section presents a recommended set of procedures for characterizing horizontal and vertical earthquake ground shaking for response analysis performed as part of a performance assessment.

The ground motion representations presented below are for unidirectional horizontal shaking. The effects of bi-directional horizontal shaking are considered by applying the earthquake shaking effects (spectral ordinate or acceleration time series) along each principal axis of the building simultaneously. When response history analysis is performed the bi-directional shaking is applied as pairs of recorded or simulated ground motion histories scaled to represent the ground shaking intensity level. Adjustments for soil type are made using the procedures of Section 5.3.

**Reader Note**: Procedures for characterizing two- and three-component earthquake histories are under development at this time, and will be published in later drafts of these *Guidelines*.

## 5.2 Building Location and Site Conditions

### *5.2.1 Seismic Environment and Hazard*

The characterization of seismic hazard due to ground shaking is dependent on the location of the site with respect to causative faults, the regional and site-specific geologic characteristics, local topographic conditions, and the chosen type of assessment.

The performance assessment procedures of these *Guidelines* address earthquake shaking hazards only. However, other seismic hazards may exist at a building site that could damage the building regardless of its ability to resist ground shaking, including fault rupture, liquefaction, lateral spreading, landslides, and inundation from offsite effects such as dam failure or tsunami (Kramer, 1996). Each of these hazards could contribute substantially to loss. As a minimum, a qualitative assessment of the potential impact of these hazards should be made and if the effects are found to be potentially significant, this should be noted and addressed.

Local topographic conditions (e.g., hills, valleys, canyons) can modify the amplitude, frequency content and duration of earthquake shaking relative to that expected at a flat, level site. Finite-element and finite-difference methods can be used to characterize the influence of topographic effects on earthquake shaking but a treatment of topographic effects is beyond the scope of these *Guidelines*.

### 5.2.2 Location

The exact location of the building site must be identified to characterize the seismic hazard for both scenario and time-based assessments. For scenario assessments, the distance from the building site to the causative fault must be known to utilize one or more attenuation relationships. The latitude and longitude of the building site must be established for time-based estimates in order to derive the seismic hazard curve. Three decimal fraction places are typically sufficient to identify the latitude and longitude.

### 5.2.3 Site Soil and Topographic Conditions

The properties of the soil (rock) at the building site must be defined for both scenario and time-based assessments and should also generally be established for an intensity-based assessment to permit an appropriate spectral shape and amplitude to be used to characterize the hazard. The soil should be characterized either by values of variables such as average shear wave velocity in the upper 30 meters that are consistent with the attenuation relationship(s) used for the hazard characterization, or by site class, so that bedrock (USGS B/C boundary) spectral demands can be adjusted for local site conditions using soil factors (see Section 5.3).

## 5.3 Spectral Adjustments for Soil Conditions

Seismic hazards are often presented for a reference site that is underlain by soft rock with a shear wave velocity of 760 *m/sec*. These demands, however calculated, must be adjusted for the local geology at the building site. One acceptable procedure for adjusting spectral demand for soil type is described below. The procedure can be used to adjust spectral demand for all three types of performance assessment. Other acceptable procedures include 1-dimensional site response analysis and soil-foundation-structure interaction analysis, as described in Appendix B.

A soil classification system is presented in Chapter 20 of ASCE-7-05. Under these procedures, the soil at a building site is classified as either A, B, C, D, E or F, depending upon its mechanical properties in the upper 30 *m* of the soil column immediately below the building foundation. The reference shear wave velocity of 760 *m/sec* lies at the boundary between Site Class B and C (the B/C boundary). In the absence of information on the soil type at a building site, and assuming that the building site is not located close to a large body of water, Site Class D (stiff soil) should be assumed for calculation.

**Sidebar: Site Characteristics.** The procedures of ASCE 7-05 permit the use of Site Class D when the site soil conditions are undefined because the characteristics of the response spectrum derived using Site Class D assumptions are generally more conservative for design than those derived using Site Class A, B or C and also because conditions associated with Site Classes E and F are relatively rare. Unlike design, in the case of performance assessments, it is desirable neither to be excessively conservative or liberal in one's assumptions. Therefore, it is recommended that sufficient investigation of the characteristics of a site be made to determine the site class, rather than using the default Site Class D assumptions.

Once the soil type at the building site has been identified, correct the USGS spectral demands computed at the B/C boundary to account for amplification of ground motion that is associated with the strength and stiffness of the soil column at the site. The correction factors presented in Tables 11.4-1 ($F_a$) and 11.4-2 ($F_v$) of ASCE-7-05 can be used for this purpose, where Table 11-4-1 presents correction factors for short-period (0.2 second) spectral response and Table 11.4-2 presents factors for long-period (1 second) response. Values of the 5% damped spectral acceleration at periods of 0.2 second and 1.0 second at the B/C boundary, for the assumed intensity of ground motion, are required to compute the correction factors.

To use the ASCE-7-05 tables to correct for soil type, the Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameters ($S_s$ and $S_1$ per ASCE-7-05) are replaced with the values of spectral acceleration at 0.2 second ($S_{0.2}$) and 1.0 second ($S_{1.0}$) computed for the selected intensity of motion. Values of the site coefficients, $F_a$ and $F_v$, are then selected from the tables, consistent with the soil type at the site. Adjusted short- and long-period spectral parameters are computed using

$$S_{a0.2} = F_a S_{0.2} \qquad (5\text{-}1)$$

$$S_{a1.0} = F_v S_{1.0} \qquad (5\text{-}2)$$

and used to calculate a transition period: $T_s = S_{a1.0} / S_{a0.2}$. If $T_1 < T_s$, the spectral demand computed at the B/C boundary is multiplied by $F_a$, otherwise it is multiplied by $F_v$.

## 5.4 USGS-based Ground Motion Computations

### *5.4.1 Ground Motion Calculator*

A USGS ground motion calculator is available as a downloadable Java application at http://earthquake.usgs.gov/research/hazmaps/design/. This applet permits the calculation of seismic hazards for sites in the conterminous 48 States. The use of this calculator to characterize seismic hazard for time-based (Section 5.4.2) and intensity-based (Section 5.4.3) assessments is described below.

### *5.4.2 Time-Based Hazard Calculations*

A screen capture is presented in Figure 5-1 that illustrates the use of the ground motion calculator for time-based assessments. The application is used here to develop probabilistic seismic hazard curves that plot spectral demand at selected periods as a function of the mean annual frequency of exceedance for this spectral demand.

The calculation illustrated in the figure is for a representative site in San Francisco (latitude 37.800 degrees and longitude -122.400 degrees). Spectral demands are computed for a reference shear wave velocity of 760 *m/sec*. (See Section 5.3 for procedures to adjust spectral demand for alternate site conditions.) In the window on the right hand side of the figure, the values of spectral acceleration at 1.0 second are presented as a function of Frequency of Exceedance per Year (mean annual frequency of exceedance). The Java application enables calculation of seismic hazard curves at period of 0, 0.1, 0.2, 0.3, 0.5, 1.0 and 2.0 seconds for a wide range of annual frequencies of exceedance (or return periods). Although several alternative hazard data sets are available within the application, the 2002 USGS data should be used for hazard computations for basic assessment.

**Sidebar: USGS seismic hazard maps.** The USGS periodically update their seismic hazard maps. The next update is scheduled for early 2008. Those maps should be used in lieu of the 2002 maps once released. Future versions of the calculator might have different output screens from those presented in these *Guidelines*.

The actions required to generate the screen capture of Figure 5-1 are as follows:

1. Set the pull-down menu at the top of the screen to *Probabilistic Hazard Curves*
2. Set the top pull-down menu on the left hand side to appropriate *Geographic Region*

3. Set the 2nd pull-down menu on the left hand side, *Data Edition*, to the appropriate national seismic hazard data set; the most current is the 2002 set (*2002 Data*)

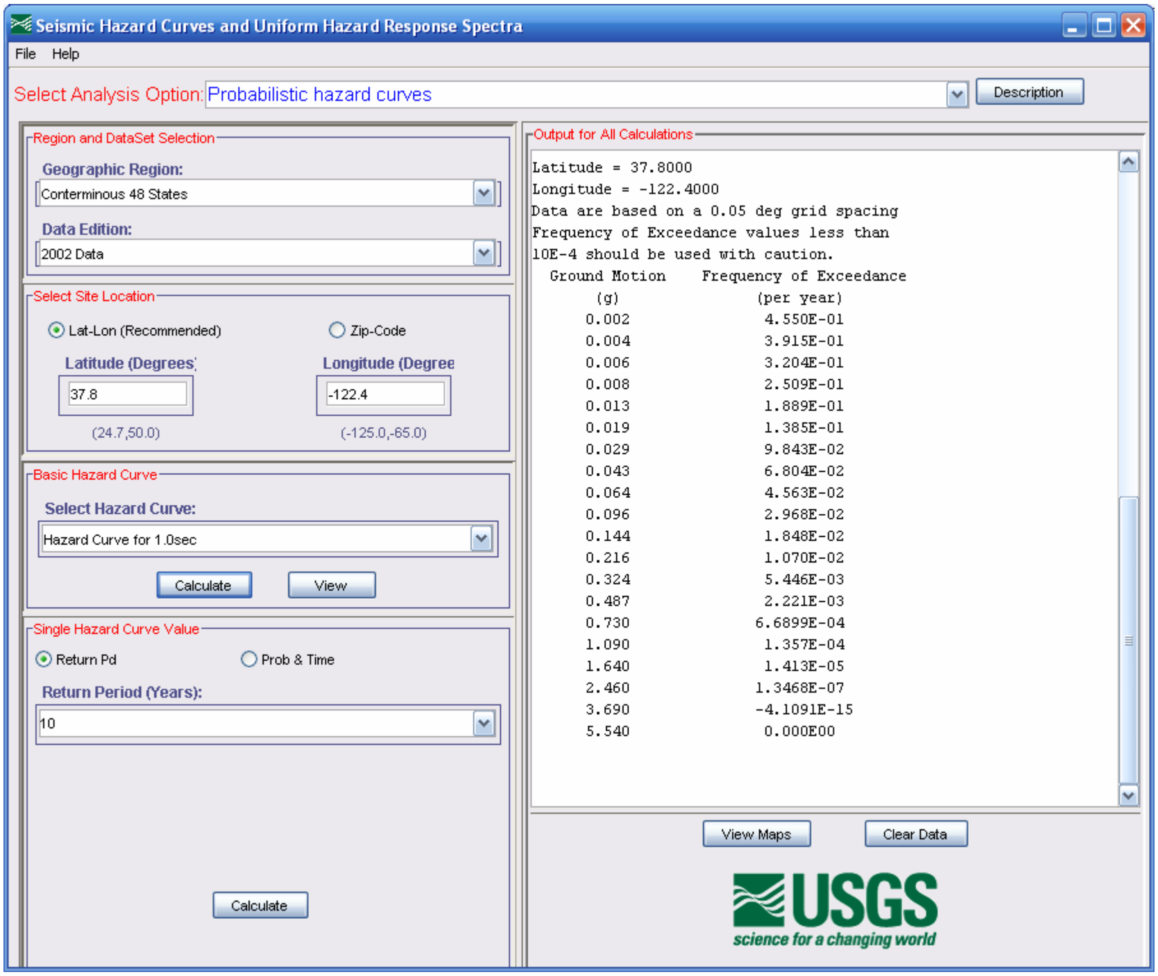



Figure 5-1 Screen capture from the USGS ground motion calculator

4. In the two input boxes at the middle of the left hand side of the page, enter the *Latitude* and *Longitude* for the building site. This data should be entered to not less than 3 decimal fraction places.

5. In the next pull-down menu, below the latitude and longitude data entry, enter the structural period for which seismic hazard data is desired. Choices include (0 second (peak ground acceleration), 0.1 second, 0.2 second, 0.3 second, 0.5 second, 1 second and 2 second.

6. Left-click on the *Calculate* button

7. Left click on the *View* button to bring up a plot of the seismic hazard data

Figure 5-2 presents the seismic hazard curve for the screen capture of Figure 5-1, plotted in log-linear format: spectral acceleration at a period of 1.0 second (*x*-axis) versus mean annual frequency of exceedance (*y*-axis). Note that the default plot format for the application is a linear plot. In order to obtain the plot in the log-linear format, use the plot options menu available at the data screen.

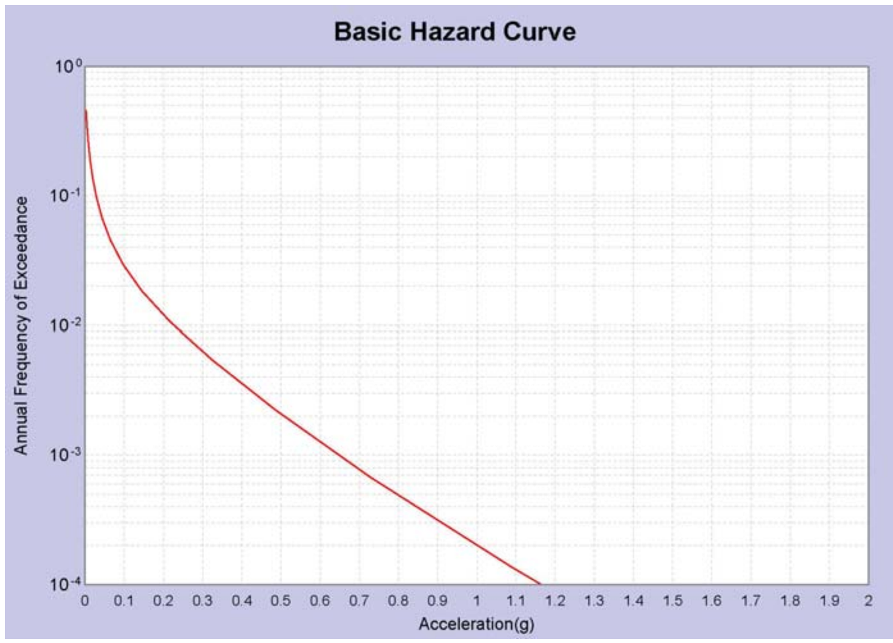



Figure 5-2 Hazard curve for data shown in Figure 5-1, plot in log-linear format

Spectral demands at selected mean annual frequencies of exceedance and periods of 0, 0.1, 0.2, 0.3, 0.5, 1.0 and 2.0 seconds can be generated using the USGS ground motion calculator. For values of $T_1$ between these periods, linear interpolation can be used to compute the spectral acceleration. For values of $T_1$ greater than 2 seconds but less than or equal to 5 seconds, the spectral acceleration can be computed as

$$S_a(T_1) = \frac{2S_a(T = 2\,\text{sec})}{T_1} \quad (2 < T_1 \leq 5\,\text{sec}) \tag{5-3}$$

### 5.4.3 Intensity-based hazard calculations

The USGS ground motion calculator can be used to generate a spectrum for intensity-based assessments although only $S_a(T_1)$ is needed for response analysis using the procedures of Chapter 6. The calculator can generate uniform hazard spectra (for 2% and 10% probability of exceedance in 50 years) and Maximum Considered Earthquake and Design Basis Earthquake spectra consistent with ASCE and other building codes and standards. Figures 5-3, 5-4 and 5-5 and the steps below illustrate the process.

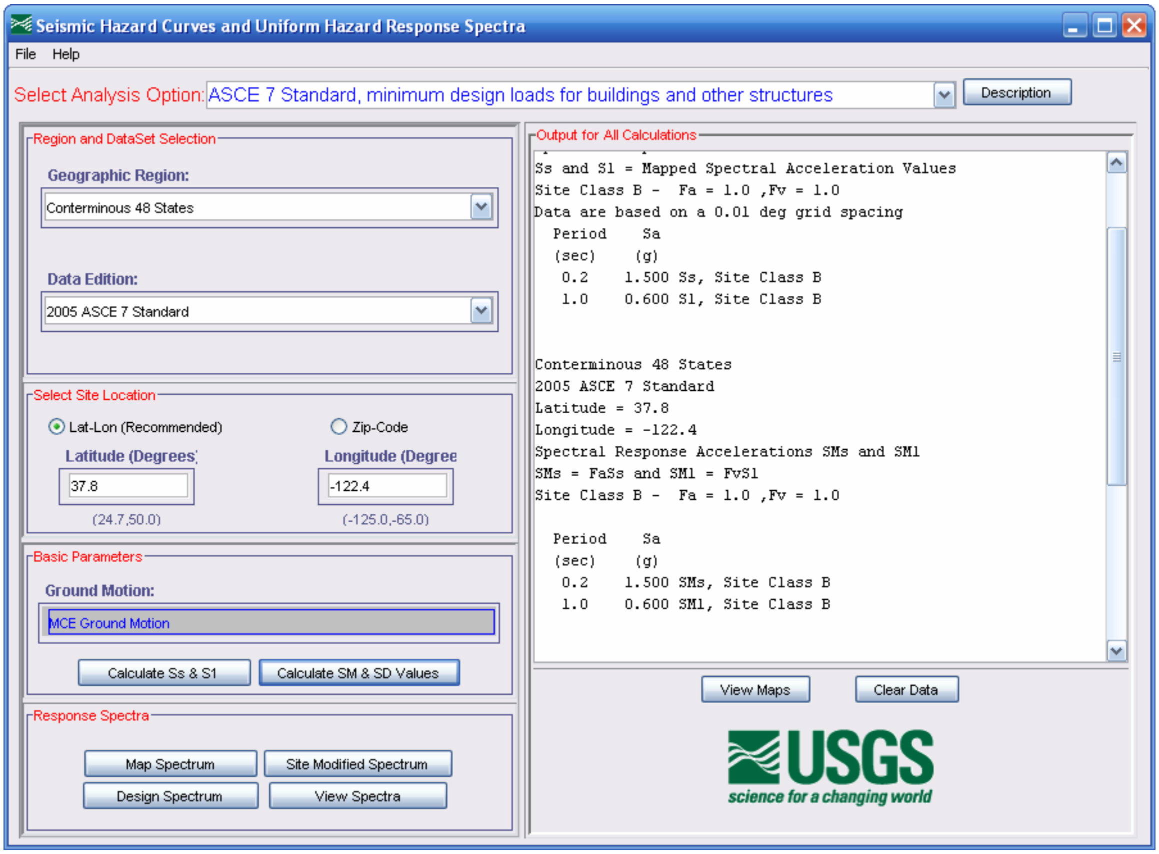



Figure 5-3 Data input to USGS ground motion calculator to generate a response spectrum

Follow the following steps:

1. In the top pull-down menu, select the type of spectrum desired, for example, that of the *ASCE Standard 7*.
2. In the upper left hand corner pull-down menu, enter the *Geographic Region* for the site
3. In the second pull-down menu on the left, select the desired *Data Set*.
4. In the data boxes on the middle of the left hand side, enter the Latitude and *Longitude* for the building site.
5. Left-click on the button labeled *Calculate Ss and S1*. The calculated values of the spectral response acceleration for Site Class B at periods of 0.2 second and at 1 second will appear in the right hand panel.
6. Left-click on the button labeled *Calculate SM & SD Values*.

After step 6 above is completed, a window like that shown in Figure 5-4 will appear and the user can adjust the spectrum for non-reference soil conditions at the site following the procedures described previously.

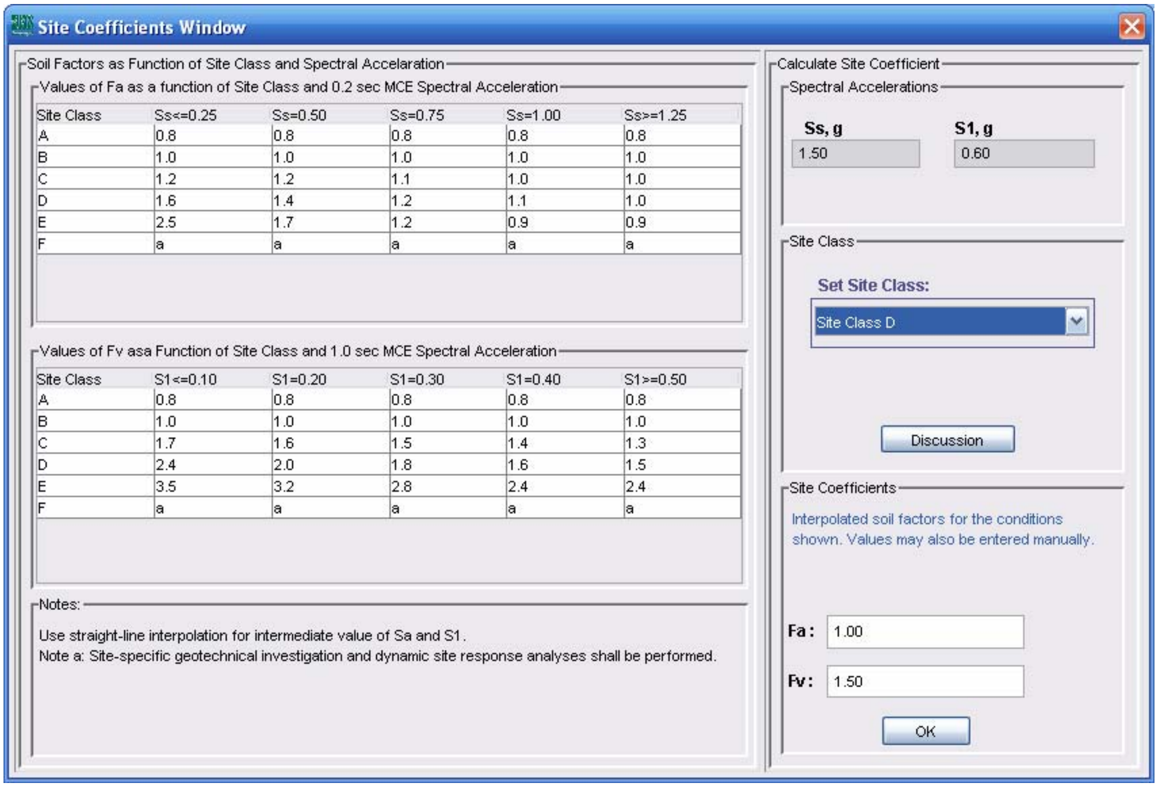



Figure 5-4 Data input to USGS ground motion calculator to modify the response spectrum for non-reference soil conditions

Follow these additional steps to adjust for site class:

7. In the pull-down menu on the right hand side, enter the appropriate *Site Class* in accordance with the building code or standard being used.
8. The site class coefficients will automatically appear in the lower right hand data boxes.
9. Left-click on the *OK* button.

After the *OK* button is clicked, the window shown in Figure 5-4 will disappear and the screen shown in Figure 5-5 will appear.

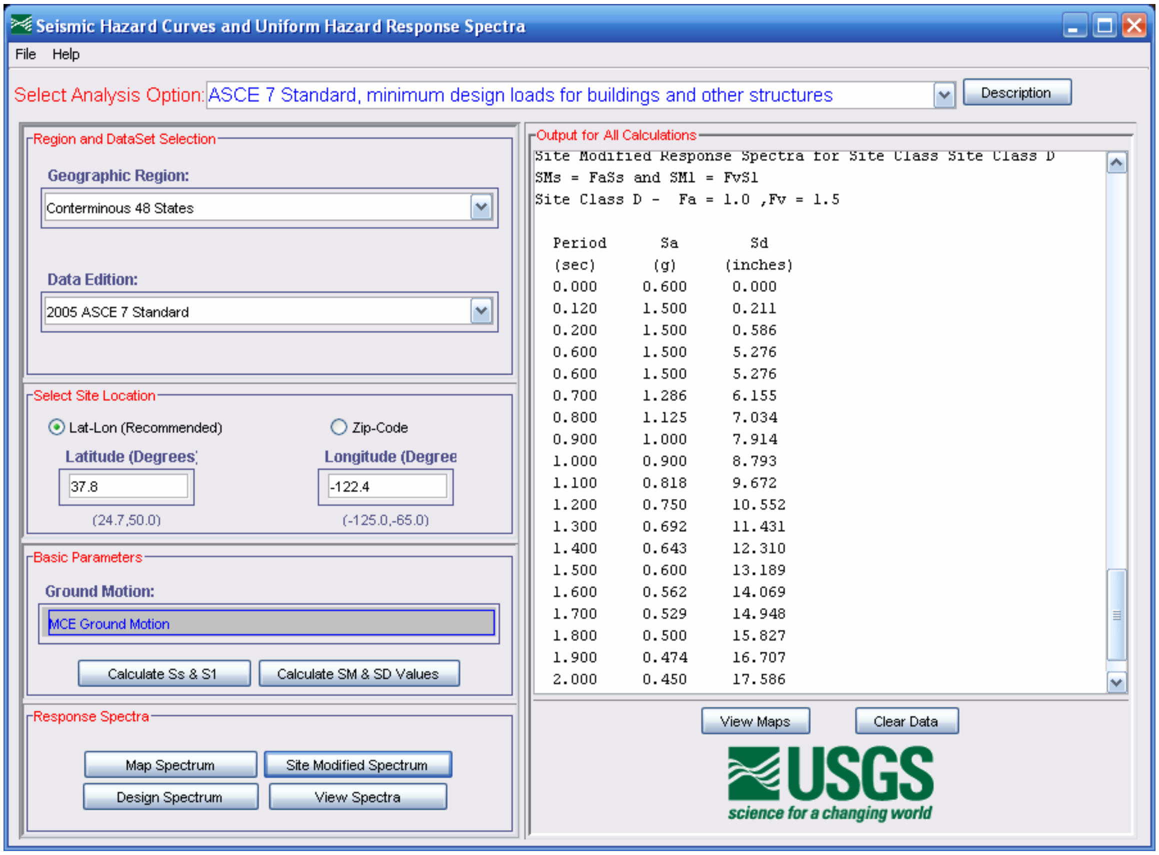



Figure 5-5 Sample response spectrum calculation for an intensity-based assessment

The right-hand window will show spectral ordinates at a variety of periods ranging from 0 second to 2.0 seconds. Linear interpolation can be used for intermediate periods. Equation (5-3) can be used to extend the response spectrum from 2 seconds to 5 seconds.

Table 5-1 presents the relationship between return period (of exceedance), annual frequency of exceedance, and % probability of exceedance in 50 years. The annual frequency of exceedance is the reciprocal of the return period. The % probability of exceedance is calculated as

$$P(SA > sa) = 1 - e^{-\lambda t} \tag{5-4}$$

where *SA* is the probability of exceedance of the Spectral Acceleration in time period *t* (equal to 50 years in the table below) that is expressed in the table below as a percentage, *sa* is the value of the spectral acceleration and $\lambda$ is the annual frequency of exceedance. One or more of these return periods or % probability of exceedance in 50 years could be used to define an alternate spectrum for an intensity-based assessment. ASCE 41 provides procedures to adjust response acceleration parameters mapped at 10% and

2% probability of exceedance in 50 years for other probabilities of exceedance.

**Table 5-1 Representations of Seismic Hazard for Intensity-Based Assessments**

| Return Period (years) | Annual Frequency of Exceedance | % Prob. of Exceedance in 50 years | Return Period (years) | Annual Frequency of Exceedance | % Prob. of Exceedance in 50 years |
|---|---|---|---|---|---|
| 10 | 0.10000 | 99.3 | 475 | 0.00211 | 10.0 |
| 20 | 0.05000 | 91.8 | 500 | 0.00200 | 9.52 |
| 30 | 0.03333 | 81.1 | 975 | 0.00103 | 5.00 |
| 40 | 0.02500 | 71.4 | 1000 | 0.00100 | 4.88 |
| 50 | 0.02000 | 63.2 | 1500 | 0.00066 | 3.28 |
| 72 | 0.01389 | 50.0 | 2475 | 0.00040 | 2.00 |
| 100 | 0.01000 | 39.4 | 4975 | 0.000201 | 1.00 |
| 200 | 0.00500 | 22.1 | 5000 | 0.000200 | 1.00 |
| 224 | 0.00446 | 20.0 | 10000 | 0.00010 | 0.50 |

## 5.5 Attenuation Relationships

### *5.5.1 Introduction*

For scenario-based assessment, seismic performance and loss are computed for a user-specified combination of earthquake moment magnitude and site-to-source distance (e.g., $M_W 7$ earthquake shaking from a source 13 *km* from the building site). Attenuation relationships are used to transform earthquake magnitude and site-to-source distance, together with information on local and regional geology and the soil profile at the site, into a median acceleration response spectrum and a period-dependant dispersion.

Information on best-estimate maximum moment magnitude for ruptures of individual and multiple fault segments in the Western United States and the Inner Mountain seismic belt can be found in the archival literature and at http://earthquake.usgs.gov/ and can be useful for selecting an appropriate scenario. However, any combination of magnitude and distance can be selected.

If the building site is within 15 *km* of the presumed zone of fault rupture and the selected earthquake magnitude is $M_W 6.5$ or greater, fault directivity effects should be considered in the spectral calculations by specifying that the scenario event has: (a) forward directivity (rupture towards the site); (b) reverse directivity (rupture away from the site); (c) null directivity; or (d) unspecified directivity (random direction of rupture). Appendix B provides guidance on fault-rupture directivity and how to include it in hazard computations.

### 5.5.2 Functional Form

One form of a ground motion attenuation relationship is

$$\ln Y = c_1 + c_2 M - c_3 \ln R - c_4 R + \varepsilon \tag{5-5}$$

where $\ln Y$ is the natural logarithm of the *median* strong-motion parameter of interest (e.g., peak ground acceleration, spectral horizontal acceleration at a particular period, etc.), *M* is the earthquake magnitude, *R* is the source-to-site distance or a term characterizing this distance, and $\varepsilon$ is the standard error term with a mean of zero and a logarithmic standard deviation of $\sigma_{\ln Y}$, or $\beta$. The term $c_2 M$ is consistent with the definition of earthquake magnitude (the *source*) as a logarithmic measure of the amplitude of the ground motion. The term $-c_3 \ln R$ (the *path*) represents the geometric spread of the seismic wave front as it propagates from the source. The value of $c_3$ will vary with distance depending on the seismic wave type (body wave, surface wave, etc). The term $-c_4 R$ treats the anelastic attenuation of seismic waves caused by material damping (treating soil as a viscoelastic materials) and scattering (a result of reflections and refractions of seismic waves due to the presence of heterogeneities and discontinuities in the earth's crust, causing multiple seismic waves to arrive at a site from different paths of differing lengths). Typical attenuation relationships are more complicated than the basic equation given above. Additional terms are used to account for other effects including near-source directivity, faulting mechanism (strike slip, reverse and normal), site conditions (different relationships), and hanging wall/footwall location of the site.

Attenuation relationships are derived using regression analysis on large ground-motion data sets. Regression analysis is used to determine the best estimate of the coefficients in the relationship, that is, the $c_1$ through $c_4$ in the basic attenuation relationship of (5-5). The value predicted by this equation is the mean value of $\ln Y$, or the 50th-percentile or median value of *Y*. The median value, by definition is exceeded by 50% of the observations[1]. To capture randomness in the value of the ground motion parameter, the probability of exceedance of the parameter can be computed using,

$$\ln Y_{1-\alpha} = \ln Y + z_\alpha \sigma_{\ln Y} \tag{5-7}$$

where $z_\alpha$ is the standard normal variable for an exceedance probability of $\alpha$. Although many attenuation relationships have been developed, most

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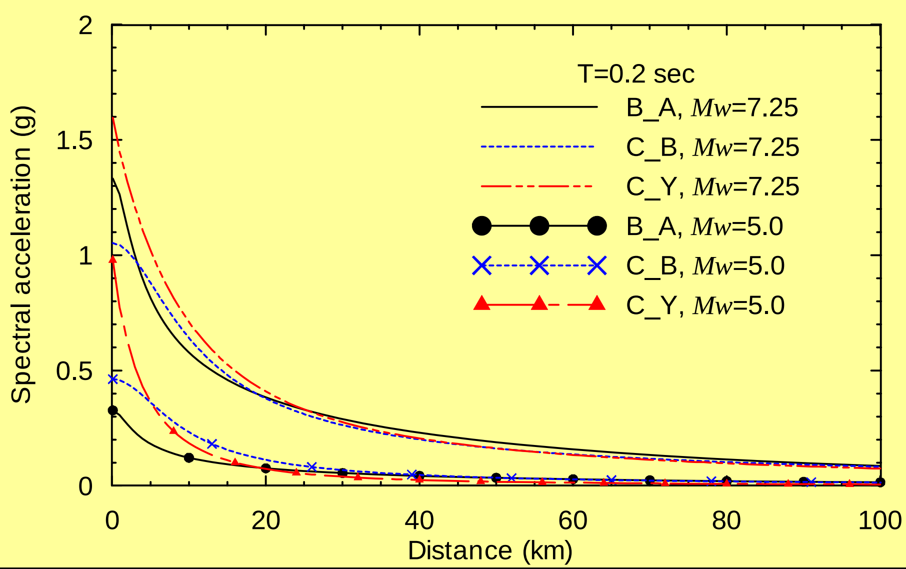
[1] See Appendix A for information on the characteristics of lognormal distributions.

apply to only a specific geographic region, a particular site class, or type of rupture mechanism, based on the data set of ground motion records used to develop the relationships.  It is important to ensure that an attenuation relationship selected for use in performance assessment is appropriate to the particular building site and earthquake source.

Selected North American attenuation relationships are presented in Appendix B.  All of the ground motion attenuation relationships described in the appendix use moment magnitude $M_W$ to define earthquake magnitude.  The Open Seismic Hazard Analysis website, http://www.opensha.org/, provides a listing of West Coast shallow crustal attenuation relationships (under the dialog box Applications). The attenuation relationship plotter, which can be downloaded from the website, can be used to generate median spectra and dispersions for the listed relationships. The three Next Generation Attenuation relationships (Boore and Atkinson, Campbell and Bozorgnia, Chiou and Youngs) developed recently for shallow crustal earthquakes on the West Coast of the United States are available.

**Sidebar: Attenuation relationships to compute spectral demands for scenario-based assessments**: Sample results of analysis using three of the so-called Next Generation Attenuation (NGA) relationships (labeled respectively: B_A, C_B, C_Y, are presented in the figure below for Moment magnitude $M_W$ 7.25 and 5.0 earthquakes, site-to-source distance in the range of 1 to 100 *km*, and 0.2-second spectral acceleration. An acceleration response spectrum can be developed for the site and a given combination of earthquake magnitude and distance by repeating the calculation for many values of period across the period range of interest (typically 0.01 to 5 second). Note the differences in spectral demands between the equally valid attenuation relationships.



**Reader Note**: An attenuation relationship calculator will be developed in the coming years of the project and included on the CD used to distribute PACT.

### 5.5.3 Median Spectrum and Dispersion

The attenuation relationships presented above return the median value $Y$ (strictly speaking, a geometric mean of two horizontal components[2]) of the spectral quantity of interest, which is typically spectral acceleration at a defined period. The dispersion (or scatter around the median) is given by $\sigma_{\ln Y}$, which is widely termed $\beta$. Both the median spectral demand and the dispersion around the median are used for scenario-based assessments.

Some attenuation relationships use shear-wave velocity in the upper 30 *m* as an input variable; others are presented for one or more reference shear wave velocities (e.g., 760 *m/sec*). If the selected attenuation relationship has an input variable for shear wave velocity in the upper 30 *m* and the shear wave velocity in the upper 30 *m* of the soil column has been computed per Chapter 20 of ASCE 7, use the site-specific value to compute the median spectral demand. If the shear wave velocity has not been computed, use 275 *m/sec*, which is the midpoint shear wave velocity for Soil Type D, as input to the attenuation relationship.

If the selected attenuation relationship uses a generic description of the reference soil type (e.g., soft rock) and reports the reference shear wave velocity, the median spectral demand can be factored by the ratio of the appropriate site class coefficients for the site-specific and reference shear wave velocities

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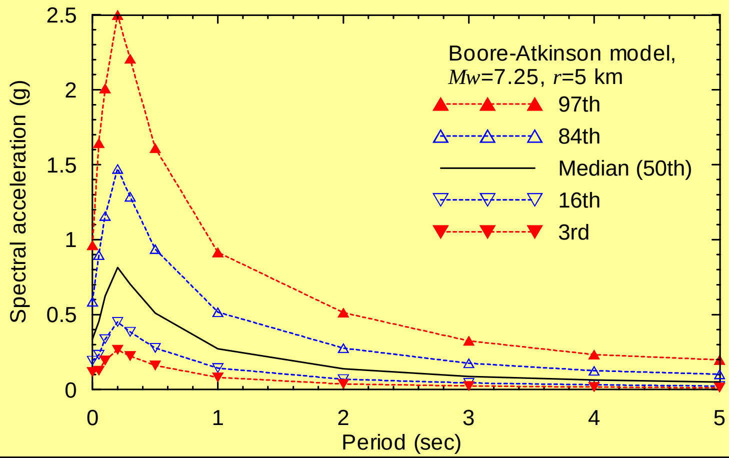
[2] For a given pair of horizontal earthquake histories, the geometric mean ($\bar{S}_g$ of the spectral ordinates of the two components ($S_x$ and $S_y$) is generally used to characterize the pair of histories: $\bar{S}_g = \sqrt{S_x S_y}$ . Since the functional form of the attenuation relationship involves the natural log of the ground motion parameter, the geometric mean of the ordinates (which is equivalent to the arithmetic mean of the logs of the ordinates) is used instead of the arithmetic mean.

**Sidebar: Median spectrum and dispersion that illustrate the variability in earthquake shaking**. The figure below illustrates the wide dispersion in spectral demand around the median demand. The figure shows median spectral demand for a $M_w$ 7.25 earthquake and a site-to-source distance of 5 *km* for the Boore-Atkinson NGA relationship together with 3rd, 16th, 84th and 97th percentiles of spectral demand.




## 5.6 Hazard Characterization for Use with Nonlinear Response-History Analysis

### 5.6.1 Introduction

This section presents one acceptable set of procedures for characterizing earthquake hazard for use with nonlinear response-history analysis. Procedures are presented for intensity-, scenario- and time-based assessments. Section 5.6.2 introduces two bins of earthquake ground motions from which seed motions can be selected for amplitude scaling. Section 5.6.3 presents information on distributions of earthquake shaking that are used to develop statistically appropriate sets of acceleration time series for response analysis. The remaining subsections present procedures for selecting and scaling earthquake ground motions for intensity-, scenario and time-based assessments.

### 5.6.2 Bins of Earthquake Ground Motions

Two bins of 50 earthquake ground motions each are provided for use in nonlinear response-history analysis of buildings. These bins are:

- Bin 1: near-fault ground motions to be used if the building site is within 15 *km* of an active fault capable of generating a moment magnitude 6.5 earthquake or greater
- Bin 2: far-field ground motions to be used if the building site is located more than 15 *km* from an active fault.

Appendix B lists the Bin 1 and Bin 2 earthquake histories in a tabular format. The acceleration time series can be downloaded from the PEER ground motion database at www.peer.berkeley.edu.

**Reader Note**: The Bin 1 and Bin 2 earthquake histories will be included with future versions of PACT.

### 5.6.3 Statistical Distributions of Earthquake Shaking

Scenario- and time-based assessments utilize distributions in spectral demand for the purpose of scaling earthquake ground motions for response-history analysis. Ground motion intensity is assumed to be lognormally distributed with median value $\theta$, and dispersion, $\beta$. For time-based assessments, two methods of representing this dispersion are provided.

Eleven values of spectral acceleration at the first mode period of the building are used to characterize the distribution of seismic demand for scenario- and Method 2 time-based assessments as follows:

$$S_{ai} = \theta e^{\beta \eta_i} \qquad i = 1, 11 \tag{5-7}$$

where $S_{ai}$ is the $i^{th}$ target spectral acceleration and values of $\eta_i$ are listed in Table 5-2.

**Table 5-2 Values of $\eta_i$ for Generating a Distribution of $S_{ai}(T_1)$**

| *i* | $\eta_i$ | *i* | $\eta_i$ | *i* | $\eta_i$ |
|---|---|---|---|---|---|
| 1 | -1.69 | 5 | -0.23 | 9 | 0.75 |
| 2 | -1.10 | 6 | 0 | 10 | 1.10 |
| 3 | -0.75 | 7 | 0.23 | 11 | 1.69 |
| 4 | -0.47 | 8 | 0.47 | | |

**Reader Note**: Studies of the appropriate number of ground motion records required to adequately capture the dispersion in the shaking intensity are continuing. Based on the studies completed to date, 11 ground motions are sufficient. The studies completed to date are documented in a project technical report by Huang et al. (2007), which is available at the ATC-58 website.

Figure 5-6 illustrates this process for a scenario earthquake having median spectral acceleration of 0.3 *g* and dispersion equal to 0.4. The figure shows the cumulative probability distribution represented by this median and dispersion. Horizontal lines across the plot divide the distribution into eleven striped-regions, each having a probability of occurrence of 9.09%. For each stripe, the central or midpoint value of the probability of exceedance is shown by × with values of 4.55%, 13.64%, 22.73%, 31.82%, 40.91%, 50%, 59.09%, 68.18%, 77.27%, 86.36% and 95.45%. A dashed horizontal line is drawn across the plot from the vertical axis to intersect the cumulative distribution function and dropped vertically to the horizontal axis, where spectral acceleration values of .153 *g*, .193 *g*, .222 *g*, 0.248 *g*, 0.274 *g*, 0.300 *g*, 0.329 *g*, 0.362 *g*, 0.405 *g*, 0.465 *g* and 0.590 *g*, respectively, are read off.

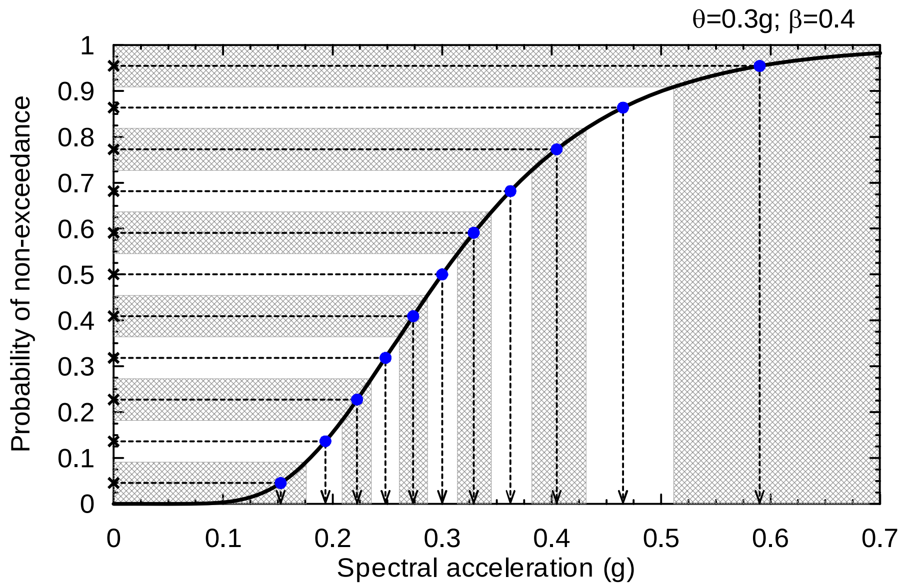



Figure 5-6 Calculation of spectral accelerations given a lognormal distribution

**Sidebar: Calculation of hazard values for scenario-based assessments.** The values of spectral acceleration, $S_{a1}$ to $S_{a11}$, characterize the distribution of spectral acceleration shown in the figure and can be obtained from the following equation:

$$S_{ai} = \theta \cdot e^{\beta \cdot \Phi^{-1}(P_i)}$$

where $\Phi^{-1}$ is the inverse standardized normal distribution and $P_i$ is the midpoint cumulative probability for region *i*. The values for $\eta_i$ in Table 5-2 are calculated from $\Phi^{-1}(P_i)$ for 11 target spectral accelerations. Alternatively, these values can be generated using an Excel spreadsheet and the function LOGINV ($p$, $\text{In}\,\theta$, $\beta$), where values of $p$ = (0.04545 ….. 0.9545), $\text{In}\,\theta$ ( In(0.3)=-1.204 ) is the mean, and $\beta$ (=0.4) is the dispersion.

### 5.6.4 *Intensity-based assessment*

Earthquake intensity is defined by a user-specified 5%-damped, elastic horizontal acceleration response spectrum, consistent with the characteristics of the site on which the building is constructed. The characterization of the earthquake hazard involves the following steps:

1. select a response spectrum
2. determine the median value of the spectral response acceleration at the fundamental period of the structure, $S_a(T_1)$
3. randomly select 11 ground motions from Bin 1 or Bin 2 per Section 5.6.2 and amplitude scale each ground motions to the median $S_a(T_1)$ ; use Bin 1 motions if the proximity of the building site to major active faults is unknown.

The product of step 3 will be 11 earthquake histories for response analysis per Chapter 6.

**Reader Note**: The use of 11 ground motions is recommended at this time for buildings responding in the inelastic range. A smaller number of ground motions will be required for buildings responding in the elastic range but that number is not available at this time.

### *5.6.5 Scenario-Based Assessment*

For scenario-based assessment, seismic performance and loss are computed for a user-specified combination of earthquake moment magnitude and site-to-source distance. Attenuation relationships are used to predict median spectral demands and dispersions as a function of period. Section 5.5 provides introductory information on these relationships. The characterization of the earthquake hazard involves the following steps:

1. select the magnitude and site-to-source distance for the scenario event
2. select an appropriate attenuation relationship (or relationships) for the region, site soil type and source characteristics
3. determine the median spectral acceleration demand, $\theta$ , and its dispersion, $\beta$ , at the fundamental period of the structure using the attenuation relationship(s) of step 2
4. compute 11 target values of spectral acceleration, $S_{ai}(T_1), i = 1, 11$, using $\theta$ and $\beta$ from step 3 and the procedures of Section 5.6.3
5. randomly select 11 ground motions per Section 5.6.2 and amplitude scale one of the 11 ground motions to one of the 11 target spectral accelerations of step 4; repeat the selection and scaling process 10 times for the remaining 10 target values of spectral acceleration so as to fully populate the distribution of $S_{ai}(T_1)$ .

The product of step 5 will be 11 earthquake histories for response analysis per Chapter 6.

### *5.6.6 Time-Based Assessments*

**Introduction**

Time-based performance assessments utilize seismic hazard curves, which can be established for a building site based on the USGS national seismic hazard maps, which is described in Section 5.4.2, or by performing site-specific probabilistic seismic hazard analysis (PSHA), which is described in Appendix B. The key product of such an analysis is a mean seismic hazard curve such as that shown in Figure 5-2. The mean hazard curve includes an explicit consideration of uncertainty. Appendix B provides summary information on the inclusion of epistemic (model) uncertainty in seismic hazard analysis.

For response-history analysis, ground motions are scaled to either the *mean* spectral demand at the selected mean annual frequency of exceedance (Method 1), or a distribution of spectral demands at the selected mean annual frequency of exceedance, which is defined by a median value, $\theta$, and a dispersion due to epistemic uncertainty, $\beta$ (Method 2). Method 1 should be used if the seismic hazard curves are established using the USGS ground motion calculator and can also be used if mean hazard curves are established using probabilistic seismic hazard analysis. Method 2 requires values of the dispersion in the seismic hazard, conditioned on the mean annual frequency of exceedance.

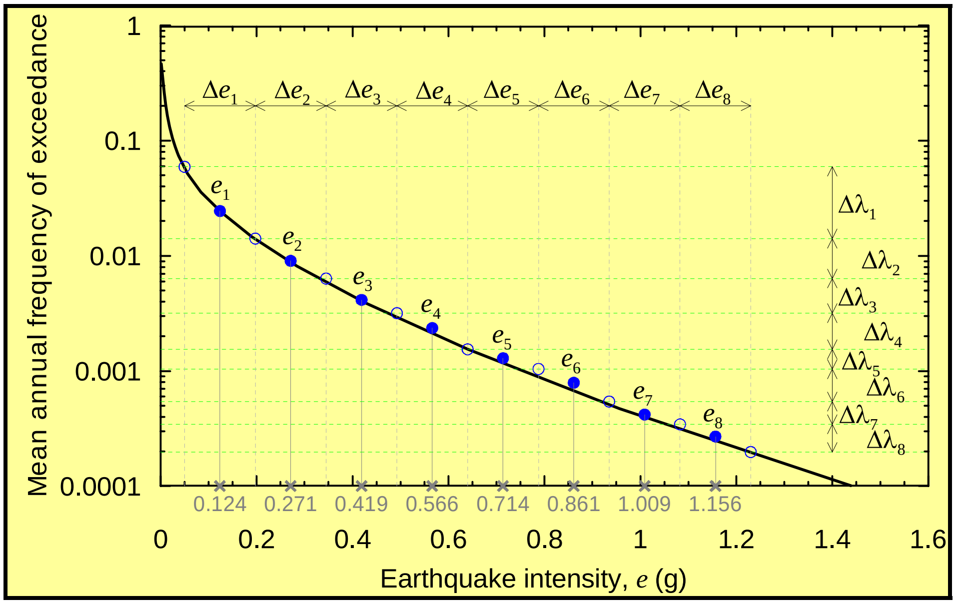
Both methods require the user to select a range of spectral demand for the assessment. One acceptable range is

- Minimum $S_a(T_1)$ = 0.05 $g$
- Maximum $S_a(T_1)$ = spectral acceleration for an annual frequency of exceedance = 0.0002 ($2\times10^{-4}$)

**Reader Note**: The range of spectral acceleration is preliminary and subject to change. The utility of this approach will be investigated in the coming phase of work and reported in subsequent drafts of these *Guidelines*. For new code-compliant buildings, collapse is not anticipated for earthquakes with an annual frequency of exceedance of 0.00040 (return period of 2500 years). The upper bound mean annual frequency of exceedance of 0.0002 was selected to drive code-compliant buildings to collapse (or the brink there-of). The writers recognize that older, non-ductile buildings are much more likely to collapse than modern code-compliant buildings and so shaking with an annual frequency of exceedance of 0.01 might be sufficient to trigger collapse–meaning that a number of the 8 simulations might result in collapse. The user is encouraged to change the range of spectral accelerations, specifically the upper limit, so that no more than 2 of the 8 simulations trigger collapse. The lower bound limit on spectral acceleration should not result in damage to either structural or nonstructural components of any vintage.

The chosen range of spectral demand is then split into 8 equal intervals and the midpoint values of spectral acceleration in these 8 intervals are used for ground motion scaling and response calculations. The sidebar illustrates the calculation of the range of spectral demand; splitting of the range into eight equal intervals, $\Delta e_1$ through $\Delta e_8$; the calculation of the midpoint values of spectral demand, $e_1$ through $e_8$; and the mean annual frequency of spectral demand in each interval, $\Delta\lambda_1$ through $\Delta\lambda_8$, which are required for loss computations.

**Sidebar: Sample calculation of spectral demands for a time-based assessment using a seismic hazard curve**. The seismic hazard curve below illustrates the calculations associated with characterizing hazard for time-based assessment. The range of mean spectral acceleration was selected as 0.05 *g* to 1.23 *g*, where 1.23 *g* is the spectral acceleration (at a given period) corresponding to a mean annual frequency of exceedance (MAFE) of 0.0002. This range of mean spectral acceleration was split into eight equal intervals, $\Delta e_i$, of 0.1475 *g*, with the boundaries of 0.050, 0.198, 0.345, 0.493, 0.640, 0.788, 0.935, 1.083 and 1.23 *g*. The midpoint value in each interval characterizes a target spectral demand for the scaling of ground motions. The 8 target spectral accelerations are identified in the figure by the symbol ☹ with values of (0.124, 0.271, 0.419, 0.566, 0.714, 0.861, 1.009, 1.156) *g*. The mean annual frequency of spectral demand in each interval is computed as the difference in the MAFEs at the boundaries of the interval (e.g., the mean annual frequency of spectral acceleration (at 1.14 seconds) being between 0.788 and 0.935 *g* is 0.000499 [=0.00104-0.000543]). The eight corresponding intervals of mean annual frequency, $\Delta\lambda_1$ through $\Delta\lambda_8$, which are required for time-based loss assessment, are 0.0452, 0.00775, 0.00318, 0.00163, 0.00050, 0.000497, 0.000199, and 0.000146, respectively.




**Reader Note**: There are numerous issues related to the selection and scaling of ground motions that have not yet been resolved by the project team, including: (a) how to select and scale ground motions for buildings in which losses accrue at widely spaced periods; (b) the use of the conditional mean spectrum (Baker and Cornell, 2006) in lieu of the uniform hazard spectrum; and (c) the extension of the conditional mean spectrum to bi- and tri-directional earthquake shaking. Developments will be included in subsequent drafts of these *Guidelines*.

### Method 1

Method 1 uses a mean seismic hazard curve computed for the fundamental period of the building. The USGS ground motion calculator can be used to establish spectral demands for periods equal to or less than 5.0 seconds. Site-specific probabilistic seismic hazard analysis can also be used to generate mean seismic hazard curves.

The characterization of the earthquake hazard using Method 1 involves the following steps:

1. develop a mean seismic hazard curve at the fundamental period of the structure that is appropriate for the soil type at the building site
2. compute the mean spectral acceleration from the mean seismic hazard curve of step 1 for an annual frequency of exceedance = 0.0002 ($2 \times 10^{-4}$) and denote that mean spectral acceleration as $S_a^{max}$
3. split the range of mean spectral acceleration, 0.05 to $S_a^{max}$ *g*, into 8 equal intervals; identify the midpoint mean spectral acceleration in each interval

4. calculate the mean annual frequency of spectral demand in each interval from the hazard curve and record these frequencies for input to PACT

5. for *each* of the 8 midpoint mean spectral accelerations, $S_{ai}(T_1)$, $i = 1, 8$, randomly select eleven ground motions from either Bin 1 or Bin 2 and amplitude scale each ground motion to $S_{ai}(T_1)$ for response calculations.

The product of step 4 will be 11×8 earthquake histories.

> **Reader Note**: The procedure for time-based assessment that is included in the version of PACT issued with the 35% draft Guideline is a variation on the steps listed above. (The above procedure will be implemented in PACT prior to the next release.) Specifically, in lieu of step 4 above, the user will enter the data that defines the seismic hazard curve into PACT per Section 4.2 and PACT will then compute the mean annual frequency of spectral demand in each interval.

**Method 2**

Method 2 uses a *median* seismic hazard curve and the dispersion due to epistemic uncertainty, computed at the fundamental period of the building. Site-specific probabilistic seismic hazard analysis is used to generate the median values and the dispersions.

The characterization of the earthquake hazard using Method 2 involves the following steps:

1. develop a median seismic hazard curve at the fundamental period of the structure that is appropriate for the soil type at the building site

2. compute the median spectral acceleration from the median seismic hazard curve of step 1 for an annual frequency of exceedance = 0.0002 ($2 \times 10^{-4}$) and denote that mean spectral acceleration as $S_a^{\max}$

3. split the range of median spectral acceleration, $\theta = 0.05$ to $S_a^{\max} g$, into 8 equal intervals; identify the midpoint median spectral acceleration in each interval, $\theta_i$, $i = 1, 8$; establish the dispersion $\beta_i$ for each $\theta_i$

4. calculate the mean annual frequency of spectral demand in each interval from the hazard curve and record these frequencies for input to PACT;

5. for *each* $\theta_i$ and $\beta_i$, identify 11 target values of spectral acceleration, $S_{ai}^{j}(T_1)$, $j = 1, 11$, using the procedures of Section 5.6.3; randomly select and amplitude scale one ground motion to one of the 11 target spectral accelerations; repeat the selection and scaling process 10 times for the remaining target values of spectral acceleration so as to

fully populate the distribution of $S_{ai}(T_1)$ at a given mean annual frequency of exceedance

6. repeat the selection and scaling procedure for the remaining seven distributions of spectral acceleration.

The product of step 5 will be 11×8 earthquake histories.

> **Reader Note**: The procedure for time-based assessment that is included in the version of PACT issued with the 35% draft Guideline is a variation on the steps listed above. (The above procedure will be implemented in PACT prior to the next release.) Specifically, in lieu of step 4 above, the user will enter the data that defines the seismic hazard curve into PACT per Section 4.2 and PACT will then compute the mean annual frequency of spectral demand in each interval.

## 5.7 Hazard Characterization for Use with Simplified Analysis

### *5.7.1 Introduction*

This section of the *Guidelines* presents acceptable procedures for characterizing earthquake hazard for simplified analysis. Procedures are presented in the following subsections for intensity-, scenario- and time-based assessments.

### *5.7.2 Intensity-Based Assessment*

Earthquake intensity is defined by a 5%-damped, elastic horizontal acceleration response spectrum. Only the value of the spectral response acceleration at the fundamental period of the building, $S_a(T_1)$, is required for response computations. An intensity-based assessment can be performed for any user-specified spectrum or spectral acceleration.

The characterization of the earthquake hazard involves the following steps:

1. select a response spectrum
2. determine the value of the spectral response acceleration at the fundamental period of the structure, $S_a(T_1)$.

### *5.7.3 Scenario-Based Assessment*

The characterization of the earthquake hazard involves the following steps:

1. select the magnitude and site source distance for the scenario event
2. select an appropriate attenuation relationship for the region, site and source characteristics

3. determine the median value of the spectral response acceleration at the fundamental period of the structure, $S_a(T_1)$ .

### *5.7.4 Time-Based Assessment*

A seismic hazard curve at the fundamental period of the structure in the direction under consideration, adjusted for soil type per Section 3.4, is used for time based assessment. The seismic hazard curve can be established using the USGS ground motion calculator, where linear interpolation can be used to establish spectral demands at a period between those available in the calculator. Site specific seismic hazard analysis can also be used.

Response analysis is performed at 8 values of spectral acceleration, $S_a(T_1)$ , assumed here to be mean $S_a(T_1)$ . The 8 values of spectral acceleration used for response computations span the range of interest in equal intervals. An acceptable range of spectral acceleration is:

- Minimum $S_a(T_1)$ = 0.05 $g$
- Maximum $S_a(T_1)$ = spectral acceleration for an annual frequency of exceedance = 0.0002 ( $2\times10^{-4}$ )

The characterization of earthquake hazard involves the following steps:

1. compute the shear wave velocity in the upper 30 m of the soil column beneath the building and identify the corresponding site class per ASCE-7-05; use Site Class D as a default
2. develop a mean seismic hazard curve at the fundamental period of the structure using the USGS ground motion calculator
3. adjust the USGS mean seismic hazard curve for soil type using the Site Class determined in step 1
4. from the adjusted mean hazard curve of step 3, compute the mean spectral acceleration for an annual frequency of exceedance = 0.0002 ( $2\times10^{-4}$ ) and denote that mean spectral acceleration as $S_a^{\max}$
5. split the range of mean spectral acceleration, 0.05 g to $S_a^{\max}$ , into 8 equal intervals; identify the midpoint mean spectral acceleration in each interval
6. calculate the mean annual frequency of spectral demand in each interval from the hazard curve and record these frequencies for input to PACT
7. use the 8 midpoint mean spectral accelerations for response calculations.

**Reader Note**: The procedure for time-based assessment that is included in the version of PACT issued with the 35% draft Guideline is a variation on the steps listed above. (The above procedure will be implemented in PACT prior to the next release.) Specifically, in lieu of step 5 above, the user will enter data that defines the seismic hazard curve into PACT per Section 4.2 and PACT will then compute the mean annual frequency of spectral demand in each interval.

# Chapter 6

# Response Analysis

## 6.1 Scope

This chapter presents acceptable procedures to determine seismic response of structural frames, elements and components, and seismic demands on nonstructural components and building contents. Section 6.2 presents procedures for nonlinear-response-history analysis. Section 6.3 presents companion information for simplified analysis. Both sections describe the response data required for input to PACT. (Example PACT input files are presented in Chapter 7.)

## 6.2 Nonlinear Response-History Analysis

### *6.2.1 Introduction*

This section presents guidance on nonlinear response-history analysis of buildings, including recommended constitutive models for structural elements; mathematical models of structural and geotechnical components and elements; analysis; and assembly of response data for input to PACT.

> **Reader Note**: The project team will develop a resource document on constitutive models, component modeling and nonlinear response-history analysis as a companion to the final *Guidelines*.

### *6.2.2 Mathematical Models of Components and Elements*

Buildings should be modeled, analyzed and assessed as a three-dimensional assembly of components and elements, including foundation and geo-components as appropriate. Nonstructural components that contribute either significant stiffness or mass should be modeled explicitly. The following subsections present guidance on mathematical modeling of structural and nonstructural components, elements and systems.

**Structural Components and Elements**

Nonlinear representation is typically used for all structural and geo-components and elements. A linear elastic component representation can be substituted for a nonlinear component model only if elastic response is anticipated and confirmed for the selected intensity of shaking. Nonlinear models of elements and components should be based on test data where available. Strength and stiffness degradation in all components and elements should be modeled explicitly unless response is insufficient to induce degradation.

Element and component models will generally be enveloped by the basic form of the force-displacement backbone relationships shown in Figure 6-1, where the generalized force (axial, shear or moment), $Q$, is normalized by the yield force, $Q_y$, taken as the statistical mean value of yield strengths for a population of similar components.

The Type 1 curve is representative of ductile behavior characterized by an elastic range (points 0 to 1 on the curve) and a plastic range (points 1 to 3). The plastic range includes a strain hardening or softening range (points 1 to 2) and a strength-degraded range with non-negligible residual strength to resist lateral and gravity loads (points 2 to 3). Loss of lateral-force-resisting capacity occurs at point 3, which is followed by loss of vertical-force-resisting capacity at point 4.

The Type 2 curve is representative of limited ductile behavior where there is an elastic range (points 0 to 1 on the curve) and a plastic range (points 1 to 3), followed by loss of lateral-force-resisting capacity at point 3 and loss of vertical-force-resisting capacity at point 4.

The Type 3 curve is representative of brittle or nonductile behavior where there is an elastic range (points 0 to 1 on the curve) followed by loss of lateral-force-resisting capacity at point 3 and loss of vertical-force-resisting capacity at point 4.

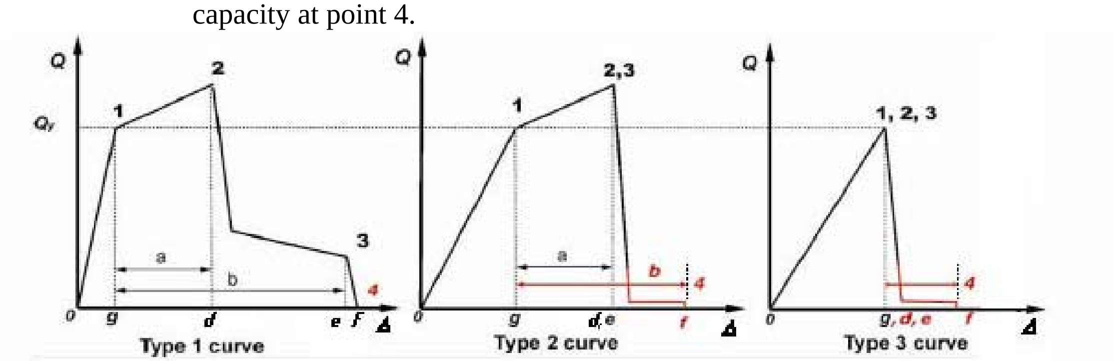
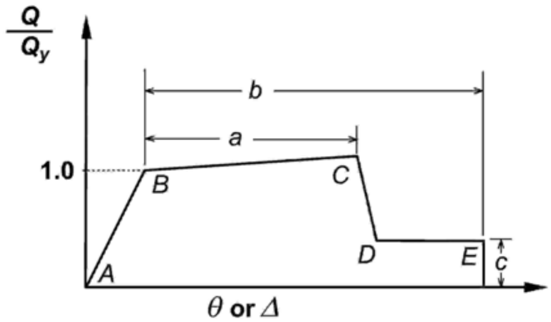
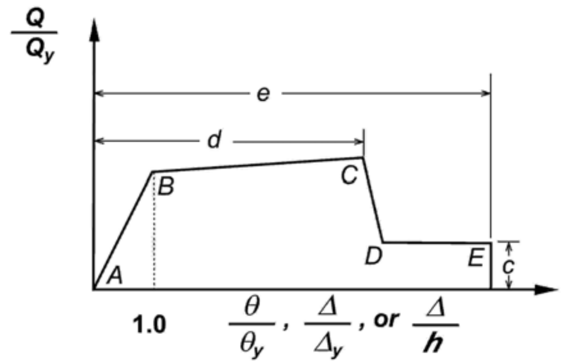



Figure 6-1 Generalized force-displacement behavior of structural components

Two measures of deformation are presented in the generalized hysteretic relationships of Figure 6-2, namely, absolute deformation (panel a) and deformation normalized by yield deformation (panel b).




**a. deformation**




**b. deformation ratio**

Figure 6-2 Generalized hysteretic relationships for structural components

> **Reader Note**: Improved component force-deformation relationships will be presented in later versions of these *Guidelines*. New force-deformation relationships will be described with statistics that represent their median (characteristic value) and dispersion. The models will address strength and stiffness degradation that is necessary to capture the nonlinear response up to the point of incipient collapse.

Floor diaphragms should be modeled as either rigid or having finite stiffness. A diaphragm can be classified as rigid if the maximum lateral deformation of the diaphragm is less than one half of the average story drift in the vertical components and elements above and below the subject diaphragm. A diaphragm that is not rigid should be modeled using appropriate finite elements. Diaphragms supporting nonstructural components that are sensitive to vertical acceleration should be modeled as flexible components using shell and line elements, as appropriate. Diaphragms in buildings containing out-of-plane offsets in the vertical seismic framing system should not be considered to be rigid and should be modeled explicitly using a sufficiently fine mesh of shell elements to capture local demands directly. Mass should be distributed across the footprint of diaphragms to capture the torsional response and the vertical dynamic response of the diaphragm and the supported nonstructural components.

Where soil-foundation-structure interaction is judged to have a significant effect on drift and acceleration response, nonlinear models should be used to represent the foundation system components. Elements composed of vertical grouped foundation components (e.g., piles associated with a single pile cap) can be used for nonlinear response analysis. The hysteretic properties of each element under generalized loadings should be developed using established principles. Foundations likely to uplift or slide should be modeled to capture these effects.

Nonlinear models of the soil in the immediate vicinity of a building should be included in the numerical model if: (a) the soil-structure-foundation interaction substantially modifies the free-field demand; (b) the soil is prone to liquefaction or ground failure; or (c) the building is embedded in the soil so as to significantly affect the free field ground shaking. Guidance on modeling soil-foundation-structure systems is presented in Appendix B.

**Reader Note**: Later drafts of these *Guidelines* will include definitive guidance on a) models for soil-foundation-structure interaction, and b) when such interaction must be addressed for performance assessment.

**Models of Nonstructural Components**

Nonstructural components that contribute significant lateral strength and/or stiffness to the building should be modeled using beam-column, membrane or shell finite elements. Distributed and discrete nonstructural components whose seismic demand can best be assessed using dynamic analysis should be included in the building model. Numerical models of nonstructural components should be based on full-scale cyclic test data or other supporting models.

### *6.2.3 Analysis Procedures*

Perform nonlinear response-history analysis using validated software and the structural and nonstructural component and system models of Section 6.2.2. For intensity-based and scenario-based assessments, nonlinear response-history analyses are conducted for eleven ground motion pairs, which are scaled to appropriate intensity values. For time-based assessments, nonlinear analyses are conducted for eight sets of eleven ground motion pairs, scaled to appropriate intensity values. If, at an intensity level, collapse occurs for one or more of the pairs of motions, additional analyses may be required to characterize collapse statistics.

### *6.2.4 Response Data Input to PACT*

For intensity-based and scenario-based assessment, eleven vectors of user-specified absolute values of maximum demand parameters (story drift and floor acceleration at each level), one per response-history analysis, are input to PACT. The format for the data input is illustrated via the examples of Chapter 7.

For time-based assessment, eight sets (one per spectral interval per Section 5.6.6.1) of eleven vectors of user-specified absolute values of maximum demand parameters (one per response-history analysis) are input to PACT, together with either the corresponding mean annual frequencies of shaking in each spectral interval (see Section 5.6.6) or the seismic hazard curve. The format for the data input is illustrated via the examples of Chapter 7.

**Reader Note**: Later drafts of these *Guidelines* will include more specific guidance on how to calculate and report collapse statistics when collapse is predicted by one or more analyses at an intensity level.

## 6.3 Simplified Analysis

### *6.3.1 Introduction*

Assessments performed in the preliminary stages of a building project will involve incomplete information. Not all structural components in the building frame will have been identified and sized, and geotechnical engineering studies to characterize the site conditions and ground motions may not have been completed. In light of this incomplete information, simplified analysis procedures have been developed to facilitate preliminary assessment and design when the rigor of nonlinear dynamic analysis is not justified given the uncertainty in building definition. The simplified procedures in this section use linear elastic mathematical models, an estimate of the yield strength of the building frame, and a simplified analysis procedure to estimate responses at global, story, floor, and component levels. Nonlinear mathematical models are not used, and nonlinear analysis is not performed using this procedure.

The use of linear mathematical models and the simplified method of analysis provide values of maximum story drift and peak floor acceleration that are representative of the median response values for a specific type of building. As such, the median values from the simplified analysis of any particular building will likely differ from the median values obtained from nonlinear response history analyses of the same building. Due to uncertainties in the building description and the use of simplified analysis, dispersions in the drift and acceleration response computed using simplified procedures will generally be greater than those associated with the nonlinear procedures of Section 6.2.

**Sidebar**: **Accuracy of Simplified Method.** Accuracy of the simplified procedures is based on a series of assumptions, including: (a) the modal properties of the linear elastic model and the corresponding nonlinear model are identical; (b) the earthquake intensities for the simplified analysis and the nonlinear analysis are identical; (c) the building is regular in plan and elevation (i.e., there are no substantial discontinuities in lateral strength and stiffness); (d) the story drifts are less than 4%, beyond which P-delta effects and strength/stiffness deterioration may become important; and (e) story drifts do not exceed 5 times the yield drift, so that the assumptions of elastic-plastic response at the component level is not compromised by excessive degradation in strength and stiffness.

### 6.3.2 Mathematical Models of Components and Elements

For meaningful assessment, the mathematical model of the building must represent the distribution of mass, lateral strength and lateral stiffness in the building. The following subsections provide guidance on modeling structural and nonstructural components for assessment using the simplified analysis procedure.

**Structural Components and Elements**

All elements that contribute significantly to either the building's lateral strength or stiffness should be considered in developing the model, whether or not these elements are considered to be structural or nonstructural, or part of the seismic framing system. ASCE-41, *Standard for Seismic Rehabilitation* (ASCE, 2007) provides guidance for modeling the strength and stiffness of typical building elements.

The simplest possible representation of a building frame is a fixed-based lumped parameter model, for which two degrees-of-freedom are used to characterize the horizontal displacement responses of each floor level in two perpendicular directions, and shear-type elements are used to represent the lateral stiffness of each story in each direction. More detailed linear models can also be used. The numerical model and framing-system data are used to establish the first mode period and shape of the building (in the direction under consideration), which is required to compute the pseudo lateral forces acting on the building frame and the resulting story drifts. An estimate of the yield strength of the building above its effective base is also required, which can be estimated for a new code-compliant building using the response modification factor, $R$ and the strength factor, $\Omega_0$ (see sidebar). Alternatively, plastic analysis can be used to estimate the yield strength.

The effects of vertical earthquake shaking and soil-foundation-structure interaction may generally be neglected if the simplified nonlinear method is used for analysis. The response of seismically isolated structures should be assessed using the equivalent linear procedures of ASCE-41.

**Sidebar: Estimation of yield strength**. For a new building structure, the building codes require design for a minimum strength given by a base shear coefficient, calculated as the product of the spectral acceleration at the first mode period of the building and the reactive weight of the building, divided by the quantity ($R$/$I$), where is $R$ is the response modification coefficient and

*I* is the occupancy importance factor. Most framing systems will have strength in excess of the minimum value for a number of reasons, including: construction using materials that are stronger than the minimum specified strength and redundancy in the frame. The factor $\Omega_0$, which varies by framing-system type, is a crude measure of the likely strength in the frame in excess of the design strength and is expressed as the ratio of the maximum strength to design strength in the building frame. The effective yield strength of a building frame should be between 1.25 and $\Omega_0$ times the code-based design strength.

**Models of Nonstructural Components**

Nonstructural components that contribute significant mass, lateral strength and lateral stiffness to the building frame should be modeled with a level of detail that is consistent with the mathematical models adopted for the structural frame. Numerical models of nonstructural components should be based on full-scale cyclic test data where available.

### *6.3.3 Analysis Procedures*

The simplified analysis procedure described in this section can be used to estimate story drifts and floor accelerations. Figure 6-3 presents the definitions of floor and story numbers and story height adopted for use with the analysis procedures of this section.

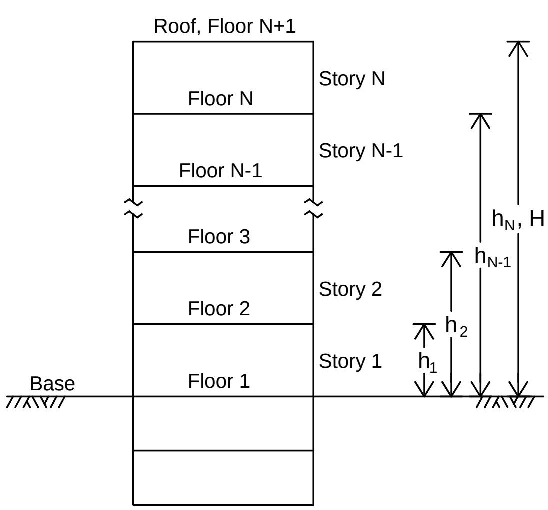



Figure 6-3 Definition of floor and story numbers and story height

**Reader Note:** The project team will continue to investigate other models for a simplified procedure in later phases of the project. At this time, the necessary adjustment factors to convert the results of such models to appropriate median estimate values of response parameters have not been developed.

### Pseudo Lateral Force

A pseudo lateral load, *V*, is used to compute median story drifts. The load *V* is computed using Equation (6-1) and distributed over the height of the mathematical model as described below.

$$V = C_1 C_2 S_a(T_1) W_1 \tag{6-1}$$

where $C_1$ is an adjustment factor for inelastic displacements; $C_2$ is an adjustment factor for cyclic degradation; $S_a(T_1)$ is the 5% damped spectral acceleration at the fundamental period of the building in the direction under consideration, for the selected level of ground shaking; and $W_1$ is the first modal weight of the building but cannot be taken as less than 80% of the total reactive weight, *W*.

The adjustment factor $C_1$ is computed as:

$$\begin{aligned} C_1 &= 1 + \frac{S-1}{0.04a} && \text{for } T_1 \le 0.2\,\text{sec} \\ &= 1 + \frac{S-1}{aT_1^2} && \text{for } 0.2 < T_1 \le 1.0\,\text{sec} \\ &= 1 && \text{for } T_1 > 1.0\,\text{sec} \end{aligned} \tag{6-2}$$

where $T_1$ is the fundamental period of the building in the direction under consideration, *a* is a function of the soil site class per Chapter 5 (= 130, 130, 90, 60 and 60 for site classes A, B, C, D and E, respectively) and $S$ is a strength ratio given by

$$S = \frac{S_a(T_1)W}{V_{y1}} \tag{6-3}$$

where $V_{y1}$ is the (estimated) story yield strength at the effective base of the building, and all other terms have been defined above. For a new code-compliant building, the value of *S* may be taken as the value of *R* tabulated in the current edition of ASCE-7 for the framing system under consideration.

The adjustment factor $C_2$ is computed as:

$$\begin{aligned} C_2 &= 1 + \frac{(S-1)^2}{32} && \text{for } T_1 \le 0.2\,\text{sec} \\ &= 1 + \frac{1}{800}\frac{(S-1)^2}{T_1^2} && \text{for } 0.2 < T_1 \le 0.7\,\text{sec} \\ &= 1 && \text{for } T_1 > 0.7\,\text{sec} \end{aligned} \tag{6-4}$$

The first modal weight, $W_1$, can be calculated as

$$W_1 = \frac{(\sum_{j=2}^{N+1} w_j \phi_{j1})^2}{\sum_{j=2}^{N+1} w_j \phi_{j1}^2} \qquad (6\text{-}5)$$

where $w_j$ is the lumped weight at floor level *j* (see Figure 6-3), $\phi_{j1}$ is the *j*th floor ordinate of the first mode deflected shape and *N* is the number of floors in the building *above* the effective base.

If the building incorporates viscous or viscoelastic damping devices, the 5% damped spectral acceleration ordinate should be replaced by the value that corresponds to the first mode damping in the building. The procedures of ASCE 41 can be used to estimate the damping in a building equipped with viscous or viscoelastic energy dissipation devices and to adjust the response spectrum for the effects of damping other than 5%.

**Story Drift Computations**

For a given spectral acceleration at the first mode period of the building, the story drifts, $\Delta_i^*$, are computed in three steps:

1. distribute the pseudo lateral force over the height of the building
2. apply the lateral forces to the elastic model of the building frame, compute the floor displacements and then compute the uncorrected story drifts, $\Delta_i$, as the difference in the displacements of adjacent floors
3. correct the story drifts to account for inelastic action and higher mode effects[1].

Each step in the drift calculation procedure is described below.

The pseudo lateral force, *V*, is distributed over the height of the building to compute the lateral loads at floor level *x*, $F_x$:

$$F_x = C_{vx} V \qquad (6\text{-}6)$$

using the vertical distribution factor, $C_{vx}$:

$$C_{vx} = \frac{w_x h_{x-1}^k}{\sum_{j=2}^{N+1} w_j h_{j-1}^k} \qquad (6\text{-}7)$$

[1] The corrections of step 3 are based on analysis of 9 buildings of varying construction with first mode periods ranging between 0.2 and 2.5 seconds. See the project technical note by Huang et al. (2007), which is available at the ATC-58 project website, for details.

where $w_j$ is the lumped weight at floor level $j$; $h_{j-1}$ ($h_{x-1}$) is the height above the effective base of the building to floor level $j$; and $k$ is equal to 2.0 for a first mode period greater than 2.5 seconds and equal to 1.0 for a first mode period less than or equal to 0.5 second (linear interpolation can be used for intermediate periods).

The lateral forces, $F_x$, are applied to the linearly elastic model of the building frame to compute the uncorrected floor displacements and story drifts, $\Delta_i$.

Correct the story drifts, $\Delta_i$, to compute demands for loss computations, $\Delta_i^*$, as follows:

$$\Delta_i^* = H_{\Delta i}(S, T_1, h_i, H)\Delta_i \tag{6-8}$$

where $H_{\Delta i}(S, T_1, h_i, H)$ is the drift correction factor for story $i$ than is computed using Equation 6-9 as a function of $S$; the first mode period, $T_1$; the height of the floor immediately above the story for which the drift is being calculated, as defined in Figure 6-3, $h_i$; and the total height of the building, $H$:

$$\ln H_{\Delta i} = a_0 + a_1 T_1 + a_2 S + a_3 \frac{h_i}{H} + a_4 \left(\frac{h_i}{H}\right)^2 \quad S \geq 1, i = 1 \text{ to N} \tag{6-9}$$

where the values of the coefficients $a_0$ through $a_4$ are presented in Table 6-1 for braced frame, moment-frame and shear-wall buildings.

**Table 6-1 Story-Drift and Floor-Acceleration Correction Factors**

| | Frame type | $a_0$ | $a_1$ | $a_2$ | $a_3$ | $a_4$ |
|---|---|---|---|---|---|---|
| Story Drift | Braced | 0.72 | 0.048 | 0.012 | -2.64 | 2.09 |
| | Moment | 0.65 | 0.027 | -0.010 | -2.58 | 2.30 |
| | Wall | 1.12 | -0.22 | -0.059 | -2.70 | 1.29 |
| Floor acceler. | Braced | 0.57 | -0.16 | -0.089 | 0 | 0 |
| | Moment | 0.67 | -0.28 | -0.080 | 0 | 0 |
| | Wall | 0.33 | 0.22 | -0.081 | 0.53 | 0 |

**Floor Acceleration Computations**

For a given peak ground acceleration (filtered at a frequency of 25 Hz), the peak acceleration at floor level $i$, $a_i^*$, is computed as:

$$a_i^* = H_{ai} a_i \quad i = 2 \text{ to N+1} \tag{6-10}$$

where $H_{ai}(S, T_1, h_i, H)$ is the acceleration correction factor for floor *i* (see Figure 6-3) calculated using Equation 6-11; and $a_i$ is the peak ground acceleration. The acceleration at the base of the building, floor level 1 per Figure 6-3, shall be set equal to the peak ground acceleration. The acceleration correction factor[2] is given by

$$\ln H_{ai} = a_0 + a_1 T_1 + a_2 S + a_3 \frac{h_{i-1}}{H} \quad S \geq 1, i = 2 \text{ to N+1} \tag{6-11}$$

where the values of the coefficients $a_0$ through $a_3$ are presented in Table 6-1 for braced frame, moment-frame and shear-wall buildings.

**Sidebar: Applicability of Simplified Procedure.** The drift and acceleration correction factors presented above are based on regression analysis of results of simplified and nonlinear analysis of 9 building frames (3 types of framing system; 3-, 5- and 9-story buildings) across a wide range of shaking intensities. Bilinear models were used for the nonlinear response-history analysis and degradation of strength and stiffness, and second-order effects were not considered. Accordingly, the utility of the simplified analysis procedure for large values of either: (a) *S* (a surrogate for damage), say greater than 5; or (b) large story drifts, say greater than 4%, for which other effects might be significant, is unknown at this time.

### Dispersions in Response Calculations

The simplified analysis procedure uses a point value (single realization) of $S_a(T_1)$ to produce a single vector of story drifts and floor accelerations. To conduct performance assessments, it is necessary to develop distributions of story drift and floor acceleration, where the distributions capture the uncertainty in ground motion, $\beta_{gm}$, analysis, $\beta_{a\Delta}$ and $\beta_{aa}$, and modeling, $\beta_m$.

Table 6-2 lists default values of the dispersion to be used with the simplified analysis procedure. The randomness in the ground motion was estimated using widely accepted attenuation relationships for Western North America (WNA), the Central and Eastern United States (CEUS), and the Pacific North West (PNW) (assuming subduction zone earthquakes). The dispersions in the drift and acceleration response resulting from the use of the simplified analysis procedure, $\beta_{a\Delta}$ and $\beta_{aa}$, respectively, were established using data presented in Huang et al. (2007) and Ruiz-Garcia and Miranda (2003). The

[2] The acceleration correction factors are based on analysis of 9 buildings of varying construction with first mode periods ranging between 0.2 and 2.5 seconds. See the project technical note by Huang et al. (2007), which is available at the ATC-58 project website, for details.

dispersion in the drift and acceleration response due to modeling assumptions, $\beta_m$, was estimated by members of the ATC-58 project team. Linear interpolation can be used to estimate values of $\beta$ for intermediate values of $T_1$ and $S$.

**Intensity-Based Assessments**

Intensity-based assessment requires a single response analysis using the value of $S_a(T_1)$, selected by the user. Compute the dispersions in the drift response, $\beta_{\text{IBA-}\Delta}$, and acceleration response, $\beta_{\text{IBA-a}}$, as follows:

$$\beta_{\text{IBA-}\Delta} = \sqrt{\beta_{a\Delta}^2 + \beta_m^2}\ ;\ \beta_{\text{IBA-a}} = \sqrt{\beta_{aa}^2 + \beta_m^2} \tag{6-12}$$

where $\beta_{a\Delta}$, $\beta_{aa}$ and $\beta_m$ are presented in Table 6-2.

**Scenario-Based Assessments**

Scenario-based assessment requires a single response analysis using the median spectral acceleration, $S_a(T_1)$, predicted by the attenuation relationship selected by the user. Compute the dispersions in the drift response, $\beta_{\text{SBA-}\Delta}$, and acceleration response, $\beta_{\text{SBA-a}}$, as

$$\beta_{\text{SBA-}\Delta} = \sqrt{\beta_{a\Delta}^2 + \beta_m^2 + \beta_{gm}^2}\ ;\ \beta_{\text{SBA-a}} = \sqrt{\beta_{aa}^2 + \beta_m^2 + \beta_{gm}^2} \tag{6-13}$$

where $\beta_{gm}$ is the dispersion in ground motion per Table 6-2 and other terms are defined above.

**Time-Based Assessments**

Time-based assessment requires 8 calculations using the mean spectral accelerations established per Section 5.7.4.

Compute the dispersions in the drift response, $\beta_{\text{TBA-}\Delta}$, and acceleration response, $\beta_{\text{TBA-a}}$, at the given intensity of shaking (and the corresponding value of $S$) as

$$\beta_{\text{TBA-}\Delta} = \sqrt{\beta_{a\Delta}^2 + \beta_m^2}\ ;\ \ \beta_{\text{TBA-a}} = \sqrt{\beta_{aa}^2 + \beta_m^2} \tag{6-14}$$

where no allowance is made for uncertainty in ground motion because this uncertainty is already captured in the hazard calculations used to derive the mean hazard curve.

### *6.3.4 Response Data Input to PACT*

For intensity-based and scenario-based assessment, one vector of absolute values of *median* maximum demand parameters is input to PACT, together with the dispersions for drift and acceleration computed per Section 6.3.3. The format for the data input is illustrated via the examples of Chapter 7.

For a time-based assessment, eight vectors of absolute values of *median* maximum demand parameters (one per spectral interval) are input to PACT, together with the corresponding dispersions for drift and acceleration and either the mean annual frequencies of shaking in each spectral interval (see Section 5.7.4) or the seismic hazard curve.  The format for the data input is illustrated via the examples of Chapter 7.

**Table 6-2 Dispersions for Use with the Simplified Analysis Procedure**

| $T_1$ (sec) | $S = \frac{S_a(T_1)W}{V_{y1}}$ | $\beta_{a\Delta}$ | $\beta_{aa}$ | $\beta_m$ | $\beta_{gm}$ [1] WNA | CEUS | PNW |
|---|---|---|---|---|---|---|---|
| 0.20 | $\leq 1.00$ | 0.05 | 0.10 | 0.10 | 0.60 | 0.53 | 0.80 |
| | 2 | 0.35 | 0.10 | 0.10 | | | |
| | 4 | 0.40 | 0.10 | 0.15 | | | |
| | 6 | 0.45 | 0.10 | 0.20 | | | |
| | $\geq 8$ | 0.45 | 0.05 | 0.20 | | | |
| 0.35 | $\leq 1.00$ | 0.10 | 0.15 | 0.10 | 0.61 | 0.55 | 0.80 |
| | 2 | 0.35 | 0.15 | 0.10 | | | |
| | 4 | 0.40 | 0.15 | 0.15 | | | |
| | 6 | 0.45 | 0.15 | 0.20 | | | |
| | $\geq 8$ | 0.45 | 0.15 | 0.20 | | | |
| 0.5 | $\leq 1.00$ | 0.10 | 0.20 | 0.10 | 0.62 | 0.55 | 0.80 |
| | 2 | 0.35 | 0.20 | 0.10 | | | |
| | 4 | 0.40 | 0.20 | 0.15 | | | |
| | 6 | 0.45 | 0.20 | 0.20 | | | |
| | $\geq 8$ | 0.45 | 0.20 | 0.20 | | | |
| 0.75 | $\leq 1.00$ | 0.10 | 0.25 | 0.10 | 0.64 | 0.58 | 0.80 |
| | 2 | 0.35 | 0.25 | 0.10 | | | |
| | 4 | 0.40 | 0.25 | 0.15 | | | |
| | 6 | 0.45 | 0.25 | 0.20 | | | |
| | $\geq 8$ | 0.45 | 0.25 | 0.20 | | | |
| 1.0 | $\leq 1.00$ | 0.15 | 0.30 | 0.10 | 0.64 | 0.60 | 0.80 |
| | 2 | 0.35 | 0.30 | 0.10 | | | |
| | 4 | 0.40 | 0.30 | 0.15 | | | |
| | 6 | 0.45 | 0.30 | 0.20 | | | |
| | $\geq 8$ | 0.45 | 0.25 | 0.20 | | | |
| 1.50 | $\leq 1.00$ | 0.15 | 0.35 | 0.10 | 0.66 | 0.60 | 0.85 |
| | 2 | 0.35 | 0.35 | 0.10 | | | |
| | 4 | 0.40 | 0.30 | 0.15 | | | |
| | 6 | 0.45 | 0.30 | 0.20 | | | |
| | $\geq 8$ | 0.45 | 0.25 | 0.20 | | | |
| 2.0+ | $\leq 1.00$ | 0.25 | 0.50 | 0.10 | 0.70 | 0.60 | 0.90 |
| | 2 | 0.35 | 0.45 | 0.10 | | | |
| | 4 | 0.40 | 0.45 | 0.15 | | | |
| | 6 | 0.45 | 0.40 | 0.20 | | | |
| | $\geq 8$ | 0.45 | 0.35 | 0.20 | | | |

1. WNA = Western North America; CEUS = Central and Eastern United States; PNW = Pacific North West. Values established using the attenuation relationships of Boore and Atkinson (2006) for WNA, Campbell (2003) for CEUS, and Atkinson and Boore (2003) for the PNW; moment magnitudes between 6.5 and 7; and rock sites.

# Chapter 7

# Example Assessments

## 7.1 Introduction

This chapter presents intensity-, scenario- and time-based performance assessments of example buildings using the procedures presented in Chapters 4, 5 and 6 of these *Guidelines*. The purpose of this presentation is to demonstrate the practical application of these procedures as they could be implemented in design practice.

> **Reader Note**: This example application makes use of a steel moment-frame building and nonlinear response-history analysis. Subsequent drafts of these *Guidelines* will include sample assessments of other types of buildings and will illustrate the use of the simplified analysis procedure.

## 7.2 Example Building #1

### *7.2.1 Introduction*

This example application uses the building shown in Figures 7-1, 7-2, and 7-3 as a prototype. This building shown is located in Berkeley, California, near the campus of the University of California. It is representative of modern Class A office space in the Greater Bay Area of California. Construction was completed in 2004. Some of the features of the actual building were modified and simplified for application in this example. The key features of the building model are:

| | |
|---|---|
| Height: | 3 stories; 14 ft. floor to floor; 42 ft total above grade; no basement |
| Area: | 22,736 sq.ft. per floor; 68,208 sq.ft. total (actual building slightly larger) |
| Occupancy: | General office space (B2) |

Figure 7-1 Photograph of the example building

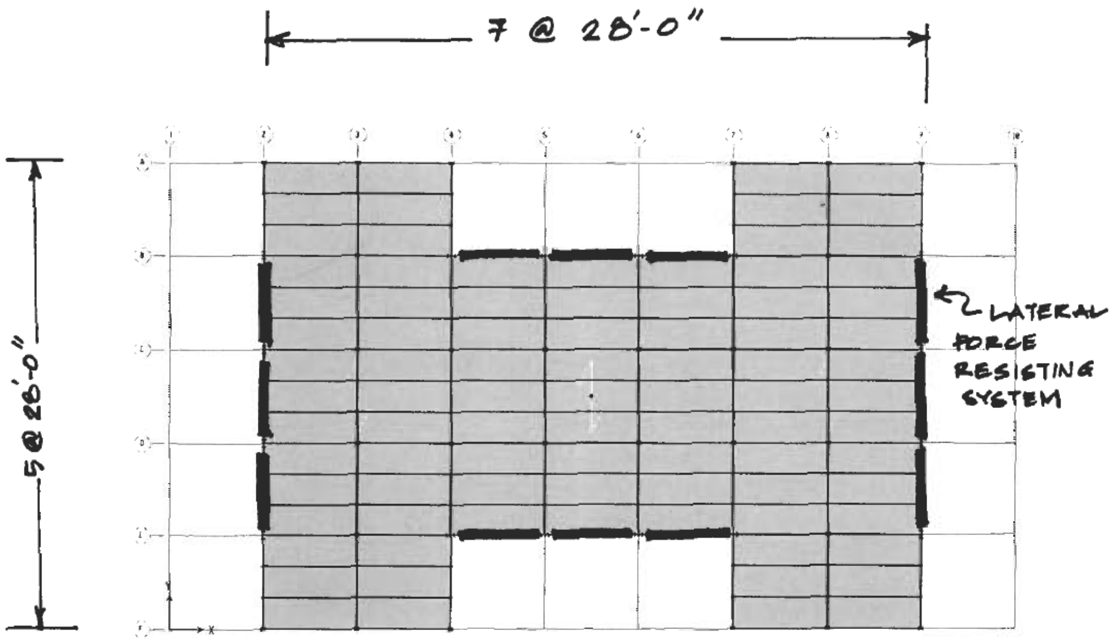



Figure 7-2 Schematic structural framing plan for example building (no scale)

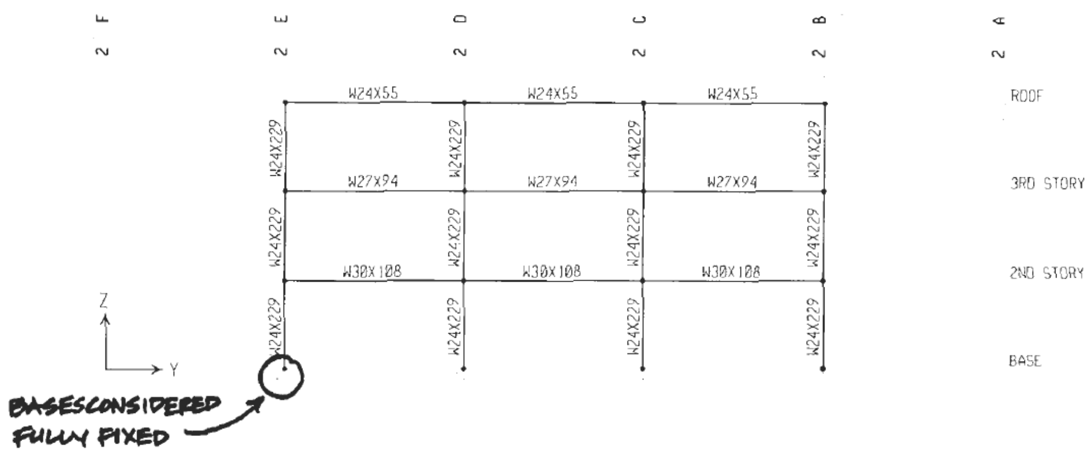



Figure 7-3 Elevation of typical steel moment frame for example building (no scale)

### *7.2.2 Seismic Hazard Characterization*

**Introduction**

This section presents the characterizations of seismic hazard for the enhanced intensity-, scenario- and time-based performance assessment of example building #1. The building is located 1 km from the Hayward Fault on a soft rock site that is classified as Site Class B per ASCE-7. The latitude/longitude coordinates of the site, which are required for intensity- and time-based assessments, are 37.875º/-122.267º. The fundamental translational period of the building, which is required information for scaling earthquake ground motions, is 1.14 seconds, along each principal axis.

**Ground Motions for Intensity-Based Assessment**

An intensity-based assessment was performed for a 5% damped design earthquake response spectrum for the site. The spectrum was developed per the 2006 International Building Code (ICC, 2006) for Site Class B. The spectral acceleration at a period of 1.14 seconds for this spectrum is 0.51 *g*. The USGS ground motion calculator and the lat/long coordinates noted above were used to compute the design earthquake spectral demand:

$$S_{M1} = 0.874\,g,\;\; S_{D1} = 0.582\,g\;,\;\; S_{D1.14} = 0.582/1.14 = 0.51\,g$$

Table 7-1 lists the eleven pairs of near-fault ground motions (Bin 1) that were randomly selected from the near-fault ground-motion bin for response-history analysis. For these pairs of motions, the moment magnitude ranges between 6.2 and 7.6 and the site-to-source distance varies between 1 and 10.7 km. The eleven pairs of seed motions in Table 7-1 were amplitude scaled to 0.51 *g* at 1.14 seconds per Section 5.6.4 of these *Guidelines*. This bin of scaled motions was denoted as Bin I1. Figure 7-4 presents the acceleration response spectra for the 22 scaled motions.

**Table 7-1 Seed Ground Motions for Response-History Analysis**

| Designation | Event | Station | $M_W$ | $r$ |
|---|---|---|---|---|
| NF3, NF4 | Loma Prieta 1989 | SAC 2/50 for Los Angeles | 7.0 | 3.5 |
| NF7, NF8 | Northridge 1994 | | 6.7 | 6.4 |
| NF13, NF14 | Elysian Park 2 (simulated) | | 7.1 | 10.7 |
| NF17, NF18 | Palos Verdes 1 (simulated) | | 7.1 | 1.5 |
| NF21, NF22 | Cape Mendocino 04/25/92 18:06 | 89156 Petrolia | 7.1 | 9.5 |
| NF27, NF28 | Chi-Chi 09/20/99 | TCU068 | 7.6 | 1.1 |
| NF35, NF36 | Erzinkan 03/13/92 17:19 | 95 Erzinkan | 6.9 | 2.0 |
| NF41, NF42 | Imperial Valley 10/15/79 23:16 | 942 El Centro Array #6 | 6.5 | 1 |
| NF43, NF44 | Kobe 01/16/95 20:46 | Takarazu | 6.9 | 1.2 |
| NF45, NF46 | Morgan Hill 04/24/84 04:24 | 57191 Halls Valley | 6.2 | 3.4 |
| NF47, NF48 | Northridge 1/17/94 12:31 | 24279 Newhall | 6.7 | 7.1 |

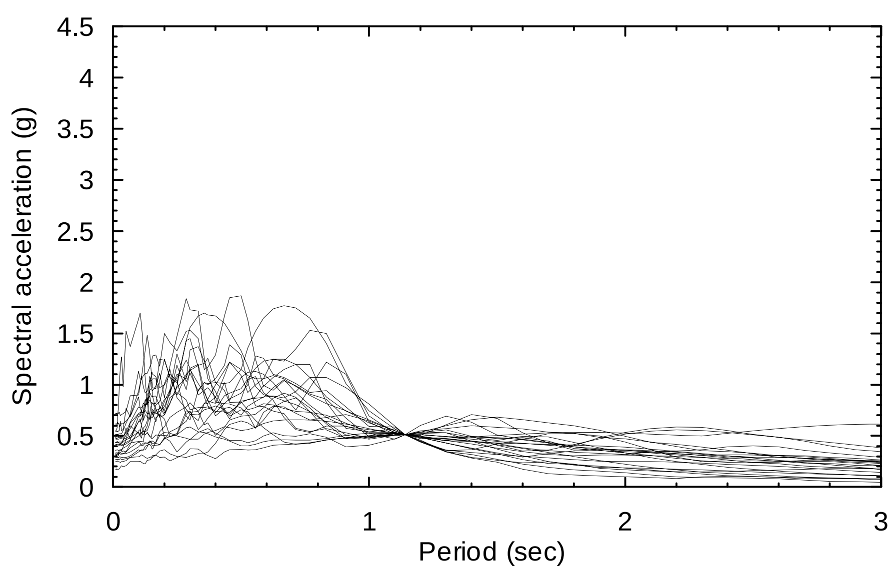



Figure 7-4 Spectral accelerations for Bin I ground motions

### Ground Motions for Scenario-Based Assessment

The building is located approximately 1 km from the Hayward Fault. The moment magnitude associated with the rupture of both segments of this fault is approximately 7.0. The scenario selected for assessment of the building was a moment magnitude 7 earthquake at a site-to-source distance of 1 km.

The Chiou-Youngs (C-Y) Next Generation Attenuation (NGA) relationship was used to predict the spectral demand for the [ $M_W$ , $r$ ] pair because this relationship predicts spectral demands at 1.0, 1.1 and 1.2 seconds (Chiou and Youngs 2006). Table 7-2 presents the median spectral acceleration ( $\theta$ ) and the logarithmic standard deviation ( $\beta$ ) at 1, 1.1 and 1.2 seconds predicted by the C-Y NGA relationship for $M_W$ = 7, $r$ = 1 km, strike-slip faulting and a shear wave velocity $v_{30} = 760$ m/s. Figure 7-5 presents the median C-Y spectral acceleration in the period range of 1 to 1.5 seconds for this combination of variables and a curve for spectral acceleration inversely proportional to period and anchored to the median C-Y spectral demand at 1 second (= 0.47 *g*). The C-Y spectral demands are virtually identical to $0.47/T$ in this period range. Accordingly, for the scenario-based assessment, $\theta(T = 1.14) = 0.47/1.14 = 0.41\,g$ and $\beta = 0.64.$ Fault rupture directivity effects were ignored for this example (but should not be ignored for project applications.)

Per Section 5.6.5 of these *Guidelines,* 11 values of target spectral acceleration are used to characterize the distribution of seismic demand for scenario-based assessments using Equation (5-7).

**Table 7-2 Spectral Demand Per the Chiou-Youngs Relationship**

| Period (sec) | $\theta$ (g) | $\beta$ |
|---|---|---|
| 1 | 0.47 | 0.63 |
| 1.1 | 0.43 | 0.64 |
| 1.2 | 0.4 | 0.64 |

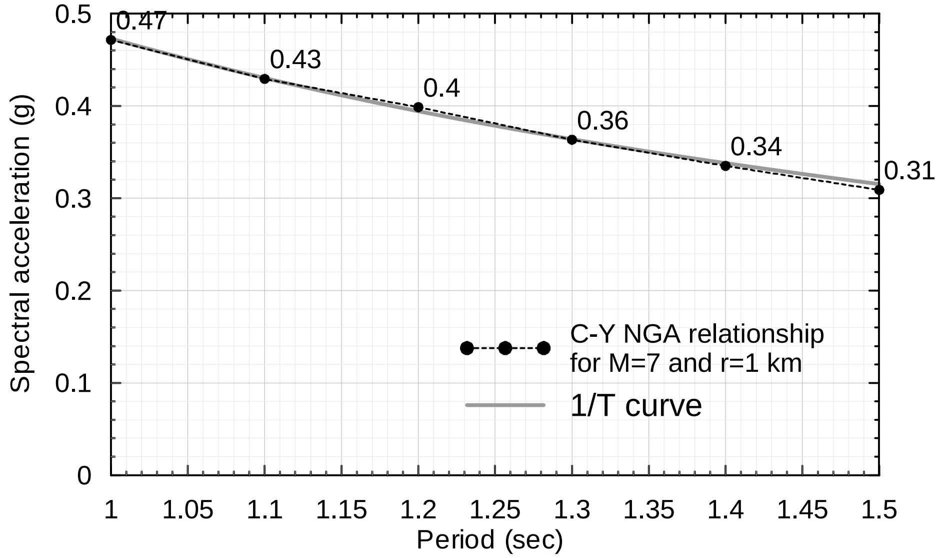



Figure 7-5 Spectral accelerations per the Chiou-Youngs NGA relationship for $M_w = 7$, $r$ = 1 km, strike-slip faulting and $v_{30}$ = 760 m/s, and varying as $0.47/T$

The eleven target spectral accelerations are (0.14, 0.20, 0.25, 0.30, 0.35, 0.41, 0.48, 0.55, 0.66, 0.83, 1.22) $g$ per Equation (5-7). Per step 5 of the scaling procedure of Section 5.6.5, a pair of motions was randomly selected from Table 7-1 and each component was amplitude scaled to one of the eleven target values. The process was repeated 10 times for the remaining target spectral ordinates and ground-motion pairs in Table 7-1. The resultant scaled motions were denoted as Bin S1 motions. Figure 7-6 presents the spectra for the Bin S1 ground motions, the 16$^{th}$, 50$^{th}$ and 84$^{th}$ percentiles of spectral acceleration and the eleven target spectral ordinates (denoted $\times$).

Figure 7-7 presents the 16$^{th}$, 50$^{th}$ and 84$^{th}$ percentiles of spectral accelerations for the Bin S1 ground motions and those predicted by the C-Y NGA relationship for $M_w = 7$, $r$ = 1 km, strike-slip faulting and $v_{30} = 760$ m/s. The distribution of spectral demand for Bin S1 ground motions is comparable to that predicted by the C-Y relationship for periods greater than 0.6 second. For periods less than 0.6 second, the median spectral accelerations are lower than those predicted by the C-Y relationship.

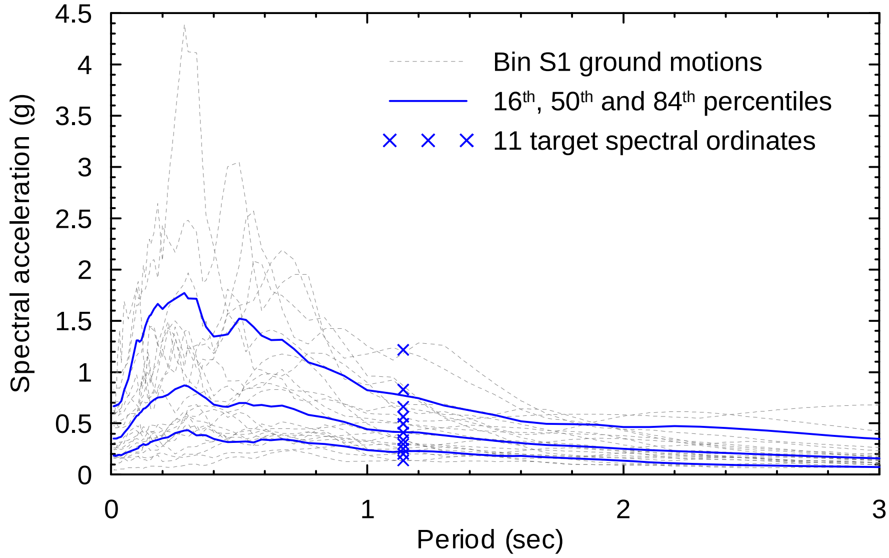



Figure 7-6 Spectral accelerations for Bin S1 ground motions, 16th, 50th and 84th percentiles of spectral acceleration and the 11 target spectral ordinates for Bin S1 motions

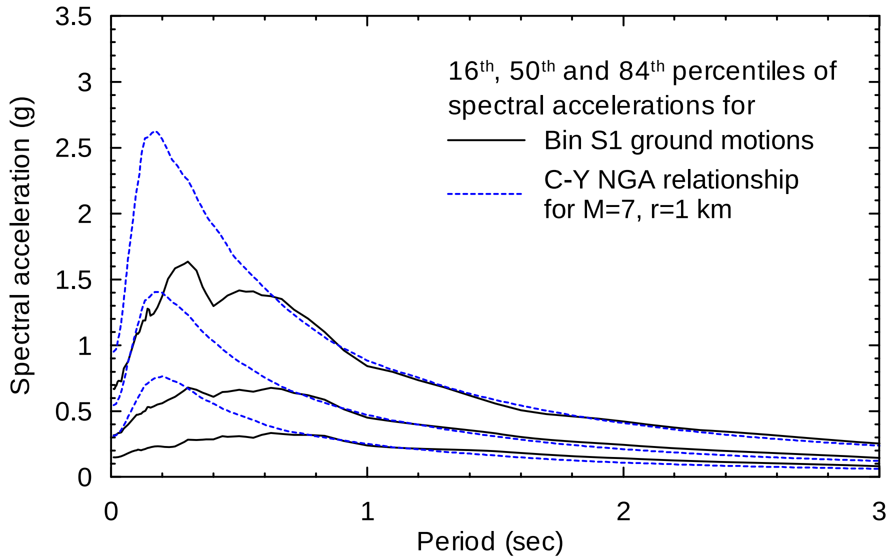



Figure 7-7 Sixteenth, 50th and 84th percentiles of spectral acceleration for Bin S1 motions and demands predicted by the Chiou-Youngs NGA relationship

**Ground Motions for Time-Based Assessment**

The USGS ground motion parameter calculator was used to generate a seismic hazard curve for the example building at a period of 1 second. Figures 7-8 and 7-9 present a screen capture from the ground motion calculator and the resultant hazard data, respectively.

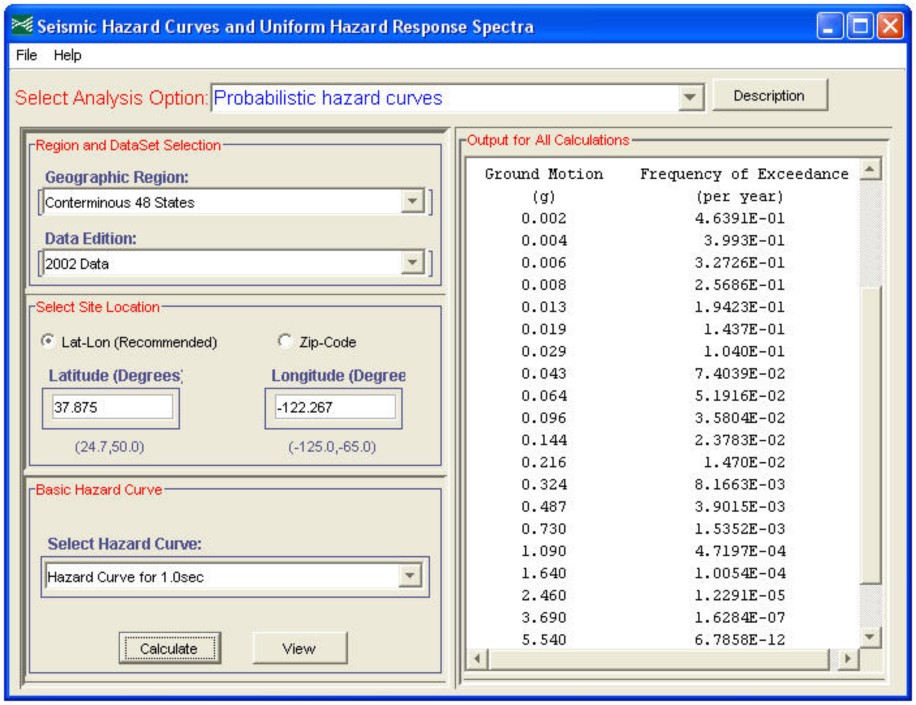



Figure 7-8 Screen capture from USGS ground motion calculator for generating a one-second hazard curve for the site of the building

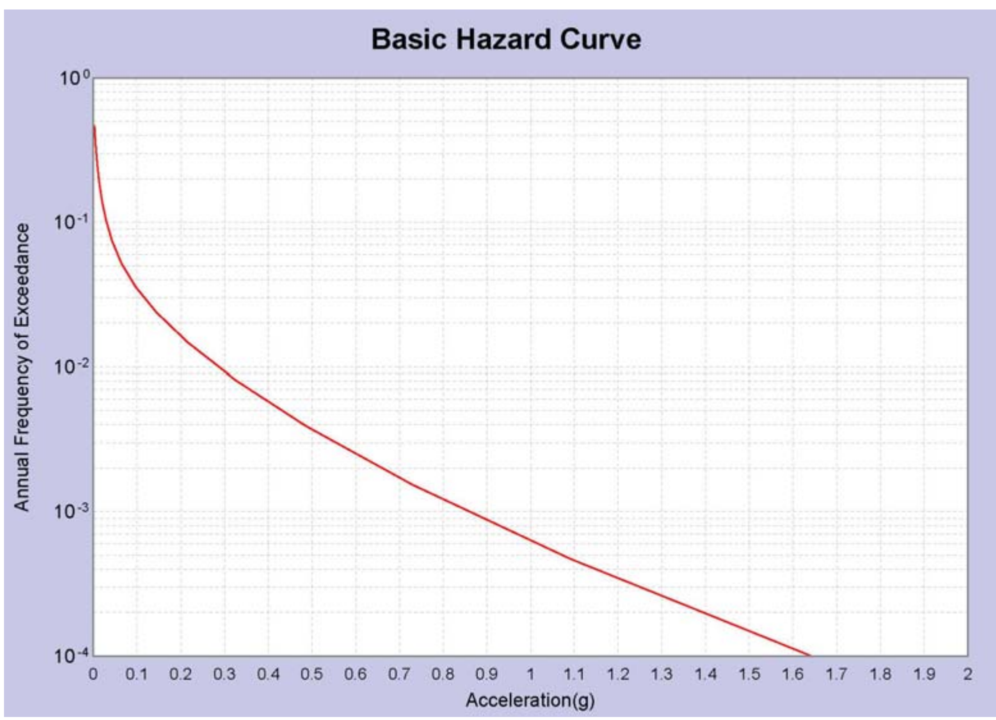



Figure 7-9 One-second seismic hazard curve

Figure 7-5 shows that the median spectral demand predicted by the C-Y NGA relationship $M_w$ = 7 and $r$ = 1 km is inversely proportional to period in the range of 1 to 1.5 seconds. Table 7-2 shows that the dispersion in spectral demand for that scenario case is essentially constant between 1 and 1.2 second. On the basis of these results (and noting that it is unlikely that the $M_w$ 7 earthquake at $r$ = 1 km will dictate the spectral demand across all mean annual frequencies of exceedance), the 1.14-second seismic hazard curve for the site was generated by dividing the 1-second values shown in Figure 7-9 by 1.14. Figure 7-10 presents four curves. Curves A and D are the 1- and 2-second seismic hazard curves, respectively, generated by the USGS ground motion calculator for the site. Curves B and C were developed by dividing the spectral demands for curve A by 1.14 and 2, respectively.

The spectral ordinates for curves C and D are virtually identical for spectral acceleration less than 0.3 *g*. The spectral demands for curve C are slightly greater than those for curve D for spectral acceleration greater than 0.3 *g*—indicating that the $S_a \propto 1/T$ is slightly conservative in the period range of 1 to 2 seconds. Curve B was used for the time-based assessment of the building. The values of spectral acceleration and the corresponding mean annual frequencies of exceedance (MAFE) for curve B are listed in Table 7-3.

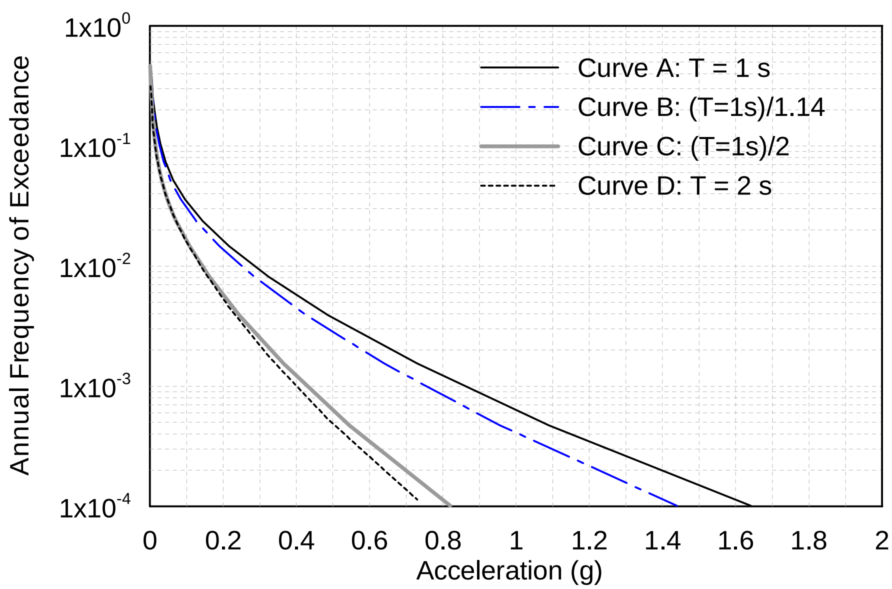



Figure 7-10 Seismic hazard curves for the site of the building

The seed ground motions in Table 7-1 were scaled per the procedure described in Section 5.6.6 of the *Guidelines* for time-based assessments. Figure 7-11 illustrates the procedure. The range of mean spectral acceleration was selected as 0.05 $g$ to 1.23 $g$, where 1.23 $g$ is the spectral acceleration at a period of 1.14 seconds corresponding to a MAFE of 0.0002. This range of mean spectral acceleration was split into eight equal intervals, $\Delta e_i$ , of 0.1475 $g$. The midpoint value in each interval characterizes a target spectral demand for the scaling of ground motions. The 8 target spectral accelerations are identified in Figure 7-11 by the symbol ⊙ with values of (0.124, 0.271, 0.419, 0.566, 0.714, 0.861, 1.009, 1.156) $g$. All of the seed ground motions of Table 7-1 were scaled to *each* of the 8 target spectral accelerations at a period of 1.14 seconds. The 22 scaled ground motions at the 8 target spectral accelerations, $e_i$ , were denoted as Bin T$i$, $i$ = 1, 8. Figure 7-12 shows the 8 target spectral ordinates and the spectra for the Bin T1 and Bin T8 ground motions.

**Table 7-3 Mean Hazard Curve**

| MAFE | Spectral acceleration (g) |
|---|---|
| 4.64E-01 | 1.75E-03 |
| 3.99E-01 | 3.51E-03 |
| 3.27E-01 | 5.26E-03 |
| 2.57E-01 | 7.02E-03 |
| 1.94E-01 | 1.14E-02 |
| 1.44E-01 | 1.67E-02 |
| 1.04E-01 | 2.54E-02 |
| 7.40E-02 | 3.77E-02 |
| 5.19E-02 | 5.61E-02 |
| 3.58E-02 | 8.42E-02 |
| 2.38E-02 | 1.26E-01 |
| 1.47E-02 | 1.89E-01 |
| 8.17E-03 | 2.84E-01 |
| 3.90E-03 | 4.27E-01 |
| 1.54E-03 | 6.40E-01 |
| 4.72E-04 | 9.56E-01 |
| 1.01E-04 | 1.44E+00 |
| 1.23E-05 | 2.16E+00 |
| 1.63E-07 | 3.24E+00 |

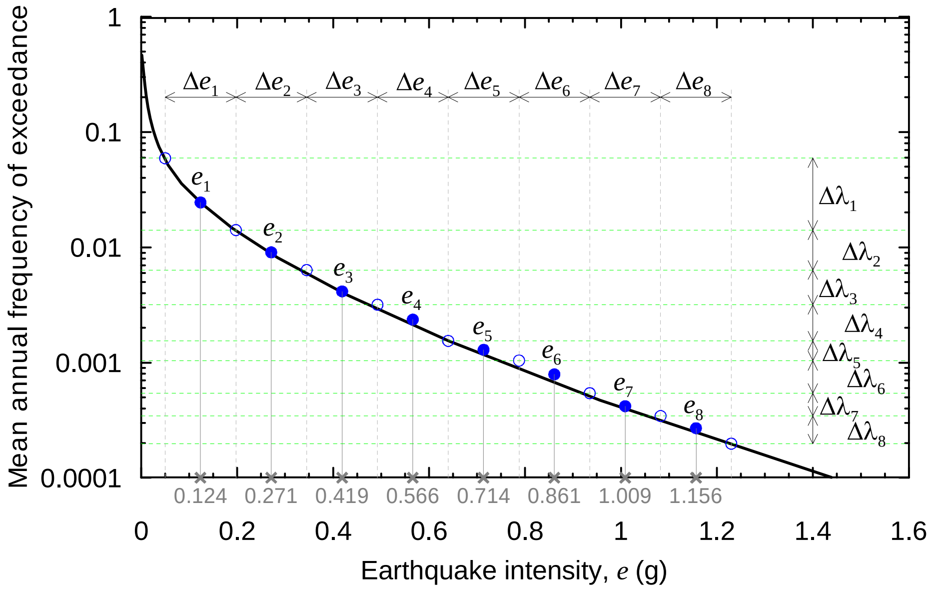



Figure 7-11 Characterizing seismic hazard for time-based assessment

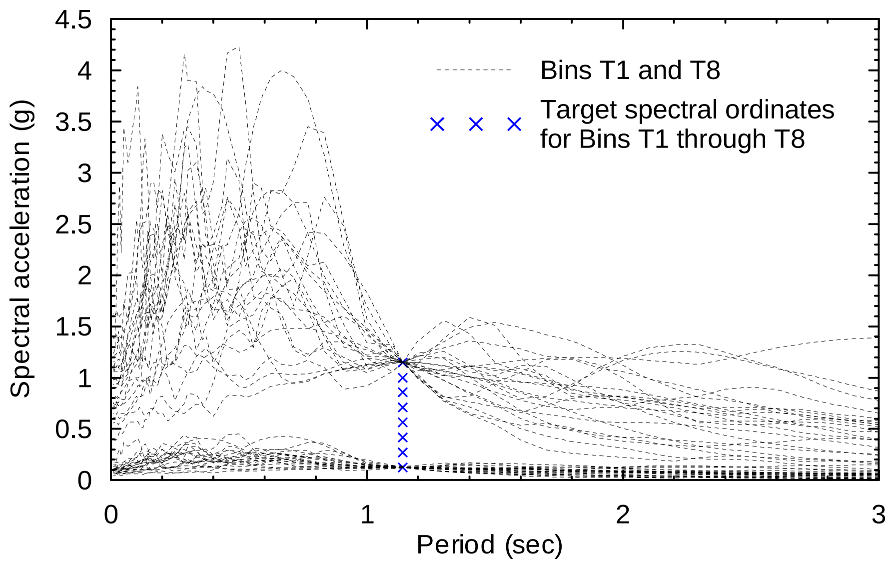



Figure 7-12 Target spectral ordinates for Bins T1 through T8 and spectra for scaled ground motions in Bins T1 and T8

The 8 values of mean annual frequency of spectral demand in interval $i$, $\Delta\lambda_i$, shown in Figure 7-11, are required for time-based assessment. Losses are computed at each of the 8 midpoint spectral demands and the annualized loss is calculated as the sum of the products of the losses conditioned on the intensity measure (midpoint spectral demand in each interval) and the corresponding mean annual frequency of shaking in the interval. The 9 values of MAFE that define the 8 intervals of spectral acceleration are listed in Table 7-4. The MAFE corresponding to a boundary between two ranges can be computed using linear interpolation and the data listed in Table 7-3. For example, the spectral boundary between $e_2$ and $e_3$ in Figure 7-11 is 0.345 *g*. The MAFE corresponding to 0.345 *g* can be computed using the shaded entries in Table 7-3 as follows:

$$\lambda_3 = 0.00817 + \frac{0.0039 - 0.00817}{0.427 - 0.284} \times (0.345 - 0.284) = 0.00635$$

The 9 values of MAFE listed in Table 7-4 were calculated in this manner. The mean annual frequency (MAF) of spectral demand in each interval is computed as the difference in the MAFEs at the boundaries of the interval. For example, the mean annual frequency of spectral acceleration at 1.14 seconds being between 0.788 and 0.935 *g* is 0.000497 (=0.00104-0.000543). Table 7-5 lists the mean annual frequencies (MAF), $\Delta\lambda_i$, and bounding spectral values (min, max) for each interval of shaking intensity.

**Table 7-4 Spectral Accelerations and MAFE ($\lambda_i$) for Boundaries on the Seismic Hazard Curve**

| *i* | Spectral acceleration (g) | $\lambda_i$ |
|---|---|---|
| 1 | 0.050 | 0.0593 |
| 2 | 0.198 | 0.0141 |
| 3 | 0.345 | 0.00635 |
| 4 | 0.493 | 0.00317 |
| 5 | 0.640 | 0.00154 |
| 6 | 0.788 | 0.00104 |
| 7 | 0.935 | 0.000543 |
| 8 | 1.083 | 0.000344 |
| 9 | 1.230 | 0.000198 |

**Table 7-5 Mean Annual Frequency (MAF) for Spectral Intervals**

| Interval | MAF | Spectral range (g)[1] |
|---|---|---|
| $\Delta\lambda_1$ | 0.0452 | 0.050, 0.198 |
| $\Delta\lambda_2$ | 0.00775 | 0.198, 0.345 |
| $\Delta\lambda_3$ | 0.00318 | 0.345, 0.493 |
| $\Delta\lambda_4$ | 0.00163 | 0.493, 0.640 |
| $\Delta\lambda_5$ | 0.000500 | 0.640, 0.788 |
| $\Delta\lambda_6$ | 0.000497 | 0.788, 0.935 |
| $\Delta\lambda_7$ | 0.000199 | 0.935, 1.083 |
| $\Delta\lambda_8$ | 0.000146 | 1.083, 1.230 |

1. Range is [min, max] spectral acceleration.

### *7.2.3 Response-History Analysis and Demand Parameter Matrices*

This section presents the matrices of demand parameters for the enhanced intensity-, scenario- and time-based performance assessment of the example building.

The scaled ground motion pairs in Bins I1, S1 and T1 through T8 were used as input to the OpenSees model of the building for intensity-, scenario- and time-based assessments, respectively. Since the building is symmetric, a two-dimensional model was used for analysis: the first component of the pair was assigned the *X* direction of the building and the second component was assigned to the *Y* direction.

Seven demand parameters, three story drifts and four peak floor accelerations were used to characterize performance. Results are attached in Tables 7-6, 7-7 and 7-8. The notation used for drift is **dr_x_y**, where **dr** denotes drift, **x** denotes the direction (1 or 2) and **y** denotes the story, with story 1 immediately above the base of the building. For accelerations, the notation is **a_x_z**, where **a** denotes acceleration, **x** denotes direction (1 or 2) and **z** denotes the floor (*g* is base and 2 is the first supported level above the base). Figure 6-3 illustrates these definitions of floor and story numbers. Story drifts and floor accelerations are organized by directions.

**Reader Note**: As noted in Chapter 5, the procedure for time-based assessment that is included in the version of PACT issued with the 35% draft *Guidelines* is a variation on that described above and in Chapter 5. In lieu of inputting the mean annual frequency of shaking in each spectral interval per Table 7-5 above, the user keys in the data required to define the seismic hazard curve per Table 7-3. (The procedure presented above that utilizes the mean annual frequency of shaking in each interval will be implemented in the next release of PACT.)

Table 7-6 **Demand Parameters for Intensity-Based Assessment**

| Number stories = | 3 | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Number of GM pairs = | 11 | Demand parameters in units of inches and g | | | | | | | | | | | | | |
| | GM pair | dr_1_1 | dr_1_2 | dr_1_3 | a_1_g | a_1_2 | a_1_3 | a_1_4 | dr_2_1 | dr_2_2 | dr_2_3 | a_2_g | a_2_2 | a_2_3 | a_2_4 |
| | 1 | 2.61 | 3.59 | 3.91 | 0.40 | 0.64 | 0.56 | 0.65 | 1.44 | 2.44 | 2.76 | 0.17 | 0.34 | 0.31 | 0.47 |
| | 2 | 4.57 | 5.66 | 5.52 | 0.62 | 0.67 | 0.82 | 0.61 | 2.64 | 2.93 | 4.02 | 0.50 | 0.91 | 0.76 | 0.69 |
| | 3 | 1.85 | 2.34 | 2.41 | 0.36 | 0.77 | 0.80 | 0.66 | 1.89 | 2.72 | 2.92 | 0.33 | 0.60 | 0.54 | 0.51 |
| | 4 | 2.91 | 3.63 | 3.39 | 0.29 | 0.46 | 0.46 | 0.68 | 2.68 | 3.67 | 3.60 | 0.27 | 0.27 | 0.33 | 0.45 |
| | 5 | 1.43 | 2.66 | 3.40 | 0.72 | 1.14 | 0.85 | 0.86 | 1.72 | 2.64 | 2.73 | 0.44 | 0.60 | 0.59 | 0.64 |
| | 6 | 2.54 | 3.28 | 3.09 | 0.32 | 0.33 | 0.42 | 0.40 | 1.94 | 2.71 | 2.50 | 0.31 | 0.36 | 0.33 | 0.38 |
| | 7 | 2.36 | 3.42 | 3.41 | 0.47 | 0.66 | 0.75 | 0.57 | 1.95 | 2.75 | 2.59 | 0.32 | 0.33 | 0.34 | 0.44 |
| | 8 | 2.43 | 3.55 | 3.73 | 0.55 | 0.58 | 0.55 | 0.53 | 3.05 | 3.52 | 3.36 | 0.54 | 0.90 | 0.78 | 0.61 |
| | 9 | 2.09 | 3.13 | 3.09 | 0.48 | 0.56 | 0.55 | 0.59 | 2.02 | 2.34 | 2.51 | 0.40 | 0.78 | 0.73 | 0.59 |
| | 10 | 1.37 | 2.32 | 2.53 | 0.51 | 1.12 | 0.87 | 0.82 | 1.71 | 2.94 | 3.52 | 0.43 | 0.55 | 0.52 | 0.53 |
| | 11 | 1.97 | 2.21 | 2.44 | 0.42 | 0.77 | 1.01 | 0.79 | 1.53 | 2.39 | 2.84 | 0.25 | 0.43 | 0.50 | 0.53 |

Table 7-7 Demand Parameters for Scenario-Based Assessment

| Number stories = | 3 | Demand parameters in units of inches and g | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **Stripe** | **dr_1_1** | **dr_1_2** | **dr_1_3** | **a_1_g** | **a_1_2** | **a_1_3** | **a_1_4** | **dr_2_1** | **dr_2_2** | **dr_2_3** | **a_2_g** | **a_2_2** | **a_2_3** | **a_2_4** |
| | **1** | 0.59 | 0.89 | 0.82 | 0.11 | 0.18 | 0.17 | 0.23 | 0.55 | 0.88 | 0.87 | 0.05 | 0.09 | 0.11 | 0.19 |
| | **2** | 1.01 | 1.37 | 1.54 | 0.25 | 0.35 | 0.40 | 0.40 | 0.99 | 1.30 | 1.76 | 0.20 | 0.40 | 0.40 | 0.43 |
| | **3** | 1.21 | 1.55 | 1.54 | 0.18 | 0.45 | 0.51 | 0.45 | 1.00 | 1.58 | 1.78 | 0.17 | 0.36 | 0.37 | 0.43 |
| | **4** | 1.21 | 1.91 | 1.81 | 0.17 | 0.27 | 0.29 | 0.34 | 1.19 | 1.92 | 1.83 | 0.16 | 0.22 | 0.30 | 0.37 |
| | **5** | 1.03 | 2.00 | 2.81 | 0.48 | 0.70 | 0.57 | 0.73 | 1.06 | 1.99 | 2.32 | 0.29 | 0.43 | 0.40 | 0.48 |
| | **6** | 1.86 | 2.62 | 2.58 | 0.26 | 0.30 | 0.39 | 0.42 | 1.57 | 2.49 | 2.50 | 0.25 | 0.29 | 0.33 | 0.39 |
| | **7** | 2.29 | 3.31 | 3.30 | 0.46 | 0.65 | 0.74 | 0.56 | 1.88 | 2.67 | 2.49 | 0.31 | 0.32 | 0.34 | 0.43 |
| | **8** | 3.14 | 4.27 | 4.37 | 0.61 | 0.60 | 0.55 | 0.55 | 3.79 | 4.34 | 4.22 | 0.60 | 0.99 | 0.81 | 0.59 |
| | **9** | 2.31 | 3.66 | 3.88 | 0.62 | 0.75 | 0.60 | 0.70 | 2.27 | 2.71 | 3.14 | 0.52 | 1.07 | 0.78 | 0.74 |
| | **10** | 1.87 | 2.93 | 3.70 | 0.83 | 1.80 | 1.30 | 0.95 | 3.13 | 5.02 | 6.20 | 0.71 | 0.90 | 0.63 | 0.72 |
| | **11** | 3.79 | 3.91 | 4.56 | 0.99 | 1.30 | 1.51 | 0.91 | 2.85 | 4.77 | 5.81 | 0.59 | 0.84 | 0.77 | 0.79 |

Table 7-8 **Demand Parameters for Time-Based Assessment**

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Number stories =** | 3 | | | | | | | | | | | | | | | |
| **Number of intensities (8 min) =** | 8 | | | | | | | | | | | | | | | |
| **Number of GM pairs =** | 11 | | **Demand parameters in units of inches and g** | | | | | | | | | | | | | |
| | **GM pair** | **MAF** | **dr_1_1** | **dr_1_2** | **dr_1_3** | **a_1_g** | **a_1_2** | **a_1_3** | **a_1_4** | **dr_2_1** | **dr_2_2** | **dr_2_3** | **a_2_g** | **a_2_2** | **a_2_3** | **a_2_4** |
| **Intensity_1** | 1 | 0.0452 | 0.53 | 0.79 | 0.73 | 0.10 | 0.16 | 0.15 | 0.21 | 0.49 | 0.79 | 0.77 | 0.04 | 0.08 | 0.10 | 0.17 |
| | 2 | | 0.61 | 0.81 | 0.92 | 0.15 | 0.22 | 0.25 | 0.25 | 0.60 | 0.79 | 1.06 | 0.12 | 0.25 | 0.25 | 0.26 |
| | 3 | | 0.62 | 0.77 | 0.78 | 0.09 | 0.23 | 0.25 | 0.23 | 0.52 | 0.79 | 0.88 | 0.08 | 0.18 | 0.18 | 0.21 |
| | 4 | | 0.50 | 0.78 | 0.73 | 0.07 | 0.11 | 0.12 | 0.17 | 0.50 | 0.79 | 0.76 | 0.07 | 0.07 | 0.12 | 0.17 |
| | 5 | | 0.39 | 0.75 | 0.98 | 0.17 | 0.22 | 0.17 | 0.32 | 0.43 | 0.74 | 0.83 | 0.11 | 0.16 | 0.15 | 0.21 |
| | 6 | | 0.55 | 0.81 | 0.81 | 0.08 | 0.11 | 0.13 | 0.19 | 0.48 | 0.77 | 0.78 | 0.07 | 0.09 | 0.11 | 0.17 |
| | 7 | | 0.54 | 0.79 | 0.90 | 0.11 | 0.18 | 0.23 | 0.25 | 0.53 | 0.78 | 0.76 | 0.08 | 0.09 | 0.14 | 0.16 |
| | 8 | | 0.55 | 0.74 | 0.88 | 0.13 | 0.19 | 0.19 | 0.22 | 0.54 | 0.76 | 0.74 | 0.13 | 0.21 | 0.17 | 0.16 |
| | 9 | | 0.58 | 0.78 | 0.72 | 0.12 | 0.17 | 0.15 | 0.16 | 0.48 | 0.77 | 0.97 | 0.10 | 0.20 | 0.23 | 0.24 |
| | 10 | | 0.48 | 0.77 | 0.76 | 0.12 | 0.28 | 0.23 | 0.24 | 0.50 | 0.76 | 0.82 | 0.10 | 0.13 | 0.15 | 0.21 |
| | 11 | | 0.69 | 0.79 | 0.98 | 0.10 | 0.26 | 0.28 | 0.28 | 0.48 | 0.78 | 0.87 | 0.06 | 0.14 | 0.15 | 0.21 |
| **Intensity_2** | 1 | 0.00775 | 1.17 | 1.68 | 1.61 | 0.21 | 0.34 | 0.34 | 0.44 | 1.03 | 1.71 | 1.72 | 0.09 | 0.17 | 0.22 | 0.35 |
| | 2 | | 1.37 | 1.94 | 2.10 | 0.33 | 0.44 | 0.50 | 0.48 | 1.34 | 1.75 | 2.33 | 0.26 | 0.49 | 0.52 | 0.49 |
| | 3 | | 1.28 | 1.62 | 1.64 | 0.19 | 0.48 | 0.54 | 0.47 | 1.04 | 1.67 | 1.87 | 0.18 | 0.39 | 0.39 | 0.44 |
| | 4 | | 1.10 | 1.72 | 1.64 | 0.15 | 0.23 | 0.25 | 0.36 | 1.08 | 1.73 | 1.65 | 0.14 | 0.14 | 0.24 | 0.35 |
| | 5 | | 0.83 | 1.61 | 2.17 | 0.38 | 0.52 | 0.41 | 0.64 | 0.91 | 1.61 | 1.83 | 0.23 | 0.35 | 0.32 | 0.43 |
| | 6 | | 1.19 | 1.79 | 1.80 | 0.17 | 0.25 | 0.28 | 0.39 | 1.03 | 1.69 | 1.74 | 0.16 | 0.19 | 0.25 | 0.37 |
| | 7 | | 1.18 | 1.75 | 1.88 | 0.25 | 0.37 | 0.48 | 0.48 | 1.13 | 1.71 | 1.67 | 0.17 | 0.19 | 0.29 | 0.34 |
| | 8 | | 1.17 | 1.64 | 2.00 | 0.29 | 0.40 | 0.40 | 0.45 | 1.19 | 1.63 | 1.56 | 0.29 | 0.47 | 0.36 | 0.35 |
| | 9 | | 1.28 | 1.69 | 1.55 | 0.25 | 0.35 | 0.32 | 0.32 | 1.03 | 1.68 | 2.15 | 0.21 | 0.43 | 0.51 | 0.47 |
| | 10 | | 1.00 | 1.63 | 1.65 | 0.27 | 0.62 | 0.51 | 0.51 | 1.07 | 1.63 | 1.79 | 0.23 | 0.29 | 0.31 | 0.42 |
| | 11 | | 1.36 | 1.54 | 2.02 | 0.22 | 0.54 | 0.61 | 0.58 | 1.03 | 1.69 | 1.92 | 0.13 | 0.27 | 0.33 | 0.44 |

Table 7-8 Demand Parameters for Time-Based Assessment (continued)

| Number of GM pairs = | 11 | | Demand parameters in units of inches and g | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | GM pair | MAF | dr_1_1 | dr_1_2 | dr_1_3 | a_1_g | a_1_2 | a_1_3 | a_1_4 | dr_2_1 | dr_2_2 | dr_2_3 | a_2_g | a_2_2 | a_2_3 | a_2_4 |
| Intensity_3 | 1 | 0.00318 | 1.81 | 2.41 | 2.83 | 0.33 | 0.52 | 0.42 | 0.58 | 1.37 | 2.39 | 2.57 | 0.14 | 0.26 | 0.28 | 0.43 |
| | 2 | | 3.03 | 3.90 | 3.65 | 0.50 | 0.57 | 0.71 | 0.53 | 2.32 | 2.66 | 3.44 | 0.41 | 0.74 | 0.69 | 0.62 |
| | 3 | | 1.55 | 1.94 | 2.24 | 0.29 | 0.66 | 0.70 | 0.61 | 1.59 | 2.55 | 2.67 | 0.27 | 0.49 | 0.51 | 0.48 |
| | 4 | | 1.99 | 2.79 | 2.69 | 0.24 | 0.39 | 0.36 | 0.55 | 1.90 | 2.82 | 2.69 | 0.22 | 0.22 | 0.29 | 0.41 |
| | 5 | | 1.22 | 2.16 | 3.21 | 0.59 | 0.90 | 0.71 | 0.80 | 1.30 | 2.13 | 2.60 | 0.36 | 0.53 | 0.50 | 0.53 |
| | 6 | | 1.89 | 2.65 | 2.61 | 0.26 | 0.30 | 0.40 | 0.41 | 1.60 | 2.51 | 2.51 | 0.25 | 0.30 | 0.33 | 0.39 |
| | 7 | | 1.88 | 2.73 | 2.71 | 0.38 | 0.57 | 0.66 | 0.54 | 1.53 | 2.22 | 2.05 | 0.26 | 0.27 | 0.35 | 0.40 |
| | 8 | | 1.67 | 2.47 | 2.86 | 0.45 | 0.49 | 0.48 | 0.51 | 2.08 | 2.65 | 2.46 | 0.44 | 0.73 | 0.60 | 0.54 |
| | 9 | | 1.78 | 2.63 | 2.55 | 0.39 | 0.52 | 0.51 | 0.51 | 1.69 | 2.21 | 2.62 | 0.33 | 0.65 | 0.68 | 0.53 |
| | 10 | | 1.28 | 2.19 | 2.31 | 0.42 | 0.95 | 0.76 | 0.73 | 1.36 | 2.41 | 2.76 | 0.36 | 0.45 | 0.46 | 0.50 |
| | 11 | | 1.63 | 1.93 | 2.26 | 0.34 | 0.65 | 0.89 | 0.73 | 1.32 | 2.16 | 2.45 | 0.20 | 0.37 | 0.46 | 0.53 |
| Intensity_4 | 1 | 0.00163 | 3.19 | 4.39 | 4.61 | 0.45 | 0.70 | 0.58 | 0.67 | 1.48 | 2.49 | 2.86 | 0.19 | 0.39 | 0.35 | 0.49 |
| | 2 | | 5.39 | 6.64 | 6.54 | 0.68 | 0.74 | 0.87 | 0.65 | 2.81 | 3.17 | 4.33 | 0.55 | 0.98 | 0.79 | 0.72 |
| | 3 | | 2.12 | 2.53 | 2.46 | 0.40 | 0.82 | 0.84 | 0.69 | 2.07 | 2.68 | 2.96 | 0.37 | 0.65 | 0.62 | 0.52 |
| | 4 | | 3.41 | 4.14 | 3.78 | 0.32 | 0.47 | 0.51 | 0.74 | 3.17 | 4.23 | 4.28 | 0.30 | 0.30 | 0.34 | 0.47 |
| | 5 | | 1.56 | 2.96 | 3.46 | 0.79 | 1.27 | 0.92 | 0.89 | 2.01 | 2.96 | 3.07 | 0.48 | 0.63 | 0.63 | 0.69 |
| | 6 | | 3.02 | 3.78 | 3.46 | 0.36 | 0.36 | 0.42 | 0.41 | 2.10 | 2.73 | 2.47 | 0.34 | 0.40 | 0.34 | 0.39 |
| | 7 | | 2.66 | 3.86 | 3.82 | 0.52 | 0.70 | 0.79 | 0.58 | 2.24 | 3.12 | 2.98 | 0.35 | 0.36 | 0.33 | 0.45 |
| | 8 | | 3.13 | 4.26 | 4.36 | 0.61 | 0.60 | 0.55 | 0.55 | 3.78 | 4.33 | 4.21 | 0.60 | 0.99 | 0.81 | 0.59 |
| | 9 | | 2.19 | 3.35 | 3.35 | 0.53 | 0.62 | 0.56 | 0.63 | 2.13 | 2.49 | 2.65 | 0.44 | 0.88 | 0.74 | 0.64 |
| | 10 | | 1.41 | 2.37 | 2.76 | 0.57 | 1.22 | 0.91 | 0.85 | 1.94 | 3.26 | 3.96 | 0.48 | 0.61 | 0.54 | 0.56 |
| | 11 | | 2.22 | 2.37 | 2.52 | 0.46 | 0.84 | 1.06 | 0.81 | 1.64 | 2.53 | 3.04 | 0.27 | 0.45 | 0.50 | 0.56 |

Table 7-8 Demand Parameters for Time-Based Assessment (continued)

| Number of GM pairs = | 11 | | Demand parameters in units of inches and g | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | GM pair | MAF | dr_1_1 | dr_1_2 | dr_1_3 | a_1_g | a_1_2 | a_1_3 | a_1_4 | dr_2_1 | dr_2_2 | dr_2_3 | a_2_g | a_2_2 | a_2_3 | a_2_4 |
| Intensity_5 | 1 | 0.000500 | 4.89 | 6.52 | 6.99 | 0.56 | 0.82 | 0.61 | 0.69 | 2.02 | 2.76 | 3.24 | 0.24 | 0.49 | 0.39 | 0.54 |
| | 2 | | 7.18 | 8.70 | 8.60 | 0.86 | 0.82 | 0.87 | 0.78 | 3.44 | 3.81 | 4.98 | 0.69 | 1.20 | 1.04 | 0.82 |
| | 3 | | 2.81 | 3.13 | 3.14 | 0.50 | 0.97 | 0.89 | 0.77 | 2.99 | 3.16 | 3.65 | 0.47 | 0.80 | 0.85 | 0.62 |
| | 4 | | 4.59 | 5.44 | 5.13 | 0.40 | 0.58 | 0.65 | 0.89 | 4.59 | 6.09 | 6.44 | 0.38 | 0.38 | 0.38 | 0.60 |
| | 5 | | 1.91 | 3.81 | 4.41 | 1.00 | 1.60 | 1.05 | 0.97 | 2.88 | 3.93 | 4.25 | 0.61 | 0.84 | 0.74 | 0.82 |
| | 6 | | 4.62 | 5.62 | 5.33 | 0.45 | 0.45 | 0.52 | 0.45 | 2.67 | 3.50 | 3.28 | 0.43 | 0.50 | 0.43 | 0.48 |
| | 7 | | 3.42 | 5.00 | 4.95 | 0.66 | 0.76 | 0.82 | 0.60 | 3.30 | 4.22 | 4.00 | 0.44 | 0.46 | 0.40 | 0.49 |
| | 8 | | 4.63 | 5.76 | 5.65 | 0.76 | 0.64 | 0.54 | 0.60 | 6.34 | 7.20 | 7.59 | 0.75 | 1.12 | 0.78 | 0.60 |
| | 9 | | 2.35 | 3.77 | 4.12 | 0.67 | 0.81 | 0.67 | 0.74 | 2.32 | 2.88 | 3.39 | 0.56 | 1.14 | 0.85 | 0.78 |
| | 10 | | 1.65 | 2.73 | 3.30 | 0.72 | 1.56 | 1.13 | 0.91 | 2.64 | 4.23 | 5.19 | 0.61 | 0.77 | 0.61 | 0.65 |
| | 11 | | 2.71 | 2.77 | 2.87 | 0.58 | 0.89 | 1.13 | 0.86 | 1.83 | 2.82 | 3.51 | 0.35 | 0.53 | 0.61 | 0.62 |
| Intensity_6 | 1 | 0.000497 | 6.07 | 7.97 | 8.83 | 0.68 | 0.91 | 0.75 | 0.74 | 2.80 | 3.60 | 3.80 | 0.29 | 0.51 | 0.42 | 0.57 |
| | 2 | | 8.79 | 10.40 | 10.02 | 1.04 | 1.01 | 0.83 | 0.86 | 4.10 | 4.34 | 5.50 | 0.83 | 1.48 | 1.31 | 0.94 |
| | 3 | | 2.92 | 3.09 | 3.49 | 0.60 | 1.09 | 0.90 | 0.82 | 3.86 | 4.35 | 5.16 | 0.56 | 0.96 | 1.03 | 0.68 |
| | 4 | | 5.63 | 6.51 | 6.61 | 0.48 | 0.66 | 0.70 | 1.01 | 6.23 | 8.34 | 8.82 | 0.45 | 0.46 | 0.46 | 0.72 |
| | 5 | | 2.20 | 4.69 | 5.67 | 1.21 | 1.82 | 1.20 | 1.13 | 3.85 | 5.01 | 5.69 | 0.74 | 1.06 | 0.83 | 0.93 |
| | 6 | | 7.06 | 8.25 | 7.85 | 0.55 | 0.52 | 0.62 | 0.48 | 3.44 | 4.43 | 4.31 | 0.52 | 0.61 | 0.51 | 0.53 |
| | 7 | | 3.97 | 5.88 | 5.88 | 0.79 | 0.86 | 0.80 | 0.67 | 4.48 | 5.52 | 5.25 | 0.54 | 0.55 | 0.48 | 0.53 |
| | 8 | | 5.79 | 6.94 | 6.78 | 0.92 | 0.83 | 0.63 | 0.68 | 10.32 | 12.06 | 11.65 | 0.91 | 1.15 | 0.90 | 0.71 |
| | 9 | | 2.41 | 4.05 | 4.77 | 0.81 | 1.00 | 0.89 | 0.84 | 2.84 | 3.23 | 4.10 | 0.67 | 1.32 | 1.05 | 0.86 |
| | 10 | | 1.95 | 2.93 | 3.78 | 0.86 | 1.84 | 1.33 | 0.96 | 3.23 | 5.21 | 6.45 | 0.73 | 0.93 | 0.64 | 0.73 |
| | 11 | | 3.04 | 2.83 | 3.22 | 0.70 | 1.00 | 1.15 | 0.89 | 2.09 | 3.33 | 4.18 | 0.42 | 0.61 | 0.68 | 0.66 |

Table 7-8 **Demand Parameters for Time-Based Assessment (continued)**

| Number of GM pairs = | 11 | | Demand parameters in units of inches and g | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | GM pair | MAF | dr_1_1 | dr_1_2 | dr_1_3 | a_1_g | a_1_2 | a_1_3 | a_1_4 | dr_2_1 | dr_2_2 | dr_2_3 | a_2_g | a_2_2 | a_2_3 | a_2_4 |
| | 1 | | 6.60 | 8.76 | 10.08 | 0.79 | 1.01 | 0.86 | 0.82 | 3.66 | 4.54 | 4.81 | 0.34 | 0.56 | 0.50 | 0.59 |
| | 2 | | 10.43 | 12.32 | 11.84 | 1.22 | 1.17 | 0.94 | 0.87 | 4.63 | 5.06 | 6.15 | 0.98 | 1.69 | 1.59 | 1.07 |
| | 3 | | 3.74 | 4.42 | 4.62 | 0.71 | 1.14 | 0.98 | 0.82 | 4.22 | 5.04 | 6.04 | 0.66 | 0.95 | 1.01 | 0.79 |
| | 4 | | 6.58 | 7.49 | 8.06 | 0.57 | 0.59 | 0.76 | 1.12 | 7.87 | 10.69 | 11.23 | 0.53 | 0.53 | 0.54 | 0.81 |
| | 5 | | 2.52 | 5.54 | 6.85 | 1.41 | 1.89 | 1.36 | 1.26 | 4.85 | 6.21 | 7.25 | 0.86 | 1.20 | 0.95 | 1.02 |
| **Intensity_7** | 6 | 0.000199 | 9.81 | 11.54 | 11.26 | 0.64 | 0.58 | 0.70 | 0.53 | 4.49 | 5.29 | 5.22 | 0.61 | 0.70 | 0.54 | 0.55 |
| | 7 | | 4.38 | 6.64 | 6.80 | 0.93 | 1.04 | 0.91 | 0.71 | 5.73 | 7.02 | 6.94 | 0.63 | 0.65 | 0.56 | 0.54 |
| | 8 | | 6.44 | 7.43 | 7.75 | 1.08 | 1.07 | 0.80 | 0.80 | 16.55 | 18.70 | 18.66 | 1.07 | 1.16 | 0.97 | 0.68 |
| | 9 | | 2.47 | 4.27 | 5.28 | 0.95 | 1.11 | 1.01 | 0.94 | 3.41 | 3.64 | 4.69 | 0.79 | 1.43 | 1.14 | 0.90 |
| | 10 | | 2.27 | 2.94 | 4.20 | 1.01 | 2.02 | 1.46 | 0.98 | 3.68 | 6.08 | 7.61 | 0.86 | 1.09 | 0.69 | 0.79 |
| | 11 | | 3.35 | 3.25 | 3.53 | 0.82 | 1.13 | 1.32 | 0.91 | 2.45 | 3.96 | 4.92 | 0.49 | 0.70 | 0.73 | 0.68 |
| | 1 | | 7.32 | 9.61 | 11.34 | 0.91 | 1.15 | 1.02 | 0.87 | 4.29 | 5.35 | 5.89 | 0.39 | 0.64 | 0.53 | 0.63 |
| | 2 | | 11.72 | 13.97 | 13.79 | 1.39 | 1.45 | 1.04 | 0.98 | 5.01 | 5.95 | 7.42 | 1.12 | 1.84 | 1.87 | 1.18 |
| | 3 | | 5.72 | 6.63 | 7.01 | 0.81 | 1.28 | 1.03 | 0.84 | 4.82 | 6.45 | 6.99 | 0.75 | 1.02 | 1.05 | 0.87 |
| | 4 | | 7.43 | 8.48 | 9.42 | 0.65 | 0.75 | 0.83 | 1.24 | 9.53 | 12.89 | 13.60 | 0.61 | 0.61 | 0.62 | 0.90 |
| | 5 | | 2.97 | 6.36 | 7.93 | 1.62 | 2.11 | 1.56 | 1.36 | 5.88 | 7.50 | 8.89 | 0.99 | 1.26 | 1.07 | 1.11 |
| **Intensity_8** | 6 | 0.000146 | 13.30 | 15.57 | 15.38 | 0.73 | 0.69 | 0.79 | 0.57 | 6.41 | 7.40 | 7.35 | 0.70 | 0.79 | 0.60 | 0.57 |
| | 7 | | 5.04 | 7.30 | 7.72 | 1.06 | 1.21 | 1.11 | 0.76 | 7.03 | 8.64 | 8.76 | 0.72 | 0.74 | 0.63 | 0.56 |
| | 8 | | 7.91 | 9.70 | 10.29 | 1.24 | 1.26 | 0.94 | 0.91 | 22.44 | 25.84 | 26.59 | 1.22 | 1.25 | 1.02 | 0.79 |
| | 9 | | 2.97 | 4.98 | 6.20 | 1.09 | 1.11 | 1.07 | 1.03 | 3.88 | 4.10 | 5.31 | 0.91 | 1.48 | 1.19 | 0.92 |
| | 10 | | 2.51 | 3.47 | 4.67 | 1.16 | 2.12 | 1.58 | 0.99 | 4.06 | 6.84 | 8.71 | 0.98 | 1.24 | 0.79 | 0.85 |
| | 11 | | 3.65 | 3.73 | 4.23 | 0.94 | 1.25 | 1.46 | 0.91 | 2.81 | 4.60 | 5.62 | 0.56 | 0.83 | 0.76 | 0.75 |

### 7.2.4 Input of Data into the Performance Assessment Calculation Tool (PACT)

The input pages for PACT are accessed through an input hub as shown in Figure 7-13. To begin click on *Edit General Info*.

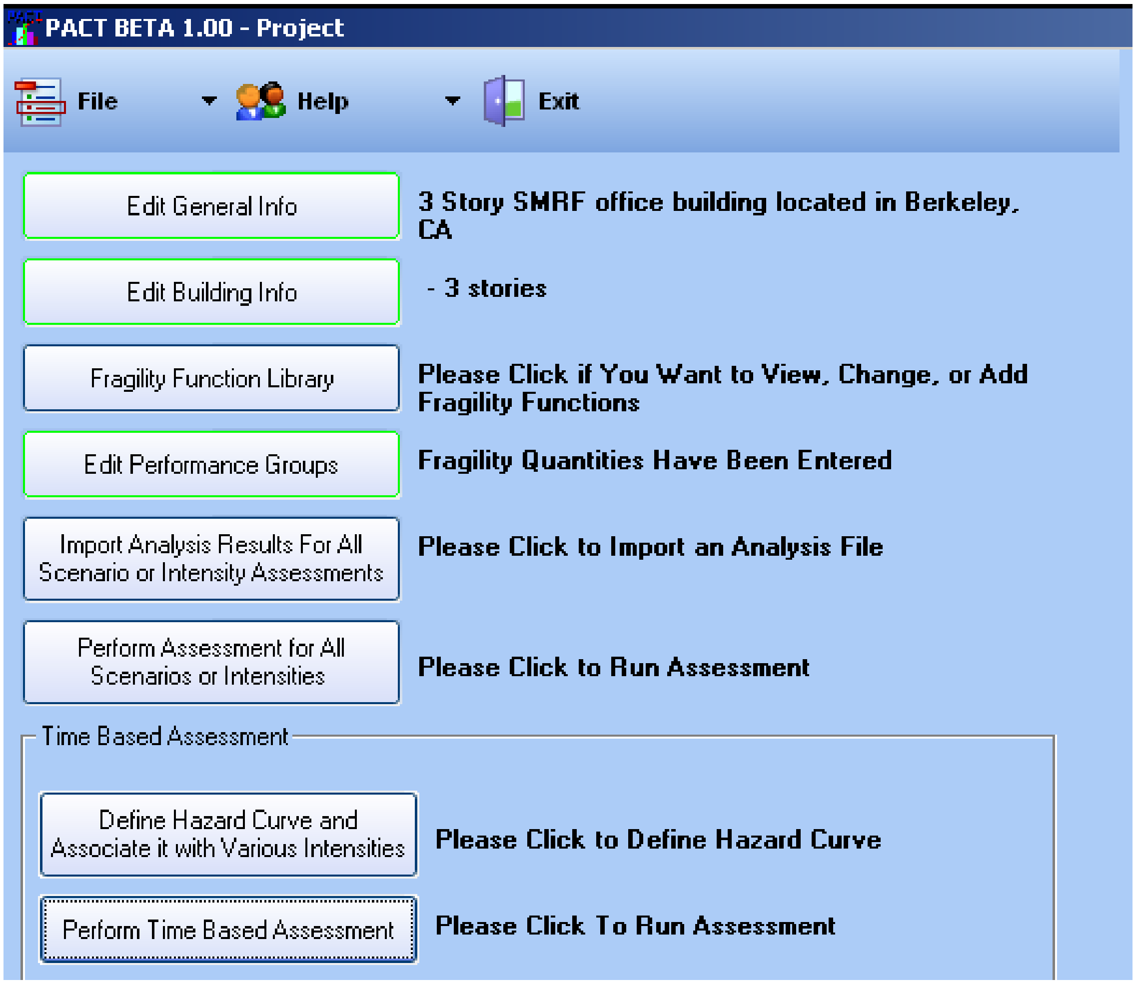



Figure 7-13 PACT Input Hub

This will bring up the general information entry screen, shown in Figure 7-14. Note that this screen will show user-specific information, based on data entered when PACT is first installed on the computer. Enter data as follows:

- Enter a project identification in the top data entry box. This can be a building name, project number or similar identification.
- In the second data entry box, enter a brief description of the building. This is not actually used by the program, but is provided to allow later identification of the data file as to the building assessed.
- In the third data entry box enter the client's name.

- In the fourth data entry box, enter the name of the engineer performing the assessment.
- Left-click on the "return" button to store the data and return to the input hub.

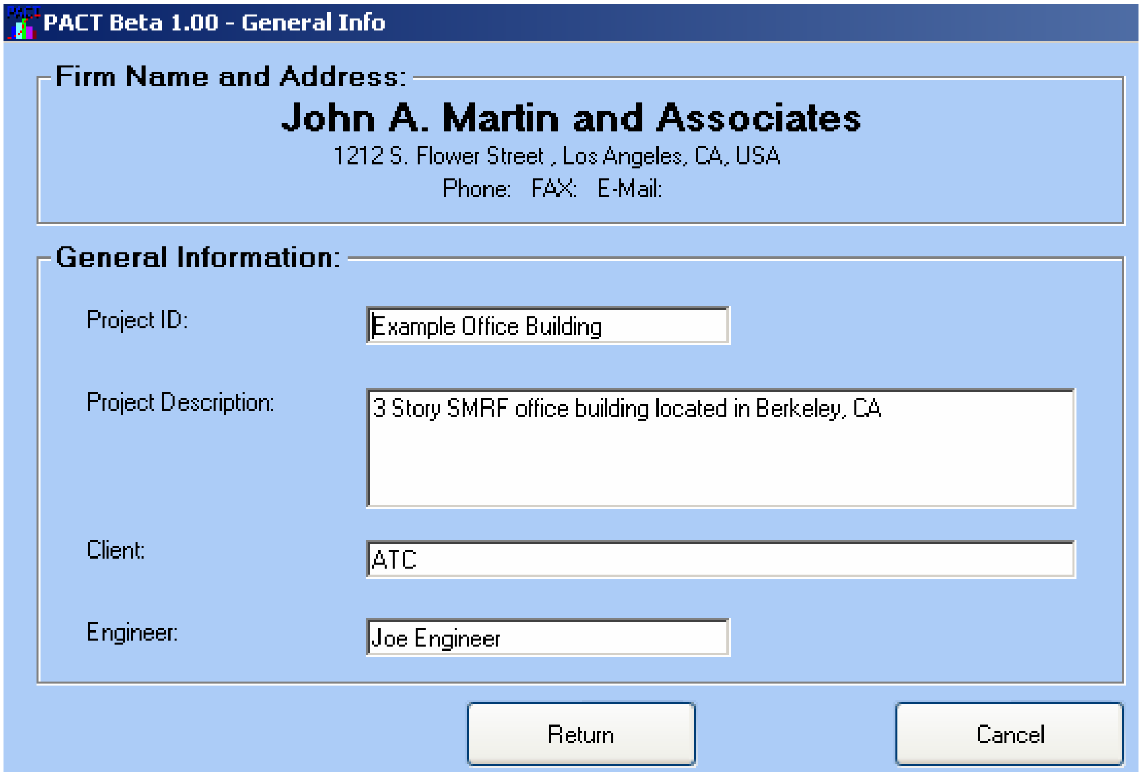


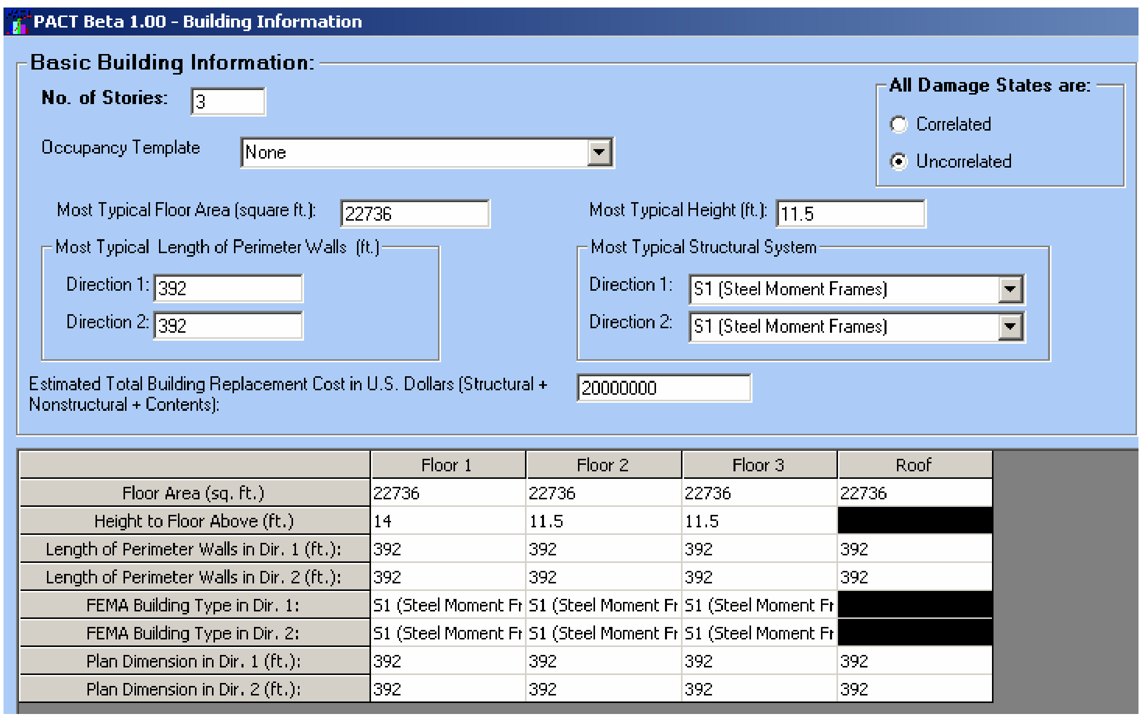

Figure 7-14 PACT General Info Page

To begin entering quantitative data on the building configuration, left-click on the "enter building info" button on the input hub screen. This will bring up the building information screen as shown in Figure 7-15.

In the Building Information Screen (Figure7-15) enter data as follows:

- In the top data left hand data entry window, enter the number of stories above grade.
- In the pull-down window below the story data entry box, select the building occupancy template. The user may specify the use of an occupancy template that is included in PACT. When this alternative is selected, PACT "seeds" the performance groups and quantities of performance groups for the building using typical quantities based on architectural standards. The user can start from scratch and simply enter the performance group information directly as explained below.

- Beneath the occupancy window, enter the typical area of a single floor in square feet. Note that for buildings with set-backs, it is possible to modify this area for different floors. PACT simplifies the data input process using the *Most Typical* quantities designation. For example, in Figure 4-1 the user has entered 11.5 as the *Most Typical Story Height*. The datasheet in the lower panel is interactive and initially includes data for each story based on the *Most Typical* quantities. It is possible to override the *Most Typical* quantities. In this example, the height for the first story has been changed from the *Most Typical* value of 11.5 feet to 14 feet.


PACT Beta 1.00 - Building Information

Basic Building Information:

No. of Stories: 3

Occupancy Template: None

All Damage States are: Correlated / Uncorrelated

Most Typical Floor Area (square ft.): 22736

Most Typical Height (ft.): 11.5

Most Typical Length of Perimeter Walls (ft.)

Direction 1: 392

Direction 2: 392

Most Typical Structural System

Direction 1: S1 (Steel Moment Frames)

Direction 2: S1 (Steel Moment Frames)

Estimated Total Building Replacement Cost in U.S. Dollars (Structural + Nonstructural + Contents): 20000000

| | Floor 1 | Floor 2 | Floor 3 | Roof |
|---|---|---|---|---|
| Floor Area (sq. ft.) | 22736 | 22736 | 22736 | 22736 |
| Height to Floor Above (ft.) | 14 | 11.5 | 11.5 | |
| Length of Perimeter Walls in Dir. 1 (ft.): | 392 | 392 | 392 | 392 |
| Length of Perimeter Walls in Dir. 2 (ft.): | 392 | 392 | 392 | 392 |
| FEMA Building Type in Dir. 1: | S1 (Steel Moment Fr | S1 (Steel Moment Fr | S1 (Steel Moment Fr | |
| FEMA Building Type in Dir. 2: | S1 (Steel Moment Fr | S1 (Steel Moment Fr | S1 (Steel Moment Fr | |
| Plan Dimension in Dir. 1 (ft.): | 392 | 392 | 392 | 392 |
| Plan Dimension in Dir. 2 (ft.): | 392 | 392 | 392 | 392 |

Figure 7-15 PACT Building Information Page

- In the data entry box located to the right of the floor area data box, enter the typical story height. Note that this can also be modified later for atypical stories.
- In the two boxes below the entry for *Typical Floor Area*, enter the total length of perimeter wall in the east-west (direction 1) and north-south (direction 2) in feet, for the most typical floor. This can be modified later for individual floors.

ta 1.00 - Fragility Quantities

on 1 ○ Direction 2 ○ Non Directional

Fill Data Based on Chosen Template

| | Fragility | Unit | Performance Group Quantities | | |
|---|---|---|---|---|---|
| | | | Story 1 (Floor 1 to 2) | Story 2 (Floor 2 to 3) | Story 3 (Floor 3 to Roof) |
| 0 | Post 1994 welded steel moment frame | Each | 12 | 12 | 12 |
| a | Exterior Wall OSB and stucco Type 3a | Sq. Ft. | 0 | 0 | 0 |
| 1 | Exterior Skin-Glass Curtainwall - Type 1 | Sq. Ft. | 5488 | 4508 | 4508 |
| .a | Interior Walls GWB on Wood studs | Ft. | 0 | 0 | 0 |
| 9a | Interior Partitions Type 9a | Ft. | 7958 | 6537 | 6537 |

ta 1.00 - Fragility Quantities

on 1 ◉ Direction 2 ○ Non Directional

Fill Data Based on Chosen Template

| | Fragility | Unit | Performance Group Quantities | | |
|---|---|---|---|---|---|
| | | | Story 1 (Floor 1 to 2) | Story 2 (Floor 2 to 3) | Story 3 (Floor 3 to Roof) |
| 0 | Post 1994 welded steel moment frame | Each | 12 | 12 | 12 |
| a | Exterior Wall OSB and stucco Type 3a | Sq. Ft. | 0 | 0 | 0 |
| 1 | Exterior Skin-Glass Curtainwall - Type 1 | Sq. Ft. | 5488 | 4508 | 4508 |
| .a | Interior Walls GWB on Wood studs | Ft. | 0 | 0 | 0 |
| 9a | Interior Partitions Type 9a | Ft. | 7958 | 6537 | 6537 |

1.00 - Fragility Quantities

1 ○ Direction 2 ◉ Non Directional

Fill Data Based on Chosen Template

| Fragility | Unit | Performance Group Quantities | | | |
|---|---|---|---|---|---|
| | | Floor 1 | Floor 2 | Floor 3 | Roof |
| Exterior Roofing Concrete tile type 2 | Sq. Ft. | 0 | 0 | 0 | 0 |
| Ceiling Systems Suspended acoustical tile type1 | Sq. Ft. | 22736 | 22736 | 22736 | 0 |
| Conveying - Hydraulic elevator | Each | 3 | 0 | 0 | 0 |
| Roof Mounted Equipment | Each | 0 | 0 | 0 | 1 |
| Miscellaneous housewares and art objects | Each | 0 | 0 | 0 | 0 |
| Home Entertainment Equipment | Each | 0 | 0 | 0 | 0 |
| Desktop Computers | Each | 57 | 57 | 57 | 0 |
| Servers and network Equipment | Each | 1 | 1 | 1 | 0 |
| Tall File Cabinet | Each | 76 | 76 | 76 | 0 |
| Unanchored Bookcase | Each | 76 | 76 | 76 | 0 |

Figu

Figure 7-16 PACT Performance Group Quantity Page(s)

- In the data entry boxes to the left, enter the structural system, in accordance with Section 2.3 in each of the east-west (direction 1) and north-south (direction 2) directions of response.
- Enter the total estimated replacement cost for the structure in current dollars.

- Left-click the “return” button to return to the input hub.

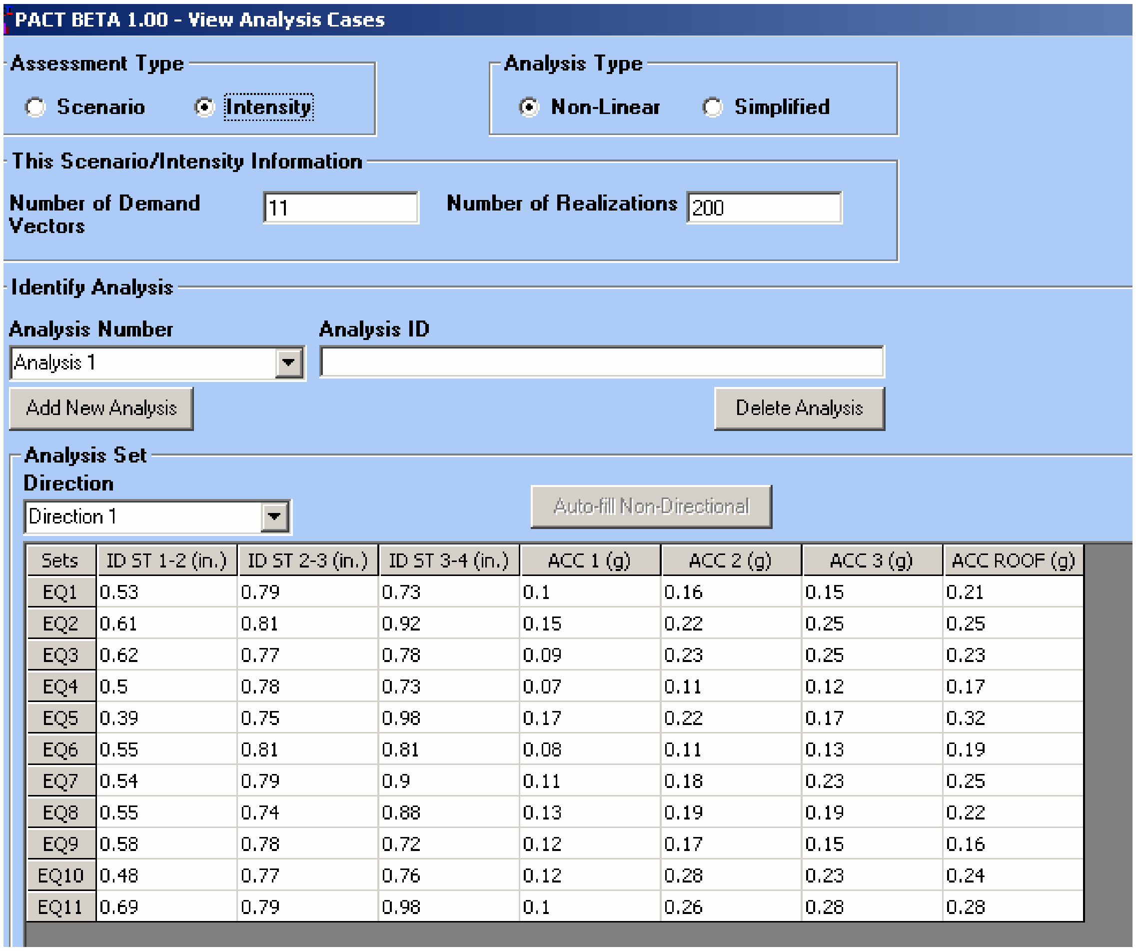
The next step in the process is to formulate performance groups for the building. Click *Edit Performance Groups* on the input hub. This brings up the *Performance Group Quantity Page(s)* as shown in Figure 7-16

In this example, PACT automatically forms default performance groups and assigns default fragility specifications based on the occupancy, structural type, and basic building information provided. The user specifies this option by clicking on “fill data based on chosen template for steel moment frame office building. Appendices D and E contain a more complete listing of the default performance groups and fragilities contained within the PACT database.

Performance groups are summarized by those subject to damage cause by displacement in each of the two orthogonal directions (Direction 1 and Direction 2) and those sensitive to acceleration in any direction (Direction-independent). The radio buttons switch among these three options. It is possible to edit any of the quantities in the performance groups and change or add fragility specifications to correct for conditions in the actual building. Left-click the *Return* button to return to the input hub.

The next step in the process is to enter the results of the response analyses. Click on “enter analysis results for all scenario or intensities. This will bring up the *View Analysis Cases* page shown in Figure 7-17.

For each intensity or scenario, the results of the response analysis, in terms of drifts and accelerations from the tables in the previous section, are entered into the program. There are results for each orthogonal direction. The nondirectional data can be “autofilled.” This results in the use of the maximum response in either direction. This is useful for acceleration sensitive performance groups. When all the data is entered click on *Return* to go back to the input hub.




| Sets | ID ST 1-2 (in.) | ID ST 2-3 (in.) | ID ST 3-4 (in.) | ACC 1 (g) | ACC 2 (g) | ACC 3 (g) | ACC ROOF (g) |
|---|---|---|---|---|---|---|---|
| EQ1 | 0.53 | 0.79 | 0.73 | 0.1 | 0.16 | 0.15 | 0.21 |
| EQ2 | 0.61 | 0.81 | 0.92 | 0.15 | 0.22 | 0.25 | 0.25 |
| EQ3 | 0.62 | 0.77 | 0.78 | 0.09 | 0.23 | 0.25 | 0.23 |
| EQ4 | 0.5 | 0.78 | 0.73 | 0.07 | 0.11 | 0.12 | 0.17 |
| EQ5 | 0.39 | 0.75 | 0.98 | 0.17 | 0.22 | 0.17 | 0.32 |
| EQ6 | 0.55 | 0.81 | 0.81 | 0.08 | 0.11 | 0.13 | 0.19 |
| EQ7 | 0.54 | 0.79 | 0.9 | 0.11 | 0.18 | 0.23 | 0.25 |
| EQ8 | 0.55 | 0.74 | 0.88 | 0.13 | 0.19 | 0.19 | 0.22 |
| EQ9 | 0.58 | 0.78 | 0.72 | 0.12 | 0.17 | 0.15 | 0.16 |
| EQ10 | 0.48 | 0.77 | 0.76 | 0.12 | 0.28 | 0.23 | 0.24 |
| EQ11 | 0.69 | 0.79 | 0.98 | 0.1 | 0.26 | 0.28 | 0.28 |

Figure 7-17 PACT View Analysis Cases Page

### 7.2.5 Loss Computations Using PACT

Once all of the data is input into PACT, the user clicks on "perform assessment for all scenarios or intensities" in the input hub. For scenario or intensity based assessments the losses are viewed using the screen shown in Figure 7-18.

The analysis to be viewed is selected from the upper drop-down menu on the left. The loss curve shows the total losses for the building subjected to the specified scenario or intensity. The losses are plotted on the horizontal axis. The probability of the losses being equal to or less than any specific value is read from the vertical axis – P(total repair cost<= $C). In the lower left of the display screen are controls that facilitate the reading of losses from the curve. If a loss value is typed into the dialog box, the corresponding probability of non-exceedance will appear in the adjacent box and the cursor will move to the designated point on the plot.

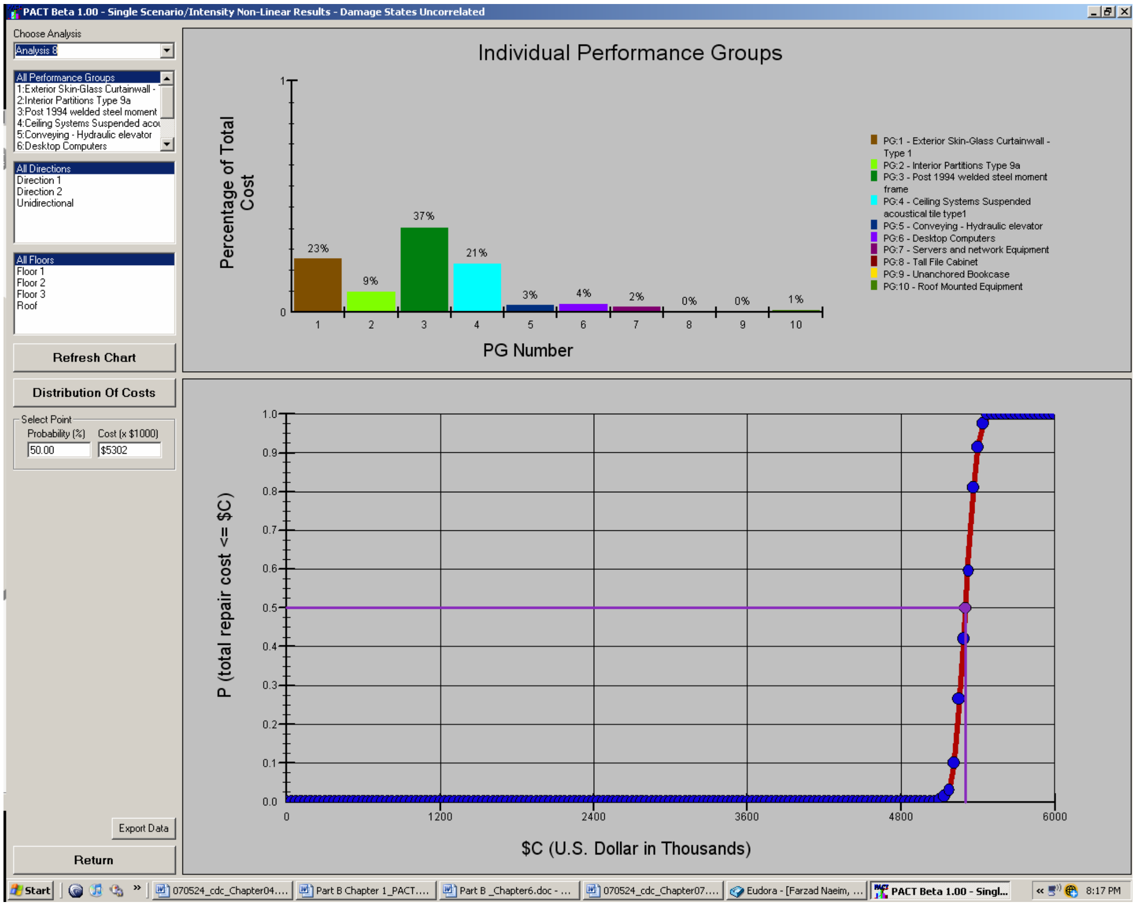



Figure 7-18 PACT Intensity and Scenarios Based Loss Page

Alternatively, the probability of non-exceedance can be entered and the loss will be automatically provided in the adjacent box. Also, it is possible to simply move the cursor to the desired point along the graph to obtain readouts in the output boxes. In Figure 7-18 the total losses for the intensity or scenario have a 50% probability of being less than or equal to $5,300,000 (median loss). The bar chart above the loss curve shows the contribution of each performance group to the loss. Below "analysis" menu there are three boxes that allow the user to select which losses are to be displayed in the bar chart. The first box lists the various types of performance groups, for which losses can be individually viewed; the second box allows specification of the direction of shaking; and the lower box designates the story within the levels of the building for which the losses are to be displayed. In the figure, as an example, direct economic losses are plotted for all performance groups from all directions, located on all floors of the building.

To view the results of a time-based assessment, click on *Perform Time-Based Assessment* on the input hub. This will bring up the page shown in Figure 7-19.

The vertical axis for a time based assessment is the annual probability that the loss on the horizontal axis will be exceeded. The integration of the time-based loss curve results in the total annualized loss (i.e. the total loss that would be expected spread out over time). This information is shown in the upper right corner of the page.

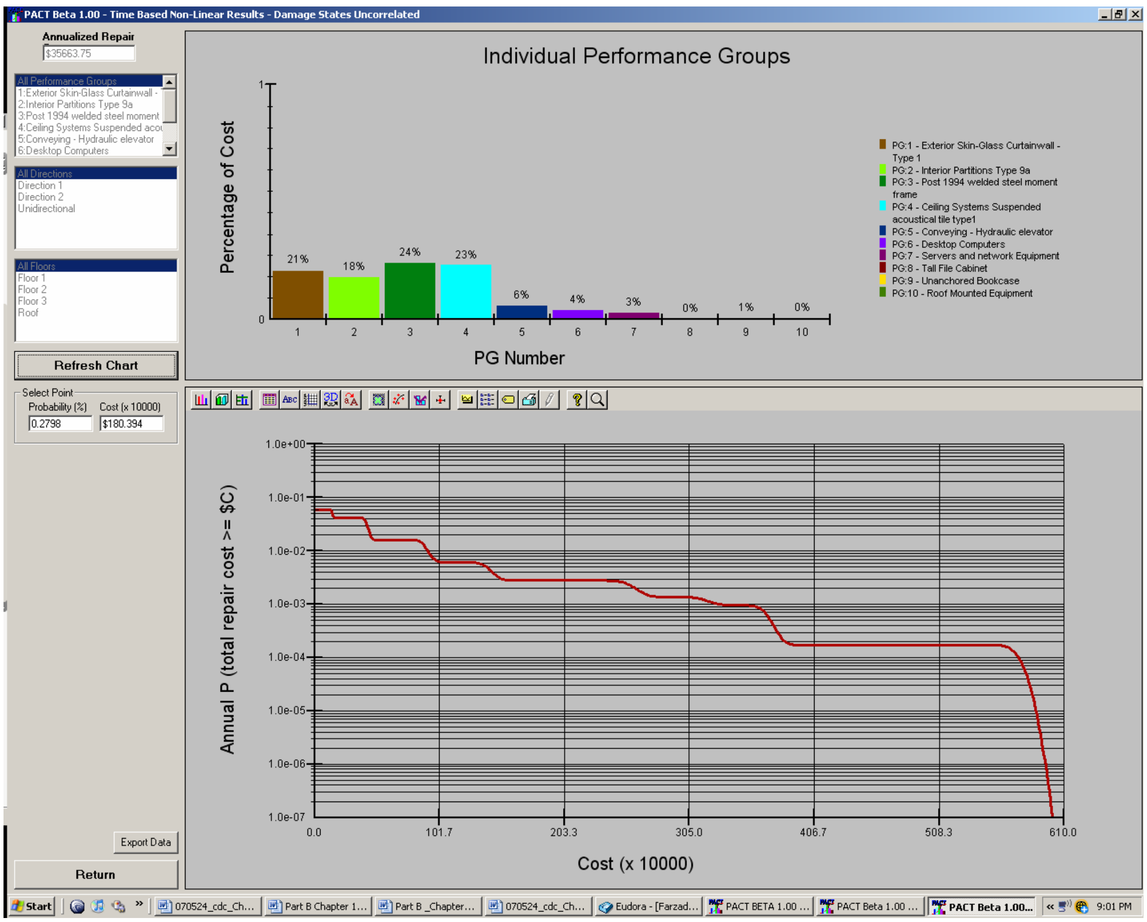



Figure 7-19 PACT Time-Based Assessment Loss Page

# Appendix A

# Probability, Statistics & Distributions

## A.1 Introduction

This appendix provides a brief tutorial on probability and statistics including methods of expressing probabilities in the form of various types of distributions. It is intended to provide readers who are unfamiliar with these topics the basic information essential to an understanding of the process used to account for the uncertainties inherent in performance assessment. Interested readers may wish to obtain additional information on this subject by reference to texts on probability and statistics and texts on structural reliability theory. In particular, the text by Benjamin and Cornell contains a wealth of information on this topic.

## A.2 Statistical Distributions

A statistical distribution is a mathematical representation of the probability of encountering a specific outcome, or an outcome that is either greater than or less than the specific outcome, given a set of possible outcomes. The set representing all possible outcomes is termed a population. There are generally two broad types of populations considered in statistical studies. The first of these is a finite population of outcomes, where each possible outcome has a discrete value representing one of the finite number of possible outcomes. The second of these is an infinite population of possible outcomes. Each of these is discussed separately, below.

### *A.2.1 Finite Populations and Discrete Outcomes*

Consider the classic case of a coin thrown in the air to determine an outcome. One of two possible outcomes will occur each time the coin is tossed. One potential outcome is that the coin will land “heads-up” and the other that it will land “heads-down.” Which way the coin will land on a given toss is a function of a number of factors including which way we hold the coin before we toss it; the technique we use to toss the coin; how hard or high we toss it; and how, it lands. We could never hope to precisely simulate each of these factors, and therefore, the occurrence of a “heads-up” or “heads-down” outcome in a given coin toss appears to be a random phenomena, that is not predictable. There is an equal change that the coin will land “heads up” or "heads down," and we can not know before tossing the coin, which way it will land.

If we toss a coin in the air one time there are two possible outcomes. These two outcomes – coin lands "heads-up" and con lands "heads-down" completely define all possibilities in one coin toss and therefore, the probability that the coin will land either heads up or heads down is 100%. In essence, this illustrates the total probability theorem, that is that the sum of the probabilities of all possible outcomes will be 1.0.

The probability that the coin will land "heads-up" is 0.5, or 50%. The probability that it will land "heads-down" is the inverse of this, calculated as one minus the probability of "heads-up," or (1-0.5) = 0.5, also 50%.

If we toss the coin in the air many times, we would expect that we would get a heads-up outcome in half of these tosses and heads-down outcome in the other half. This does not mean that every second toss of the coin will have a "heads-up" outcome. We might obtain several successive "heads-up" outcomes or several successive "heads-down" outcomes, however, over a great many tosses, we should have approximately the same number of each possible outcome.

### *A.2.2 Combined Probabilities*

A combined probability is the probability of experiencing a specific combination of two or more independent outcomes. We can calculate combined probability of two independent events as the product of the probability of outcome 1 and the probability of outcome 2. That is:

$$P(A+B) = P(A) * P(B) \tag{A-1}$$

To illustrate this, if we toss the coin into the air two times, there is a 50% chance, each time that the coin will land "heads-up." The chance that the coin will land "heads-up" both times we toss the coin is calculated, using equation A-1, as the probability of landing "heads-up" the first time (0.5) multiplied by the probability that it will land "heads-up" the seconds time (0.5), or (0.5 x 0.5) = 0.25, or 25%.

This probability means that if we toss a coin into the air twice in succession, a large number of times, and record the number of "heads-up" outcomes, in each pair of tosses, approximately 25% of the total number of pairs of tosses will have two successive heads-up outcomes. The 25% probability does not mean that every fourth time we make a pair of coin tosses we will get two successive heads-up outcomes. There is some possibility that we will get two successive heads-up outcomes several times in a row and there is also a possibility that we will have to make more than four pairs of coin tosses to obtain an outcome of two heads-up in any of the pairs of tosses. However, over a very large number of pairs of tosses, one fourth of the pairs should be successive heads-up outcomes.

If we toss the coin into the air three times, there is again a 50% chance on each toss that the coin will land "heads-up." The probability that the coin will land "heads-up" all three times is given by the probability that it will land "heads-up" the first two times (25%) multiplied by the probability that it will land "heads-up" the third time (50%), or (0.25 x 0.5) = 0.125, or 12.5%. If we repeat this exercise with 4 coin tosses, the probability that all four will land "heads-up" will be the probability that the first three tosses will land "heads-up" (12.5%) times the probability that the fourth toss will land "heads-up" or (0.125 x 0.5 = 0.0625) or 6.25%. That is, there is approximately a 6% chance that we will have two successive pairs of coin tosses both having two heads-up outcomes.

### *A.2.3 Mass Distributions*

A probability mass distribution is a plot of the probability of occurrence of each of the possible outcomes in a finite population of discrete outcomes, such as a coin landing either "heads-up" or "heads-down" a specified number of times in N-throws.

Consider the case of four coin tosses. As shown in the previous section, there is a 6.25% chance that all four coin tosses will land "heads-up." There is also a 6.25% chance that all four coin tosses will land "head-down" or that none of the coin tosses will land "heads-up." The chance that three of the four coin tosses will land "heads-up" is equal to the combined chance of any of the following outcomes: "T, H, H, H"; "H, T, H, H"; "H, H, T, H" or "H, H, H, T." The chance of each of these outcomes is the same as having all four tosses landing "heads-up" or 6.25%. Therefore, the chance of exactly three out of four tosses landing "heads-up" is equal to 6.25% for the "T, H, H, H" combination, plus 6.25% for the "H, T, H, H" combination, plus 6.25% for the "H, H, T, H" combination, plus 6.25% for the "H, H, H, T" combination, or a total of 25% (6.25% + 6.25% + 6.25% +6.25%). Similarly, the probability that exactly three of the four tosses will be "heads-down," or that only one of the tosses will land "heads-up" is 25%. We can use similar approaches, to show that the probability that exactly half the tosses will land "heads-up" is 37.5%.

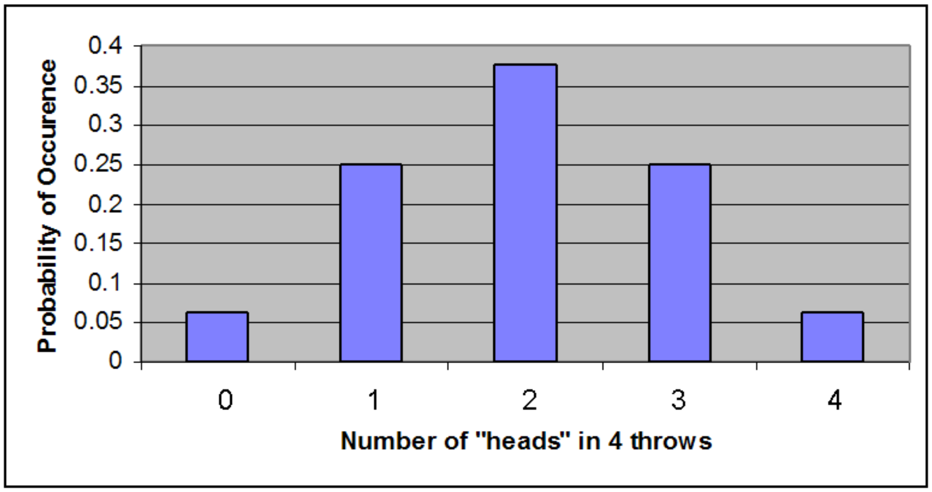



Figure A-1 Probability mass function indicating the probability of "n" numbers of "heads-up" outcomes in four successive coin tosses

Figure A-1 is a plot that shows the probability of obtaining zero, one, two, three, or four "heads-up" outcomes from four coin tosses. Plots of this type are termed probability mass distributions. By entering the plot along the horizontal axis, at a particular outcome, for example – "1 heads-up" out of four throws, we can read vertically to see the probability of this outcome, which is 25%. The probability of having not more than one "heads-up" toss in four tosses, is calculated as the sum of the probability of no "heads-ups", which is 6.25% plus the probability of 1 "heads-up" which is 25%, for a cumulative probability of 31.25%. Another way to say this is that the probability of nonexceedance for "1 heads-up in 4 coin tosses" is 31.25%. The inverse of this, that is, the cumulative probability of more than "1 heads-up" is 1-.3125 or 68.75%. This could also be called the probability of exceedance of "1 heads-up toss in 4 tosses."

### *A.2.4 Continuous Distributions*

The distribution of possible "heads-up" throws in a finite number of coin tosses is an example of a discrete distribution. That is, there are only a finite number of possible outcomes, and these have discrete values, in the example above, 0, 1, 2, 3 or 4 "heads-up" outcomes in 4 coin tosses.

For many situations, the possible outcomes are not a finite number of discrete possibilities but rather a continuous range of possibilities. An example of such a continuous distribution is that of possible compressive strengths obtained from concrete cylinder compression tests, where all cylinders are from concrete mixed using the same mix design. Such a distribution will have the form shown in Figure A-2, where the dispersion in strength is due to minor variability in the amounts of cement, amount of water, strength of aggregates, cylinder-casting technique, curing technique etc. There are an infinite number of possible outcomes for the strength of any particular cylinder test. The form of the distribution shown in Figure A-2 is termed a probability density function.

The area under the curve of a probability density function, between any two points along the horizontal axis gives the probability that the value of any member of the population will be within the range defined by the two values. Figure A-3 illustrates this. In the figure, the probability that a single cylinder test, conforming to the population represented by Figure A-2 will have strength that is greater than 4,000 psi but less than or equal to 5,000 psi is given by the area under the curve between the two strength values, which in this case has a calculated probability of 54%. That is, there is a 54% chance that any cylinder test in this population will have strength between 4,000 psi and 5,000 psi.

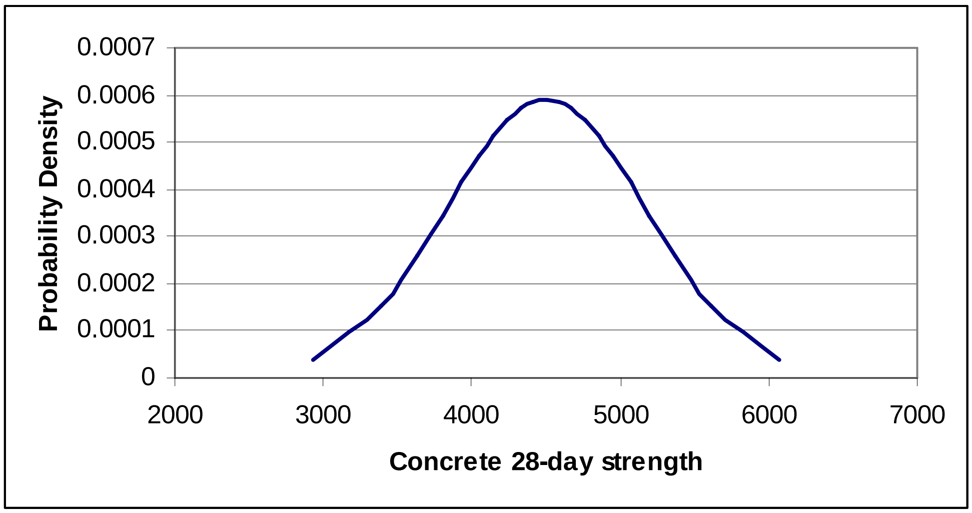



Figure A-2 Distribution of possible concrete cylinder strengths for a hypothetical mix design

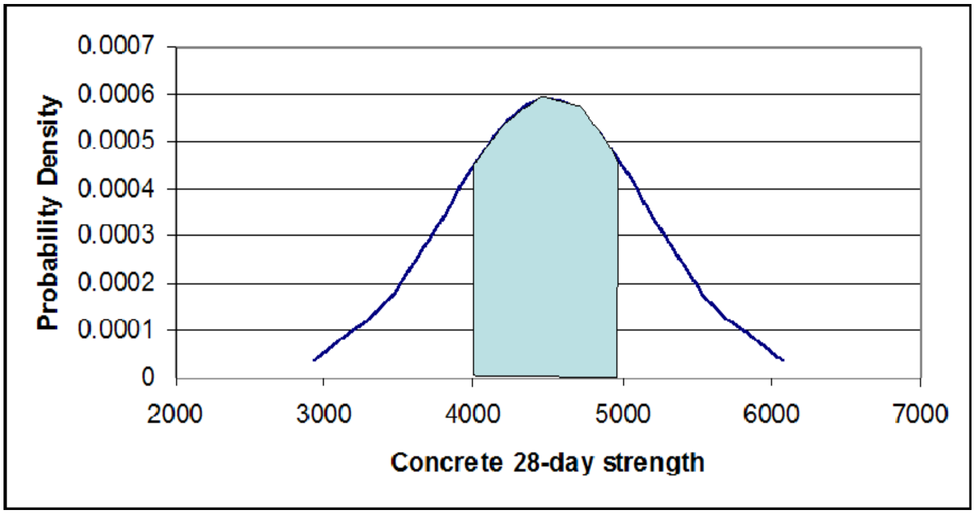



Figure A-3 Calculation of probability that a member of the population will have a value within a defined range.

## A.3 Common Forms of Distributions

### *A.3.1 Normal Distributions*

The probability density function illustrated in Figure A-2 has a special set of properties known as a normal distribution. First, the "median" outcome, that is, the outcome that is exceeded 50% of the time is also equal to the average or "mean" outcome, which is the total value of all possible outcomes, divided by the number of possible outcomes. In the example above, the median outcome is that the concrete has a compressive strength of 4,500 psi. The average strength of all cylinders tested is also 4,500 psi. Normal distributions are also symmetric. That is, there is an equal probability of having a value at a defined measure above the average, say 1.5 times the average as there is of having a value the same defined distance below the average, say .5 times the average.

Two parameters are used to uniquely specify the characteristics of a normal distribution, namely, the mean value $\bar{x}$ and standard deviation, $\sigma$. Equations A-2 and A-3, define these values for a random variable *x* (e.g.,, compressive strength of concrete as measured by cylinder testing) and a sample of size *N*:

$$\bar{x} = \frac{\sum_{i=1}^{N} x_i}{N} \qquad \text{(A-2)}$$

$$\sigma = \sqrt{\frac{1}{N-1}\sum_{i=1}^{N}\left(x_i - \bar{x}\right)^2} \qquad \text{(A-3)}$$

where, $N$ is the size of the population. If a parameter is normally distributed, that is, its population of possible outcomes is represented by a normal distribution, having mean value $\bar{x}$ and standard deviation $\sigma$, it is possible to determine the value $x$ of an outcome that has a specific probability of exceedance, based on the number of standard deviations that the value $x$ lies away from the mean, $\bar{x}$. For example, there is a 97.7% chance that the value of any single outcome $x$ will be greater than $\bar{x}$ -2$\sigma$, an 84.1% chance that the value of any single outcome $x$ will be greater than $\bar{x}$ - $\sigma$, a 50% chance it will be greater than $\bar{x}$, a 15.8% chance it will exceed $\bar{x} + \sigma$ and a 2.3% chance that it will be greater than $\bar{x}$ +2$\sigma$. Standard tables that are available in most texts on probability theory indicate the probability that the value of any outcome in a normally distributed population of potential outcomes will exceed a value that is a defined number of standard deviations from the mean. The number of standard deviations above or below the mean that will have a specified probability of exceedance is sometimes termed the Gaussian variate and the probability tables that give these values, Gaussian tables.

Structural engineers use normal distribution to represent the distribution of values for many random quantities (e.g., concrete compressive strength). In addition to the mean, median and standard deviation, another often-used parameter to characterize the properties of a normal distribution is the coefficient of variation (COV). The coefficient of variation is calculated as the standard deviation, $\sigma$, divided by the mean value $\bar{x}$. It is useful because it represents a normalized measure of the scatter inherent in a normally distributed population. Figure A-4 plots probability density functions for several normal distributions, all having mean values of 1.0 and coefficients of variation of 0.1, 0.25, and 0.5, respectively.

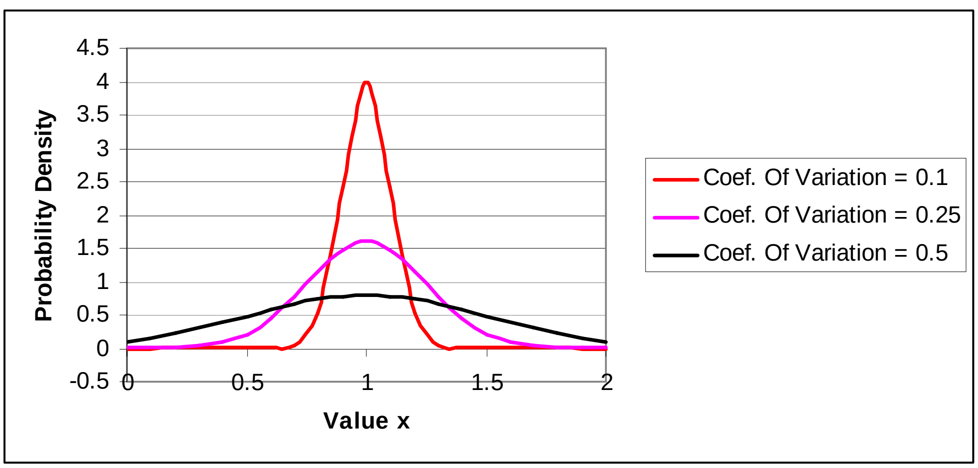



Figure A-4 Probability density function plots of normal distributions with mean values of 1.0 and coefficients of variation of 0.1, 0.25 and 0.5.

### *A.3.2 Cumulative Probability Functions*

An alternative means of plotting probability distributions is in the form known as a cumulative probability function. A cumulative probability function plot shows the probability that an outcome in the population of possible outcomes will have a value that is less than or equal to the specified value *x*. This probability is sometimes termed the probability of non-exceedance. Cumulative density plots are obtained by integrating over the probability density function to determine the area under the curve between an *x* value of zero and any other value of *x*. Figure A-5 presents cumulative probability plots for the same normally distributed populations previously shown in Figure A-4. From either series of plots, it is possible to observe that the larger the coefficient of variation becomes, the greater the amount of scatter in possible outcome values.

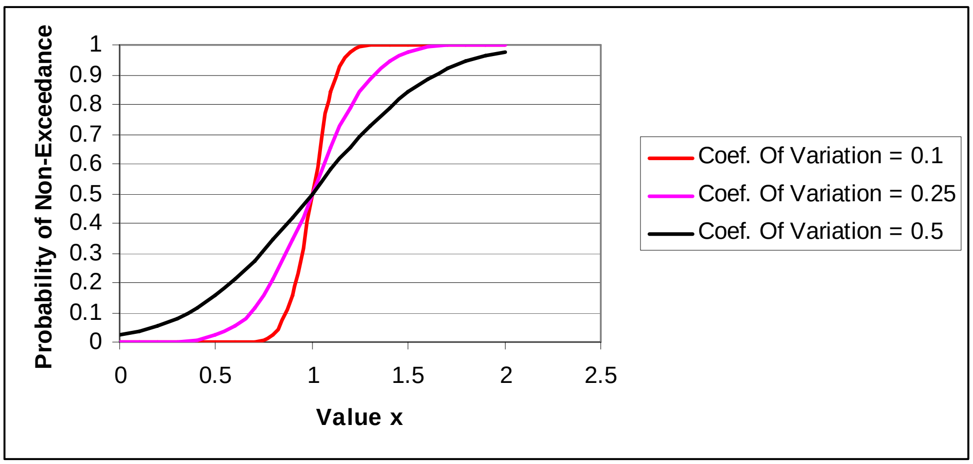



Figure A-5 Cumulative probability plots of normal distributions with coefficients of variation of 0.1, 0.25, and 0.5.

### *A.3.3 Lognormal Distributions*

Although normal distributions represent some random variables well, they do not represent all variables well. Some variables have skewed distributions. In skewed distributions, the mean value, $\bar{x}$, will be either greater than or less than the median value. In structural reliability applications, such as performance assessment, it is common to use a specific type of skewed distribution known as a lognormal distribution. This is because the skew inherent in lognormal distributions can reasonably represent the distributions observed in many structural engineering phenomena, such as the distribution of strength in laboratory specimens.

The lognormal distribution has the property that the natural logarithm of the values of the population $\text{In}(x)$ are normally distributed. The mean and the median of the natural logarithms of the population have the same value, which is equal to $\ln(\theta)$, where $\theta$ is the median value. In these guidelines, the standard deviation of the natural logarithms of the values $\sigma_{\ln(x)}$ is called the dispersion and is denoted by the symbol $\beta$. For relatively small values of dispersion, the value of $\beta$ is approximately equal to the coefficient of variation for $x$. Together, the values of $\theta$ and $\beta$ completely define the lognormal distribution for a population.

Figure A-6 plots probability density functions for three lognormally distributed populations having median values of 1.0 and dispersions of 0.1, 0.25 and 0.5 respectively. Figure A-7 plots this same data in the form of cumulative probability functions.

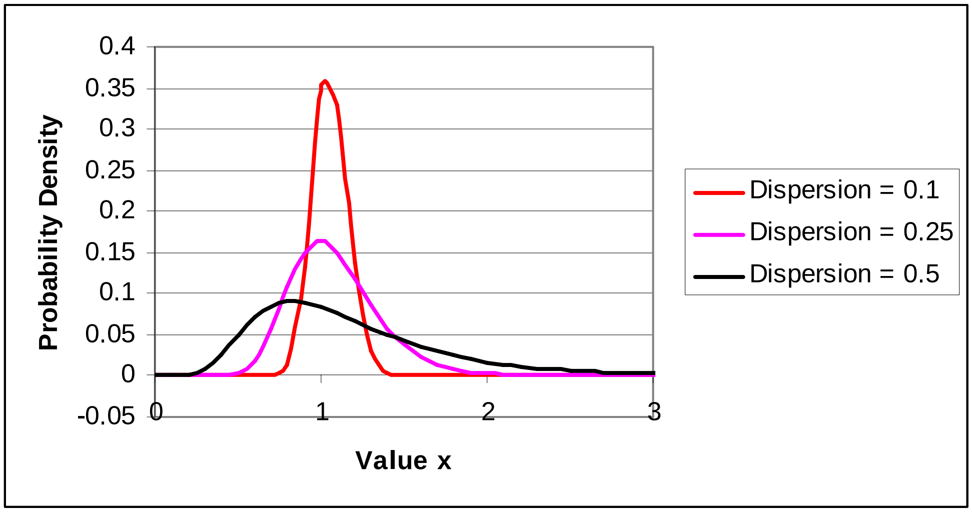



Figure A-6 Probability density function plots of lognormal distributions with median values of 1.0 and dispersions of 0.1, 0.25 and 0.5.

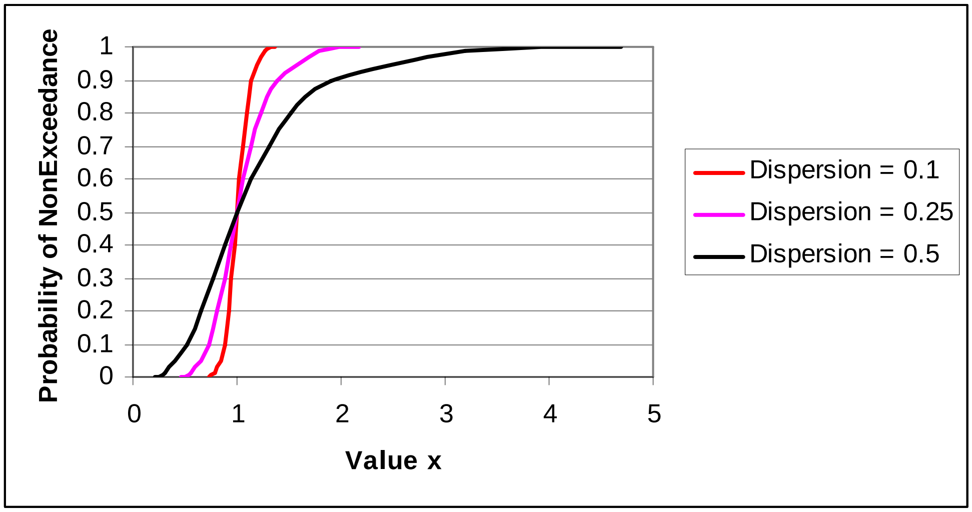



Figure A-7 Cumulative probability plots of lognormal distributions with median values of 1.0 and dispersions of 0.1, 0.25, and 0.5.

As can be seen from the plots, for small values of the dispersion, the distribution approaches the shape of the normal distribution, but as the dispersion increases in value, the distribution becomes more and more skewed. This skew is such that there is nil probability of incurring a negative value in the distribution (because the logarithm of a negative number is positive) and extreme values above the median are substantially more probable than extreme values below the median. Consider the distribution of possible actual yield strengths of various steel parts, all conforming to a specific ASTM specification and grade. Since this variable (yield strength) cannot take on values of less than zero, the lognormal distribution could be used to describe the variation in possible steel strength using a median value and dispersion. In this example, the median value will be substantially higher than the minimum specified value, which, is intended to have a very low probability of non-exceedance by any steel conforming to the specification.

The procedures presented in these guidelines use lognormal distributions to represent the distributions of intensity for a scenario earthquake; the values of response parameters given an intensity; the probability of incurring a damage state as a function of response parameter, herein termed a fragility; and the probability of incurring a specific level of direct economic loss, downtime and casualties, given a damage state. These lognormal distributions are derived in a variety of ways. Specifically,

- Intensity distributions are obtained based on statistical analysis of actual ground motion data recorded from past earthquakes, and represented in the form of equations known as attenuation relationships
- Response parameter distributions are obtained by performing suites of structural analyses using multiple ground motion recordings and varying the strength, stiffness, damping, and ductility capacity of the structural elements
- Fragility distributions are obtained primarily through laboratory testing or observation of actual damage in past earthquakes
- Loss distributions are obtained through performing multiple calculations of loss, varying the factors that affect the loss, such as contractor efficiency, and number of persons likely to be occupying a building at the time of the earthquake

Regardless of how these distributions are derived, they are represented by two variables, the median value, $\theta$ and dispersion $\beta$.

The mean $m_Y$ and standard deviation $\sigma_Y$ of a lognormal distribution *Y*, can be calculated from $\theta_Y$ and $\beta_Y$ as follows:

$$m_Y = \theta_Y \exp\left(\frac{\beta_Y^2}{2}\right) \tag{A-4}$$

$$\sigma_Y^2 = m_Y^2\left[\exp\left(\beta_Y^2\right) - 1\right] \tag{A-5}$$

The coefficient of variation, $\nu_Y$, is given by

$$\nu_Y = \frac{\sigma_Y}{m_Y} = \sqrt{\exp(\beta_Y^2) - 1} \tag{A-6}$$

Many common spreadsheet applications also have embedded within them, functions that will automatically solve lognormal distribution problems. For example, in Microsoft Excel, the LOGNORMDIST function will determine the cumulative probability of non-exceedance of any value in a population *Y*, based on input of the value, *y*, the natural logarithm of the median $\ln\theta_Y$ and the dispersion, $\beta_Y$. The Excel input is of the form =LOGNORMDIST (*y*, $\ln\theta_Y$, $\beta_Y$). Similarly, the LOGINV function can be used to determine the value of *y*, at a given cumulative probability of non-exceedance, based on the desired probability, *p*, which must be less than 1.0, the natural logarithm of the median $\ln\theta_Y$ and the dispersion, $\beta_Y$. The Excel input is of the form =LOGINV(*p*, $\ln\theta_Y$, $\beta_Y$).

## Appendix B
# Ground Shaking Hazards

## B.1 Scope

This appendix presents supplemental information on the characterization of ground shaking hazards.

## B.2 Geomean, Maximum and Minimum Horizontal Shaking

**Reader Note**: Text that describes the relationships between geometric mean (geomean), rotated geometric mean, maximum, minimum, fault normal and fault parallel shaking will be presented in the next draft of this *Guideline*. The presentation will draw on work currently under way in BSSC Project 07, which will define the characterization of seismic hazard in the 2008 *NEHRP Recommended Provisions*.

## B.3 Vertical Earthquake Shaking

### *B.3.1 Introduction*

For most buildings, where important vertical periods of response are less than or equal to 1.0 second, vertical earthquake spectra can be derived from the horizontal spectra using the procedures of this section. For buildings with important natural periods of vertical response in excess of 1.0 second, a site-specific analysis should be conducted.

Sections B.3.2 and B.3.3 below present procedures for Site Classes A, B and C, and Site Classes D and E, per ASCE-7-05, respectively.

### *B.3.2 Procedure for Site Classes A, B and C*

For structural periods less than or equal to 1.0 second, the vertical response spectrum can be constructed by scaling the corresponding ordinates of the horizontal response spectrum, $S_a$, as follows.

For periods less than or equal to 0.1 second, the vertical spectral acceleration, $S_v$, can be taken as

$$S_v = S_a \tag{B-1}$$

For periods between 0.1 and 0.3 second, the vertical spectral acceleration, $S_v$, can be taken as

$$S_v = (1 - 1.048[\log(T) + 1])S_a \tag{B-2}$$

For periods between 0.3 and 1.0 second, the vertical spectral acceleration, $S_v$, can be taken as

$$S_v = 0.5 S_a \tag{B-3}$$

### B.3.3 Procedure for Site Classes D and E

For structural periods less than or equal to 1.0 second, the vertical response spectrum can be constructed by scaling the corresponding ordinates of the horizontal response spectrum, $S_a$, as follows.

For periods less than or equal to 0.1 second, the vertical spectral acceleration, $S_v$, can be taken as

$$S_v = \eta S_a \tag{B-4}$$

For periods between 0.1 and 0.3 second, the vertical spectral acceleration, $S_v$, can be taken as

$$S_v = (\eta - 2.1(\eta - 0.5)[\log(T) + 1])S_a \tag{B-5}$$

For periods between 0.3 and 1.0 second, the vertical spectral acceleration, $S_v$, can be taken as

$$S_v = 0.5 S_a \tag{B-6}$$

In equations (B-4) through (B-6), $\eta$ can be taken as 1.0 for $S_s \le 0.5g$; 1.5 for $S_s \le 1.5g$; and $(1 + 0.5(S_s - 0.5))$ for $0.5g \le S_s \le 1.5g$.

## B.4 Attenuation Relationships

Attenuation relationships relate ground motion parameters to the magnitude of an earthquake and the distance away from the fault rupture. Relationships have been established for many ground motion parameters including

- Peak horizontal ground acceleration, velocity, displacement and corresponding spectral terms
- Peak vertical ground acceleration, velocity, displacement and corresponding spectral terms

Attenuation relationships are developed by statistical evaluation of large sets of ground motion data. Relationships have been developed for different regions of the United States (and other countries) and different fault types (i.e., strike-slip, dip-slip and subduction). These relationships are only as good as the dataset from which the relationships were derived; the greater the size of the data set, the more robust the relationship.

The basic construction of a ground motion attenuation relationship is presented in Equation 3-5 of Part B and is not repeated here. Attenuation relationships generally return a geometric mean[1] of two horizontal spectral ordinates and corrections are required to compute maximum and minimum demands, as defined in Section B.2 above.

Selected North American ground motion attenuation relationships are presented in Table B-1. These attenuation relationships use moment magnitude $M_W$ to define earthquake magnitude. The attenuation relationships of Table B-1 use different definitions of site-to-source distance; some of the definitions are illustrated in Figure B-1 that is adapted from Abrahamson and Shedlock (1997). The seismogenic depth is defined here as the depth of the surface materials.

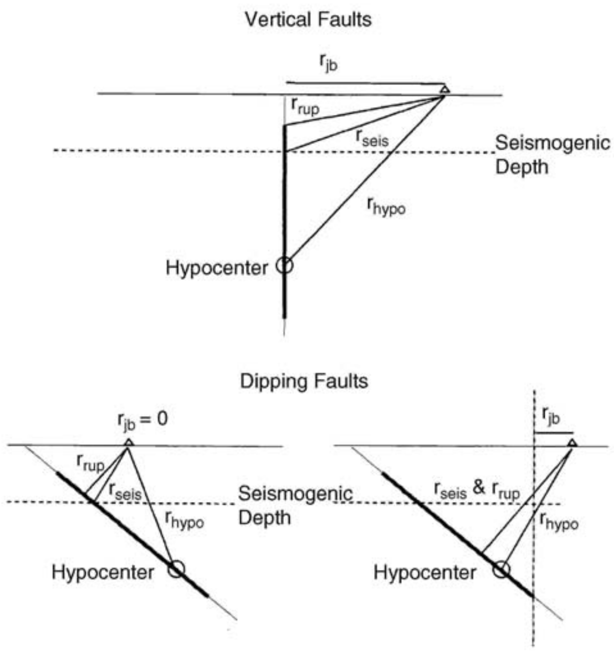



Figure B-1 Site-to-source distance definitions (Abrahamson and Shedlock, 1997)

Reader Note: This discussion will be updated in the next draft of this *Guideline* to report on the attenuation relationships adopted by the USGS to develop seismic hazard maps for the 2008 *NEHRP Recommended Provisions*. Many of the attenuation relationships in Table B-1 will be retired

[1] For a given pair of horizontal earthquake histories, the geometric mean ($\overline{S}_g$ of the spectral ordinates of the two components ($S_x$ and $S_y$) is generally used to characterize the pair of histories: $\overline{S}_g = \sqrt{S_x S_y}$ . Because the functional form of the attenuation relationship involves the natural log of the ground motion parameter, the geometric mean of the ordinates (which is equivalent to the arithmetic mean of the logs of the ordinates) is used instead of the arithmetic mean.

and replaced with updated relationships, including the Next Generation Attenuation (NGA) relationships for WNA.

**Table B-1 Ground Motion Attenuation Relationships**

| Model | Calculated[1] | Site Conditions | Variables[2] | Ranges | | |
|---|---|---|---|---|---|---|
| | | | | $T_n$ (secs) | $r$ (km) | $M_W$ |
| **Western North America (WNA)** | | | | | | |
| Abrahamson and Silva, 1997 | PHA, PVA, Sah, Sav | Rock, Deep Soil | $M$, $r_{rup}$, F, HW | 0-5 | 0-100 | 4-8 |
| Boore, Joyner, Fumal, 1997 | PHA, Sah | $v_S$ in upper 30m | $M$, $r_{jb}$, F | 0-2 | 0-80 | 5.5-7.5 |
| Campbell, 1997 | PHA, PVA, PHV, PVV, Sah, Sav | Hard rock, Soft rock, Soil | $M$, $r_{seis}$, F, D | 0-4 | 0-100 | 4-9.5 |
| Campbell and Bozorgnia, 2003 | PHA, PVA, Sah, Sav | Rock, Deep Soil | $M$, $r_{sei}$, HW + | 0-4 | 0-100 | 4.7-8 |
| Sadigh et al., 1997 | PHA, Sah | Rock, Deep Soil | $M$, $r_{rup}$, F, HW | 0-4 | 0-100 | 4-8 |
| **Central and Eastern North America (CENA)** | | | | | | |
| Atkinson & Boore, 1997 | PHA, Sah | Rock | $M$, $r_{hypo}$ | 0-2 | 10-300 | 4-9.5 |
| Campbell, 2003 | PHA, Sah | Hard rock | $M$, $r_{rup}$ | 0-4 | 1-1000 | 5-8.2 |
| Toro et al., 1997 | PHA, Sah | Rock | $M$, $r_{jb}$ | 0-2 | 1-100 | 5-8 |
| **Subduction Zones** | | | | | | |
| Atkinson & Boore, 2003 | PHA, Sah | Rock to poor soil | $M$, $r_{hypo}$, + | 0-3 | 10-300 | 5.5-8.3 |
| Youngs et al., 1997 | PHA, Sah | Rock, Soil | $M$, $r_{rup}$, F, H | 0-4 | 0-100 | 4-9.5 |

1. PHA = peak horizontal ground acceleration, PHV = peak vertical ground acceleration, PHV = peak horizontal ground velocity, PVV = peak vertical ground velocity, Sah = horizontal spectral acceleration, Sav = vertical spectral acceleration
2. $r_{rup}$ = closest distance to the rupture surface, $r_{jb}$ = closest horizontal distance to the vertical projection of the rupture, $r_{hypo}$ = hypocentral distance, $r_{seis}$ = closest distance to the seismogenic rupture zone, $M$ = magnitude, $F$ = fault type, $H$ = hanging wall.

### B.5 Fault Rupture Directivity

Rupture directivity causes spatial variations in the amplitude and duration of ground motions around faults. Propagation of rupture towards a site produces larger amplitudes of shaking at periods longer than 0.6 second and shorter strong-motion durations than for average directivity conditions.

Somerville et al. (1997) developed modifications to the empirical attenuation relations of Abrahamson and Silva (1997), see Table B-1, to account for these variations. Somerville (1997) identified fault rupture directivity parameters $\theta$ and $X$ for strike-slip faults and $\phi$ and $Y$ for dip slip faults as shown in Figure B-2. Somerville developed three ground motion parameters to characterize directivity: (1) *Amplitude factor*: bias in average horizontal response spectrum acceleration with respect to Abrahamson and Silva (1997); (2) Duration factor: bias in duration of acceleration with respect to Abrahamson and Silva; and (3) *Fault-normal/Average amplitude*: ratio of fault normal to average (directivity) horizontal response spectrum acceleration. Bounds were set on the range of applicability of the directivity model.

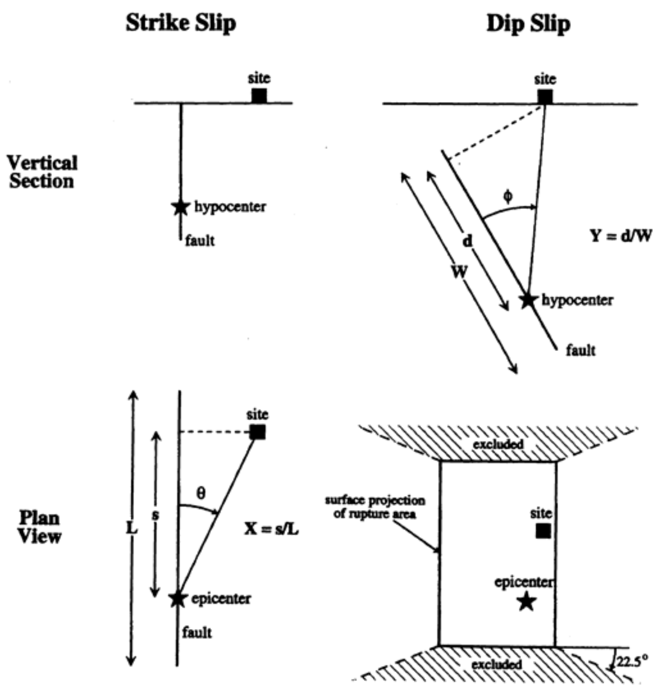



Figure B-2 Fault rupture directivity parameters (Somerville et al., 1997)

Abrahamson (2000) identified aspects of the spatial component of the Somerville et al. (1997) rupture directivity model (parameter 1) that could be improved to make the correction procedure more amenable for probabilistic seismic hazard assessment. Specifically, Abrahamson proposed the following model to incorporate rupture directivity effects:

$$\text{In}Y_{Dir} = \text{In}Y + f_1(DR,\xi)T(r_{rup})T(M_w) + f_2(r_{rup}, M_W, \xi) \qquad \text{(B-7)}$$

where $Y$ is the average horizontal component of the ground-motion parameter with null directivity effects (the Abrahamson and Silva relationships of 1997) and $Y_{Dir}$ is the value of Y accounting for rupture directivity effects; $f_1(.)$ accounts for the spatial variability and $f_2(.)$ accounts for orientation with respect to the strike of the fault.

For strike-slip faulting:

$$\begin{aligned} f_1(DR,\xi) &= c_1 + 1.88c_2(s/L)\cos\theta \qquad &&\text{for } s/L \le 0.4 \\ &= c_1 + 0.75c_2\cos\theta &&\text{for } s/L > 0.4 \end{aligned} \qquad \text{(B-8)}$$

and for dip-slip faulting

$$f_1(DR,\xi) = c_1 + c_2(d/W)\cos\phi \qquad \text{(B-9)}$$

and where

$$\begin{aligned} f_2(r_{rup}, M_W, \xi) &= 0.5(\cos 2\xi)[c_3 + c_4\text{In}(r_{rup}+1) + c_5(M_W - 6)] \qquad &&\text{for fault-normal} \\ &= \text{-}0.5(\cos 2\xi)[c_3 + c_4\text{In}(r_{rup}+1) + c_5(M_W - 6)] &&\text{for fault-parallel} \\ &= 0 &&\text{for } \xi \ge 45^0 \end{aligned} \qquad \text{(B-10)}$$

$$\begin{aligned} T(r_{rup}) &= 1 \qquad &&\text{for } r_{rup} \le 30 \text{ km} \\ &= 1\text{-}(r_{rup} - 30)/30 &&\text{for } 30 < r_{rup} < 60 \text{ km} \\ &= 0 &&\text{for } r_{rup} \ge 60 \text{ km} \end{aligned}$$

$$\begin{aligned} T(M_W) &= 1 \qquad &&\text{for } M_W \ge 6.5 \\ &= 1\text{-}(6.5 - M_W)/0.5 &&\text{for } 6.0 < M_W < 6.5 \\ &= 0 &&\text{for } M_W \le 6.0 \end{aligned}$$

In the above equations, $r_{rup}$ is the closest distance to the rupture plane (*km*); the length and width ratios, $DR = s/L; d/W$ , are defined as the fraction of the fault rupture length *L* and fault rupture width *W* that ruptures towards the site for strike-slip and dip-slip faults, respectively; and $\xi = \theta;\phi$ are the azimuth and zenith angles between the fault rupture plane and the ray path to the site for strike-slip and dip-slip faults, respectively. The standard deviation of the predicted strong-motion parameter when directivity effects are accounted for is given by

$$\sigma_{\text{In}Y,Dir} = \sigma_{\text{In}Y} - 0.05c_2/1.333 \qquad \text{(B-11)}$$

where $\sigma_{\ln Y,Dir}$ is the standard deviation of $\ln Y_{Dir}$ and $\sigma_{\ln Y}$ is the standard deviation of $\ln Y$. Values for coefficients $c_1$ through $c_5$ are presented in Table B-2 from Bozorgnia and Bertero (2004).

**Table B-2 Directivity Coefficients (Bozorgnia and Bertero, 2004)**

| | Strike Slip | | Dip Slip | | | | |
|---|---|---|---|---|---|---|---|
| $T$(s) | $c_1$ | $c_2$ | $c_1$ | $c_2$ | $c_3$ | $c_4$ | $c_5$ |
| 0.6 | 0 | 0 | 0 | 0 | 0.027 | −0.0069 | 0 |
| 0.7 | — | — | — | — | 0.050 | −0.0127 | 0 |
| 0.75 | −0.084 | 0.185 | −0.045 | 0.008 | 0.061 | −0.0155 | 0 |
| 0.8 | — | — | — | — | 0.070 | −0.0178 | 0 |
| 0.9 | — | — | — | — | 0.088 | −0.0220 | 0 |
| 1.0 | −0.192 | 0.423 | −0.104 | 0.178 | 0.104 | −0.0255 | 0 |
| 1.5 | −0.344 | 0.759 | −0.186 | 0.318 | 0.164 | −0.0490 | 0.034 |
| 2.0 | −0.452 | 0.998 | −0.245 | 0.418 | 0.207 | −0.0613 | 0.059 |
| 2.5 | — | — | — | — | 0.280 | −0.0816 | 0.078 |
| 3.0 | −0.605 | 1.333 | −0.327 | 0.559 | 0.353 | −0.1007 | 0.093 |
| 3.5 | — | — | — | — | 0.415 | −0.1172 | 0.106 |
| 4.0 | −0.713 | 1.571 | −0.386 | 0.659 | 0.456 | −0.1282 | 0.118 |
| 4.5 | — | — | — | — | 0.462 | −0.1307 | 0.128 |
| 5.0 | −0.797 | 1.757 | −0.431 | 0.737 | 0.450 | −0.1269 | 0.137 |
| 6.0 | — | — | — | — | 0.424 | −0.1223 | 0.152 |

**Reader Note:** This discussion will be updated in the next draft of this *Guideline*. The Next Generation Attenuation (NGA) relationships that will be used by the USGS to map seismic hazard in WNA address the spatial near-fault correction described above. BSSC Project 07 is addressing the computation of maximum and minimum spectral demands given geomean demands. Within 5 km of a fault, maximum shaking is represented well by fault-normal shaking. Beyond 5 km, the orientation of maximum shaking cannot be assumed to be normal to the fault. The procedures under development by Project 07 to correct geomean demands to maximum and minimum demands will be included in the next draft of the *Guideline*.

## B.6 Probabilistic Seismic Hazard Assessment

### *B.6.1 Introduction*

Performance-based seismic design will often utilize a site-specific characterization of the ground shaking associated with different probabilities of exceedance or return periods. Such characterizations are routinely performed using Probabilistic Seismic Hazard Assessment (PSHA). The following subsections provide introductory information on PSHA, and draw substantially from Kramer (1996), McGuire (2004) and Bozorgnia and Bertero (2004).

### *B.6.2 PSHA Calculations*

**General**

For Probabilistic Seismic Hazard Assessment (PSHA), probability distributions are determined for the magnitude of each earthquake on each source, $f_M(m)$, the location of the earthquake in or along each source, $f_R(r)$, and the prediction of the response parameter of interest $P(\text{pga} > \text{pga}' | m, r)$. Kramer describes PHSA as a four-step process that is enumerated below and depicted in Figure B-3.

1. Identify and characterize (geometry and potential $M_W$) all earthquake sources capable of generating significant shaking (say $M_W \geq 4.5$) at the site. Develop the probability distribution of rupture locations within each source. Combine this distribution with the source geometry to obtain the probability distribution of source-to-site distance for each source.
2. Develop a seismicity or temporal distribution of earthquake occurrence for each source using a recurrence relationship.
3. Using predictive (attenuation) relationships, determine the ground motion produced at the building site (including the uncertainty) by earthquakes of any possible size or magnitude occurring at any possible point in each source zone.
4. Combine the uncertainties in earthquake location, size, and ground motion prediction to obtain the probability that the chosen ground motion parameter (e.g., peak horizontal ground acceleration, spectral acceleration at a specified frequency) will be exceeded in a particular time period (say 10% in 50 years).

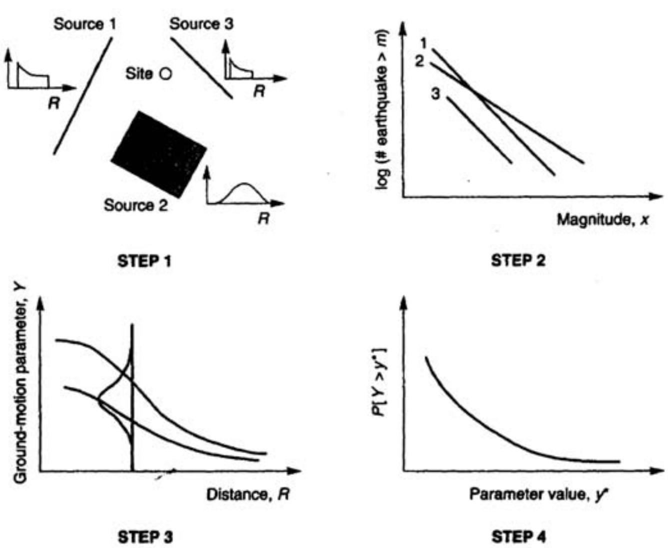



Figure B-3 Steps in probabilistic seismic hazard assessment (Kramer, 1996)

Summary information on parts of each step in the process described above is presented below. The reader is referred to Kramer (1996) and McGuire (2004) for much additional information.

### Earthquake Source Characterization

The characterization of an earthquake source (and there might be a number of sources for a given site) requires consideration of the spatial characteristics of the source, the distribution of earthquakes within that source, of the distribution of earthquake size within that source, and of the distribution of earthquakes with time. Each of these characteristics involves some degree of uncertainty and such uncertainty is addressed explicitly by PSHA.

### Spatial Uncertainty

The geometries of earthquake sources are typically characterized as *point sources* (e.g., volcanoes), two-dimensional *areal sources* (e.g., a well-defined fault plane) and three-dimensional *volumetric sources* (e.g., areas where earthquake mechanisms are poorly defined such as the Central and Eastern USA). Source zones might be similar to or different from the actual source, depending on the relative geometry of the source and the site of interest, as shown in Figure B-4 below from Kramer (1996).

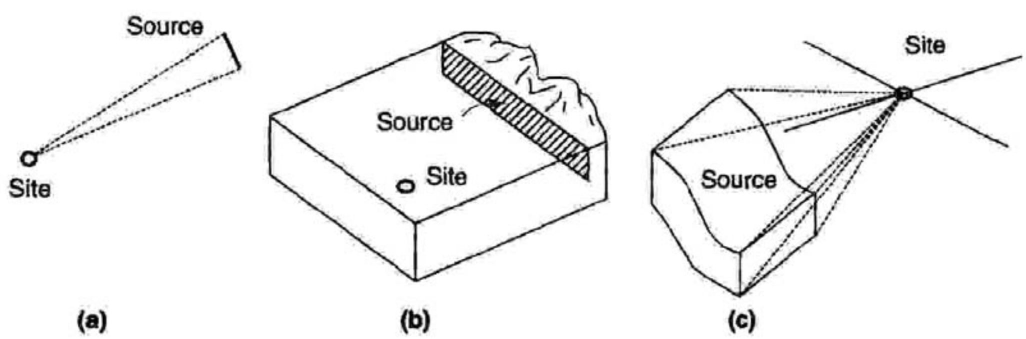



Figure B-4 Source zone geometries (Kramer, 1996)

Since attenuation relationships express ground motion parameters in terms of a measure of the source-to-site distance, the spatial uncertainty must be described with respect to the appropriate distance parameter. The uncertainty in source-to-site distance can be described by a probability density function as shown in Figure B-5.

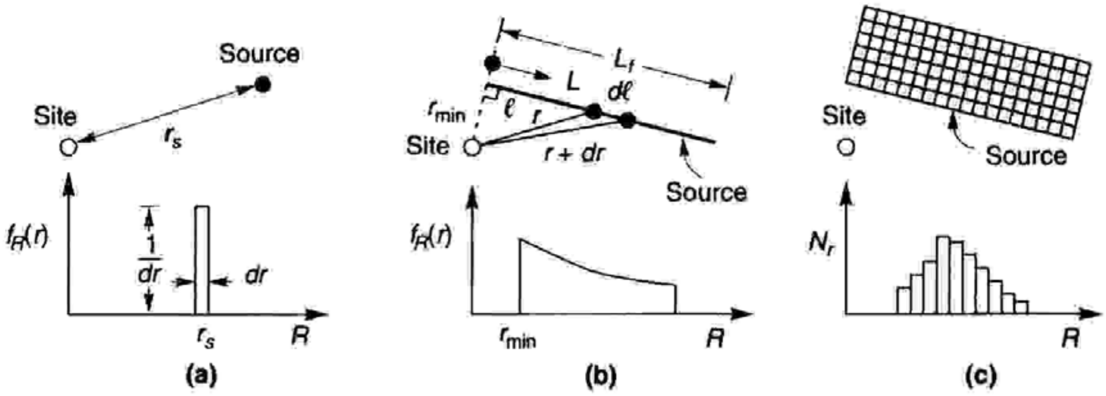



Figure B-5 Variations in site-to-source distance for three source zone geometries (Kramer, 1996)

For the point source above, the distance *R* is $r_s$ and the probability that $R = r_s$ is 1.0 and $R \neq r_s$ is 0. For more complex source zones, it is easier to evaluate $f_R(r)$ by numerical integration. For example, the source zone of part c. of the figure above is broken up into a large number of discrete elements of the same area. A histogram that approximates $f_R(r)$ can be constructed by tabulating the values of *R* that correspond to the center of each element.

**Size Uncertainty**

The distribution of earthquake sizes in a given period is described by a *recurrence law*. One basic assumption of PSHA is that the recurrence law obtained on the basis of past seismicity is appropriate for the preduction of future seismicity. The best known recurrence law is that of Gutenberg and Richter (1944), who collected data from Southern Californian earthquakes over a period of years and plotted the data according to the number of earthquakes that exceeded different magnitudes during that period. The number of exceedances of each magnitude was divided by the length of the time period used to assemble the data to define a *mean annual rate of exceedance* $\lambda_m$ of an earthquake of magnitude *m*. The reciprocal of the mean annual rate of exceedance of a particular magnitude is termed the return period of earthquakes exceeding that magnitude. Guttenberg-Richter plotted the logarithm of the annual rate of exceedance (of earthquakes in Southern California) against earthquake magnitude and the resulting relationship was linear, namely,

$$\log \lambda_m = a - bm \qquad \text{(B-12)}$$

where $\lambda_m$ is defined above, $10^a$ is the mean yearly number of earthquakes of magnitude greater than or equal to 0, and *b* describes the relative likelihood of large and small earthquakes. As the value of *b* increases, the number of larger magnitude earthquakes relative to smaller magnitude earthquakes decreases. The values of *a* and *b* are generally obtained by regression analysis on a database of seismicity from the source zone of interest. The mean rate of small earthquakes is often underpredicted because historical records are often used to supplement the instrumental records and only the larger magnitude events from part of the historical record.

The Guttenberg-Richter recurrence law can also be expressed as

$$\lambda_m = 10^{a-bm} = \exp(2.303a - 2.303b) \qquad \text{(B-13)}$$

which shows that the law implies that earthquake magnitudes are exponentially distributed and that the range of magnitude is from $-\infty$ to $\infty$. Small magnitude earthquakes are of little significance to the built environment and can be ignored in terms of hazard assessment. The law also predicts non-zero mean rates of exceedance from magnitudes up to $\infty$, which is not physically possible. Bounded (lower and upper) recurrence laws have been proposed to deal with these practical bounds on magnitude,

The Guttenberg-Richter law was originally developed from regional data and not for specific source zones. Paleoseismic studies over the past 30 years have indicated that individual points on faults (or fault segments) tend to move by approximately the same distance in each earthquakes, suggesting that individual faults repeatedly generate earthquakes of a similar (with 0.5 magnitude unit) size, known as *characteristic earthquakes* at or near their maximum magnitude. Geological evidence indicates that characteristic earthquakes occur more frequently than that would be implied by extrapolation of the law from high exceedance rates (of low magnitude events) to low exceedance rates (of high magnitude), resulting in a more complex recurrence law than that given by equation B-12.

**Attenuation Relationships**

Predictive (or attenuation) relationships are generally obtained empirically by least-squares regression on a strong-motion dataset. Scatter or randomness in the results is inevitable because of differences in site conditions, travel path and fault rupture mechanics. The scatter can be characterized by confidence limits or by the standard deviation of the predicted parameter.

The probability that a ground motion parameter *Y* exceeds a certain value *y* for an earthquake of magnitude *m*, occurring at a distance *r* is given by

$$P[Y > y \mid m, r] = 1 - F_y(y) \qquad \text{(B-14)}$$

where $F_Y(y)$ is the value of the CDF of $Y$ at $m$ and $r$. The value of $F_Y(y)$ depends on the probability distribution used to describe $Y$. As noted previously, ground motion parameters are generally assumed to be lognormally distributed. Figure B-6 illustrates the conditional probability of exceeding a particular value of the ground motion parameter for a given combination of $m$ and $r$.

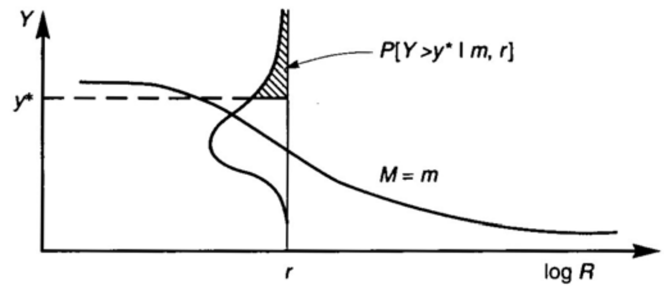



Figure B-6 Conditional probability calculation (Kramer, 1996)

**Temporal Uncertainty**

The distribution of earthquake occurrence with time must be computed or assumed to calculate the probabilities of different earthquake magnitudes occurring in a given time period. Earthquakes are assumed to occur randomly with time and the assumption of random occurrence permits the use of simple probability models.

The temporal occurrence of earthquakes is commonly described as a *Poisson* process: one that yields values of a random variable describing the number of occurrences of a particular event during a given time interval (or spatial region). In a Poisson process, a) the number of occurrences in one time interval are independent of the number that occur in any other time interval; b) the probability of occurrence during a very short time interval is proportional to the length of the time interval; and c) the probability of more than one occurrence in a very short time interval is negligible. Events in a Poisson process occur randomly, with no memory of the time, size or location of any preceding events. Cornell (19**) showed that the Poisson model is useful for PSHA unless the hazard is dominated by a single source zone that produces characteristic earthquakes and the time period since the previous significant event exceeds the average inter-event time.

For a Poisson process, the probability of a random variable $N$, representing the number of occurrences of a particular event in a given time period is given by

$$P[N=n]=\frac{\mu^n e^{-\mu}}{n!} \tag{B-15}$$

where $\mu$ is the average number of occurrences of the event in the time period. To characterize the temporal distribution of earthquake recurrence for PSHA, the Poisson probability is normally expressed as

$$P[N=n]=\frac{(\lambda t)^n e^{-\lambda t}}{n!} \tag{B-16}$$

where $\lambda$ is the average rate of recurrence of the event and *t* is the time period. When the event is the exceedance of a particular earthquake magnitude, the Poisson model can be combined with a suitable recurrence law to predict the probability of at least one exceedance in a period of *t* years by

$$P[N \geq 1]=1-e^{-\lambda_m t} \tag{B-17}$$

**Probability Computations and Seismic Hazard Curves**

The development of *seismic hazard curves*, which identify the annual probability of exceedance of different values of a selected ground motion parameter, involves probabilistic calculations that combine the uncertainties in earthquake size, location and frequency for each potential earthquake source that could impact shaking at the site under study. The seismic hazard curves can then be used to compute the probability of exceeding the chosen ground motion parameter in a specified period of time.

The seismic hazard curve calculations are (somewhat) straightforward once the uncertainties in earthquake size, location and frequency are established but much bookkeeping is involved. The probability of exceeding a particular value *y* of a ground motion parameter *Y* is calculated for one possible source location and then multiplied by the probability of that magnitude earthquake occurring at that particular location. The calculation is then repeated for all possible magnitudes and locations and the probabilities of each are summed to compute the $P[Y > y]$ at the site. A summary of the presentation of Kramer (1996) is presented below. The reader is referred to Kramer (1996) and McGuire (2004) for much additional information.

For a given earthquake occurrence, the probability that a ground motion parameter *Y* will exceed a particular value *y* can be computed using the total probability theorem (Cornell and Benjamin, 1968), namely,

$$P[Y > y]=P[Y > y \mid \mathbf{X}]\,P[\mathbf{X}]=\int P[Y > y \mid \mathbf{X}]\, f_x(\mathbf{X})\,dx \tag{B-18}$$

where $\mathbf{X}$ is a vector of random variables that influence *Y*. In most cases, the quantities in $\mathbf{X}$ are limited to the magnitude *M* and distance *R*. Assuming that *M* and *R* are independent, the probability of exceedance can be written as

$$P[Y > y] = \iint P[Y > y | m, r]\, f_M(m)\, f_R(r)\, dm\, dr \qquad \text{(B-19)}$$

where $P[Y > y | m, r]$ is obtained from the predictive relationship and $f_M(m)$ and $f_R(r)$ are the probability density functions for magnitude and distance, respectively.

If the building site is in a region of $N_s$ potential earthquake sources, each of which has an average rate of threshold exceedance $\nu_i = \exp(\alpha_i - \beta_i m)$, the total average exceedance rate for the region is given by the equation below, which is typically solved by numerical integration.

$$\lambda_y = \sum_{i=1}^{N_s} P[Y > y | m, r]\, f_{Mi}(m)\, f_{Ri}(r)\, dm\, dr \qquad \text{(B-20)}$$

One approach that is described by Kramer as simple rather than efficient is to divide the possible ranges of magnitude and distance into $N_M$ and $N_R$ segments, respectively. The average exceedance rate can then be calculated using a multi-level summation as follows:

$$\begin{aligned} \lambda_y &= \sum_{i=1}^{N_s} \sum_{j=1}^{N_m} \sum_{k=1}^{N_R} \nu_i P[Y > y | m_j, r_k]\, f_{Mi}(m_j)\, f_{Ri}(r_k)\, \Delta m\, \Delta r \\ &= \sum_{i=1}^{N_s} \sum_{j=1}^{N_m} \sum_{k=1}^{N_R} \nu_i P[Y > y | m_j, r_k]\, P[M = m_j]\, P[R = r_k] \end{aligned} \qquad \text{(B-21)}$$

where the terms are $m_j = m_0 + (j - 0.5)(m_{\max} - m_0) / N_M$, $\Delta r = (r_{\max} - r_{\min}) / N_r$, $r_k = r_{\min} + (k - 0.5)(r_{\max} - r_{\min}) / N_R$ and $\Delta m = (m_{\max} - m_0) / N_m$. The above statement is equivalent to assuming that each source is capable of generating only $N_M$ different earthquakes of magnitude $m_j$ at only $N_R$ different source-to-site distances of $r_k$. The accuracy of the above method increases with smaller segments and thus larger values of $N_M$ and $N_R$.

Figure B-7 presents a sample seismic hazard curve for peak ground acceleration at a site in Berkeley, California (McGuire, 2004). The contributions to the annual frequency of exceedance from 9 seismic sources are shown.

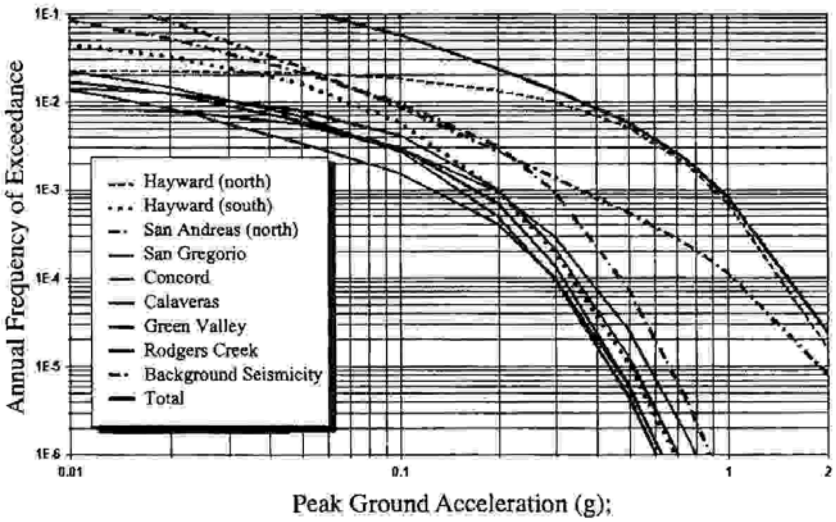



Figure B-7 Seismic hazard curve for Berkeley, California (McGuire, 2004)

Probabilities of exceedance in a selected time period can be computed using seismic hazard curves combined with the Poisson model. The probability of exceedance of *y* in a time period *T* is

$$P[Y_T > y] = 1 - e^{-\lambda_y T} \qquad \text{(B-22)}$$

As an example, the probability that a peak horizontal ground acceleration of 0.10 *g* will be exceeded in a 50-year time period for the site characterized by the hazard curve above:

$$P[\text{PHA} > 0.1g \text{ in 50 years}] = 1 - e^{-\lambda_y T} = 1 - e^{-(0.060)(50)} = 0.950 = 95\% \quad \text{(B-23)}$$

An alternate, often made, computation is the value of the ground motion parameter corresponding to a particular probability of exceedance in a given time period. For example, the acceleration that has a 10% probability of exceedance in a 50-year period would be that with an annual rate of exceedance, calculated by re-arranging the second-to-last equation, namely

$$\lambda_y = -\frac{\text{In}(1 - P[Y_T > y])}{T} = -\frac{\text{In}(1 - 0.10)}{50} = 0.00211 \qquad \text{(B-24)}$$

Using the hazard curve from Figure B-7 for all nine sources, the corresponding zero-period spectral acceleration is approximately 0.75 *g*.

Hazard curves can be developed for specific oscillator periods (0.2 second and 1.0 second are widely used) and the above calculations of probability of exceedance can be extended to the spectral domain, namely, to develop probabilistic estimates of spectral demands for different probabilities of exceedance or return periods. Hazard curves have been developed for many different response quantities and characterizations of earthquake ground motion.

### B.6.3 Inclusion of Rupture Directivity Effects

Rupture directivity effects should be considered in PSHA if the building site is located within 20 *km* of an active fault capable of generating a moment magnitude 6.5 earthquake or greater.

Rupture directivity effects can be included directly in PSHA if the attenuation relationships are either a) constructed using a database of recorded ground motions that include directivity effects (i.e., the NGA relationships described previously), or b) corrected to account for rupture directivity in a manner similar to that described in Section B-5. Note that the ratio of median maximum demand to geomean demand in the near-fault region is period-dependant but might be as great as 2.

### B.6.4 Deaggregation of Seismic Hazard Curves

The probabilistic seismic hazard analysis procedures described previously enable the calculation of annual rates of exceedance of ground motion parameters (e.g., spectral acceleration at a selected period) at a particular site based on aggregating the risks from all possible source zones. The rate of exceedance computed in the probabilistic seismic hazard analysis is therefore not associated with any particular earthquake magnitude *m* or distance *r*.

For a given building site and hazard curve, the combinations of magnitude, distance and source that contribute most to the hazard curve can be established. This process is termed de-aggregation (or disaggregation). Hazard deaggregation requires that the mean annual rate of exceedance be expressed as a function of magnitude and distance as follows:

$$\lambda_y(m_j, r_k) = P[M = m_j]\,P[R = r_k]\sum_{i=1}^{N_s} \nu_i P[Y > y \mid m_j, r_k] \tag{B-25}$$

Detailed information on the deaggregation of seismic hazard curves is provided in McGuire (2004).

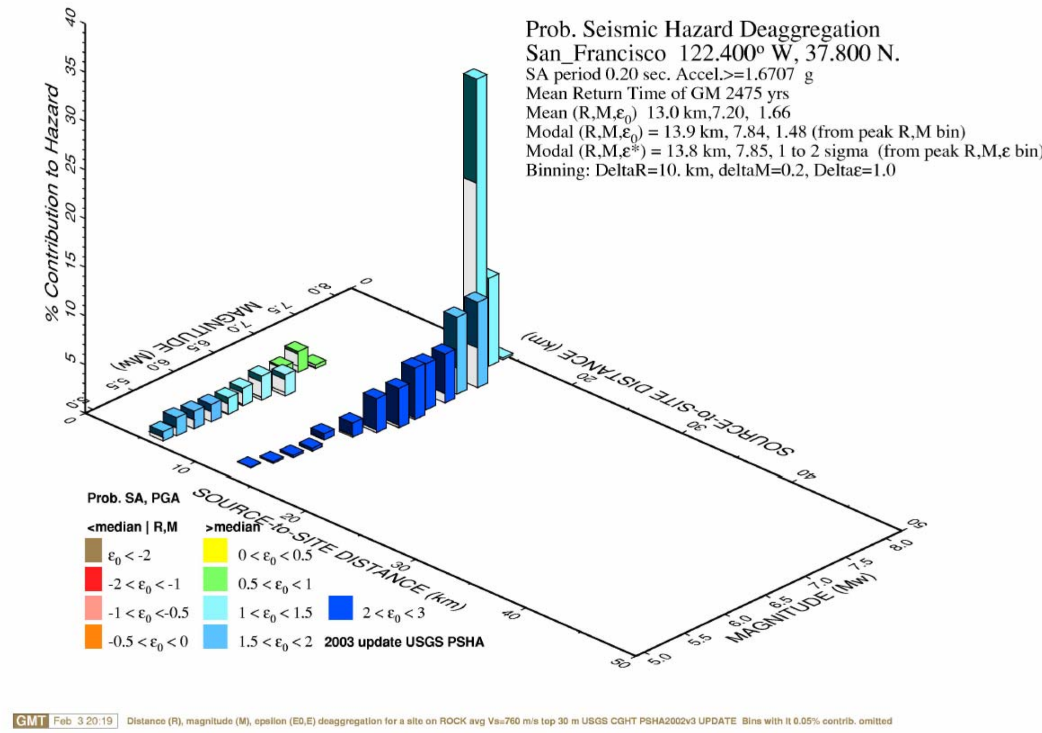
Sample deaggregation results for horizontal spectral acceleration at periods of 0.2 second and 1.0 second for a site in San Francisco, Latitude = 37.8º, Longitude = -122.4 º , for a 2% probability of exceedance in 50 years, are shown in Figure B-8 below. This site is identical to that used in part B, Section 3 to demonstrate the USGS ground motion calculator. To generate similar results, visit the USGS website below and click on the Interactive deaggregation button under the headings of 2002 and Hazard Values, and enter the Lat and Long for the building site and the type of deaggregation required: http://earthquake.usgs.gov/research/hazmaps/products_data/48_States/index.php.

Consider the 1-second deaggregation data of part b of Figure B-8. The plot shows the contribution to the 1-second uniform hazard spectral ordinate for a 2% probability of exceedance in 50 years as a function of moment magnitude and distance. The figure shows that approximately 50% of the total 1-second seismic demand can be ascribed to a moment magnitude 7.8 earthquake at a distance of 14 *km*–the [magnitude, distance] pair that dominates the 1-second spectral demand at this site is [7.8, 14].

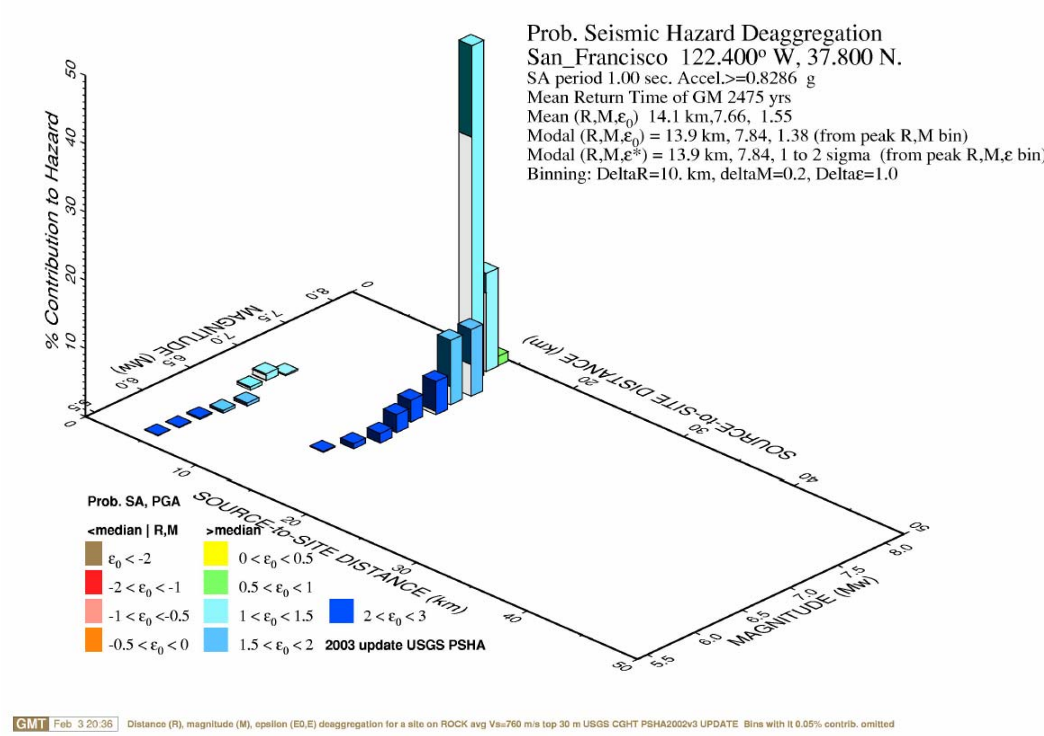
Importantly, this figure introduces another important ground motion variable, epsilon, $\varepsilon$. One straightforward definition of $\varepsilon$ is as follows:

$$\varepsilon = \frac{S_a - \theta}{\beta} \tag{B-20}$$

where all variables vary as a function of period; $S_a$ is the spectral acceleration computed by PSHA for a given probability (e.g., 2%) of exceedance in a specified time period (e.g., 50 years) and equal to 0.829*g* in this instance at a period of 1 second; $\theta$ is the median value of spectral acceleration computed by an appropriate attenuation relationship(s) for the dominant [magnitude, distance] pair (equal to [7.8, 14] in this instance), and $\beta$ is the dispersion in the attenuation relationship. In this example, and using the modal [magnitude, distance, $\varepsilon$] triple, we see that $\varepsilon$ ranges between 1 and 2, meaning that less than 15% of moment magnitude 7.8 earthquakes at a distance of 14 *km* would produce *geomean* spectral demands in excess of 0.829 *g*. Herein, we term ground motions that generate spectral demands for which $\varepsilon > 2$ as *high-fractile*–see Section B.8.3.




a. 0.2 second deaggregation




b. 1.0 second deaggregation

Figure B-8 Sample de-aggregation of a hazard curve (from www.usgs.gov)

## B.7 Soil-Foundation-Structure Interaction

### *B.7.1 General*

The response of a structure to earthquake shaking is affected by the response of the dynamic properties of the three components of the soil-foundation-structure system and the interactions between the three components. A soil-foundation-structure interaction (SFSI) analysis involves the direct or indirect analysis of the three-component system to prescribed free-field inputs, which are typically imposed as bedrock motions.

In traditional structural analysis, the base of the structure is assumed to be rigid (components fixed at their base) and the free-field motions at the base of the foundation are derived assuming one of the soil categories described in Part B, Section 3. SFSI effects are absent from such an analysis. Figure B-9a illustrates the traditional structural model. The fixed base model is inappropriate for structural framing systems that incorporate stiff vertical components for lateral resistance (e.g., reinforced concrete shear walls, steel braced frames) because the response of such systems can be sensitive to small base rotations and translations that are neglected with a fixed base. Relatively flexible lateral framing systems such as steel moment-resisting frames are often not significantly affected by SFSI.

### *B.7.2 Direct Soil-Foundation-Structure-Interaction Analysis*

SFSI can be analyzed directly using the finite-element method and response history solutions, where the soil is discretized with solid finite elements. Full nonlinear analysis is also possible using this method although typical analysis is performed using equivalent linear soil properties. Direct analysis can address the three key effects of SFSI, namely, foundation flexibility, kinematic effects, and foundation damping, although solution of the kinematic interaction problem is often difficult without customized finite element codes because typical finite element codes cannot account for wave inclination and wave incoherence effects.

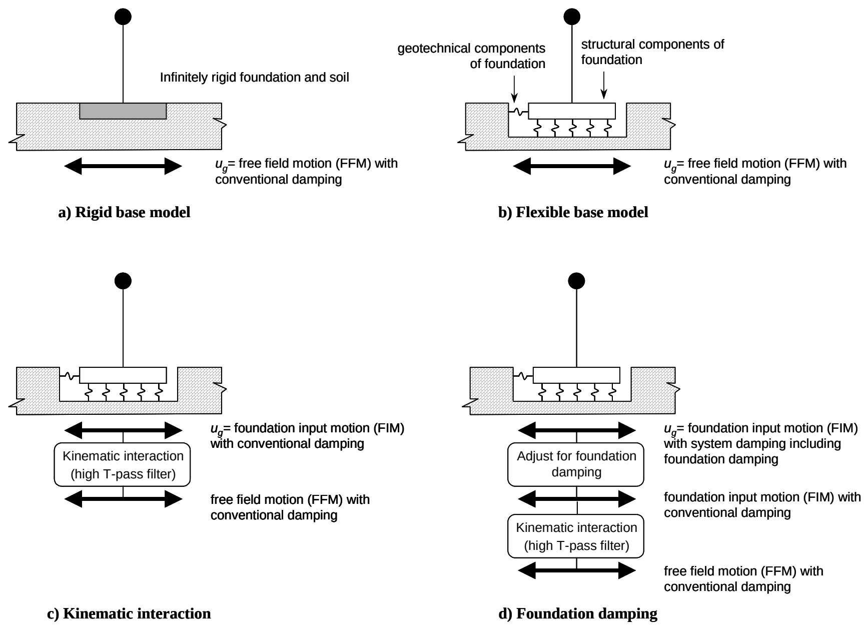



Figure B-9 Analysis for soil foundation structure interaction (FEMA, 2005)

Response-history analysis will require the development of a two- or three-dimensional numerical model of the soil-foundation-structure system. Stress-strain or constitutive models (linear, nonlinear or equivalent linear) for the soils in the model should be developed based on test data. Appropriate and consistent frequency-dependant stiffness and damping matrices should be developed for the edges or boundaries of the soil in the numerical model.

**Reader Note:** More guidance will be provided in later drafts of this *Guideline*, including a discussion of 2D versus 3D modeling, basin effects and topographic effects.

### *B.7.3 Simplified Soil-Foundation-Structure-Interaction Analysis*

**Simplified Procedures**

Simplified procedures for including the effects of SFSI in response analysis are presented in this section. The procedures are based on those presented in FEMA 440 (FEMA, 2005). More rigorous procedures are available in Appendix A of FEMA 440.

For the discussion below, soil-foundation-structure interaction is parsed into three key effects:

- foundation flexibility
- kinematic effects (filtering of the ground shaking to the building)

- foundation damping (dissipation of energy from the soil structure system through radiation and hysteretic soil damping).

Figure B-9b illustrates the incorporation of foundation flexibility into the numerical model of a building frame. Current analysis procedures in guidelines such as FEMA 356 (FEMA, 2000) and ATC 40 (ATC, 1996) partially address the flexible foundation effect by providing guidance on the stiffness and strength of the geotechnical (soil) components of the foundation in the structural analysis model. However, these analysis procedures do not characterize the reduction of the shaking demand on the structural framing system relative to the free-field motion due to kinematic interaction or the foundation damping effect, both of which are described below. Guidance on including these effects in a simplified manner for nonlinear dynamic response analysis is provided below. The product of numerical simulation using the model of Figure B-9b is a global response that includes elastic and inelastic deformations in the structural and geotechnical parts of the foundation system. These deformations are sometimes referred to as an *inertial soil-structure-interaction effect*. The inclusion of foundation flexibility in the numerical model can lead to both significant changes to the responses computed assuming a fixed base (Figure B-9a) and more accurate representation of probable structural response. Foundation responses and failures (e.g., rocking, soil bearing failure, pier/pile slip) can be explicitly evaluated using a numerical model that explicitly includes foundation flexibility.

*Kinematic soil-structure interaction* (Figure B-9c) results from the presence of relatively stiff foundation elements atop or in soil that causes the foundation motions to deviate from free-field motions. Base slab averaging and embedment are two kinematic effects. Ground motion shaking is spatially variable (i.e., not each point beneath a building footprint, in the absence of the foundation, would experience identical shaking at the same instant in time). Placement of a structure and foundation across these spatially variable motions produces an averaging effect and the weighted or overall motion experienced by the building is less than the localized maxima that would have occurred in the free-field. The embedment effect is associated with the reduction of ground motion with depth into a soil deposit. Both base slab averaging[2] and embedment[3] affect the characteristics of the

[2] Base slab averaging occurs to some extent in virtually all buildings. The slab averaging effect occurs at the foundation level for mats or spread footings interconnected by either grade beams or reinforced concrete slabs. Even if a laterally stiff foundation system is not present, the averaging can occur at the first elevated level of buildings with rigid diaphragms. The

foundation-level motion (sometimes called the foundation input motion) in a manner that is independent of the superstructure (i.e., the portion of the structure above the foundation) with one exception. The effects are strongly period dependent, being maximized at small periods. The effects can be visualized as a filter applied to the high-frequency components of the free-field ground motion. The impact of those effects on superstructure response will tend to be greatest for short-period buildings. A description of a simplified procedure to reduce the free field motion to a foundation input motion is presented below. The foundation input motion can be applied to a fixed base model or combined with a flexible base model. Kinematic effects tend to be important for buildings with relatively short fundamental periods (e.g., less than 0.5 second), large plan dimensions and basements embedded 10 feet or more in soil materials.

Figure B-9d illustrates foundation damping effects that are another result of *inertial soil-structure interaction*. Foundation damping results from the relative movements of the foundation and free-field soil. It is associated with radiation of energy away from the foundation and hysteretic damping within the soil. The result is an effective decrease in the spectral ordinates of ground motion experienced by the structure. Foundation damping effects tend to be greatest for stiff structural framing systems (e.g., reinforced concrete shear walls, steel braced frames) and relatively soft foundation soils (e.g., Site Classes D and E per ASCE-7-05). Simplified procedures for incorporating foundation damping in a numerical model for nonlinear dynamic response analysis are described below.

### Simplified Procedure for Kinematic Interaction

A ratio-of-response-spectra (RRS) factor can be used to represent kinematic interaction effects.  An RRS is the ratio of the response spectral ordinates imposed on the foundation (i.e., the foundation input motion, FIM) to the free-field spectral ordinates. Two phenomena should be considered in evaluating the RRS: base-slab averaging and foundation embedment. Foundation embedment effects should be considered for buildings with basements. The simplified procedure of Chapter 8 of FEMA 440 (FEMA, 2005) can be used for assessment of kinematic interaction. The following information is needed for such an assessment: dominant building period (as

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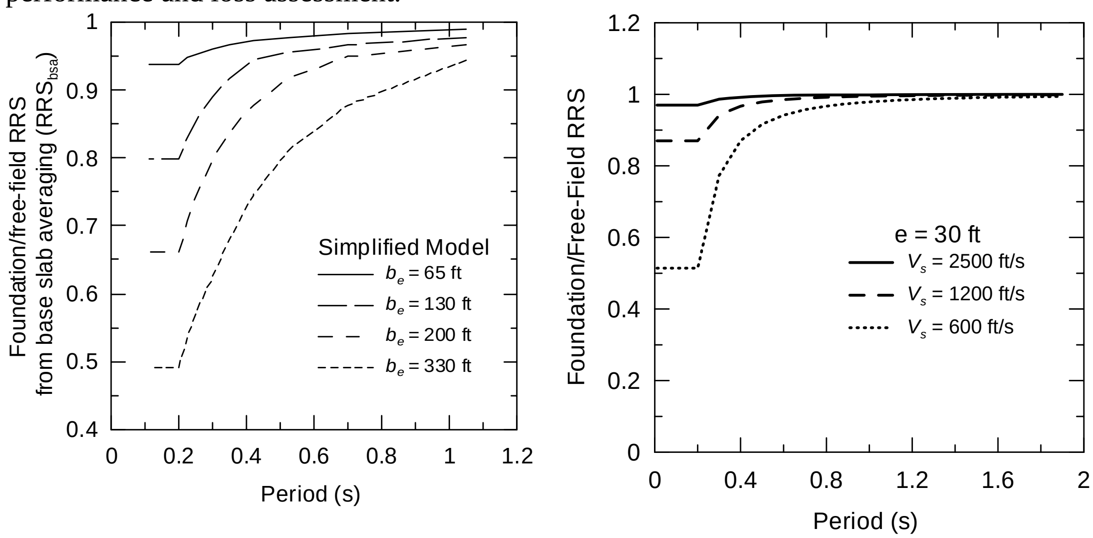
only case where base slab averaging effects should be neglected is for buildings without a laterally connected foundation system and with flexible floor and roof diaphragms.

[3] Embedment effects should not be considered for buildings without basements, even if the footings are embedded. Embedment effects tend to be significant when the depth of basements is greater than about 10 feet.

measured by % mass participating in the elastic base shear computation), building foundation plan dimension, foundation embedment, peak horizontal ground acceleration, and shear wave velocity. Limitations on the use of the simplified procedure are presented in Section 8 of FEMA 440. Figures B-10a and B-10b (from FEMA 440) provide summary information on the reductions in spectral demand due to base-slab averaging and embedment. The 5% damped free-field spectrum is multiplied by the product of the period-dependant reduction factors for base-slab averaging and embedment to derive the foundation input motion spectrum. Since the embedment computation is dependant on the peak horizontal ground acceleration, the computation must be repeated for every level of hazard considered for the performance and loss assessment.




a. base slab averaging b. embedment

Figure B-10 Reductions in spectral demand due to kinematic interaction

**Simplified Procedure for Foundation Damping**

Damping related to soil-foundation interaction due to hysteretic soil damping and radiation damping can significantly increase the effective damping in a structural frame.

Herein, the effects of foundation radiation damping serve to reduce the ordinates of a 5% damped acceleration response spectrum, which is generally used as the basis for generating earthquake histories for response-history analysis. The simplified procedure of FEMA 440 can be used for assessment of foundation damping. Hysteretic damping in the soil should be captured directly through the use of nonlinear soil springs. The following information is needed for such an assessment: dominant (first mode) building period (as measured by % mass participating in the elastic base shear computation), $T_1$; first modal mass; building foundation plan dimension; building height;

foundation embedment; strain-degraded soil shear modulus and soil Poisson’s ratio. Limitations on the use of the simplified procedure are presented in Section 8 of FEMA 440. The 5% damped free-field spectrum is modified in the period range $T \geq 0.75T_1$ to account for the additional damping provided by the foundation.

## B.8 Selecting and Scaling Ground Motions for Response Analysis

### *B.8.1 Bins of Earthquake Histories*

Two bins of 25 pairs of earthquake histories are presented in this Guideline for use in selecting ground motions. These bins of motions were used for the ATC-58 ground motion studies referenced in the body of this *Guideline*.

Bin 1 presents near-fault ground motions that should be used if the building site is within 15 km of an active fault capable of generating a moment magnitude 6.5 earthquake or greater; see Table B-3. The Bin 2 motions of Table B-4 are far-field ground motions.

### *B.8.2 General Procedures for Scaling Ground Motions*

The general procedures for scaling ground motions for *unidirectional* response-history analysis are based on ATC-58 ground motions studies that are summarized in Huang et al. (2007). The reader is referred to that project report for detailed information.

> Reader Note: General procedures for selecting and scaling ground motions for bidirectional response-history analysis are under development and will be presented in later drafts of this *Guideline*.

### *B.8.3 Procedures for Scaling High-Fractile Ground Motions*

> Reader Note: High-fractile ground motions were defined in Section B.6.4. The spectral shape of such ground motions are being studied at this time in the ATC-63 project and by Professor Baker at Stanford University. We plan to leverage these on-going activities to develop rules for selecting and scaling two- and three-component sets of earthquake histories for mean annual frequencies of exceedance of 0.001 and smaller. These procedures will be included in later drafts of the *Guideline*.

**Table B-3 Bin 1–Near-Fault Ground Motions**

| Designation | Event | Station | $M$ [1] | $r$ [1] |
|---|---|---|---|---|
| NF1, NF2 | Kobe 1995 | SAC 2/50 for Los Angeles | 6.9 | 3.4 |
| NF3, NF4 | Loma Prieta 1989 | | 7.0 | 3.5 |
| NF5, NF6 | Northridge 1994 | | 6.7 | 7.5 |
| NF7, NF8 | Northridge 1994 | | 6.7 | 6.4 |
| NF9, NF10 | Tabas 1974 | | 7.4 | 1.2 |
| NF11, NF12 | Elysian Park 1 (simulated) | | 7.1 | 17.5 |
| NF13, NF14 | Elysian Park 2 (simulated) | | 7.1 | 10.7 |
| NF15, NF16 | Elysian Park 3 (simulated) | | 7.1 | 11.2 |
| NF17, NF18 | Palos Verdes 1 (simulated) | | 7.1 | 1.5 |
| NF19, NF20 | Palos Verdes 2 (simulated) | | 7.1 | 1.5 |
| NF21, NF22 | Cape Mendocino 04/25/92 | 89156 Petrolia | 7.1 | 9.5 |
| NF23, NF24 | Chi-Chi 09/20/99 | TCU053 | 7.6 | 6.7 |
| NF25, NF26 | Chi-Chi 09/20/99 | TCU056 | 7.6 | 11.1 |
| NF27, NF28 | Chi-Chi 09/20/99 | TCU068 | 7.6 | 1.1 |
| NF29, NF30 | Chi-Chi 09/20/99 | TCU101 | 7.6 | 11.1 |
| NF31, NF32 | Chi-Chi 09/20/99 | TCUWGK | 7.6 | 11.1 |
| NF33, NF34 | Duzce 11/12/99 | Duzce | 7.1 | 8.2 |
| NF35, NF36 | Erzinkan 03/13/92 17:19 | 95 Erzinkan | 6.9 | 2.0 |
| NF37, NF38 | Imperial Valley 10/15/79 | 5057 El Centro Array #3 | 6.5 | 9.3 |
| NF39, NF40 | Imperial Valley 10/15/79 | 952 El Centro Array #5 | 6.5 | 1 |
| NF41, NF42 | Imperial Valley 10/15/79 | 942 El Centro Array #6 | 6.5 | 1 |
| NF43, NF44 | Kobe 01/16/95 20:46 | Takarazu | 6.9 | 1.2 |
| NF45, NF46 | Morgan Hill 04/24/84 04:24 | 57191 Halls Valley | 6.2 | 3.4 |
| NF47, NF48 | Northridge 1/17/94 12:31 | 24279 Newhall | 6.7 | 7.1 |
| NF49, NF50 | Northridge 1/17/94 12:31 | 0637 Sepulveda VA | 6.7 | 8.9 |

1. $M$ = moment magnitude; $r$ = closest site-to-fault-rupture distance

Table B-4 Bin 2–Far-Field Ground Motions

| Designation | Event | Station | $M$ [1] | $r$ [1] |
|---|---|---|---|---|
| FF1, FF2 | Cape Mendocino 04/25/92 | 89509 Eureka—Myrtle & West | 7.1 | 44.6 |
| FF3, FF4 | Cape Mendocino 04/25/92 | 89486 Fortuna—Fortuna Blvd | 7.1 | 23.6 |
| FF5, FF6 | Coalinga 1983/05/02 | 36410 Parkfield—Cholame 3W | 6.4 | 43.9 |
| FF7, FF8 | Coalinga 1983/05/02 | 36444 Parkfield—Fault Zone 10 | 6.4 | 30.4 |
| FF9, FF10 | Coalinga 1983/05/02 | 36408 Parkfield—Fault Zone 3 | 6.4 | 36.4 |
| FF11, FF12 | Coalinga 1983/05/02 | 36439 Parkfield—Gold Hill 3E | 6.4 | 29.2 |
| FF13, FF14 | Imperial Valley 10/15/79 | 5052 Plaster City | 6.5 | 31.7 |
| FF15, FF16 | Imperial Valley 10/15/79 | 724 Niland Fire Station | 6.5 | 35.9 |
| FF17, FF18 | Imperial Valley 10/15/79 | 6605 Delta | 6.5 | 43.6 |
| FF19, FF20 | Imperial Valley 10/15/79 | 5066 Coachella Canal #4 | 6.5 | 49.3 |
| FF21, FF22 | Landers 06/28/92 | 22074Yermo Fire Station | 7.3 | 24.9 |
| FF23, FF24 | Landers 06/28/92 | 12025 Palm Springs Airport | 7.3 | 37.5 |
| FF25, FF26 | Landers 06/28/92 | 12149 Desert Hot Springs | 7.3 | 23.2 |
| FF27, FF28 | Loma Prieta 10/18/89 | 47524 Hollister—South & Pine | 6.9 | 28.8 |
| FF29, FF30 | Loma Prieta 10/18/89 | 47179 Salinas—John &Work | 6.9 | 32.6 |
| FF31, FF32 | Loma Prieta 10/18/89 | 1002 APEEL 2—Redwood City | 6.9 | 47.9 |
| FF33, FF34 | Northridge 01/17/94 | 14368 Downey—Co Maint Bldg | 6.7 | 47.6 |
| FF35, FF36 | Northridge 01/17/94 | 24271 Lake Hughes #1 | 6.7 | 36.3 |
| FF37, FF38 | Northridge 01/17/94 | 14403 LA—116th St School | 6.7 | 41.9 |
| FF39, FF40 | San Fernando 02/09/71 | 125 Lake Hughes #1 | 6.6 | 25.8 |
| FF41, FF42 | San Fernando 02/09/71 | 262 Palmdale Fire Station | 6.6 | 25.4 |
| FF43, FF44 | San Fernando 02/09/71 | 289 Whittier Narrows Dam | 6.6 | 45.1 |
| FF45, FF46 | San Fernando 02/09/71 14:00 | 135 LA—Hollywood Stor Lot | 6.6 | 21.2 |
| FF47, FF48 | Superstition Hills (A) 11/24/87 | 5210Wildlife Liquef. Array | 6.3 | 24.7 |
| FF49, FF50 | Superstition Hills (B) 11/24/87 | 5210Wildlife Liquef. Array | 6.7 | 24.4 |

1. $M$ = moment magnitude; $r$ = closest site-to-fault-rupture distance

# Appendix C

# Fragility Development

## C.1 Introduction

### *C.1.1 Purpose*

This appendix provides guidelines for development of fragility functions for individual building components for use in building performance assessment. It may be used to set fragility functions for either structural or nonstructural components, elements or systems.

### *C.1.2 Fragility Function Definition*

Fragility functions are probability distributions that are used to indicate the probability that a component, element or system will be damaged to a given or more severe damage state as a function of a single predictive demand parameter such as story drift or floor acceleration. Here, fragility functions take the form of lognormal cumulative distribution functions, having a median value $\theta$ and logarithmic standard deviation, or dispersion, $\beta$. The mathematical form for such a fragility function is:

$$F_i(D) = \Phi\left(\frac{\ln\left(D/\theta_i\right)}{\beta_i}\right) \quad \text{(C-1)}$$

where: $F_i(D)$ is the conditional probability that the component will be damaged to damage state "i" or a more severe damage state as a function of demand parameter, $D$; $\Phi$ denotes the standard normal (Gaussian) cumulative distribution function, $\theta_i$ denotes the median value of the probability distribution, and $\beta_i$ denotes the logarithmic standard deviation. Both $\theta$ and $\beta$ are established for each component type and damage state using one of the methods presented in Section C.2.

The conditional probability that a component will be damaged to damage state "i" and not to a more or less severe state, given that it experiences demand, $D$ is given by:

$$P\left[i|D\right] = F_{i+1}\left(D\right) - F_i\left(D\right) \quad \text{(C-2)}$$

where $F_{i+1}(D)$ is the conditional probability that the component will be damaged to damage state "i+1" or a more severe state and $F_i(D)$ is as previously defined. Note that, when $\beta_{i+1}$ is unequal to $\beta_i$, Equation C-2 can produce a meaningless negative probability at some levels of $D$. This case is addressed in Section C.3.4.

Figure C-1 (a) shows the form of a typical fragility function when plotted in the form of a cumulative distribution function and (b) the calculation of the probability that a component will be in damage state "i" at a particular level of demand, $d$.

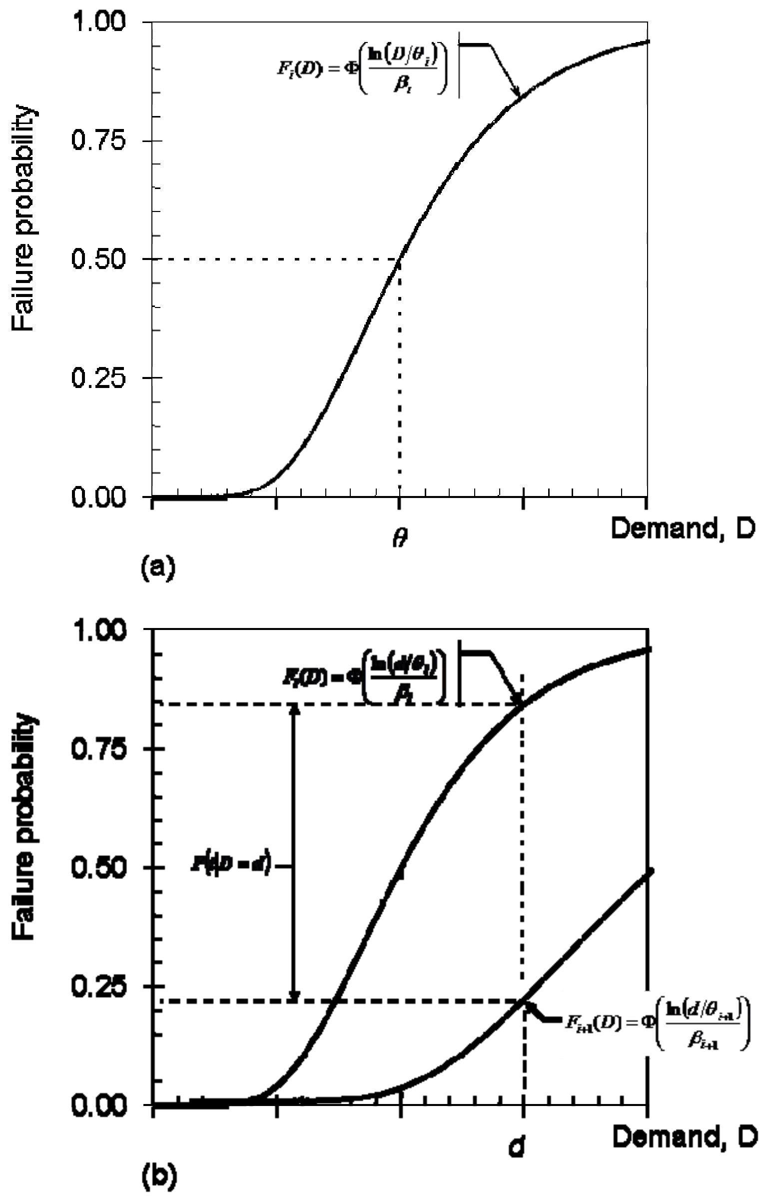



Figure C-1 Illustration of (a) fragility function, and (b) evaluating individual damage-state probabilities

The dispersion, $\beta$, represents uncertainty in the actual value of demand, $D$, at which a damage state is likely to initiate in a component . This uncertainty is a result of variability in the quality of construction and installation of the components in a building, as well as variability in the loading history that the component may experience before it fails. When fragility parameters are determined on the basis of a limited set of test data, two components of the dispersion should be considered. The first of these, termed herein $\beta_r$, represents the random variability that is observed in the available test data from which the fragility parameters are determined. The second portion, $\beta_u$, represents uncertainty that the tests represent the actual conditions of installation and loading that a real component in a building will experience, or that the available test data is an inadequate sample to accurately represent the true random variabiltiy. The dispersion parameter $\beta$, is computed as:

$$\beta = \sqrt{\beta_r^{\ 2} + \beta_u^{\ 2}} \qquad \text{(C-3)}$$

In these guidelines, the following minimum values of the uncertainty parameter $\beta_u$ are recommended for use: A minimum value of 0.25 should be used if any of the following apply:

- Test data are available for five (5) or fewer specimens
- In an actual building, the component can be installed in a number of different configurations, however, all specimens tested had the same configuration.
- All specimens were subjected to the same loading protocol
- Actual behavior of the component is expected to be dependent on two or more demand parameters, e.g. simultaneous drift in two orthogonal directions, however, specimens were loaded with only one of these parameters.

If none of the above conditions apply, a value of $\beta_u$ of 0.10 may be used.

### *C.1.3 Derivation Methods*

Fragility functions can best be derived when there is a large quantity of appropriate test data available on the behavior of the component of interest at varying levels of demand. *FEMA 461* provides recommended protocols for performing such tests and recording the data obtained. Since testing is expensive and time consuming, there is not a great body of test data presently available to serve as the basis for determining fragility functions for many building components. Therefore, these guidelines provide procedures for developing the median ($\theta$) and dispersion ($\beta$) values for a fragility under five different conditions of data. These are:

a. Actual Demand Data: When test data is available from $M$ number of specimens and each tested component actually experienced the damage state of interest at a known value of demand, $D$.

b. Bounding Demand Data: When test data or earthquake experience data are available from $M$ number of specimens, however, the damage state of interest only occurred in some specimens. For the other specimens, testing was terminated before the damage state occurred or the earthquake did not damage the specimens. The value of the maximum demand, $D_i$, to which each specimen was subjected is known for each specimen. This maximum demand need not necessarily be the demand at which the damage state initiated.

c. Capable Demand Data: When test data or earthquake experience data are available from $M$ number of specimens, however, the damage state of interest did not occur in any of the specimens. The maximum value of demand, $D_i$, to which each specimen was subjected is known.

d. Derivation (analysis): When no test data are available, however, it is possible to model the behavior and estimate the level of demand at which the damage state of interest will occur.

e. Expert Opinion: When no data are available and analysis of the behavior is not feasible, however, one or more knowledgeable individuals can offer an opinion as to the level of demand at which damage is likely to occur, based either on experience or judgment.

In addition, a procedure is presented for updating existing fragility functions as more data become available. Section C2.6 provides this guidance.

### *C.1.4 Documentation*

This section provides recommendations for documenting the basis for fragility functions. Each fragility function should be accompanied by documentation of the sources of data and procedures used to establish the fragility parameters in sufficient detail that others may evaluate the adequacy of the process and findings. As a minimum, it is recommended that the documentation include the following:

1. *Description of applicability.* Describe the type of component that the fragility function addresses including any limitations on the type of installation to which the fragility applies.
2. *Description of specimens*. Describe the specimens used to establish the fragility including identifying the number of specimens examined, their

locations, and the specific details of the specimen fabrication/construction, mounting and installation.

3. *Demands and Load Application*. Detail the loading protocol or characteristics of earthquake motion applied to each specimen. Identify the demand parameters examined that might be most closely related to failure probability and define how demand is calculated or inferred from the loading protocol or excitation. Indicate whether the reported demand quantities are the value at which damage occurred (Method A data) or the maximum to which each specimen was subjected.

4. *Damage state.* Fully describe each damage state for which fragilities are developed including the kinds of physical damage observed and any force-deformation quantities recorded. Define damage states quantitatively in terms of the repairs required or potential downtime or casualty consequences.

5. *Observation summary, analysis method, and results.* Present a tabular or graphical listing of specimens, demand parameters, and damage states. Identify the method(s) used to derive the fragility parameters per Section C.2 of this Appendix. Present resulting fragility function parameters $\theta$ and $\beta$ and results of tests to establish fragility function quality (discussed below). Provide sample calculations.

## C.2 Fragility Parameter Derivation

### *C.2.1 Actual Demand Data*

This section defines the procedures for deriving fragility parameters ($\theta$, $\beta$) when data is available from a suitable series of tests and in each specimen, the damage state of interest was initiated at a known value of the demand. In this case, the median value of the demand at which the damage state is likely to initiate, $\theta$, is given by the equation:

$$\theta = e^{\left(\frac{1}{M}\sum_{i=1}^{M}\ln d_i\right)} \tag{C-3}$$

where:

$M$ = total number of specimens tested to at least the initiation of the damage state

$d_i$ = demand in test "i" at which the damage state was first observed to occur.

The value of the random dispersion, $\beta_r$, is given by:

$$\beta_r = \sqrt{\left(\frac{1}{M-1}\sum_{i=1}^{M}\left(\ln\left(d_i/\theta\right)\right)^2\right)^2} \quad \text{(C-4)}$$

where $M$, $r_i$ and $\theta$ are as defined above.

If one or more of the $r_i$ data appear to lie far from the bulk of the data, either above or below, apply the procedure specified in Section C3.2. Finally, test the resulting fragility parameters using the Lilliefors goodness-of-fit test (Section C3.3). If it passes at the 5% significance level, the fragility function may be deemed acceptable.

**Example**: Determine the parameters $\theta$ and $\beta$, from a series of 10 tests, all of which produced the damage state of interest. Demands at which the damage state initiated are respectively story drifts of: 0.9, 0.9, 1.0, 1.1, 1.1, 1.2, 1.3, 1.4, 1.7, and 2 percent.

| Test # | Demand di | ln(di) | ln(di/θi) | ln(di/θi)^2 |
|---|---|---|---|---|
| 1 | 0.9 | -0.10536 | -0.30384 | 0.092321 |
| 2 | 0.9 | -0.10536 | -0.30384 | 0.092321 |
| 3 | 1 | 0 | -0.19848 | 0.039396 |
| 4 | 1.1 | 0.09531 | -0.10317 | 0.010645 |
| 5 | 1.1 | 0.09531 | -0.10317 | 0.010645 |
| 6 | 1.2 | 0.182322 | -0.01616 | 0.000261 |
| 7 | 1.3 | 0.262364 | 0.063881 | 0.004081 |
| 8 | 1.4 | 0.336472 | 0.137989 | 0.019041 |
| 9 | 1.7 | 0.530628 | 0.332145 | 0.11032 |
| 10 | 2 | 0.693147 | 0.494664 | 0.244692 |
| Σ | | 1.984833 | | 0.623723 |

$$\theta = e^{\left(\frac{1}{M}\sum_{i=1}^{M}\ln d_i\right)} = e^{\left(\frac{1}{10}(1.9848)\right)} = 1.22$$

$$\beta_r = \sqrt{\frac{1}{M-1}\sum_{i=1}^{M}\left(\ln\left(d_i/\theta\right)\right)^2} = \sqrt{\frac{1}{(10-1)}(0.6237)} = 0.26$$

### *C.2.2 Bounding Demand Data*

This section defines the procedures for deriving fragility parameters ($\theta$, $\beta$) when data are available from a suitable series of tests or earthquake experience records, however, the damage state of interest was initiated in only some of the specimens. For the other specimens, loading applied during the testing or earthquake shaking was insufficient to intiate the damage state of interest occurred. For each specimen "i", it is necessary to know the maximum value of the demand, $d_i$ to which the specimen was subjected, and whether or not the damage state did occur in the specimen.

Divide the data into a series of $N$ bins. It is suggested, but not essential, that $N$ be taken as the largest integer that is less than or equal to the square root of $M$, where $M$ is the total number of specimens available, as this will usually result in an appropriate number of approximately equally sized sets.

In order to divide the specimens into the several bins, sort the specimen data in order of ascending maximum demand value, $d_i$, for each test, then divide the list into $N$ groups of approximately equal size. Each group "$j$" will have $M_j$ specimens, where:

$$\sum_{i=1}^{N} M_j = M \tag{C-5}$$

Next, determine the average value of the maximum demand for each bin of specimens:

$$\bar{d}_j = \frac{1}{M_j} \sum_{k=1}^{M_j} d_k \tag{C-6}$$

and $x_j$, the natural logarithm of $d_j$ (i.e., $\ln(d_j)$). Also determine the number of specimens within each bin, $m_j$, in which the damage state of interest was achieved and the inverse standard normal distribution, $y_j$, of the failed fraction specimens in the bin:

$$y_j = \Phi^{-1}\left(\frac{m_j + 1}{M_j + 1}\right) \tag{C-7}$$

That is, determine the number of standard deviations, above the mean that the stated fraction lies, assuming a mean value, $\mu$=0 and a standard deviation, $\sigma$=1. This can easily be determined using the "normsinv" function on a Microsoft Excel spreadsheet or by referring to standard tables of the normal distribution. Next, fit a straight line to the data points, $x_j$, $y_j$, using a least-squares approach. The straight line will have the form:

$$y = bx + c \tag{C-8}$$

where, $b$ is the slope of the line and $c$ is the y intercept. The slope $b$ is given by:

$$b = \frac{\sum_{i=1}^{M} \left(x_j - \bar{x}\right)\left(y_j - \bar{y}\right)}{\sum_{i=1}^{M} \left(x_j - \bar{x}\right)^2} \tag{C-9}$$

$$\bar{x} = \frac{1}{M}\sum_{j=1}^{M} x_j \tag{C-10}$$

$$\bar{y} = \frac{1}{M}\sum_{j=1}^{M} y_j \tag{C-11}$$

Determine the value of the random dispersion, $\beta_r$ as:

$$\beta_r = \frac{1}{b} = \frac{\sum_{i=1}^{M}\left(x_j - \bar{x}\right)}{\sum_{i=1}^{M}\left(x_j - \bar{x}\right)\left(y_j - \bar{y}\right)} \tag{C-12}$$

The value of the median, $\theta$, is taken as:

$$\theta = e^{-c\beta_r} = e^{\left(\bar{x} - \bar{y}\beta_r\right)} \tag{C-13}$$

**Example:** Consider the damage statistics shown in the figure below. The figure depicts the hypothetical performance of motor control centers (MCCs) observed after various earthquakes in 45 facilities. Each box represents one specimen. Several damage states are represented. Crosshatched boxes represent MCCs that experienced a noticeable earthquake effect such as shifting but that remained operable. Black boxes represent those that were found to be inoperable following the earthquake. Each stack of boxes represents one facility. Calculate the fragility function using *PGA* as the demand parameter, binning between halfway points between *PGA* values shown in the figure.

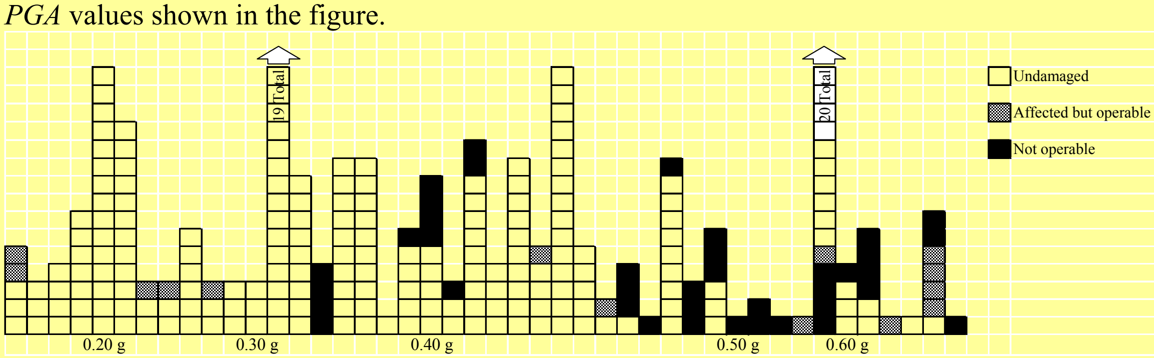



**Figure Hypothetical observed earthquake damage data for motor control centers**

The number of bins, *N*, and the lower demand bounds $a_j$, are dictated by the available data: *N* is taken as 5 with lower bounds, $a_i$ of 0.15g, 0.25g, 0.35g, 0.45g, and 0.55g respectively. The damage state of interest is loss of post-earthquake functionality (black boxes in figure). The values of $M_j$ and $m_j$ are found by counting all boxes and black boxes, respectively, in the figure in each bin, and are shown in the table below. The value of $M$ is found by summing: $M = \Sigma M_j = 260$. Values $x_j$ and $y_j$ are calculated as $x_j = \ln(\bar{r}_j)$, and $y_j = \Phi^{-1}((m_j+1)/(M_j+1))$. Average values are calculated as shown: $\bar{x} = –0.99$, $\bar{y} = –1.05$, according to Equations C-10 and C-11. For each bin, the values of $x_j - \bar{x}$ and $y_j - \bar{y}$ are calculated as shown.

**Table Example solution data**

| $j$ | $a_j$ (g) | $\bar{r}_j$ (g) | $M_j$ | $m_j$ | $x_j$ | $y_j$ | $x_j - \bar{x}$ | $y_j - \bar{y}$ | $\left(x_j - \bar{x}\right)^2$ | $\left(x_j - \bar{x}\right)\left(y_j - \bar{y}\right)$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.15 | 0.2 | 52 | 0 | -1.61 | -2.08 | -0.623 | -1.031 | 0.388 | 0.642 |
| 2 | 0.25 | 0.3 | 48 | 4 | -1.20 | -1.27 | -0.217 | -0.223 | 0.047 | 0.049 |
| 3 | 0.35 | 0.4 | 84 | 8 | -0.92 | -1.25 | 0.070 | -0.202 | 0.005 | -0.014 |
| 4 | 0.45 | 0.5 | 35 | 15 | -0.69 | -0.14 | 0.294 | 0.907 | 0.086 | 0.266 |
| 5 | 0.55 | 0.6 | 41 | 12 | -0.51 | -0.50 | 0.476 | 0.549 | 0.226 | 0.261 |
| Σ = | | | 260 | | -4.93 | -5.23 | | | 0.753 | 1.204 |
| | | Average = | | | -0.99 | -1.05 | | | | |

Then, $\beta$ and $\theta$ are calculated as:

$$\beta_r = \frac{\sum_{j=1}^{N}\left(x_j - \bar{x}\right)^2}{\left(x_j - \bar{x}\right)\left(y_j - \bar{y}\right)} = \frac{0.753}{1.204} = 0.63$$

$$\theta = e^{\left(\bar{x} - \bar{y}\beta_r\right)} = e^{\left(-0.99+1.05(0.63)\right)} = e^{(-0.329)} = 0.72g$$

### *C.2.3 Capable Demand Data*

This section defines the procedures for deriving fragility parameters ($\theta$, $\beta$) when data is available from a suitable series of specimens, however, the damage state of interest was not initiated in any of the specimens. For each available specimen, ”$i$,” the maximum demand at which the specimen was loaded, “$d_i$”, and whether or not the specimen experienced any distress or damage must be known.

From the data for $M$ specimens, determine the maximum demand experienced by each specimen, $d_{max}$, and the minimum demand for any of the specimens that exhibited any distress or damage, $d_{min}$. Determine $d_a$ as the smaller of $d_{min}$ or $0.7d_{max}$. Determine $M_A$ as the number of specimens that did not exhibit distress or damage, but that were loaded with demands, $d_i \geq d_a$; $M_B$ as the number of specimens that exhibited distress or damage, but which did not appear to be initiating or on the verge of initiating the damage state of interest; and $M_C$ as the number of specimens appeared to be on the verge of initiating the damage state of interest.

If none of the specimens in any of the tests exhibited any sign of distress or damage, take the value of $d_m$ as $d_{max}$. If one or more of the specimens exhibited distress or damage of some type, take $d_m$ as:

$$d_m = \frac{d_{\max} + d_a}{2} \tag{C-14}$$

Determine the subjective failure probability $S$ at $d_m$ as:

$$S = \frac{0.5M_C + 0.1M_B}{M_A + M_B + M_C} \tag{C-15}$$

Take the logarithmic standard deviation, $\beta$, as having a value of 0.4. Determine the median, $\theta$, as:

$$\theta = d_m e^{-0.4z} \tag{C-16}$$

where, $z$ is determined from Table C-1 based on the value of $M_A$ and $S$.

**Table C-1. Values of z**

| Conditions | $Z$ |
|---|---|
| $M_A \geq 3$ and $S = 0$ | -2.326 |
| $M_A < 3$ and $S \leq 0.075$ | -1.645 |
| $0.075 < S \leq 0.15$ | -1.282 |
| $0.15 < S \leq 0.3$ | -0.842 |
| $S > 0.3$ | -0.253 |

**Example**: Determine the parameters $\theta$ and $\beta$, from tests of 10 specimens. Five of the specimens had maximum imposed drift demands of 1% with no observable signs of distress. Three of the specimens had maximum imposed drift demands of 1.5% and exhibited minor distress, but did not appear to be at or near the initiation of the damage state of interest. Two of the specimens had maximum imposed drift demands of 2%, did not enter the damage state of interest during the test, but appeared to be about to sustain such damage. From the given data determine: $d_{max}$ = 2%, $d_{min}$ = 1.5%. $d_a$ is the smaller of $0.7d_{max}$ or $d_{min}$ and therefore, is 0.7(2%) = 1.4%. $M_A$ = 0, $M_B$=3, and $M_C$=2.

$$d_m = \frac{d_{\max} + d_a}{2} = \frac{2\% + 1.4\%}{2} = 1.7\%$$

$$S = \frac{0.5M_C + 0.1M_B}{M_A + M_B + M_C} = \frac{0.5(2) + 0.1(3)}{0 + 3 + 2} = 0.26$$

From Table 2-1, $z$ is taken as -0.842. Therefore,

$$\theta = d_m e^{-0.4z} = 1.7\% e^{-0.4(-0.842)} = 2.4\%$$

and $\beta$ is taken as 0.4.

### *C.2.4 Derivation*

There are two methods available for analytical derivation of fragility parameters. The first of these uses a single calculation of the probable capacity and a default value of the logarithmic standard deviation. The second method uses Monte Carlo analysis to explore the effect of variation in material strength, construction quality and other random variables.

**Single calculation**

Calculate the capacity of the component, $Q$ in terms of a demand parameter, *d,* using average material properties and dimensions and estimates of workmanship. Resistance factors should be taken as unity and any conservative bias in code equations, if such equations are used, should be removed. The logarithmic standard deviation, $\beta$, is taken as having a value of 0.4. The median capacity θ is taken as:

$$\theta = 0.92Q \qquad \text{(C-17)}$$

**Monte Carlo simulation**

Identify all those factors, important to predicting the capacity that are uncertain including material strength, cross section dimensions, member straightness, workmanship. Estimate a median value and dispersion for each of these random variables. Conduct sufficient analyses, randomly selecting the values of each of these random variables in accordance with their estimated distribution properties, each time calculating the capacity. Determine the median value of the capacity as that capacity exceeded in 50% of the calculations. Determine the random logarithmic standard deviation, $\beta_r$, as the standard deviation of the natural logarithm of the calculated capacity values. Use equation 1-3 to determine the total logarithmic standard deviation, β, assuming a value of $\beta_u$ of 0.25.

### *C.2.5 Expert Opinion*

Select one or more experts with professional experience in the design or post-earthquake damage observation of the component of interest. Solicit their advice using the format shown in

. Note the suggested inclusion of representative images, which should be recorded with the responses. If an expert refuses to provide estimates or limits them to certain conditions, either narrow the component definition accordingly and iterate, or ignore that expert's response and analyze the remaining ones.

Calculate the median value, $\theta$, as:

$$\theta = \frac{\sum_{i=1}^{N} w_i^{1.5}\theta_i}{\sum_{i=1}^{N} w_i^{1.5}} \tag{C-18}$$

where, $N$ is the number of experts providing an opinion; $\theta_i$ is expert "i's" opinion as to the median value, and $w_i$ is expert "i's" level of expertise, on a 1-5 scale.

Calculate the lower bound value for the capacity as:

$$d_l = \frac{\sum_{i=1}^{N} w_i^{1.5} d_{li}}{\sum_{i=1}^{N} w_i^{1.5}} \tag{C-19}$$

where, $d_{li}$ is expert "i's" opinion as to the lower bound value and other terms are as previously defined. The value of the logarithmic standard deviation, $\beta$, is taken as:

$$\beta = \frac{\ln(\theta / d_l)}{1.28} \tag{C-20}$$

If this calculation produces an estimate of $\beta$ that is less than 0.4, either justify the $\beta$, or take $\beta$ as having a value of 0.4 and recalculate $\theta$ as:

$$\theta = 1.67 d_l \tag{C-21}$$

***Objective***. This form solicits your judgment about the values of a demand parameter (*D*) at which a particular damage state occurs to a particular building component. Judgment is needed because the component may contribute significantly to the future earthquake repair cost, fatality risk, or post-earthquake operability of a building, and because relevant empirical and analytical data are currently impractical to acquire. Your judgment is solicited because you have professional experience in the design or post-earthquake damage observation of the component of interest.

***Definitions.*** Please provide judgment on the damageability of the following component and damage state. Images of a representative sample of the component and damage state may be attached. It is recognized that other demand parameters may correlate better with damage, but please consider only the one specified here.

Component name: ______________________________________________________________
Component definition:__________________________________________________________
Damage state name: ____________________________________________________________
Damage state definition: ___
________________________________________________________________________________
Relevant Demand Parameter_
Definition of *Demand Parameter*_________________________________________________
________________________________________________________________________________

***Uncertainty; no personal stake.*** Please provide judgment about this general class of components, not any particular instance, and not one that you personally designed, constructed, checked, or otherwise have any stake in. There is probably no precise threshold level of demand that causes damage, because of variability in design, construction, installation, inspection, age, maintenance, interaction with nearby components, etc. Even if there were such a precise level, nobody might know it with certainty. To account for these uncertainties, **please provide two values of demand at which damage occurs: median and lower bound.**

***Estimated median capacity*** _______ *Definition*. Damage would occur at this level of demand in 5 cases out of 10, or in a single instance, you judge there to be an equal chance that your median estimate is too low or too high.

***Estimated lower-bound capacity*** _______*Definition*. Damage would occur at this level of demand in 1 case in 10. In a single case, you judge there to be a 10% chance that your estimate is too high. *Judge the lower bound carefully*. Make an initial guess, then imagine all the conditions that might make the actual threshold demand lower, such as errors in design, construction or installation, substantial deterioration, poor maintenance, more interaction with nearby components, etc. Revise accordingly and record your revised estimate. Research shows that without careful thought, expert judgment of the lower bound tends to be too close to the median estimate, so think twice and do not be afraid of showing uncertainty.

On a 1-to-5 scale, please judge your expertise with this component and damage state, where 1 means “no experience or expertise” and 5 means “very familiar or highly experienced.”
Your level of expertise:__________

Your name:____________________ Date: _____________________________________

Figure C-2 Form for soliciting expert judgment on component fragility

### C.2.6 Updating

This section addresses procedures for re-evaluating fragility parameters for a building component as additional data become available. The pre-existing and updated fragility parameters are respectively termed $\theta$, $\beta$, $\theta'$, and $\beta'$. The additional data are assumed to be a set of *M* specimens with known maximum demand and damage states. It is not necessary that any of the specimens experienced damage.

Calculate the revised median, $\theta'$ and logarithmic standard deviation $\beta'$ as follows:

$$\theta' = e^{\sum_{j=1}^{5} w'_j \ln(d_j)} \tag{C-21}$$

$$\beta' = \sum_{j=1}^{5} w'_j \beta_j \tag{C-22}$$

where:

$$w'_j = \frac{w_j \prod_{i=1}^{M} L(i,j)}{\sum_{j=1}^{5} w_j \prod_{i=1}^{M} L(i,j)} \tag{C-23}$$

where Π denotes the product of the terms that come after it, and

$L(i,j) = 1 - \Phi\left(\frac{\ln(d_i/x_j)}{0.707 s \beta_j}\right)$ if specimen "i" did not experience the damage state of interest, and

$L(i,j) = \Phi\left(\frac{\ln(d_i/x_j)}{0.707 s \beta_j}\right)$ if specimen "i" did experience the damage state of interest, or a more severe state. and

$x_1 = x_4 = x_5 = \theta$

$x_2 = \theta e^{-1.22\beta}$

$x_3 = \theta e^{1.22\beta}$

$\beta_1 = \beta_2 = \beta_3 = \beta$

$\beta_4 = 0.64\beta$

$\beta_5 = 1.36\beta$

$w_1 = 1/3$

$w_2 = w_3 = w_4 = w_5 = 1/6$

## C.3 Assessing Fragility Function Quality

This section provides procedures that can be used to assess the quality of fragility parameters.

### *C.3.1 Competing Demand Parameters*

The behavior of some components may be dependent on several types of demands, for example in-plane and out-of-plane drift, or both drift and acceleration. It may not be clear which demand is the best single predictor of component damage. Assuming that data are available to create fragility functions for each possibly relevant demand, do so. Choose the fragility function that has the lowest $\beta$.

### *C.3.2 Dealing with Outliers using Pierce's Criterion*

When fragilities are determined on the basis of actual demand data (Section C2.1), it is possible that one or more tests reported spurious values of demand, $d_i$, and reflect experimental errors rather than the true demands at which the specimens failed. In cases where one or more $d_i$ values in the data set are obvious outliers from the bulk of the data, investigate whether the data reflects real issues in the damage process that may recur, especially where $d_i << \theta$ for these outliers. If there is no indication that these data reflect a real recurring issue in the damage process, apply the following procedure (Peirce's criterion) to test and eliminate doubtful observations of $d_i$.

1. Calculate ln($\theta$) and $\beta$ of the complete data set.
2. Let $D$ denote the number of doubtful observations, and let $R$ denote the maximum distance of an observation from the body of the data, defined as:

$$R = \frac{\left|\ln(d) - \ln\theta\right|}{\beta} \qquad \text{(C-24)}$$

where $\theta$, $\beta$, and $M$ are as previously defined, $d$ is a measured demand value, and $R$ is as shown in Table C-. Assume $D$ = 1 first, even if there appears to be more than one doubtful observation.

3. Calculate the maximum allowable deviation: $|\ln(d) - \ln(\theta)|_{max}$. Note that this can include $d >> \theta$ and $d << \theta$.
4. For any suspicious measurement $d_i$, obtain $|\ln(d_i) - \ln(\theta)|$.
5. Eliminate the suspicious measurements if: $\left|\ln(d_i) - \ln(\theta)\right| > \left|\ln(d) - \ln(\theta)\right|_{max}$
6. If this results in the rejection of one measurement, assume $D$=2, keeping the original values of $\theta$ and $\beta$, and go to step 8.
7. If more than one measurement is rejected in the above test, assume the next highest value of doubtful observations. For example, if two

measurements are rejected in step 5, assume the case of $D = 3$, keeping the original values of $\theta$, and $\beta$, as the process is continued.

8. Repeat steps 2 – 5, sequentially increasing $D$ until no more data measurements are eliminated.
9. Obtain $\theta$ and $\beta$ of the reduced data set as for the original data.

**Table C-2. Parameters for Applying Peirce's Criterion**

| *M* | *D*=1 | *D*=2 | *D*=3 | *D*=4 | *D*=5 | *D*=6 | *D*=7 | *D*=8 | *D*=9 |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 1.1960 | | | | | | | | |
| 4 | 1.3830 | 1.0780 | | | | | | | |
| 5 | 1.5090 | 1.2000 | | | | | | | |
| 6 | 1.6100 | 1.2990 | 1.0990 | | | | | | |
| 7 | 1.6930 | 1.3820 | 1.1870 | 1.0220 | | | | | |
| 8 | 1.7630 | 1.4530 | 1.2610 | 1.1090 | | | | | |
| 9 | 1.8240 | 1.5150 | 1.3240 | 1.1780 | 1.0450 | | | | |
| 10 | 1.8780 | 1.5700 | 1.3800 | 1.2370 | 1.1140 | | | | |
| 11 | 1.9250 | 1.6190 | 1.4300 | 1.2890 | 1.1720 | 1.0590 | | | |
| 12 | 1.9690 | 1.6630 | 1.4750 | 1.3360 | 1.2210 | 1.1180 | 1.0090 | | |
| 13 | 2.0070 | 1.7040 | 1.5160 | 1.3790 | 1.2660 | 1.1670 | 1.0700 | | |
| 14 | 2.0430 | 1.7410 | 1.5540 | 1.4170 | 1.3070 | 1.2100 | 1.1200 | 1.0260 | |
| 15 | 2.0760 | 1.7750 | 1.5890 | 1.4530 | 1.3440 | 1.2490 | 1.1640 | 1.0780 | |
| 16 | 2.1060 | 1.8070 | 1.6220 | 1.4860 | 1.3780 | 1.2850 | 1.2020 | 1.1220 | 1.0390 |
| 17 | 2.1340 | 1.8360 | 1.6520 | 1.5170 | 1.4090 | 1.3180 | 1.2370 | 1.1610 | 1.0840 |
| 18 | 2.1610 | 1.8640 | 1.6800 | 1.5460 | 1.4380 | 1.3480 | 1.2680 | 1.1950 | 1.1230 |
| 19 | 2.1850 | 1.8900 | 1.7070 | 1.5730 | 1.4660 | 1.3770 | 1.2980 | 1.2260 | 1.1580 |
| 20 | 2.2090 | 1.9140 | 1.7320 | 1.5990 | 1.4920 | 1.4040 | 1.3260 | 1.2550 | 1.1900 |
| >20 | $a\ln M + b$ | | | | | | | | |
| *a* | 0.4094 | 0.4393 | 0.4565 | 0.4680 | 0.4770 | 0.4842 | 0.4905 | 0.4973 | 0.5046 |
| *b* | 0.9910 | 0.6069 | 0.3725 | 0.2036 | 0.0701 | -0.0401 | -0.1358 | -0.2242 | -0.3079 |

### *C.3.3 Goodness of Fit Testing*

Fragility parameters that are developed based on actual demand data (Section C2.1) should be tested for goodness of fit in accordance with this section. Calculate

$$D = \max_x \left| F_i(d) - S_M(d) \right| \tag{C-25}$$

where $S_M(d)$ denotes the sample cumulative distribution function

$$S_M(d) = \frac{1}{M} \sum_{i=1}^{M} H\left(d_i - d\right) \tag{C-26}$$

and $H$ is taken as:

- 1.0 if $d_i - d$ is positive
- ½ if $d_i - d$ is zero
- 0 if $d_i - d$ is negative.

If $D > D_{\text{crit}}$ from Table C-, the fragility function fails the goodness of fit test. This result is used in assigning a quality level to the fragility function. Use α = 0.05.

**Table C-3 Critical Values for the Lilliefors Test**

| Significance Level | $D_{crit}$ |
|---|---|
| α = 0.15 | 0.775 / ($M^{0.5}$ – 0.01 + 0.85$M^{-0.5}$) |
| α = 0.10 | 0.819 / ($M^{0.5}$ – 0.01 + 0.85$M^{-0.5}$) |
| α = 0.05 | 0.895 / ($M^{0.5}$ – 0.01 + 0.85$M^{-0.5}$) |
| α = 0.025 | 0.995 / ($M^{0.5}$ – 0.01 + 0.85$M^{-0.5}$) |

### *C.3.4 Fragility Functions that Cross*

Some components will have two or more possible damage states, with a defined fragility function for each. For any two (cumulative lognormal) fragility functions $i$ and $j$ with medians $\theta_j > \theta_i$ and logarithmic standard deviations $\beta_i \neq \beta_j$, the fragility functions will cross at extreme values. In such a case, adjust the fragility functions by one of the following two methods;
Method 1: adjust the fragility functions such that.

$$F_i(D) = \max_j \left\{ \Phi\left( \frac{\ln\left(D/\theta_i\right)}{\beta_i} \right) \right\} \quad \text{for all } j \geq i \tag{C-27}$$

This has the effect that for the damage state with the higher median value, the probability of failure, $F_i(D)$ is never taken as less than the probability of failure for a damage state with a lower median value.

Method 2: First establish $\theta$ and $\beta$ values for the various damage states independently. Next calculate the average of the dispersion values for each of the damage states with crossing fragility curves as:

$$\beta_i' = \frac{1}{N}\sum_{i=1}^{N}\beta_i \qquad \text{(C-28)}$$

This average logarithmic standard deviation is used as a replacement for the independently calculated values. An adjusted median value must be calculated for each of the crossing fragilities as:

$$\theta_i' = e^{\left(1.28\left(\beta'-\beta_i\right)+\ln\theta_i\right)} \qquad \text{(C-29)}$$

### C.3.5 Assigning a Single Quality Level to a Fragility Function

Assign each fragility function a quality level of high, medium, or low, as shown in Table C-4.

**Table C-4 Fragility Function Quality Level**

| Quality | Method | Peer reviewed* | Number of specimens | Other |
|---|---|---|---|---|
| High | A | Yes | ≥ 5 | Passes Lilliefors test at 5% significance level. Examine and justify (a) differences of greater than 20% in $\theta$ or $\beta$, compared with past estimates, and (b) any case of $\beta < 0.2$ or $\beta > 0.6$. |
| | B | Yes | ≥ 20 | Examine and justify (a) differences of greater than 20% in $\theta$ or $\beta$, compared with past estimates, and (b) any case of $\beta < 0.2$ or $\beta > 0.6$. |
| | U | Yes | ≥ 6 | Prior was at least moderate quality |
| Moderate | A | | ≥ 3 | Examine and justify any case of $\beta < 0.2$ or $\beta > 0.6$. |
| | B | | ≥ 16 | Examine and justify any case of $\beta < 0.2$ or $\beta > 0.6$. |
| | C | Yes | ≥ 6 | |
| | D | Yes | | |
| | E | Yes | | At least 3 experts with $w \geq 3$ |
| | U | | ≥ 6 | or prior was moderate quality |
| Low | | | | All other cases |

* Data and derivation published in a peer-reviewed archival journal.

# Appendix D

# Default Structural Fragility Data

## D.1 Introduction

ASCE-7-05, *Minimum Design Loads on Buildings and Other Structures* (ASCE, 2005) identifies seventy-eight lateral-force-resisting systems for new building construction. Many other framing systems have also been constructed over the course of the past 50 years. In addition, these various types of lateral-force-resisting systems can be combined with many gravity-load carrying systems to form a very large number of possible building types. Although it is possible to use the methodology contained in these *Guidelines* to a building of any construction type, to do so would require a combination of laboratory test data and expert opinion on the behavior of components of each structural system. Many users of these *Guidelines* are unlikely to have either sufficient laboratory test data or expert opinions at their disposal to develop such fragility specifications, so default fragility data for common structural components and systems are provided.

A classification is used to group all possible framing systems into a modest number of systems for which default fragility and loss data are provided. The classification system includes categorization of the gravity-load-resisting system as well as two components of seismic structural framing, vertical and horizontal (diaphragms).

**Reader Note**: This draft of the *Guidelines* contains default data for only two structural systems: moment-resisting steel frames supporting concrete filled metal deck and light wood frame bearing wall construction with plywood sheathed diaphragms. Subsequent versions of these *Guidelines* will contain additional fragility data.

## D.2 Vertical Seismic Framing Systems

The classification system for vertical seismic framing systems is presented in Table D-1. The classification system is composed of the following four levels:

*Level I: Structural Material/System Designation*: defines the system based on the primary construction materials and system type/configuration.

*Level II: Seismic Ductility*: up to four seismic ductility classes are identified for each structural system/material type, which will generally reflect the extent to which inelastic system behavior will be limited to predictable and ductile behavioral modes. In modern design procedures, capacity design and detailing rules are invoked to control these properties for some structural systems. However many modern buildings, particularly in zones of low and moderate seismicity and most archaic buildings have not been designed in this manner. Given the wide variation in deformation capacities among various structural system/material types, the seismic ductility classes are defined primarily in a relative sense, although, some consideration is given to parity between systems and materials. The four seismic ductility classes are as follows:

- **Limited** (that a structure has definable lateral strength but no specific detailing or provisions to ensure ductility)
- **Low** (limited detailing to provide inelastic response capability, comparable to requirements for *ordinary* systems requirements per ASCE 7-05)
- **Moderate** (comparable to requirements for *intermediate* systems per ASCE 7-05)
- **High** (extensive detailing and other provisions to assure ductile behavior, comparable to requirements for *special* systems per ASCE 7-05)

*Level III: Seismic Behavior Characteristics*: distinguishes between behavioral aspects that are not sufficiently reflected in the seismic performance class. For example, the level III designation is used to distinguish between alternative bracing configurations for concentrically braced steel frames (e.g. "X"-braced and "V"-braced patterns) that might significantly impact behavior and are reflected in current building codes.

*Level IV: Numerical Designation*: provides a structured alphanumeric description for each framing system of level III.

The fifth column in Table D-1 provides supplemental guidance to the user for the purpose of selecting appropriate default fragility curves to represent a particular building structure. The guidance is not absolute because adherence to a specific code or guideline does not guarantee a certain level of performance because the performance of code-compliant buildings has never been quantified. The sixth column identifies each Level IV vertical seismic framing system by number.

**Reader Note**: Default fragility specifications will be eventually provided for all listed Level II systems and some Level III systems.

## D.3 Horizontal Seismic Framing Systems (Diaphragms)

Table D-2 presents the classification system for horizontal framing systems. The intent of the classifications presented in Table D-2 are to articulate attributes that distinguish between diaphragm systems insofar as they affect the overall performance of the seismic framing system, including the likelihood of partial or total building collapse, and affect the assessment of structural damage and the associated repair measures and losses. The floor diaphragm systems can be matched with any vertical seismic framing system (Table D-1) and gravity framing system (Table D-3).

The diaphragm classification system focuses on defining the materials and characteristics of the system, particularly with regard to toughness and tendency for damage either at joints within the diaphragm or at attachments to the supporting gravity and lateral force resisting systems. The classification system is not intended to explicitly address diaphragm flexibility and its effect on building response, since the calculation of diaphragm deformations should be addressed by analysis (see Chapter 6). Whereas stiffness and the damageability are related, the two aspects of response are uncoupled herein. The diaphragm classification system is composed of the following three levels:

*Level I: Structural Material/System Designation*: defines the system based on the primary construction materials and diaphragm configuration.

*Level II: Seismic Toughness*: toughness of a diaphragm is classified as low, moderate or high. Toughness is a somewhat subjective classification, which relates to whether the diaphragms and their connections are expected to be a weak link in the seismic framing system. For systems with *high toughness*, diaphragm damage should be minimal. For systems designated as having *low toughness*, the diaphragm performance and damage to its components are expected to have a major effect on the damage and loss. Systems with *moderate toughness* lie between these two extremes.

*Level III: Seismic Behavioral Characteristics*: provides a mechanism to distinguish between behavioral aspects that are not sufficiently reflected in the seismic toughness class.

## D.4 Gravity Framing Systems

The intent of the gravity framing system classifications are to articulate attributes that distinguish between gravity systems insofar as they affect the overall building seismic performance. The gravity framing system classifications can be matched with various lateral and diaphragm systems, although in practice the number of variants is limited.

The key aspects of the gravity framing system classification schemes relate to their ability to undergo deformations associated with seismic framing system and their tendency for damage at the slab/beam-to-column connections or for damage in the vertical support elements (columns or walls). The proposed classification system is composed of the following three levels:

*Level I: Structural Material/System Designation*: defines the system based on the primary construction materials and configuration.

*Level II: Deformation Class:* three classes of deformation capacity are used, low, moderate and high, which relate to the ability of the system to sustain deformations associated with story drift before major damage is reached.

*Level III: Seismic Behavior Characteristics*: identifies the components of the system associated with the designated deformation class for common construction.

## D.5 Default Fragility Data

Figures D-1 and D-2 present default fragility data for steel moment-resisting frames and light wood frame bearing wall construction, respectively.

**Table D-1 Classification System for Vertical Seismic Framing Systems**

| Material/System Designation | Seismic Ductility | Seismic Behavior Characteristics | Numerical Designation | Code Equivalency | No. |
|---|---|---|---|---|---|
| Steel Moment Frame (S1) | Limited | Basic strength design w/o special detailing | S1-MF-1a | 2005 AISC Specification (w/o seismic detailing); pre-1985 UBC (strength design) | 1 |
| | Low | S1-MF-1a with masonry or concrete infill walls | S1-MF-2a | | 2 |
| | | Strength design with limited rotation capacity in beam-column connections. | S1-MF-2b | 2005 AISC – OMF | 3 |
| | | Seismically designed for high ductility but with pre-Northridge connection details | S1-MF-2c | 1985-1994 UBC (pre-Northridge) | 4 |
| | Moderate | Strength design with pre-qualified capacity design provisions for beam-column connections and member slenderness. | S1-MF-3 | 2005 AISC – IMF | 5 |
| | High | Seismically designed and detailed with pre-qualified capacity design provisions. W-shape columns shallower than W18 or box columns | S1-MF-4a | 2005 AISC - SMF | 6 |
| | | S1-MF-4a with W-shape columns deeper than W18 | S1-MF-4b | | 7 |
| Steel Braced Frame (S2) | Limited | Concentric or eccentrically braced system. Basic strength design w/o special detailing | S2-BF-1 | 2005 AISC Specification (w/o seismic detailing) | 8 |
| | Low | Concentrically braced system; strength design with capacity design procedures for connections, compact bracing member compactness. | S2-CBF-2a | 2005 AISC – O-CBF | 9 |
| | | S2-CBF-2a with braces other than HSS members | S2-CBF-2b | | 10 |
| | | S2-CBF-2a with K-brace configuration | S2-CBF-2c | | 11 |
| | Moderate | Concentrically braced system. Seismically designed and detailed with slenderness/compactness limits on braces and capacity design requirements for connections, beams, and columns. | S2-CBF-3a | 2005 AISC – S-CBF | 12 |
| | | S2-CBF-3a with braces other than HSS members | S2-CBF-3b | | 13 |

Table D-1 Classification System for Vertical Seismic Framing Systems

| Material/System Designation | Seismic Ductility | Seismic Behavior Characteristics | Numerical Designation | Code Equivalency | No. |
|---|---|---|---|---|---|
| | | S2-CBF-3a with chevron configuration braces | S2-CBF-3c | | 14 |
| | | S2-CBF-3a with X-configuration braces | S2-CBF-3d | | 15 |
| | | S2-CBF-3a with beam-brace-column connections designed as moment resisting | S2-CBF-3e | | 16 |
| | High | Concentrically braced system with buckling restrained braces and capacity design of connections, beams, and columns. | S2-CBF-4 | 2005 AISC – BRBF | 17 |
| Steel Eccentrically Braced Frame (S3) | Moderate | Eccentrically braced system designed and detailed with capacity design provisions; flexural links remote from column | S3-EBF-3a | 2005 AISC – EBF | 18 |
| | | Eccentrically braced system designed and detailed with capacity design provisions; flexural links adjacent to column | S3-EBF-3b | 2005 AISC – EBF | 19 |
| | High | Eccentrically braced system designed and detailed with capacity design provisions. Shear links adjacent to column | S3-EBF-4a | 2005 AISC – EBF | 20 |
| | | S3-EBF-1a but with shear links away from column | S3-EBF-4b | 2005 AISC – EBF | 21 |
| | | S3-EBF-1a but with shear links away from column with beam-brace-column connections designed as moment resisting | S3-EBF-4c | 2005 AISC – EBF | 22 |
| Steel Light Frame (S4) | Limited | Steel moment frame structure with light gravity load (i.e., a metal building). Basic strength design w/o special detailing | S4-LMF-1 | 2005 AISC Specification (w/o seismic detailing) | 23 |
| | Low | Steel framed structure with light gravity load (i.e., a metal building). Basic strength design with capacity design of connections. | S4-LMF-2 | 2005 AISC – OMF for systems with low gravity load | 24 |

Table D-1 Classification System for Vertical Seismic Framing Systems

| Material/System Designation | Seismic Ductility | Seismic Behavior Characteristics | Numerical Designation | Code Equivalency | No. |
|---|---|---|---|---|---|
| Steel Moment Frame with Concrete Infill Walls (S5) | Low | Basic strength design w/o special detailing. | S5-RCIF-2 | 2005 AISC Specification and ACI-318 Building Code (w/o seismic detailing) | 25 |
| Steel Moment Frame with Unreinforced Masonry Infill Walls (S6) | Limited | Basic strength design w/o special detailing. | S5-URMIF-1 | 2005 AISC Specification and Masonry Code? (w/o seismic detailing) | 26 |
| Concrete Moment Frame (C1) | Limited | Basic strength design w/o special detailing | C1-MF-1 | 2005 ACI-318 (w/o seismic detailing) | 27 |
| | Low | Basic strength design with continuous rebar at beam-column connections. | C1-MF-2 | 2005 ACI-318, OMF | 28 |
| | Moderate | Strength design with continuous rebar at beam-column connections and column confinement rebar. | C1-MF-3 | 2005 ACI-318, IMF | 29 |
| | High | Seismically designed and detailed with capacity design provisions, including precast systems that utilize pre-qualified connections. | C1-MF-4 | 2005 ACI-318, SMF | 30 |
| Concrete Shear Wall (C2) | Limited | Basic strength design with deficient (non-conforming) reinforcement details | C2-W-1a | Pre-1976 UBC (strength design) | 31 |
| | Low | Low gravity load, basic strength design w/o seismic detailing | C2-W-2a | 2005 ACI-318 (non-seismic) | 32 |
| | | High gravity load, basic strength design w/o seismic detailing | C2-W-2b | | 33 |
| | | Precast shear wall (including tilt-up), basic strength design w/o seismic detailing | C2-W-2c | | 34 |
| | Moderate | Low gravity load, squat shear wall, strength design. | C2-W-3a | 2005 ACI-318, including Chapter 21 provisions | 35 |
| | | Low gravity load, slender shear wall, strength design with seismic detailing for shear and boundary zone confinement | C2-W-3b | | 36 |

**Table D-1 Classification System for Vertical Seismic Framing Systems**

| Material/System Designation | Seismic Ductility | Seismic Behavior Characteristics | Numerical Designation | Code Equivalency | No. |
|---|---|---|---|---|---|
| | | Same as C2-W-3b but with coupling beams | C2-W-3c | | 37 |
| | | High gravity load, squat shear wall, strength design. | C2-W-3d | | 38 |
| | | High gravity load, slender shear wall, strength design with seismic detailing for shear and boundary zone confinement | C2-W-3e | | 39 |
| | | Same as C2-W-3e but with coupling beams | C2-W-3f | | 40 |
| | | Precast shear wall, strength design with seismic detailing for wall anchorage, shear and boundary zone confinement | C2-W-3g | | 41 |
| Concrete Frame with Unreinforced Masonry Infill Walls (C3) | Limited | Basic strength design w/o special detailing. | C3-1a | 2005 ACI-318 and Masonry Code (w/o seismic detailing) | 42 |
| Reinforced Masonry Bearing Walls (RM) | Limited | Basic strength design w/o special detailing | RM-1a | | 43 |
| | Moderate | Strength design with seismic detailing | RM-2a | | 44 |
| Unreinforced Masonry Bearing Walls (URM) | Limited | Basic strength design w/o special detailing | URM-1a | Existing URM construction | 45 |
| | | Same as URM-1a, but with tiebacks to diaphragm to help avoid out of plane failures | URM-1b | Retrofitted URM construction in high seismic regions, or new URM in low seismic regions | 46 |
| Light Wood Frame Shear Wall (W1) | Limited | Structural panel sheathing (plywood or OSB) shear walls with interior gypsum wall board, basic strength design | WSW-1a | | 47 |
| | | Structural panel sheathing with stucco exterior and gypsum wall board interior, basic strength design | WSW-1b | | 48 |
| | | Stucco on gypsum wallboard | WSW-1c | | 49 |

Table D-1 Classification System for Vertical Seismic Framing Systems

| Material/System Designation | Seismic Ductility | Seismic Behavior Characteristics | Numerical Designation | Code Equivalency | No. |
|---|---|---|---|---|---|
| | Low | Structural panel sheathing (plywood or OSB) shear walls with interior gypsum wall board, strength design with seismic tie downs and nail/screw details | WSW-2a | | 50 |
| | | Structural panel sheathing (plywood or OSB) shear walls with stucco exterior and gypsum wall board interior, strength design with seismic tie downs and nail/screw details | WSW-2b | | 51 |
| Light Wood Frame, Diagonal Strut Bracing | Limited | Basic strength design | WDS-1 | | 52 |

Table D-2 Classification System for Horizontal Seismic Framing Systems (Diaphragms)

| Material/System Designation | Seismic Toughness | Seismic Behavioral Characteristics | Numerical Designation | No. |
|---|---|---|---|---|
| Concrete Slab on Ribbed Steel Deck | Moderate | Welded wire fabric reinforcement; decking attached to framing by puddle welds | CSD-2a | 1 |
| | High | Connected to supporting framing through welded shear studs, reinforcing bars at critical locations for shear and in-plane flexure in slab | CSD-3a | 2 |
| Ribbed Steel Deck | Low | Connected by intermittent plates and screws to seismic framing system. | SD-1a | 3 |
| | Moderate | Connected by studs or puddle welds to provide diaphragm continuity and connection to the seismic framing system. | SD-2a | 4 |
| Cast-In-Place Concrete Slab | Moderate | Connected to supporting framing by nominal reinforcement. | CS-2a | 5 |
| | High | Connected to supporting framing by reinforcing bars or other anchorage, slab reinforcing bars at critical locations for shear and in-plane flexure in slab | CS-3a | 6 |
| Precast Concrete Slab or Slab/Joists | Low | Connections between slab/joists and at boundary connections designed for strength only. | PC-1a | 7 |
| | Moderate | Detailed for ductile connections between slab/joists and at boundary connections | PC-2a | 8 |
| | | Same as PC-3a, but with cast in place topping slab with provisions made for bond/shear transfer between topping and precast panels with welded wire fabric. | PC-2b | 9 |
| | High | Same as PC-3a, but with cast in place topping slab with provisions made for bond/shear transfer between topping and precast units using rebar. | PC-3a | 10 |
| Wood Sheathing with Structural Panels | Low | Structural sheathing panels with minimum nailing to supporting framing. | SPA-1a | 11 |
| | Moderate | Structural sheathing panels with close nailing to supporting framing to provide continuity and shear transfer. | SPA-2a | 12 |

**Table D-2 Classification System for Horizontal Seismic Framing Systems (Diaphragms)**

| | | | | |
|---|---|---|---|---|
| | | Structural sheathing panels with close nailing to supporting framing to provide continuity and shear transfer with concrete topping slab | SPA-2b | 13 |
| Wood Sheathing with Structural Planks | Low | Diagonal or straight plank sheathing with nailing designed for shear transfer. | SPL-1a | 14 |
| | Moderate | Diagonal or straight plank sheathing with nailing designed for shear transfer and with a concrete topping slab | SPL-2a | 15 |
| Steel Bracing | Low | Diagonal rod or other steel bracing | SR-1a | 16 |
| | Moderate | Diagonal rod or other steel member bracing designed using capacity design principles to prevent connection failure prior to significant yielding either in bars or the seismic lateral system | SR-2a | 17 |

**Table D-3 Classification System for Gravity Framing Systems**

| Material/System Designation | Deformation Class | Seismic Behavior Characteristics | Numerical Designation | No. |
|---|---|---|---|---|
| Composite Metal Deck Supported by Steel Beams, Girders and Columns | Moderate | Concrete over metal deck or metal deck supported by rolled steel beams or open-web joists and HSS section columns | MD-2a | 1 |
| | High | Concrete over metal deck or metal deck supported by rolled steel beams or open- web joists and rolled steel columns | MD-3a | 2 |
| Cast-in-Place Concrete Two-Way, Flat Slab Supported by Concrete Columns | Low | Lightly reinforced and post-tensioned slabs not designed to transfer shear/flexural loads into columns | CIP-1a | 3 |
| | Moderate | Lightly reinforced and post-tensioned slabs not specifically designed to transfer shear/flexural loads into columns, but include structural integrity reinforcement | CIP2-2a | 4 |
| | High | Lightly reinforced and post-tensioned slabs specifically designed to transfer shear/flexural loads into columns, including structural integrity reinforcement | CIP2-3a | 5 |
| Cast-in-Place Concrete One-Way Slab Supported by Concrete Joists, Beams, and Columns | Low | One-way slab with reinforcement in one direction supported by concrete joists, beams, and columns | CIP1-1a | 6 |
| | Moderate | One-way slab with reinforcement in both directions supported by concrete joists, beams, and columns | CIP1-2a | 7 |
| Precast Concrete Planks or Beams Supported by Concrete Beams and Columns | Low | Precast concrete planks without topping slab supported by concrete beams and columns | PC-1a | 8 |
| | Moderate | Precast concrete planks with topping slab integrating the planks and the supporting beams, supported by concrete columns | PC-2a | 9 |
| Wood Joists Supported by Beams, Columns, and Bearing Walls | Moderate | Wood structural flooring supported by wood joists beams, columns, and bearing walls | W-2a | 10 |
| | High | Wood structural flooring supported by wood joists, beams, and columns | W-3a | 11 |

| Fragility, damage measures, and consequence functions for<br>**Post 1994 welded steel moment frame** | | | |
|---|---|---|---|
| BASIC COMPOSITION | | | |
| NORMATIVE QUANTITIES | Specific user inoput of no. of joints (each side of column counts as a joint at interior frame) in each direction | | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | | |
| | DS1 | DS2 | DS3 |
| DESCRIPTION | Local beam flange and web buckling. | DS1 plus lateral-torsoinal distortion of beam in hinge region | Low-cycle fatigue fracture of beam flanges in hinge region. |
| ILLUSTRATION<br>(example photo or drawing) | | | |
| MEDIAN EDP<br>(Interstory drift in specified direction) | 3.00% | 4.0% | 5.0% |
| BETA | 0.35 | 0.35 | 0.35 |
| CORRELATION (%) | Moderate | Moderate | Moderate |
| REPAIR MEASURES | Heat straightening of buckled region. | Heat straightening and/or replacement of portion of beam around hinge region. | Replace large portion of beam in distroted and fractured region. Will likely requried engineered shoring of beam during repairs. |
| CONSEQUENCE FUNCTION<br>Cost per joint (beam to column) | | | |
| Max. cost | $ 8,000 | $ 15,000 | $ 60,000 |
| up to lower quantity | 6 | 6 | 6 |
| Min cost over upper quantity | $ 5,000 | $ 10,000 | $ 45,000 |
| over upper quantity | 12 | 12 | 12 |
| Beta (cost) | 0.3 | 0.3 | 0.4 |

Figure D-1 Fragility specification for post-1994 welded steel moment frame

Fragility, damage measures, and consequence functions for

## Exterior Wall OSB and stucco Type 3a B2011.003a

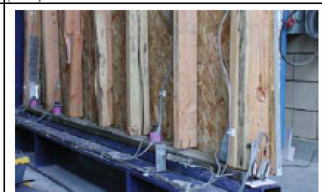
| | | | |
|---|---|---|---|
| BASIC COMPOSITION | 7/8" thick, 3-coat, Stucco with OSB exterior sheathing and interior GWB per CUREE design drawings, Contains windows and doors | | |
| NORMATIVE QUANTITIES | Square feet of stucco wall oriented in a specified *direction* per floor | | |
| DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS | | | |
| | DS1 | DS2 | DS3 |
| DESCRIPTION | Cracking of Stucco and Gypboard | Severe cracking of stucco, cracked gypboard, cracked glass | Wood stud failure, sill plate splitting and failure, stucco/gypboard wall panel failure, glass fallout (DS3) |
| ILLUSTRATION (example photo or drawing) |  |  |  |
| MEDIAN EDP (interstory drift in specified direction) | 0.20% | 1.0% | 4.0% |
| BETA | 0.40 | 0.400 | 0.4 |
| CORRELATION (%) | | | |
| REPAIR MEASURES | Repair stucco crack and repaint, repaint gypboard crack | Grout stucco crack and repaint; tape, mud, sand and paint gypboard cracks, replace cracked glass panel(s) and "cleanup" | Replace entire wall (Wood framing, Stucco, OSB and gypboard); replace glass panels |
| CONSEQUENCE FUNCTION Cost per sq ft of wall | | | |
| Max. cost up to lower quantity | $ 2.37 | $ 9.62 | $ 49.76 |
| | 1,000 | 1,000 | 1,000 |
| Min cost over upper quantity | $ 2.05 | $ 9.00 | 42.44 |
| | 10,000 | 10,000 | 10,000 |
| Beta (cost) | 0.40 | 0.40 | 0.40 |
| TIMEFRAME FOR REPAIRS | Stucco patching can be done in 1-2 days. Painting can follow depending on patching methods, and reccommended drying times | Taping and painting interior walls can take 5-10 days depending on crew size and amount of area to be repaired. | New stucco walls take minimum 35 days to stucco in 3 coats, due to 7 day drying tim ebetween brown and scratch coat, and 28 days between brown and finish. Painting should not take place until a moisture meter indicates sufficient dryness ( usually 20 days depending on weather) Additional time required for reframing, plywood and glass panels. These repairs could take 2 weeks for installation, and 4-6 weeks to receive new glass and windows. |

Figure D-2 Fragility specification for exterior wall with structural sheathing and cement plaster

| Fragility, damage measures, and consequence functions for<br>**Interior Walls GWB on Wood studs C1011.001a** | | | |
|---|---|---|---|
| **BASIC COMPOSITION** | Gypsum Wall Board constructed as follows; ½-in GWB, laid horizontally, #6 1- ¼ in standard drywall screws 16 in OC vertically to 2x4 framing (i.e., not fixed at T&B), 1st level has sillplate anchored to slab w ½ in Abs 32 in OC, bottom plate of upper-level ptns are nailed to diaphragm w 16d @ 16 in OC, Top connections at bottom level: A35 @ 32 in OC | | |
| **NORMATIVE QUANTITIES** | Square feet of wall area in a given level in a specific direction | | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | | |
| | **DS1** | **DS2** | **DS3** |
| **DESCRIPTION** | cracked GWB | Major cracks, buckling of gypsum wallboards at corners of walls | Wood stud failure, sill plate splitting and failure, gypboard wall panel failure |
| **ILLUSTRATION (example photo or drawing)** |  |  |  |
| **MEDIAN EDP (interstory drift in specified direction)** | 0.0020 | 0.010 | 0.030 |
| **BETA** | 0.40 | 0.40 | 0.40 |
| **CORRELATION (%)** | | | |
| **REPAIR MEASURES** | Taping, sanding, and painting.<br>Includes no mechanical repairs<br>Includes no door repairs | Replacing gypboard panels, and then taping, sanding and painting | Replace entire partition walls |
| **CONSEQUENCE FUNCTION Cost per sq ft of curtain wall** | | | |
| Max. cost up to lower quantity | $ 3.33 | $ 5.56 | 13.22 |
| | 100 | 100 | 100 |
| Min cost over upper quantity | $ 2.94 | $ 4.76 | $ 11.22 |
| | 10,000 | 10,000 | 10,000 |
| Beta (cost) | 0.20 | 0.20 | 0.20 |
| **TIMEFRAME FOR REPAIRS** | Minor repairs can be done in a couple days | Demo, new gyp, taping and painting will take at least one week | New framing, possible mechanical inspection and/or repair, new gyp, taping and painting will take at least 2 weeks. |
| | The picture above for DS3 shows a 2x6 wall. Repair costs are for 2x4 per the damage state info given to ARG | The picture above for DS3 shows a 2x6 wall. Repair costs are for 2x4 per the damage state info given to ARG | The picture above for DS3 shows a 2x6 wall. Repair costs are for 2x4 per the damage state info given to ARG |

Figure D-3 Fragility specification for interior wall with wood studs and gypsum board sheathing

# Appendix E

# Default Fragility Assignment Tables, Normative Quantity Logic, and Nonstructural Fragility Specifications

## E.1 Occupancy Default Assignment Tables

Table E-1 and E-2 itemize default descriptions and fragility IDs corresponding to the default inventory quantities assigned to low-rise commercial office buildings and low-rise, multi-family residential buildings. The logic used to determine default quantities is provided in Section E.2, and default fragility values are provided in Section E.3.

**Reader note:** At this stage only 2 tables are provided. In subsequent versions of these guidelines, it is planned that the tables we will be provided for all 8 occupancies identified in Sec 2.2.2 of Part B and for each of the factors that would affect the seismic fragilities. It is also expected that more items will be considered in the list of default inventory items. When completed, it is expected there will be several hundred of these tables that will be included in updated versions of PACT.

**Table E-1 Default Assignment Commercial Office**

| | | |
|---|---|---|
| **General** | Age of Construction = New<br>Number of Stories = Less than or equal to 5<br>Local Seismic Design Practice = High<br>Local Seismic Installation Practice = Average<br>Facility Seismic Safety Management Practice = Low | |
| **Default Inventory Item** | **Assigned Description** | **Fragility ID** |
| Exterior Enclosure – Walls | Highrise curtain wall system with annealed glass – square corners | B2022.001 |
| Interior Walls | Drywall finish, 5/8-in., 2 sides on 3-5/8-in metal studs, screw – no slipjoint – electrical embedded | C1011.001a |
| Interior Ceilings | lightweight acoustical ceiling 4'x2' , alumiinum tee-bar grid – installed, in accordance with ASCE 7- | C3032.001 |

| | | |
|---|---|---|
| | 05 for high seismic – sprinkler drops protruding | |
| Conveying – Elevator | Hydraulic Passenger Elevator | D1011.002 |
| HVAC – Package Units | Heating/Cooling Air Handling Units | D3063.000 |
| Desk top Computers | Unsecured computer and CRT monitor | E2022.011 |
| Servers and Network Equip. | 6 foot high unsecured rack of equipment | E2022.011a |
| Tall Filing Cabinets | 4 high front loading filling cabinets – not anchored. | E2022.026a |
| Tall Book Cases | 7 foot high unanchored book case | E2022.029a |

**Table E-2 Default Assignment Multi-family Residential**

| **General** | Age of Construction = New<br>Construction Type = Wood frame<br>Number of Stories = Less than or equal to 2<br>Local Seismic Design Practice = High<br>Local Seismic Installation Practice = Average<br>Personal Seismic Safety Management Practice = Low | |
|---|---|---|
| **Default Inventory Item** | **Assigned Description** | **Fragility ID** |
| Roof Covering | 10 psf concrete, clay or slate roofing that are individually fastened to the roof sheathing per manufacturers instructions | B3011.002 |
| Ceilings | Gypsum Wall Board on wood joists | C3033.001 |
| Furniture & Accessories | Unsecured china cabinet and refrigerator | E2022.000 |
| Household Entertainment | Unsecured television and stereo units | E2022.004 |
| Desk top Computers | Unsecured computer and CRT monitor | E2022.011 |

## E.2 Occupancy Default Normative Quantity Logic

### *E.2.1 Commercial Office*

Table E-3 below indicates the default normative quantities for nonstructural components related to commercial office occupancies. These quantities apply only to buildings that are 5 stories or less in height. The numbering system as based on the Uniformat II Classification System that is presented in Section E4. The fragility data corresponding to the fragility specification is provided in Section E.3.

**Table E-3 Normative Quantities for Commercial Office Occupancies**

| Fragility Specification | | DIRECTION | NORMATIVE QUANTITIES |
|---|---|---|---|
| Number | Name | | |
| B2022.001 | Highrise curtain-wall systems with annealed glass | Direction 1 | Direction 1 perimeter times story height at each level |
| | | Direction 2 | Direction 2 perimeter times story height at each level |
| C1011.009a | Drywall finish, 5/8-in., 2 sides, on 3-5/8-in metal stud, screws | Direction 1 | 0.025 times the floor area times story height; includes one door and one window per 600sf floor area |
| | | Direction 2 | 0.025 times the floor area times story height; assume one door and one window per 600sf floor area |
| D1011.002 | Hydraulic passenger elevators | Unidirectional | one elevator per 30,000 floor area (except for single floor |
| C3032.001 | Lightweight acoustical ceiling 4'-x-2' aluminum tee-bar grid SEE COMMENT | Unidirectional | Floor area below |
| D3063.000 | Heating/Cooling Air Handling Units, all | Unidirectional | For 70000sf bldg: Hundred ton cooling tower + one 600 cfm fan 20000 cfm air handler, one 15 T multi zone AC. Prorate by sf for larger or smaller by decimal unit (e.g. 100,000sf =100,000/70,000=1.4 units |
| E2022.011 | Desktop computer system unit and CRT monitor | Unidirectional | Floor area divided by 400 |
| E2022.026a | Tall file cabinets | Unidirectional | Floor area divided by 300 |
| E2022.029 | Unanchored bookcases | Unidirectional | Floor area divided by 300 |
| E2022.011a | Computer system servers and network equipment | Unidirectional | One office system per 20,000sf ($50k).Prorate by sf for larger or smaller by decimal unit (e.g. 100,000sf =100,000/20,000=5.0 units |

### *E.2.2 Residential*

Table E-4 indicates the default normative quantities for nonstructural components related to residential occupancies. These quantities apply only to low-rise buildings.

## E.3 Default Nonstructural Fragility Data Tables

Figures E-1 through E-13 present the default fragilities contained within the PACT database.

Table E-4 Normative Quantities for Residential Occupancy

| Fragility Specification | | DIRECTION | NORMATIVE QUANTITIES |
|---|---|---|---|
| Number | Name | | |
| B3011.002 | Concrete, clay, and slate roofing tiles that are individually fastened to the roof sheathing | Unidirectional | 1.15 times the upper floor area |
| E2022.011 | Desktop computer system unit and CRT monitor | Unidirectional | 1.0 set per 600sf |
| E2022.004 | Household entertainment equipment | Unidirectional | 1.0 set per 600sf |
| E2022.000 | Furniture & Accessories, all | | 1.0 units per 1200sf with $20k value |

| Fragility, damage measures, and consequence functions for **Interior Partitions Type 9a C1011.009a** | | | |
|---|---|---|---|
| BASIC COMPOSITION | Full height 5/8 inch gypsumboard screwed on metal studs. No slip track or window panels. | | |
| NORMATIVE QUANTITIES | Square feet of wall area at given level in a specfied direction | | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | | |
| | DS1 | DS2 | DS3 |
| DESCRIPTION | Visible damage and small cracks in gypsum wallboard that can be repaired with taping, patching and painting. No window and door damage | Extensive cracking or crushing in gypsum wallboard and minimal or no damage to metal studs. Re-hang door. | Severe damage to gypsum wallboard and enough damage to metal studs and runners to require replacement |
| ILLUSTRATION (example photo or drawing) |  |  |  |
| MEDIAN EDP (interstory drift in specified direction) | 0.0025 | 0.0060 | 0.014 |
| BETA | 0.70 | 0.50 | 0.40 |
| CORRELATION (%) | low | low | low |
| REPAIR MEASURES | Taping, patching and painting | Replacing the gypsum boards, and then taping, and painting | Remove damaged materials<br>Reframe walls<br>Repair damaged electrical<br>Install new gypsum wallboard<br>Tape, sand and paint |
| | | Re-hang doors | Includes some door repairs |
| | | | Includes some minor mechanical repairs |
| CONSEQUENCE FUNCTION Cost per sq ft of interior partition | | | |
| Max. cost up to lower quantity | $3.33 | $5.56 | $12.92 |
| | 100 | 100 | 100 |
| Min cost over upper quantity | $2.94 | $4.76 | $10.92 |
| | 10,000 | 10,000 | 10,000 |
| Beta (cost) | 0.20 | 0.30 | 0.30 |
| TIMEFRAME FOR REPAIRS | Minor repairs can be done in a couple of days, including repainting patches | Larger repairs take min. 5 days to remove damage, patch and paint | Major reframing of walls and mechanical damage repairs will take a minimum of 10 days, and possible longer depending on crew size, amount of damage, and availability of mechanical parts for repairs |

Figure E-1 Fragility for interior partitions

**Fragility, damage measures, and consequence functions for**

## Exterior Skin-Glass Curtainwall-Type 1 B2022.001

| | |
|---|---|
| **BASIC COMPOSITION** | Full height glass curtain wall made of annealed glass with square glazed corners, thicknesses in the range of 6 mm with height to width ratio of about 1.2 and with 11 mm clearances. |
| **NORMATIVE QUANTITIES** | Square feet of curtain wall oriented in a specified *direction* per floor |

### DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS

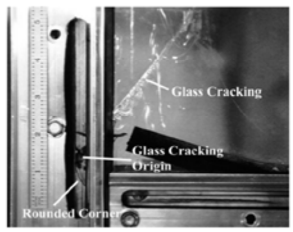
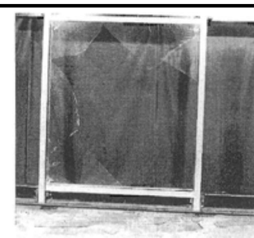
| | DS1 | DS2 |
|---|---|---|
| **DESCRIPTION** | Glass cracking | Glass fallout |
| **ILLUSTRATION (example photo or drawing)** |  |  |
| **MEDIAN EDP (interstory drift in specified direction)** | 0.031 | 0.034 |
| **BETA** | 0.3 | 0.3 |
| **CORRELATION (%)** | low | low |
| **REPAIR MEASURES** | Replace the cracked glass panel(s) | Replace the glass panel(s) where glass has fallen out or cracked<br>Set up scaffold to access façade<br>Repair damaged framework |
| **CONSEQUENCE FUNCTION Cost per sq ft of curtain wall** | | |
| Max. cost | $ 54.00 | $ 59.74 |
| lower quantity | 250 | 250 |
| Min cost | $ 29.00 | $ 32.74 |
| upper quantity | 10,000 | 10,000 |
| Beta (cost) | 0.20 | 0.20 |
| **TIMEFRAME FOR REPAIRS** | Lead time on framework is min. 15 days. Tempured glass is min. 10 days | Lead time on framework is min. 15 days. Tempured glass is min. 10 days |

Figure E-2 Fragility for unitized glazed curtainwall

**Fragility, damage measures, and consequence functions for**

## Ceiling Systems Suspended acoustical tile type1 C3032.001

| | | |
|---|---|---|
| **BASIC COMPOSITION** | Suspended ceiling system with T bars and acoustical ceiling tiles (2x4), 16x16 ft in plan, ceiling tiles not exceeding 2 lbs/sf, installed in accordance with ICBO standard 25-2 or IBC (CISCA zones 3 and 4 and ASTM 635 and 636), no tile retainer clip. | |
| **NORMATIVE QUANTITIES** | Square feet of floor area at a specified level. | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | |
| | **DS1** | **DS2** |
| **DESCRIPTION** | Some tiles displaced and fallen, usually in the perimeter of the room. | Extensive tile fallout and T bar grid failure. Mechanical systems damaged |
| **ILLUSTRATION (example photo or drawing)** |  Photo courtesy National Information Service for Earthquake Engineering, University of California, Berkeley |  Photo courtesy National Information Service for Earthquake Engineering, University of California, Berkeley |
| **MEDIAN EDP (peak floor accelerartion)** | 0.55g | 1.00g |
| **BETA** | 0.40 | 0.40 |
| **CORRELATION (%)** | | |
| **ASSUMPTIONS** | Assumed leakage of sprinklers associated with ceiling collapse, but at a higher level. | Assumed leakage of sprinklers associated with ceiling collapse, but at a higher level. |
| **REPAIR MEASURES** | Turn off all mechanical systems. Remove fallen debris. Remove any residual water. Verify sound condition of ducting. Verify that structure above is sound. Install new ceiling tiles. | Remove all damaged materials<br>Inspect structure for damage<br>Inspect all mechanical systems<br>Reinstall new T-bar<br>Repair damaged mechanical system<br>Install new ceiling tile |
| **CONSEQUENCE FUNCTION** | Replace displaced tiles, and replace damaged tiles | Remove all damaged tile and grid<br>Repair mechanical damage<br>Replace grid, tile and mechanical trims as necessary. |
| **Cost per sq ceiling system** | | |
| Max. cost up to lower quantity | $5.26 | $26.81 |
| | 100 | 100 |
| Min cost over upper quantity | $3.89 | 25 |
| | 10,000 | 10,000 |
| Beta (cost) | 0.40 | 0.40 |
| **TIMEFRAME FOR REPAIRS** | Minimum 1 day for tile replacement only | Minimum 3 days for tile and grid |

Figure E-3 Fragility for suspended acoustic ceiling systems

| Fragility, damage measures, and consequence functions for<br>**Ceiling GWB on wood joists C3033.001** | | | |
|---|---|---|---|
| **BASIC COMPOSITION** | ½ in gypsum wallboard ceilings installed same as walls.<br>#6 1- ¼ in standard drywall screws 11 in OC parallel to joists or rafters | | |
| **NORMATIVE QUANTITIES** | Square feet of ceiling area per floor | | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | | |
| | **DS1** | **DS2** | |
| **DESCRIPTION** | Cracking of ceiling to wall joints | Total ceiling Collapse | |
| **ILLUSTRATION**<br>**(example photo or drawing)** |  | | |
| **MEDIAN EDP**<br>**(peak story drift in specified direction)** | 0.010 | 0.050 | |
| **BETA** | 0.30 | 0.60 | |
| **CORRELATION (%)** | | | |
| **REPAIR MEASURES** | Taping and painting | Total ceiling replacement<br>HVAC, lighting, and sprinkler drops will need repair measures, and inspection. | |
| **CONSEQUENCE FUNCTION**<br>**Cost per sq ft of curtain wall** | | | |
| Max. cost up to lower quantity | $ 2.86 | $ 4.99 | |
| Min cost over upper quantity | $ 2.33 | $ 4.05 | |
| Beta (cost) | 1 | 2 | |
| **TIMEFRAME FOR REPAIRS** | One week to patch and repaint entire ceiling area | One- two weeks for reframing and new sheetrock. Indeterminate time for mechanical repairs. If mechanical repairs are minor they could be done in 5 days ready for inspection and close up. | |

Figure E-4 Fragility for gypsum ceiling on wood joists

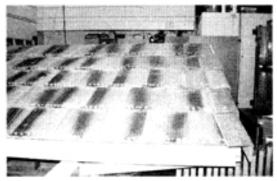
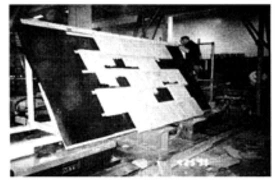
| Fragility, damage measures, and consequence functions for<br>**Exterior Roofing Concrete tile type 2 B3011.002** | | | |
|---|---|---|---|
| **BASIC COMPOSITION** | Concrete tile weighing 10 psf attached per mfg. recommendations, each tile nailed to roof | | |
| **NORMATIVE QUANTITIES** | Square feet of roof area | | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | | |
| | **DS1** | **DS2** | |
| **DESCRIPTION** | Tile dislodged – requires repair | Major portion of tiles dislodged, requires full roof replacement | |
| **ILLUSTRATION (example photo or drawing)** |  Image from Xiao, Y. and H.W Yun, 1998. Dynamic testing of full-scale concrete and clay tile roof models. ASCE Journal of Structural Engineering, 124 (5), May 1998, 482-489 |  Image from Xiao, Y. and H.W Yun, 1998. Dynamic testing of full-scale concrete and clay tile roof models. *ASCE Journal of Structural Engineering*, 124 (5), May 1998, 482-489 | |
| **MEDIAN EDP (peak horizontal roof acceleration)** | 1.50g | 1.90g | |
| **BETA** | 0.40 | 0.40 | |
| **CORRELATION (%)** | | | |
| **REPAIR MEASURES** | Repair or replace dislodged tiles. Assume repair to 20% 0f roof area. | Replace full roof<br>Assumes that plywood underlayment is intact and needs no repairs | |
| **CONSEQUENCE FUNCTION**<br>**Cost per sq ft of repair/replacement** | | | |
| Max. cost | $ 5.06 | $ 10.85 | |
| up to lower quantity | 1,000 | 1,000 | |
| Min cost | $ 4.03 | $ 9.85 | |
| over upper quantity | 10,000 | 10,000 | |
| Beta (cost) | 0.30 | 0.2 | |
| **TIMEFRAME FOR REPAIRS** | Assume miunimum 1 day for tile replacement. Procurement could take 2-6 weeks depending on pattern and availability | Scaffold installation and roof demo could take 10 days.<br>Assume miunimum 5 days for tile replacement. Procurement could take 2-6 weeks depending on pattern and availability | |

Figure E-5 Fragility for concrete tile roofs

| Fragility, damage measures, and consequence functions for<br>**Conveying- Hydraulic elevator D1011.002** | | | |
|---|---|---|---|
| **BASIC COMPOSITION** | Single Cab Hydraulic 3 stop passenger elevator 3500 lb capacity w/ standard finishes | | |
| **NORMATIVE QUANTITIES** | Number of Elevators per Building | | |
| **DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS** | | | |
| | **DS1** | **DS2** | **DS3** |
| **DESCRIPTION** | Elevator does not work (because of various reasons, see related table). Major repairs needed | | TBD |
| **ILLUSTRATION**<br>**(example photo or drawing)** | | | |
| **MEDIAN EDP**<br>**(Peak Ground Acceleration)** | 0.40 g | | |
| **BETA** | 0.30 | | |
| **CORRELATION (%)** | | | |
| **REPAIR MEASURES** | Repair elevator (depends on the type of damage, see related table). | | |
| **CONSEQUENCE FUNCTION**<br>**Cost per unit of conveyance** | | | |
| Max. cost | $ 56,000.00 | | |
| up to lower quantity | 3 | | |
| Min cost | $ 33,600.00 | | |
| over upper quantity | 5 | | |
| Beta (cost) | 0.20 | | |
| **Timeframe for Repairs**<br>**Cost per unit of conveyance** | Assume that parts will take a minimum of 10 days. Some parts could take up to 12 weeks | | |

Figure E-6 Fragility for hydraulic elevators

**Fragility, damage measures, and consequence functions for**

## D3063.000 Roof Mounted Equipment

| | | | |
|---|---|---|---|
| **BASIC COMPOSITION** | Chillers, fans, air handlers on vibration isolators with restraints and anchorage designed and installed per ASCE 7 requirements for normal occupancy. Flexible utility lines provided. | | |
| **NORMATIVE QUANTITIES** | Number of HVAC Units | | |

**DAMAGES STATES, FRAGILITIES, AND CONSEQUENCE FUNCTIONS**

| | DS1 | | |
|---|---|---|---|
| **DESCRIPTION** | Inoperative | | |
| **ILLUSTRATION (example photo or drawing)** |  Photo credit: Mason Industries (2002) | | |
| **MEDIAN EDP (peak horizontal acceleration of roof)** | 1.60g | | |
| **BETA** | 0.50 | | |
| **CORRELATION (%)** | | | |
| **ASSUMPTIONS** | | | |
| **REPAIR MEASURES** | | | |
| **CONSEQUENCE FUNCTION** | | | |
| **Cost per each** | | | |
| Max. cost up to lower quantity | $ 220,000.00 | | |
| | 2 | | |
| Min cost over upper quantity | $ 150,000.00 | | |
| | 8 | | |
| Beta (cost) | 0.6 | | |
| **TIMEFRAME FOR REPAIRS** | | | |

Figure E-7 Fragility for roof-mounted mechanical equipment

## Fragility, damage measures, and consequence functions for

# E2022.000 Miscellaneous housewares and art objects

| | | | |
|---|---|---|---|
| BASIC COMPOSITION | Miscellaneous houswares,'China and art objects, includes China cabinet – assumed unanchored | | |
| NORMATIVE QUANTITIES | Cost of Replacement per 1200 square feet | | |
| **DAMAGES STATES, FRAGILITIES, AND CONSEQUENCE FUNCTIONS** | | | |
| | DS1 | | |
| DESCRIPTION | Fall off shelf, shelf over turns, objects break | | |
| ILLUSTRATION (example photo or drawing) |  Filiatrault et al. (2001) | | |
| MEDIAN EDP (peak horizontal floor acceleration) | 0.20g | | |
| BETA | 0.50 | | |
| CORRELATION (%) | low | | |
| ASSUMPTIONS | | | |
| REPAIR MEASURES | Replace | | |
| CONSEQUENCE FUNCTION | | | |
| Cost per each | | | |
| Max. cost up to lower quantity | 20k | | |
| Min cost over upper quantity | 20k | | |
| Beta (cost) | 0.4 | | |
| TIMEFRAME FOR REPAIRS | | | |

Figure E-8 Fragility for miscellaneous housewares and art objects

**Fragility, damage measures, and consequence functions for**

## E2022.004 Home Entertainment Equipment

| | | | |
|---|---|---|---|
| BASIC COMPOSITION | Speakers or televisions resting on shelves, stereos, etc. - assume unanchored. | | |
| NORMATIVE QUANTITIES | 1 set per 600 square of floor area | | |
| **DAMAGES STATES, FRAGILITIES, AND CONSEQUENCE FUNCTIONS** | | | |
| | DS1 | | |
| DESCRIPTION | Falls off wall, slides off shelf, does not function | | |
| ILLUSTRATION (example photo or drawing) |  Filiatrault et al. (2001) | | |
| MEDIAN EDP (peak horizontal floor acceleration) | 0.20g | | |
| BETA | 0.50 | | |
| CORRELATION (%) | | | |
| ASSUMPTIONS | | | |
| REPAIR MEASURES | Replace | | |
| CONSEQUENCE FUNCTION | | | |
| Cost per each | | | |
| Max. cost up to lower quantity | $2,500.00 | | |
| Min cost over upper quantity | $2,500.00 | | |
| Beta (cost) | 0.4 | | |
| TIMEFRAME FOR REPAIRS | | | |

Figure E-9 Fragility for home entertainment equipment

| Fragility, damage measures, and consequence functions for<br>**E2022.011 Desktop Computers** | | | |
|---|---|---|---|
| **BASIC COMPOSITION** | Monitors and computers - assume unsecured and computer are on desk and not on floors. | | |
| **NORMATIVE QUANTITIES** | Number of sets per floor | | |
| **DAMAGES STATES, FRAGILITIES, AND CONSEQUENCE FUNCTIONS** | | | |
| | **DS1** | | |
| **DESCRIPTION** | Computer falls, becomes inoperative | | |
| **ILLUSTRATION (example photo or drawing)** | <br>Photo courtesy National Information Service for Earthquake Engineering, University of California, Berkeley | | |
| **MEDIAN EDP (peak horizontal floor acceleration)** | 1.20g | | |
| **BETA** | 0.60 | | |
| **CORRELATION (%)** | | | |
| **ASSUMPTIONS** | Computer hard drive parks when the unit falls (as in Mac Powerbooks) or when power is lost (as in most other computers), so system unit is rugged (nondamageable). | | |
| **REPAIR MEASURES** | Repair/replace | | |
| **CONSEQUENCE FUNCTION**<br>**Cost per each** | | | |
| Max. cost | $2,500.00 | | |
| up to lower quantity | 10 | | |
| Min cost over upper quantity | $1,000.00 | | |
| over upper quantity | 100 | | |
| Beta (cost) | 0.4 | | |
| **TIMEFRAME FOR REPAIRS** | | | |

Figure E-10 Fragility for desktop computer equipment

| Fragility, damage measures, and consequence functions for<br>**E2022.011a Servers and network Equipment** | | | |
|---|---|---|---|
| BASIC COMPOSITION | Servers and network devices | | |
| NORMATIVE QUANTITIES | One set pet floor in Office occupancy | | |
| DAMAGES STATES, FRAGILITIES, AND CONSEQUENCE FUNCTIONS | | | |
| | DS1 | | |
| DESCRIPTION | Equipment overturns and is rendered inoperative | | |
| ILLUSTRATION<br>(example photo or drawing) | <br>Image courtesy of National Information Service for Earthquake Engineering, University of California, Berkeley | | |
| MEDIAN EDP<br>(peak horizontal floor acceleration) | 0.8g | | |
| BETA | 0.50 | | |
| CORRELATION (%) | | | |
| ASSUMPTIONS | | | |
| REPAIR MEASURES | | | |
| CONSEQUENCE FUNCTION | | | |
| Cost per each | | | |
| Max. cost up to lower quantity | $50,000.00<br>2 | | |
| Min cost over upper quantity | $40,000.00<br>6 | | |
| Beta (cost) | 0.4 | | |
| TIMEFRAME FOR REPAIRS | | | |

Figure E-11 Fragility for servers and network equipment

Fragility, damage measures, and consequence functions for

## E2022.026a Tall File Cabinet

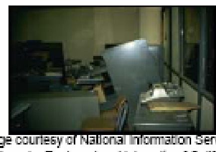
| | | | |
|---|---|---|---|
| BASIC COMPOSITION | 3- or 4-drawer tall, 1-drawer wide metal file cabinet, full of paper, freestanding in both directions. Drawers are not locked, may not all be latched shut. | | |
| NORMATIVE QUANTITIES | | | |
| **DAMAGES STATES, FRAGILITIES, AND CONSEQUENCE FUNCTIONS** | | | |
| | DS1 | | |
| DESCRIPTION | File cabinet overturns. Drawers may open before overturning, causing overturning. Some papers fall out. Cabinet may be damaged. | | |
| ILLUSTRATION (example photo or drawing) |  Image courtesy of National Information Service for Earthquake Engineering, University of California, Berkeley | | |
| MEDIAN EDP (peak horizontal floor acceleration) | 1.0g | | |
| BETA | 0.70 | | |
| CORRELATION (%) | | | |
| ASSUMPTIONS | | | |
| REPAIR MEASURES | | | |
| CONSEQUENCE FUNCTION | | | |
| Cost per each | | | |
| Max. cost up to lower quantity | $250.00 | | |
| | 6 | | |
| Min cost over upper quantity | $100.00 | | |
| | 50 | | |
| Beta (cost) | 0 | | |
| TIMEFRAME FOR REPAIRS | | | |

Figure E-12 Fragility for tall filing cabinets

| Fragility, damage measures, and consequence functions for<br>**E2022.029 Unanchored Bookcase** | | | |
|---|---|---|---|
| **BASIC COMPOSITION** | Includes 6 foot and 8 foot high book cases. Fully loaded, not secured to wall or floor, ½ inch gap between back of bookcase and gypboard-metal stud wall. | | |
| **NORMATIVE QUANTITIES** | Number of Bookcases per floor | | |
| **DAMAGES STATES, FRAGILITIES, AND CONSEQUENCE FUNCTIONS** | | | |
| | **DS1** | | |
| **DESCRIPTION** | Overturns, contents fall out, fragile contents break. | | |
| **ILLUSTRATION (example photo or drawing)** | <br>Image courtesy of National Information Service for Earthquake Engineering, University of California, Berkeley | | |
| **MEDIAN EDP (peak horizontal floor acceleration)** | 0.40g | | |
| **BETA** | 0.30 | | |
| **CORRELATION (%)** | | | |
| **ASSUMPTIONS** | | | |
| **REPAIR MEASURES** | | | |
| **CONSEQUENCE FUNCTION**<br>**Cost per each** | | | |
| Max. cost | $150.00 | | |
| up to lower quantity | 6 | | |
| Min cost | $75.00 | | |
| over upper quantity | 50 | | |
| Beta (cost) | 0.20 | | |
| **TIMEFRAME FOR REPAIRS** | | | |

Figure E-13 Fragility for unanchored bookcases

## E.4 UNIFORMAT II Classification System

Table E-5 presents the top three levels of the UNIFORMAT classification system used to categorize nonstructural components and fragilities in these guidelines.

**Table E-5 ASTM Uniformat Classification for Building Elements**

| Level 1<br>Major Group Elements | Level 2<br>Group Elements | Level 3<br>Individual Elements |
|---|---|---|
| A SUBSTRUCTURE | A10 Foundations | A1010 Standard Foundations<br>A1020 Special Foundations<br>A1030 Slab on Grade |
| | A20 Basement Construction | A2010 Basement Excavation<br>A2020 Basement Walls |
| B SHELL | B10 Superstructure | B1010 Floor Construction<br>B1020 Roof Construction |
| | B20 Exterior Enclosure | B2010 Exterior Walls<br>B2020 Exterior Windows<br>B2030 Exterior Doors |
| | B30 Roofing | B3010 Roof Coverings<br>B3020 Roof Openings |
| C INTERIORS | C10 Interior Construction | C1010 Partitions<br>C1020 Interior Doors<br>C1030 Fittings |
| | C20 Stairs | C2010 Stair Construction<br>C2020 Stair Finishes |
| | C30 Interior Finishes | C3010 Wall Finishes<br>C3020 Floor Finishes<br>C3030 Ceiling Finishes |
| D SERVICES | D10 Conveying | D1010 Elevators & Lifts<br>D1020 Escalators & Moving Walks<br>D1090 Other Conveying Systems |
| | D20 Plumbing | D2010 Plumbing Fixtures<br>D2020 Domestic Water Distribution<br>D2030 Sanitary Waste<br>D2040 Rain Water Drainage<br>D2090 Other Plumbing Systems |
| | D30 HVAC | D3010 Energy Supply<br>D3020 Heat Generating Systems<br>D3030 Cooling Generating Systems<br>D3040 Distribution Systems<br>D3050 Terminal & Package Units<br>D3060 Controls & Instrumentation<br>D3070 Systems Testing & Balancing<br>D3090 Other HVAC Systems & Equipment |
| | D40 Fire Protection | D4010 Sprinklers<br>D4020 Standpipes<br>D4030 Fire Protection Specialties<br>D4090 Other Fire Protection Systems |
| | D50 Electrical | D5010 Electrical Service & Distribution<br>D5020 Lighting and Branch Wiring<br>D5030 Communications & Security<br>D5090 Other Electrical Systems |
| E EQUIPMENT & FURNISHINGS | E10 Equipment | E1010 Commercial Equipment<br>E1020 Institutional Equipment<br>E1030 Vehicular Equipment<br>E1090 Other Equipment |
| | E20 Furnishings | E2010 Fixed Furnishings<br>E2020 Movable Furnishings |
| F SPECIAL CONSTRUCTION & DEMOLITION | F10 Special Construction | F1010 Special Structures<br>F1020 Integrated Construction<br>F1030 Special Construction Systems<br>F1040 Special Facilities<br>F1050 Special Controls and Instrumentation |
| | F20 Selective Building Demolition | F2010 Building Elements Demolition<br>F2020 Hazardous Components Abatement |

Table E-6 present the complete list of UNIFORMAT coded building components. Not all of these components are contained within the default databases. However, this UNIFORMAT coding system should be used to identify components for which user-defined fragility or loss data are input as is illustrated in Section E.3.

**Table E-6 Uniformat II Classification System**

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| | | | |
| Wall Foundations | 000 | A1011.000 | Wall Foundations, all |
| Column Foundations & Pile Caps | 000 | A1012.000 | Column Foundations & Pile Caps, all |
| Perimeter Drainage & Insulation | 000 | A1013.000 | Perimeter Drainage & Insulation, all |
| Pile Foundations | 000 | A1021.000 | Pile Foundations, all |
| Grade Beams | 000 | A1022.000 | Grade Beams, all |
| Caissons | 000 | A1023.000 | Caissons, all |
| Underprinting | 000 | A1024.000 | Underprinting, all |
| Dewatering | 000 | A1025.000 | Dewatering, all |
| Raft Foundations | 000 | A1026.000 | Raft Foundations, all |
| Pressure Injected Grouting | 000 | A1027.000 | Pressure Injected Grouting, all |
| Other Special Conditions | 000 | A1029.000 | Other Special Conditions, all |
| Standard Slab on Grade | 000 | A1031.000 | Standard Slab on Grade, all |
| Structural Slab on Grade | 000 | A1032.000 | Structural Slab on Grade, all |
| Inclined Slab on Grade | 000 | A1033.000 | Inclined Slab on Grade, all |
| Trenches, Pits & Bases | 000 | A1034.000 | Trenches, Pits & Bases, all |
| Under-Slab Drainage & Insulation | 000 | A1035.000 | Under-Slab Drainage & Insulation, all |
| Excavation for Basements | 000 | A2011.000 | Excavation for Basements, all |
| Structure Back Fill & Compaction | 000 | A2012.000 | Structure Back Fill & Compaction, all |
| Shoring | 000 | A2013.000 | Shoring, all |
| Basement Wall Construction | 000 | A2021.000 | Basement Wall Construction, all |
| Moisture Protection | 000 | A2022.000 | Moisture Protection, all |
| Basement Wall Insulation | 000 | A2023.000 | Basement Wall Insulation, all |
| Interior Skin | 000 | A2024.000 | Interior Skin, all |
| Suspended Basement Floors Construction | 000 | B1011.000 | Suspended Basement Floors Construction, all |
| Upper Floors Construction | 000 | B1012.000 | Upper Floors Construction, all |
| Balcony Floors Construction | 000 | B1013.000 | Balcony Floors Construction, all |
| Ramps | 000 | B1014.000 | Ramps, all |
| Exterior Stairs and Fire Escapes | 000 | B1015.000 | Exterior Stairs and Fire Escapes, all |
| Floor Raceway Systems | 000 | B1016.000 | Floor Raceway Systems, all |
| Other Floor Construction | 000 | B1019.000 | Other Floor Construction, all |
| Flat Roof Construction | 000 | B1021.000 | Flat Roof Construction, all |
| Pitched Roof Construction | 000 | B1022.000 | Pitched Roof Construction, all |
| Canopies | 000 | B1023.000 | Canopies, all |
| Other Roof Systems | 000 | B1029.000 | Other Roof Systems, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Steel Columns | 000 | B1031.000 | Steel Columns, all |
| Steel Beams | 000 | B1032.000 | Steel Beams, all |
| Steel Braces | 000 | B1033.000 | Steel Braces, all |
| Steel Shearwalls | 000 | B1034.000 | Steel Shearwalls, all |
| Steel Connections | 000 | B1035.000 | Steel Connections, all |
| Steel Connections | 001 | B1035.001 | Pre-Northridge welded-steel moment-frame connection |
| Reinforced Concrete or Composite Columns | 000 | B1041.000 | Reinforced Concrete or Composite Columns, all |
| Reinforced Concrete or Composite Columns | 001 | B1041.001 | Nonductile cast-in-place reinforced-concrete column Ag in [250,500) in^2, L in [100,150) in |
| Reinforced Concrete or Composite Beams | 000 | B1042.000 | Reinforced Concrete or Composite Beams, all |
| Reinforced Concrete or Composite Beams | 001 | B1042.001 | Nonductile cast-in-place reinforced-concrete beam Ag in [100, 250) in^2, L in [150, 300) in |
| Reinforced Concrete or Composite Braces | 000 | B1043.000 | Reinforced Concrete or Composite Braces, all |
| Reinforced Concrete or Composite Shearwalls | 000 | B1044.000 | Reinforced Concrete or Composite Shearwalls, all |
| Reinforced Concrete Slab-Column Connections | 000 | B1045.000 | Reinforced Concrete Slab-Column Connections (all) |
| Reinforced Concrete Slab-Column Connections | 001 | B1045.001 | Reinforced concrete slab-column connection pre-1976 RC flat plate |
| Reinforced Concrete Slab-Column Connections | 002 | B1045.002 | Reinforced concrete slab-column connection post-1976 RC flat plate |
| Reinforced Concrete Slab-Column Connections | 003 | B1045.003 | Reinforced concrete slab-column connection post-1976 PT flat plate |
| Reinforced Concrete Beam-Column Connections | 000 | B1046.000 | Reinforced concrete beam-column joint (all) |
| Reinforced Concrete Beam-Column Connections | 001 | B1046.001 | Reinforced concrete beam-column joint pre-1967 |
| Reinforced Concrete Beam-Column Connections | 002 | B1046.002 | Reinforced concrete beam-column joint post-1967 not compying with ACI 318 2002 |
| Reinforced Concrete Beam-Column Connections | 003 | B1046.003 | Reinforced concrete beam-column joint complying with ACI 318 2002 |
| Exterior Wall Construction | 000 | B2011.000 | Exterior Wall Construction, all |
| Exterior Wall Construction | 001 | B2011.001 | Exterior shearwall, 3/8 C-D ply, 2x4, 16" OC, 7/8" stucco ext, no int finish |
| Exterior Wall Construction | 002 | B2011.002 | Exterior shearwall, 15/32 C-D ply, 2x4, 16" OC, 7/8" stucco ext, no int finish |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Exterior Wall Construction | 003 | B2011.003 | Exterior shearwall, 7/16 OSB, 2x4, 16" OC, 7/8" stucco ext, no int finish |
| Exterior Wall Construction | 003 | B2011.003a | Exterior shearwall, 7/16 OSB, 2x4, 16" OC, 7/8" stucco ext, GWB interior side |
| Exterior Wall Construction | 004 | B2011.004 | Exterior wall, no structural sheathing, 2x4, 16" OC, 7/8" stucco ext, no int finish |
| Exterior Wall Construction | 005 | B2011.005 | Stucco finish, 7/8", on 3-5/8″ mtl stud, 16"OC, typ quality |
| Parapets | 000 | B2012.000 | Parapets, all |
| Exterior Louvers, Screens, and Fencing | 000 | B2013.000 | Exterior Louvers, Screens, and Fencing, all |
| Exterior Louvers, Screens, and Fencing | 001 | B2013.001 | Non-engineered concrete block freestanding walls |
| Exterior Louvers, Screens, and Fencing | 002 | B2013.002 | Engineered concrete block freestanding walls |
| Exterior Sun Control Devices | 000 | B2014.000 | Exterior Sun Control Devices, all |
| Balcony Walls & Handrails | 000 | B2015.000 | Balcony Walls & Handrails, all |
| Exterior Soffits | 000 | B2016.000 | Exterior Soffits, all |
| Windows | 000 | B2021.000 | Windows, all |
| Windows | 001 | B2021.001 | Window, aluminum frame, sliding, standard glass, 1-25 sf pane |
| Windows | 002 | B2021.002 | Window, aluminum frame, fixed, standard glass, 80"x80" pane |
| Windows | 003 | B2021.003 | Windows, wood, double hung, standard glass, 3'-1.5"x4' |
| Windows | 004 | B2021.004 | Window, aluminum frame, sliding, heavy sheet glass, 4'-0x2'-6"x3/16" |
| Curtain Walls | 000 | B2022.000 | Curtain Walls, all |
| Curtain Walls | 001 | B2022.001 | Highrise curtain-wall systems with annealed glass |
| Curtain Walls | 002 | B2022.002 | Highrise curtain-wall systems with tempered, wired, or laminated glass, or glass with shatter-resistant film |
| Storefronts | 000 | B2023.000 | Storefronts, all |
| Storefronts | 001 | B2023.001 | Lowrise storefront windows with annealed glass |
| Storefronts | 002 | B2023.002 | Lowrise storefront windows with tempered, wired, or laminated glass, or glass with shatter-resistant film |
| Glazed Doors & Entrances | 000 | B2031.000 | Glazed Doors & Entrances, all |
| Glazed Doors & Entrances | 001 | B2031.001 | Doors, sliding, patio, aluminum, std, 6'-0"x6'-8", wood frame, insulated glass |
| Solid Exterior Doors | 000 | B2032.000 | Solid Exterior Doors, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Revolving Doors | 000 | B2033.000 | Revolving Doors, all |
| Overhead Doors | 000 | B2034.000 | Overhead Doors, all |
| Other Doors & Entrances | 000 | B2039.000 | Other Doors & Entrances, all |
| Roof Finishes | 000 | B3011.000 | Roof Finishes, all |
| Roof Finishes | 001 | B3011.001 | Concrete, clay, and slate roofing tiles that are not individually fastened to the roof sheathing |
| Roof Finishes | 002 | B3011.002 | Concrete, clay, and slate roofing tiles that are individually fastened to the roof sheathing |
| Roof Finishes | 003 | B3011.003 | Lightweight roofing |
| Traffic Toppings & Paving Membranes | 000 | B3012.000 | Traffic Toppings & Paving Membranes, all |
| Roof Insulation & Fill | 000 | B3013.000 | Roof Insulation & Fill, all |
| Flashings & Trim | 000 | B3014.000 | Flashings & Trim, all |
| Roof Eaves and Soffits | 000 | B3015.000 | Roof Eaves and Soffits, all |
| Gutters and Downspouts | 000 | B3016.000 | Gutters and Downspouts, all |
| Glazed Roof Openings | 000 | B3021.000 | Glazed Roof Openings, all |
| Roof Hatches | 000 | B3022.000 | Roof Hatches, all |
| Gravity Roof Ventilators | 000 | B3023.000 | Gravity Roof Ventilators, all |
| Wall Finishes to Exterior | 000 | B4041.000 | Wall Finishes to Exterior, all |
| Wall Finishes to Exterior | 001 | B4041.001 | Paint on exterior stucco or concrete |
| Wall Finishes to Exterior | 002 | B4041.002 | Brick masonry veneer without ties to the supporting wall |
| Wall Finishes to Exterior | 003 | B4041.003 | Brick masonry veneer that is tied to the supporting wall |
| Wall Finishes to Exterior | 004 | B4041.004 | Stone veneer attached with mortar spots |
| Wall Finishes to Exterior | 005 | B4041.005 | Stone veneer tied to the supporting wall |
| Fixed Partitions | 000 | C1011.000 | Fixed Partitions, all |
| Fixed Partitions | 001 | C1011.001 | GWB partition, no structural sheathing, 1/2" GWB one side, 2x4, 16" OC |
| Fixed Partitions | 001 | C1011.001a | GWB partition, no structural sheathing, 1/2" GWB two sides, 2x4, 16" OC |
| Fixed Partitions | 002 | C1011.002 | GWB finish, 1/2", one side, on 2x4, 16"OC |
| Fixed Partitions | 003 | C1011.003 | Interior shearwall, 3/8 C-D ply, 2x4, 16" OC, 1/2" GWB finish one side |
| Fixed Partitions | 004 | C1011.004 | Interior shearwall, 15/32 C-D ply, 2x4, 16" OC, 1/2" GWB finish one side |
| Fixed Partitions | 005 | C1011.005 | Interior sheathing, 3/8 C-D ply, 1/2" GWB finish one side, on 2x4 16" OC |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Fixed Partitions | 006 | C1011.006 | Interior sheathing, 15/32 C-D ply, 1/2" GWB finish one side, on 2x4, 16" OC |
| Fixed Partitions | 007 | C1011.007 | Interior shearwall, 7/16 OSB, 2x4, 16" OC, 1/2" GWB finish one side |
| Fixed Partitions | 008 | C1011.008 | Interior sheathing, 7/16 OSB, 1/2" GWB finish one side, on 2x4 16" OC |
| Fixed Partitions | 009 | C1011.009 | Drywall finish, 5/8-in., 1 side, on 3-5/8-in metal stud, screws |
| Fixed Partitions | 009 | C1011.009a | Drywall finish, 5/8-in., 2 sides, on 3-5/8-in metal stud, screws |
| Fixed Partitions | 010 | C1011.010 | Drywall partition, 5/8-in., 1 side, with 3-5/8-in metal stud, screws |
| Demountable Partitions | 000 | C1012.000 | Demountable Partitions, all |
| Retractable Partitions | 000 | C1013.000 | Retractable Partitions, all |
| Site Built Toilet Partitions | 000 | C1014.000 | Site Built Toilet Partitions, all |
| Site Built Compartments Cubicles | 000 | C1015.000 | Site Built Compartments Cubicles, all |
| Interior Balustrades and Screens | 000 | C1016.000 | Interior Balustrades and Screens, all |
| Interior Windows & Storefronts | 000 | C1017.000 | Interior Windows & Storefronts, all |
| Interior Doors | 000 | C1021.000 | Interior Doors, all |
| Interior Door Frames | 000 | C1022.000 | Interior Door Frames, all |
| Interior Door Hardware | 000 | C1023.000 | Interior Door Hardware, all |
| Interior Door Wall Opening Elements | 000 | C1024.000 | Interior Door Wall Opening Elements, all |
| Interior Door Sidelights & Transoms | 000 | C1025.000 | Interior Door Sidelights & Transoms, all |
| Interior Hatches & Access Doors | 000 | C1026.000 | Interior Hatches & Access Doors, all |
| Door Painting & Decoration | 000 | C1027.000 | Door Painting & Decoration, all |
| Fabricated Toilet Partitions | 000 | C1031.000 | Fabricated Toilet Partitions, all |
| Fabricated Compartments & Cubicles | 000 | C1032.000 | Fabricated Compartments & Cubicles, all |
| Storage Shelving and Lockers | 000 | C1033.000 | Storage Shelving and Lockers, all |
| Ornamental Metals and Handrails | 000 | C1034.000 | Ornamental Metals and Handrails, all |
| Identifying Devices | 000 | C1035.000 | Identifying Devices, all |
| Closet Specialties | 000 | C1036.000 | Closet Specialties, all |
| General Fittings & Misc. Metals | 000 | C1037.000 | General Fittings & Misc. Metals, all |
| Regular Stairs | 000 | C2011.000 | Regular Stairs, all |
| Curved Stairs | 000 | C2012.000 | Curved Stairs, all |
| Spiral Stairs | 000 | C2013.000 | Spiral Stairs, all |
| Stair Handrails and Balustrades | 000 | C2014.000 | Stair Handrails and Balustrades, all |
| Stair, Tread, and Landing Finishes | 000 | C2021.000 | Stair, Tread, and Landing Finishes, all |
| Stair Soffit Finishes | 000 | C2022.000 | Stair Soffit Finishes, all |
| Stair Handrail & Balustrade Finishes | 000 | C2023.000 | Stair Handrail & Balustrade Finishes, all |
| Wall Finishes to Inside Exterior | 000 | C3011.000 | Wall Finishes to Inside Exterior, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Wall Finishes to Inside Exterior | 001 | C3011.001 | Paint on interior of exterior walls |
| Wall Finishes to Inside Exterior | 002 | C3011.002 | Ceramic tile veneer over interior of exterior walls |
| Wall Finishes to Inside Exterior | 003 | C3011.003 | Wallpaper on interior of exterior walls |
| Wall Finishes to Inside Exterior | 004 | C3011.004 | Vinyl wall coverings on interior of exterior walls |
| Wall Finishes to Interior Walls | 000 | C3012.000 | Wall Finishes to Interior Walls, all |
| Wall Finishes to Interior Walls | 001 | C3012.001 | Paint on interior concrete, drywall or plastser |
| Wall Finishes to Interior Walls | 002 | C3012.002 | Paint on interior partitions |
| Wall Finishes to Interior Walls | 003 | C3012.003 | Ceramic tile veneer over interior partitions |
| Wall Finishes to Interior Walls | 004 | C3012.004 | Wallpaper on interior partitions |
| Wall Finishes to Interior Walls | 005 | C3012.005 | Vinyl wall coverings on interior partitions |
| Column Finishes | 000 | C3013.000 | Column Finishes, all |
| Floor Toppings | 000 | C3021.000 | Floor Toppings, all |
| Traffic Membranes | 000 | C3022.000 | Traffic Membranes, all |
| Hardeners and Sealers | 000 | C3023.000 | Hardeners and Sealers, all |
| Flooring | 000 | C3024.000 | Flooring, all |
| Carpeting | 000 | C3025.000 | Carpeting, all |
| Bases, Curbs and Trim | 000 | C3026.000 | Bases, Curbs and Trim, all |
| Access Pedastal Flooring | 000 | C3027.000 | Access Pedastal Flooring, all |
| Ceiling Finishes | 000 | C3031.000 | Ceiling Finishes, all |
| Suspended Ceilings | 000 | C3032.000 | Suspended Ceilings, all |
| Suspended Ceilings | 001 | C3032.001 | Lightweight acoustical ceiling 4'-x-2' aluminum tee-bar grid SEE COMMENT |
| Suspended Ceilings | 002 | C3032.002 | Suspended ceilings lacking either diagonal braces, compression struts or both |
| Suspended Ceilings | 003 | C3032.003 | Suspended ceilings with braces and compression struts |
| Other Ceilings | 000 | C3033.000 | Other Ceilings, all |
| Other Ceilings | 1 | C3033.001 | GWB on wood joists |
| Passenger Elevators | 000 | D1011.000 | Passenger Elevators, all |
| Passenger Elevators | 001 | D1011.001 | Traction passenger elevators |
| Passenger Elevators | 002 | D1011.002 | Hydraulic passenger elevators |
| Passenger Elevators | 003 | D1011.003 | Traction passenger elevators meeting seismic reqts UBC 1994 |
| Passenger Elevators | 004 | D1011.004 | Traction passenger elevators exceeding UBC 1994 |
| Freight Elevators | 000 | D1012.000 | Freight Elevators, all |
| Freight Elevators | 001 | D1012.001 | Traction freight elevators |
| Freight Elevators | 002 | D1012.002 | Hydraulic freight elevators |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Freight Elevators | 003 | D1012.003 | Traction freight elevators meeting seismic reqts UBC 1994 |
| Freight Elevators | 004 | D1012.004 | Traction freight elevators exceeding seismic reqts UBC 1994 |
| Lifts | 000 | D1013.000 | Lifts, all |
| Escalators | 000 | D1021.000 | Escalators, all |
| Moving Walks | 000 | D1022.000 | Moving Walks, all |
| Dumbwaiters | 000 | D1091.000 | Dumbwaiters, all |
| Pneumatic Tube Systems | 000 | D1092.000 | Pneumatic Tube Systems, all |
| Hoists & Cranes | 000 | D1093.000 | Hoists & Cranes, all |
| Conveyors | 000 | D1094.000 | Conveyors, all |
| Chutes | 000 | D1095.000 | Chutes, all |
| Turntables | 000 | D1096.000 | Turntables, all |
| Baggage Handling & Loading Systems | 000 | D1097.000 | Baggage Handling & Loading Systems, all |
| Transportation Systems | 000 | D1098.000 | Transportation Systems, all |
| Water Closets | 000 | D2011.000 | Water Closets, all |
| Urinals | 000 | D2012.000 | Urinals, all |
| Lavatories | 000 | D2013.000 | Lavatories, all |
| Sinks | 000 | D2014.000 | Sinks, all |
| Bathtubs | 000 | D2015.000 | Bathtubs, all |
| Wash Fountains | 000 | D2016.000 | Wash Fountains, all |
| Showers | 000 | D2017.000 | Showers, all |
| Drinking Fountains and Coolers | 000 | D2018.000 | Drinking Fountains and Coolers, all |
| Bidets and Other Plumbing Fixtures | 000 | D2019.000 | Bidets and Other Plumbing Fixtures, all |
| Cold Water Service | 000 | D2021.000 | Cold Water Service, all |
| Cold Water Service | 001 | D2021.001 | Pumps, Cold Water Service, Motor-Driven (horizontal, 10 – 200 horsepower) |
| Hot Water Service | 000 | D2022.000 | Hot Water Service, all |
| Hot Water Service | 001 | D2022.001 | Pumps, Hot Water Service, Motor-Driven (horizontal, 10 – 200 horsepower) |
| Domestic Water Supply Equipment | 000 | D2023.000 | Domestic Water Supply Equipment, all |
| Domestic Water Supply Equipment | 001 | D2023.001 | Pumps, Domestic Water Supply, Motor-Driven (horizontal, 10 – 200 horsepower) |
| Waste Piping | 000 | D2031.000 | Waste Piping, all |
| Vent Piping | 000 | D2032.000 | Vent Piping, all |
| Floor Drains | 000 | D2033.000 | Floor Drains, all |
| Sanitary Waste Equipment | 000 | D2034.000 | Sanitary Waste Equipment, all |
| Pipe Insulation | 000 | D2035.000 | Pipe Insulation, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Pipe & Fittings | 000 | D2041.000 | Pipe & Fittings, all |
| Roof Drains | 000 | D2042.000 | Roof Drains, all |
| Rainwater Drainage Equipment | 000 | D2043.000 | Rainwater Drainage Equipment, all |
| Pipe Insulation | 000 | D2044.000 | Pipe Insulation, all |
| Gas Distribution | 000 | D2091.000 | Gas Distribution, all |
| Acid Waste Systems | 000 | D2092.000 | Acid Waste Systems, all |
| Interceptors | 000 | D2093.000 | Interceptors, all |
| Pool Piping and Equipment | 000 | D2094.000 | Pool Piping and Equipment, all |
| Decorative Fountain Piping Devices | 000 | D2095.000 | Decorative Fountain Piping Devices, all |
| Other Piping Systems | 000 | D2099.000 | Other Piping Systems, all |
| Oil Supply System | 000 | D3011.000 | Oil Supply System, all |
| Gas Supply System | 000 | D3012.000 | Gas Supply System, all |
| Coal Supply System | 000 | D3013.000 | Coal Supply System, all |
| Steam Supply System | 000 | D3014.000 | Steam Supply System, all |
| Hot Water Supply System | 000 | D3015.000 | Hot Water Supply System, all |
| Hot Water Supply System | 001 | D3015.001 | Electric water heater, residential, 100F rise, 50 gal, 9 kW 37 GPH |
| Solar Energy System | 000 | D3016.000 | Solar Energy System, all |
| Wind Energy System | 000 | D3017.000 | Wind Energy System, all |
| Boilers | 000 | D3021.000 | Boilers, all |
| Boiler Room Piping & Specialties | 000 | D3022.000 | Boiler Room Piping & Specialties, all |
| Auxiliary Equipment | 000 | D3023.000 | Auxiliary Equipment, all |
| Insulation | 000 | D3024.000 | Insulation, all |
| Chilled Water Systems | 000 | D3031.000 | Chilled Water Systems, all |
| Chilled Water Systems | 001 | D3031.001 | Packaged Circulating Water Chillers |
| Direct Expansion Systems | 000 | D3032.000 | Direct Expansion Systems, all |
| Air Distribution Systems | 000 | D3041.000 | Air Distribution Systems, all |
| Air Distribution Systems | 001 | D3041.001 | Fan |
| Air Distribution Systems | 002 | D3041.002 | HVAC ductwork rod hung |
| Air Distribution Systems | 003 | D3041.003 | HVAC ductwork with sway braces |
| Exhaust Ventilation Systems | 000 | D3042.000 | Exhaust Ventilation Systems, all |
| Exhaust Ventilation Systems | 001 | D3042.001 | Unreinforced brick chimneys |
| Exhaust Ventilation Systems | 002 | D3042.002 | Reinforced masonry and precast reinforced concrete chimneys |
| Exhaust Ventilation Systems | 003 | D3042.003 | Lightweight (insulated metal-lined) flues in woodframe chimneys |
| Steam Distribution Systems | 000 | D3043.000 | Steam Distribution Systems, all |
| Hot Water Distribution | 000 | D3044.000 | Hot Water Distribution, all |
| Chilled Water Distribution | 000 | D3045.000 | Chilled Water Distribution, all |
| Change-over Distribution System | 000 | D3046.000 | Change-over Distribution System, all |
| Glycol Distribution Systems | 000 | D3047.000 | Glycol Distribution Systems, all |
| Terminal Self-Contained Units | 000 | D3051.000 | Terminal Self-Contained Units, all |
| Package Units | 000 | D3052.000 | Package Units, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Heating Generating Systems | 000 | D3061.000 | Heating Generating Systems, all |
| Cooling Generating Systems | 000 | D3062.000 | Cooling Generating Systems, all |
| Heating/Cooling Air Handling Units | 000 | D3063.000 | Heating/Cooling Air Handling Units, all |
| Exhaust & Ventilating Systems | 000 | D3064.000 | Exhaust & Ventilating Systems, all |
| Hoods and Exhaust Systems | 000 | D3065.000 | Hoods and Exhaust Systems, all |
| Terminal Devices | 000 | D3066.000 | Terminal Devices, all |
| Energy Monitoring & Control | 000 | D3067.000 | Energy Monitoring & Control, all |
| Building Automation Systems | 000 | D3068.000 | Building Automation Systems, all |
| Other Controls & Instrumentation | 000 | D3069.000 | Other Controls & Instrumentation, all |
| Piping System Testing & Balancing | 000 | D3071.000 | Piping System Testing & Balancing, all |
| Air Systems Testing & Balancing | 000 | D3072.000 | Air Systems Testing & Balancing, all |
| HVAC Commissioning | 000 | D3073.000 | HVAC Commissioning, all |
| Other Systems Testing and Balancing | 000 | D3079.000 | Other Systems Testing and Balancing, all |
| Special Cooling Systems & Devices | 000 | D3091.000 | Special Cooling Systems & Devices, all |
| Special Humidity Control | 000 | D3092.000 | Special Humidity Control, all |
| Dust & Fume Collectors | 000 | D3093.000 | Dust & Fume Collectors, all |
| Air Curtains | 000 | D3094.000 | Air Curtains, all |
| Air Purifiers | 000 | D3095.000 | Air Purifiers, all |
| Paint Spray Booth Ventilation | 000 | D3096.000 | Paint Spray Booth Ventilation, all |
| General Construction Items (HVAC) | 000 | D3097.000 | General Construction Items (HVAC), all |
| Sprinkler Water Supply | 000 | D4011.000 | Sprinkler Water Supply, all |
| Sprinkler Water Supply | 001 | D4011.001 | Unbraced automatic sprinklers |
| Sprinkler Water Supply | 002 | D4011.002 | Braced automatic sprinklers |
| Sprinkler Water Supply | 003 | D4011.003 | Automatic sprinklers that are noncompliant with NFPA-13 (1991) |
| Sprinkler Water Supply | 004 | D4011.004 | Automatic sprinklers that are compliant with NFPA-13 (1991) |
| Sprinkler Water Supply | 005 | D4011.005 | Pre-action or deluge sprinklers |
| Sprinkler Water Supply | 006 | D4011.006 | Halon or other non-water-based fire-suppression systems |
| Sprinkler Pumping Equipment | 000 | D4012.000 | Sprinkler Pumping Equipment, all |
| Dry Sprinkler System | 000 | D4013.000 | Dry Sprinkler System, all |
| Standpipe Water Supply | 000 | D4021.000 | Standpipe Water Supply, all |
| Pumping Equipment | 000 | D4022.000 | Pumping Equipment, all |
| Standpipe Equipment | 000 | D4023.000 | Standpipe Equipment, all |
| Fire Hose Equipment | 000 | D4024.000 | Fire Hose Equipment, all |
| Fire Extinguishers | 000 | D4031.000 | Fire Extinguishers, all |
| Fire Extinguisher Cabinets | 000 | D4032.000 | Fire Extinguisher Cabinets, all |
| Carbon Dioxide Systems | 000 | D4091.000 | Carbon Dioxide Systems, all |
| Foam Generating Equipment | 000 | D4092.000 | Foam Generating Equipment, all |
| Clean Agent Systems | 000 | D4093.000 | Clean Agent Systems, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Dry Chemical System | 000 | D4094.000 | Dry Chemical System, all |
| Hood & Duct Fire Protection | 000 | D4095.000 | Hood & Duct Fire Protection, all |
| High Tension Service & Dist. | 000 | D5011.000 | High Tension Service & Dist., all |
| High Tension Service & Dist. | 001 | D5011.001 | Unit substation transformer (Typically 12 kilovolts to 480 volts, 500 – 5,000 kVA) |
| High Tension Service & Dist. | 002 | D5011.002 | Med voltage switchgear (Typically 4 – 12 kilovolt) |
| Low Tension Service & Dist. | 000 | D5012.000 | Low Tension Service & Dist., all |
| Low Tension Service & Dist. | 001 | D5012.001 | Unanchored electrical cabinet |
| Low Tension Service & Dist. | 002 | D5012.002 | Low voltage switchgear (4800 volt) |
| Low Tension Service & Dist. | 003 | D5012.003 | Electrical cabinet well anchored |
| Low Tension Service & Dist. | 004 | D5012.004 | Electrical cabinet nominally anchored |
| Low Tension Service & Dist. | 005 | D5012.005 | Electrical cabinet unanchored |
| Branch Wiring Devices | 000 | D5021.000 | Branch Wiring Devices, all |
| Lighting Equipment | 000 | D5022.000 | Lighting Equipment, all |
| Lighting Equipment | 001 | D5022.001 | Lay-in fluorescent lighting fixtures without two or more slack safety wires |
| Lighting Equipment | 002 | D5022.002 | Lay-in fluorescent lighting fixtures with two or more slack safety wires |
| Lighting Equipment | 003 | D5022.003 | Stem-hung pendant fluorescent fixtures without safety wires in the stems |
| Lighting Equipment | 004 | D5022.004 | Stem-hung pendant fluorescent fixtures with safety wires in the stems |
| Lighting Equipment | 005 | D5022.005 | High-intensity-discharge gas vapor lights |
| Public Address & Music Systems | 000 | D5031.000 | Public Address & Music Systems, all |
| Intercommunication & Paging Syst. | 000 | D5032.000 | Intercommunication & Paging Syst., all |
| Telephone Systems | 000 | D5033.000 | Telephone Systems, all |
| Call Systems | 000 | D5034.000 | Call Systems, all |
| Television Systems | 000 | D5035.000 | Television Systems, all |
| Clock and Program Systems | 000 | D5036.000 | Clock and Program Systems, all |
| Fire Alarm Systems | 000 | D5037.000 | Fire Alarm Systems, all |
| Security and Detection Systems | 000 | D5038.000 | Security and Detection Systems, all |
| Local Area Networks | 000 | D5039.000 | Local Area Networks, all |
| Grounding Systems | 000 | D5091.000 | Grounding Systems, all |
| Emergency Light & Power Systems | 000 | D5092.000 | Emergency Light & Power Systems, all |
| Emergency Light & Power Systems | 001 | D5092.001 | Standby generator (50 kilowatts – 20 megawatts) |
| Floor Raceway Systems | 000 | D5093.000 | Floor Raceway Systems, all |
| Other Special Systems & Devices | 000 | D5094.000 | Other Special Systems & Devices, all |
| Other Special Systems & Devices | 001 | D5094.001 | Motor control center |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Other Special Systems & Devices | 002 | D5094.002 | Unbraced motor installation |
| General Construction Items (Elect.) | 000 | D5095.000 | General Construction Items (Elect.), all |
| General Construction Items (Elect.) | 002 | D5095.002 | Electrical distribution panel |
| General Construction Items (Elect.) | 003 | D5095.003 | Inverter |
| Security & Vault Equipment | 000 | E1011.000 | Security & Vault Equipment, all |
| Teller and Service Equipment | 000 | E1012.000 | Teller and Service Equipment, all |
| Registration Equipment | 000 | E1013.000 | Registration Equipment, all |
| Checkroom Equipment | 000 | E1014.000 | Checkroom Equipment, all |
| Mercantile Equipment | 000 | E1015.000 | Mercantile Equipment, all |
| Laundry & Dry Cleaning Equipment | 000 | E1016.000 | Laundry & Dry Cleaning Equipment, all |
| Vending Equipment | 000 | E1017.000 | Vending Equipment, all |
| Office Equipment | 000 | E1018.000 | Office Equipment, all |
| Ecclesiastical Equipment | 000 | E1021.000 | Ecclesiastical Equipment, all |
| Library Equipment | 000 | E1022.000 | Library Equipment, all |
| Theater & Stage Equipment | 000 | E1023.000 | Theater & Stage Equipment, all |
| Instrumental Equipment | 000 | E1024.000 | Instrumental Equipment, all |
| Audio-visual Equipment | 000 | E1025.000 | Audio-visual Equipment, all |
| Detention Equipment | 000 | E1026.000 | Detention Equipment, all |
| Laboratory Equipment | 000 | E1027.000 | Laboratory Equipment, all |
| Medical Equipment | 000 | E1028.000 | Medical Equipment, all |
| Other Institutional Equipment | 000 | E1029.000 | Other Institutional Equipment, all |
| Vehicular Service Equipment | 000 | E1031.000 | Vehicular Service Equipment, all |
| Parking Control Equipment | 000 | E1032.000 | Parking Control Equipment, all |
| Loading Dock Equipment | 000 | E1033.000 | Loading Dock Equipment, all |
| Other Vehicular Equipment | 000 | E1039.000 | Other Vehicular Equipment, all |
| Maintenance Equipment | 000 | E1091.000 | Maintenance Equipment, all |
| Solid Waste Handling Equipment | 000 | E1092.000 | Solid Waste Handling Equipment, all |
| Food Service Equipment | 000 | E1093.000 | Food Service Equipment, all |
| Residential Equipment | 000 | E1094.000 | Residential Equipment, all |
| Unit Kitchens | 000 | E1095.000 | Unit Kitchens, all |
| Window Washing Equipment | 000 | E1097.000 | Window Washing Equipment, all |
| Other Equipment | 000 | E1099.000 | Other Equipment, all |
| Fixed Artwork | 000 | E2011.000 | Fixed Artwork, all |
| Fixed Casework | 000 | E2012.000 | Fixed Casework, all |
| Blinds and Other Window Treatment | 000 | E2013.000 | Blinds and Other Window Treatment, all |
| Fixed Floor Grilles and Mats | 000 | E2014.000 | Fixed Floor Grilles and Mats, all |
| Fixed Multiple Seating | 000 | E2015.000 | Fixed Multiple Seating, all |
| Fixed Interior Landscaping | 000 | E2016.000 | Fixed Interior Landscaping, all |
| Movable Artwork | 000 | E2021.000 | Movable Artwork, all |
| Furniture & Accessories | 000 | E2022.000 | Furniture & Accessories, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Furniture & Accessories | 001 | E2022.001 | Large freestanding storage furniture subject to overturning |
| Furniture & Accessories | 002 | E2022.002 | Large freestanding household electrical appliances |
| Furniture & Accessories | 003 | E2022.003 | Small countertop household electrical appliances |
| Furniture & Accessories | 004 | E2022.004 | Household entertainment equipment |
| Furniture & Accessories | 005 | E2022.005 | Floor-standing furniture subject to crushing |
| Furniture & Accessories | 006 | E2022.006 | Heaters and air conditioning equipment subject to crushing or overturning |
| Furniture & Accessories | 007 | E2022.007 | Indoor accessories, e.g., curtains, health and medical equipment, sporting goods, bags, shoes, carpets |
| Furniture & Accessories | 008 | E2022.008 | Tableware |
| Furniture & Accessories | 009 | E2022.009 | Small home entertainment items subject to falling, e.g., clocks, cameras, lights, records, CDs, toys |
| Furniture & Accessories | 010 | E2022.010 | Clothing and bedclothes subject to damage or contamination by broken glass or other foreign matter |
| Furniture & Accessories | 011 | E2022.011 | Desktop computer system unit and CRT monitor |
| Furniture & Accessories | 012 | E2022.012 | Countertop contents with low base friction (coeff of static friction ≤ 0.50) and low weight (≤ 20 lb) |
| Furniture & Accessories | 013 | E2022.013 | Countertop contents with low base friction (coeff of static friction ≤ 0.50) and med weight (20-400 lb) |
| Furniture & Accessories | 014 | E2022.014 | Countertop contents with high base friction (coeff of static friction ≤ 0.50) and low weight (≤ 20 lb) |
| Furniture & Accessories | 015 | E2022.015 | Countertop contents with high base friction (coeff of static friction ≤ 0.50) and med weight (20-400 lb) |
| Furniture & Accessories | 016 | E2022.016 | Shelved contents with low base friction (coeff of static friction ≤ 0.50), low weight (≤ 20 lb), ≤ 4 ft above floor |
| Furniture & Accessories | 017 | E2022.017 | Shelved contents with low base friction (coeff of static friction ≤ 0.50), low weight (≤ 20 lb), > 4 ft above floor |
| Furniture & Accessories | 018 | E2022.018 | Shelved contents with low base friction (coeff of static friction ≤ 0.50), med weight (20-400 lb), ≤ 4 ft above floor |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Furniture & Accessories | 019 | E2022.019 | Shelved contents with low base friction (coeff of static friction ≤ 0.50), med weight (20-400 lb), > 4 ft above floor |
| Furniture & Accessories | 020 | E2022.020 | Shelved contents with high base friction (coeff of static friction > 0.50), low weight (≤ 20 lb), ≤ 4 ft above floor |
| Furniture & Accessories | 021 | E2022.021 | Shelved contents with high base friction (coeff of static friction > 0.50), low weight (≤ 20 lb), > 4 ft above floor |
| Furniture & Accessories | 022 | E2022.022 | Shelved contents with high base friction (coeff of static friction > 0.50), med weight (20-400 lb), ≤ 4 ft above flr |
| Furniture & Accessories | 023 | E2022.023 | Shelved contents with high base friction (coeff of static friction > 0.50), med weight (20-400 lb), > 4 ft above floor |
| Furniture & Accessories | 024 | E2022.024 | Library shelving not braced to the building frame |
| Furniture & Accessories | 025 | E2022.025 | Library shelving that is braced to the building frame |
| Furniture & Accessories | 026 | E2022.026 | Contents in cabinets without positive mechanical or strong magnetic catches |
| Furniture & Accessories | 027 | E2022.027 | Contents in cabinets with positive mechanical or strong magnetic catches |
| Furniture & Accessories | 028 | E2022.028 | Mechanically restrained light contents and light contents on shelves with bungy-cord or spring-mounted wire restraint |
| Furniture & Accessories | 029 | E2022.029 | Unanchored bookcases |
| Movable Rugs and Mats | 000 | E2023.000 | Movable Rugs and Mats, all |
| Movable Interior Landscaping | 000 | E2024.000 | Movable Interior Landscaping, all |
| Air Supported Structures | 000 | F1011.000 | Air Supported Structures, all |
| Pre-engineered Structures | 000 | F1012.000 | Pre-engineered Structures, all |
| Other Special Structures | 000 | F1013.000 | Other Special Structures, all |
| Integrated Assemblies | 000 | F1021.000 | Integrated Assemblies, all |
| Special Purpose Rooms | 000 | F1022.000 | Special Purpose Rooms, all |
| Other Integrated Construction | 000 | F1023.000 | Other Integrated Construction, all |
| Sound, Vibration & Seismic Const. | 000 | F1031.000 | Sound, Vibration & Seismic Const., all |
| Radiation Protection | 000 | F1032.000 | Radiation Protection, all |
| Special Security Systems | 000 | F1033.000 | Special Security Systems, all |
| Vaults | 000 | F1034.000 | Vaults, all |

| Level 4 description | Level 5 extension | Level 5 ID | Level 5 description |
|---|---|---|---|
| Other Special Construction Systems | 000 | F1039.000 | Other Special Construction Systems, all |
| Aquatic Facilities | 000 | F1041.000 | Aquatic Facilities, all |
| Ice Rinks | 000 | F1042.000 | Ice Rinks, all |
| Site Constructed Incinerators | 000 | F1043.000 | Site Constructed Incinerators, all |
| Kennels & Animal Shelters | 000 | F1044.000 | Kennels & Animal Shelters, all |
| Liquid & Gas Storage Tanks | 000 | F1045.000 | Liquid & Gas Storage Tanks, all |
| Liquid & Gas Storage Tanks | 001 | F1045.001 | Liquid oxygen tank, light anchors |
| Liquid & Gas Storage Tanks | 002 | F1045.002 | Liquid oxygen tank, well anchored |
| Other Special Facilities | 000 | F1049.000 | Other Special Facilities, all |
| Recording Instrumentation | 000 | F1051.000 | Recording Instrumentation, all |
| Building Automation System | 000 | F1052.000 | Building Automation System, all |
| Other Special Controls & Instrumentation | 000 | F1059.000 | Other Special Controls & Instrumentation, all |
| Building Interior Demolition | 000 | F2011.000 | Building Interior Demolition, all |
| Building Exterior Demolition | 000 | F2012.000 | Building Exterior Demolition, all |
| Removal of Hazardous Components | 000 | F2021.000 | Removal of Hazardous Components, all |
| Encapsulation of Hazardous Components | 000 | F2022.000 | Encapsulation of Hazardous Components, all |

# Appendix F
# Generation of Realizations for Loss Computations

## F.1 Loss Computations

The loss-computation algorithm in PACT generates a large number of realizations (or vectors) of demand per intensity level to develop a loss curve. This appendix describes the two algorithms in PACT that are used to transform the demands computed by nonlinear response-history analysis (multiple demand vectors per intensity level) or simplified analysis (1 vector per intensity level), into a large number of realizations[1]. Section F2 presents the algorithm for basic assessment, which uses nonlinear response-history analysis, and Section F3 presents the algorithm for simplified assessment, which uses a simplified analysis procedure. For both types of assessment, the input vector(s) of demand parameters is (are) not included in the generated realizations used for loss computations.

## F.2 Realizations for Assessment Using Nonlinear Response-History Analysis

### *F.2.1 Introduction*

Section F2.2 introduces the algorithm implemented in PACT to transform a limited number of vectors of demand parameters established by response-history analysis to the large number of demand vectors used for loss calculations. Section F2.3 presents sample calculations to illustrate the

[1] In this *Guideline*, nonlinear response-history analysis is recommended for basic assessment, a linear static procedure is provided for simplified assessment, and 200 realizations are generated per intensity level for loss computations. Eleven simulations per intensity level are recommended for nonlinear analysis and one simulation is used for linear analysis. These procedures, numbers of analyses and number of realizations are not mandatory and others can be used. This appendix uses the default procedures, number of realizations and numbers of demand vectors per intensity level, 11 for nonlinear response-history analysis (for basic assessment) and 1 for linear analysis (for simplified assessment), to illustrate the algorithms included in PACT to generate a large number of realizations for loss computations.

procedure. A Matlab script is presented in F2.4 to enable the reader to generate correlated vectors outside of PACT.

### *F.2.2 Algorithm*

The technical basis for the algorithm described herein was developed by Yang, Moehle and DerKiureghian (see Yang, 2006).

For basic assessment, a limited number, *m*, of response analyses (the default number is 11) are performed at each intensity level. For each analysis, the peak absolute value of each demand parameter (e.g., third story drift, roof acceleration) is assembled into a row vector with *n* entries, where *n* is the number of demand parameters. The *m* row vectors are catenated to form an $m \times n$ matrix (rows × columns; simulations × demand parameters)–each column presents *m* values of one demand parameter.

Denote the matrix of demand parameters $\mathbf{X}$. The entries in $\mathbf{X}$ are assumed to be jointly lognormal (see Appendix A for a discussion of joint probability distributions). The natural logarithm of each entry in $\mathbf{X}$ is computed to form an $11 \times n$ matrix $\mathbf{Y}$. The entries in $\mathbf{Y}$ are assumed to be jointly normal and can be characterized by $1 \times n$ mean vector, $\mathbf{M_Y}$, a $n \times n$ correlation coefficient matrix, $\mathbf{R_{YY}}$, and a $n \times n$ diagonal matrix of standard deviations, $\mathbf{D_Y}$.

A vector of demand parameters, $\mathbf{Z}$, can be generated with the same statistical distribution as $\mathbf{Y}$ using a vector of uncorrelated standard normal random variables, $\mathbf{U}$, with a mean of 0 and a standard deviation of 1 for each random variable. For this case

$$\mathbf{Z} = \mathbf{AU} + \mathbf{B} = \mathbf{D_Y L_Y U} + \mathbf{M_Y} \tag{F-1}$$

where $\mathbf{A}$ is a matrix of constant coefficients that linearly transforms $\mathbf{U}$ to $\mathbf{Z}$ and $\mathbf{B}$ is a vector of constant coefficients that translates $\mathbf{U}$ to $\mathbf{Z}$. Matrix $\mathbf{A}$ and vector $\mathbf{B}$ are derived in Yang (2006). The matrix $\mathbf{L_Y}$ is the transposed Cholesky decomposition[2] of $\mathbf{R_{YY}}$, all other terms are defined above.

One realization of demand is generated by 1) computing $\mathbf{M_Y}$, $\mathbf{D_Y}$ and $\mathbf{L_Y}$ by sampling $\mathbf{Y}$; 2) populating $\mathbf{U}$ by random sampling each demand parameter on a distribution with a mean of 0 and a standard deviation of 1.0; 3) computing $\mathbf{Z}$ per (F-1); and 4) taking the exponential of every entry in $\mathbf{Z}$

[2] If matrix $\mathbf{K}$ is symmetric and positive-definite (e.g., a stiffness matrix), it can be decomposed into a lower triangular matrix, $\mathbf{L}$, the Cholesky triangle, and the transpose of the lower triangular matrix, such that $\mathbf{K} = \mathbf{LL^T}$. To solve $\mathbf{Ku} = \mathbf{R}$, one solves first $\mathbf{Ly} = \mathbf{R}$ for $\mathbf{y}$ and then $\mathbf{L^T u} = \mathbf{y}$ for $\mathbf{u}$.

to recover the demand parameters. Two hundred vectors of $\mathbf{U}$ are required for the 200 realizations: one vector per realization. The process is computationally efficient because $\mathbf{M_Y}$, $\mathbf{D_Y}$ and $\mathbf{L_Y}$ are computed just once for each intensity of shaking. The process is illustrated in Figure F-1 (from Yang 2006) below.

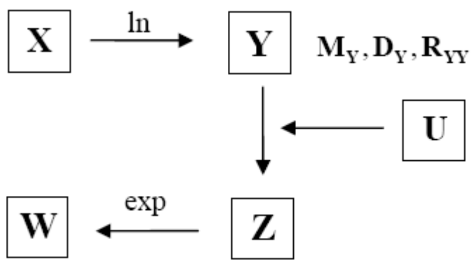



Figure F-1 Generation of vectors of correlated demand parameters (Yang 2006)

### *F.2.3 Sample Application of the Algorithm*

#### F.2.3.1 Description

The sample building is the three story steel moment frame that is described in Section 7 of the *Guideline*. Seven demand parameters are used for performance assessment for this building: 1st, 2nd and 3rd story drift (expressed as a percentage of the story height); and 1st, 2nd and 3rd floor and roof acceleration: $\delta_1$, $\delta_2$, $\delta_3$, $a_1$, $a_2$, $a_3$ and $a_r$, respectively. The matrix $\mathbf{X}$ of demand parameters, 11 rows (one per simulation) × 7 columns (one per demand parameter), is presented in Table F-1.

#### F2.3.2 Probability distributions

Consider the matrix $\mathbf{X}$ above, which can be described as $\mathbf{X} = [\mathbf{X}_1, \mathbf{X}_2 ......\mathbf{X}_7]$: 7 column vectors of demand, with 11 entries per vector. For each vector, the mean (expected) value, $\mu_x$ and the variance, $s_x^2$, can be established as follows:

$$\mu_x = \frac{1}{n}\sum_{i=1}^{i=n} x_i \tag{F-2}$$

$$s_x^2 = \frac{1}{n-1}\sum_{i=1}^{i=n} (x_i - \mu_x)^2 \tag{F-3}$$

The mean and variance of $\mathbf{X} = [\mathbf{X}_1, \mathbf{X}_2 ......\mathbf{X}_7]$ are presented in Table F-2.

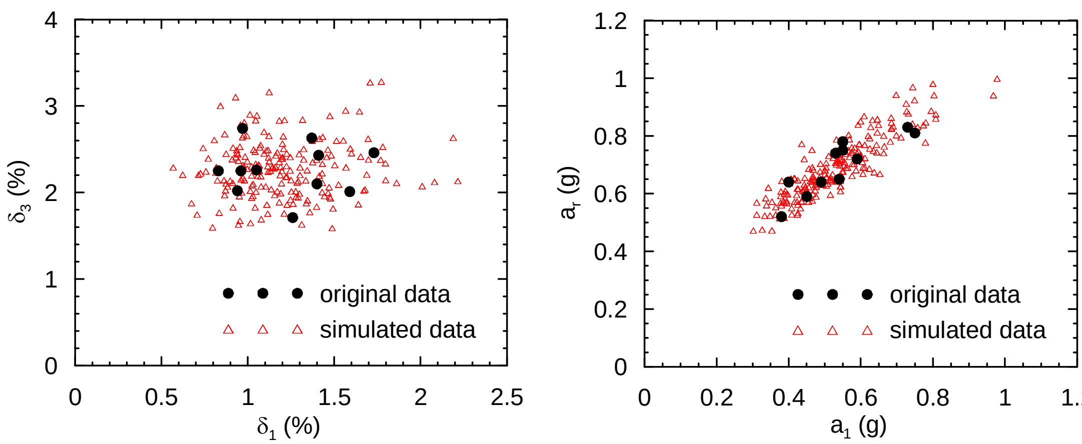
**Table F-1 Matrix of Demand Parameters, $\mathbf{X}$**

| | $\delta_1$ (%) | $\delta_2$ (%) | $\delta_3$ (%) | $a_1$ (g) | $a_2$ (g) | $a_3$ (g) | $a_r$ (g) |
|---|---|---|---|---|---|---|---|
| 1 | 1.26 | 1.45 | 1.71 | 0.54 | 0.87 | 0.88 | 0.65 |
| 2 | 1.41 | 2.05 | 2.43 | 0.55 | 0.87 | 0.77 | 0.78 |
| 3 | 1.37 | 1.96 | 2.63 | 0.75 | 1.04 | 0.89 | 0.81 |
| 4 | 0.97 | 1.87 | 2.74 | 0.55 | 0.92 | 1.12 | 0.75 |
| 5 | 0.94 | 1.8 | 2.02 | 0.40 | 0.77 | 0.74 | 0.64 |
| 6 | 1.73 | 2.55 | 2.46 | 0.45 | 0.57 | 0.45 | 0.59 |
| 7 | 1.05 | 2.15 | 2.26 | 0.38 | 0.59 | 0.49 | 0.52 |
| 8 | 1.40 | 1.67 | 2.1 | 0.73 | 1.50 | 1.34 | 0.83 |
| 9 | 1.59 | 1.76 | 2.01 | 0.59 | 0.94 | 0.81 | 0.72 |
| 10 | 0.83 | 1.68 | 2.25 | 0.53 | 1.00 | 0.9 | 0.74 |
| 11 | 0.96 | 1.83 | 2.25 | 0.49 | 0.90 | 0.81 | 0.64 |

**Table F-2 Mean and Variance of $\mathbf{X}$**

| | $\delta_1$ (%) | $\delta_2$ (%) | $\delta_3$ (%) | $a_1$ (g) | $a_2$ (g) | $a_3$ (g) | $a_r$ (g) |
|---|---|---|---|---|---|---|---|
| $\mu_x$ | 1.2282 | 1.8882 | 2.2600 | 0.5418 | 0.9064 | 0.8364 | 0.6973 |
| $s_x^2$ | 0.0878 | 0.0849 | 0.0885 | 0.0139 | 0.0615 | 0.0627 | 0.0094 |

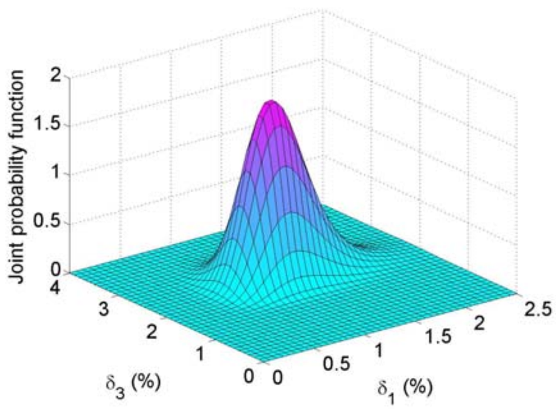
Joint probability distributions can be developed for pairs of vectors, for examples, between a) $\mathbf{X}_1$ (first story drift) and $\mathbf{X}_3$ (third story drift), and b) $\mathbf{X}_4$ (ground floor acceleration) and $\mathbf{X}_7$ (roof acceleration). The black solid circles in Figure F-2 below presents the relationships between $\mathbf{X}_1$ and $\mathbf{X}_3$ (part a) and $\mathbf{X}_4$ and $\mathbf{X}_7$ (part b) constructed using the data of Table F-1: $\mathbf{X}_1$ and $\mathbf{X}_3$ are weakly correlated and $\mathbf{X}_4$ and $\mathbf{X}_7$ are strongly correlated.



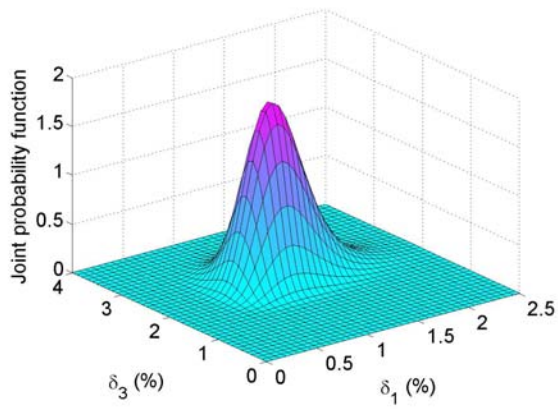

a. $\mathbf{X}_1$ and $\mathbf{X}_3$ b. $\mathbf{X}_4$ and $\mathbf{X}_7$

Figure F-2 Relationships between demand parameters

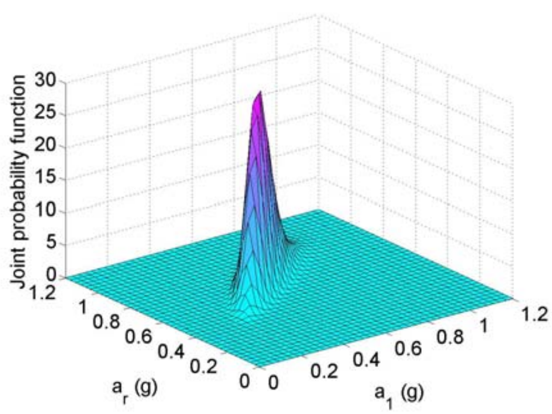
A joint probability density function of the type shown in Figure F-3 can be constructed for any pair of vectors given two means, two variances, a covariance and type of distribution (normal, log-normal). This figure presents the joint probability density functions for $\mathbf{X}_1$ and $\mathbf{X}_3$ (part a) and $\mathbf{X}_4$ and $\mathbf{X}_7$ (part c) constructed using the data of Table F-2—termed the *original* data hereafter.



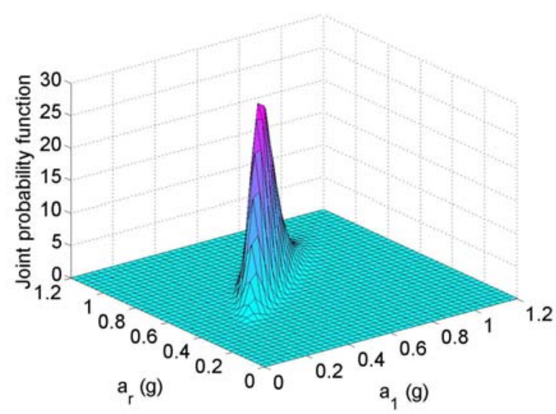

a. joint pdf for $\mathbf{X}_1$ and $\mathbf{X}_3$, original data




b. joint pdf for $\mathbf{X}_1$ and $\mathbf{X}_3$, simulated data




c. joint pdf for $\mathbf{X}_4$ and $\mathbf{X}_7$, original data




d. joint pdf for $\mathbf{X}_4$ and $\mathbf{X}_7$, simulated data

Figure F-3 Joint probability density functions

### F2.3.3 Analysis

The entries in matrix $\mathbf{X}$ are assumed to be jointly lognormal. The log of each entry in $\mathbf{X}$ is used to construct the entries in matrix $\mathbf{Y}$, which are now assumed to be jointly normal. Table F-3 below presents the entries in $\mathbf{Y}$; the last two rows in the table present the mean and variance of each column vector in $\mathbf{Y}$. (Note that the natural logarithm of any number less than 1.00 will be negative.)

The transpose of the row vector $\mu_y$ in Table F-3 is the mean vector $\mathbf{M_Y}$ of (F-1). The diagonal matrix of standard deviations $\mathbf{D_Y}$ in (F-1) is formed by first taking the square root of each entry in the last row of Table F-3 (standard deviation $s_y$ is the square root of the variance $s_y^2$) and placing the first entry ($0.2431 = \sqrt{0.059}$) in cell (1,1), the second entry in cell (2,2) and so on. Matrix $\mathbf{D_Y}$ for this example is presented in Table F-4.

**Table F-3 Demand Parameters, $\mathbf{Y}$**

| | $\delta_1$ | $\delta_2$ | $\delta_3$ | $a_1$ | $a_2$ | $a_3$ | $a_r$ |
|---|---|---|---|---|---|---|---|
| 1 | 0.231 | 0.372 | 0.536 | -0.616 | -0.139 | -0.128 | -0.431 |
| 2 | 0.344 | 0.718 | 0.888 | -0.598 | -0.139 | -0.261 | -0.248 |
| 3 | 0.315 | 0.673 | 0.967 | -0.288 | 0.039 | -0.117 | -0.211 |
| 4 | -0.030 | 0.626 | 1.008 | -0.598 | -0.083 | 0.113 | -0.288 |
| 5 | -0.062 | 0.588 | 0.703 | -0.916 | -0.261 | -0.301 | -0.446 |
| 6 | 0.548 | 0.936 | 0.900 | -0.799 | -0.562 | -0.799 | -0.528 |
| 7 | 0.049 | 0.765 | 0.815 | -0.968 | -0.528 | -0.713 | -0.654 |
| 8 | 0.336 | 0.513 | 0.742 | -0.315 | 0.405 | 0.293 | -0.186 |
| 9 | 0.464 | 0.565 | 0.698 | -0.528 | -0.062 | -0.211 | -0.329 |
| 10 | -0.186 | 0.519 | 0.811 | -0.635 | 0.000 | -0.105 | -0.301 |
| 11 | -0.041 | 0.604 | 0.811 | -0.713 | -0.105 | -0.211 | -0.446 |
| $\mu_y$ | 0.179 | 0.625 | 0.807 | -0.634 | -0.131 | -0.222 | -0.370 |
| $s_y^2$ | 0.059 | 0.022 | 0.018 | 0.046 | 0.070 | 0.0993 | 0.021 |

**Table F-4 Matrix $\mathbf{D_Y}$ for the Sample Problem**

| | | | | | | |
|---|---|---|---|---|---|---|
| 0.243 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 0.000 | 0.149 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 0.000 | 0.000 | 0.135 | 0.000 | 0.000 | 0.000 | 0.000 |
| 0.000 | 0.000 | 0.000 | 0.215 | 0.000 | 0.000 | 0.000 |
| 0.000 | 0.000 | 0.000 | 0.000 | 0.265 | 0.000 | 0.000 |
| 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.315 | 0.000 |
| 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.144 |

The correlation coefficient matrix, $\mathbf{R_{YY}}$, is a symmetric matrix with entries $\rho_{i,j}$, which is the correlation coefficient between random variables $Y_i$ and $Y_j$. The range for $\rho_{i,j}$ is -1 to +1, where values close to a) 1.0 indicate a positive linear relationship between the random variables, b) -1 indicate a negative linear relationship between the random variables, and c) values close to or equal to 0 indicate no linear relationship between the random variables. Table F-5 presents the correlation coefficient matrix for this sample problem. The values on the leading diagonal are equal to 1.000, which is an expected result because $\rho_{i,i}$ is the correlation of a given random variable (7 in this example) with itself. Correlation coefficients are dimensionless.

**Table F-5 Matrix $\mathbf{R_{YY}}$ for the Sample Problem**

| | | | | | | |
|---|---|---|---|---|---|---|
| 1.000 | 0.339 | -0.019 | 0.375 | -0.022 | -0.193 | 0.145 |
| 0.339 | 1.000 | 0.656 | -0.353 | -0.646 | -0.723 | -0.376 |
| -0.019 | 0.656 | 1.000 | 0.136 | -0.094 | -0.066 | 0.220 |
| 0.375 | -0.353 | 0.136 | 1.000 | 0.839 | 0.731 | 0.881 |
| -0.022 | -0.646 | -0.094 | 0.839 | 1.000 | 0.934 | 0.863 |
| -0.193 | -0.723 | -0.066 | 0.731 | 0.934 | 1.000 | 0.820 |
| 0.145 | -0.376 | 0.220 | 0.881 | 0.863 | 0.820 | 1.000 |

From this table one can judge the linear dependence of one random variable on another. Consider $\rho_{1,3}$, the correlation coefficient between first story drift and third story drift, in the log space. In this example, $\rho_{1,3} = -0.019$, which indicates negligible dependence of $\mathbf{Y}_3$ on $\mathbf{Y_1}$ (and vice-versa). Consider now the linear dependence of the second and third floor and roof accelerations on the ground acceleration, $\rho_{4,5}$, $\rho_{4,6}$, $\rho_{4,7}$, respectively. All values are close to 1.0, which indicates strong linear dependence.

The correlation coefficient is closely related to the covariance, which is a measure of how much two random variables vary together. (Variance is a measure of how much a single random variable varies—see the last row of Table F-3 that presents the variance in $\mathbf{Y_i}$ for all seven demand parameters). The unit of covariance is the product of the units of the random variables. The covariance of random variables $\mathbf{Y_i}$ and $\mathbf{Y_j}$, $\text{cov}(\mathbf{Y_i}, \mathbf{Y_j})$, is computed as

$$\text{cov}(\mathbf{Y_i}, \mathbf{Y_j}) = E(\mathbf{Y_i} \cdot \mathbf{Y_j}) - \mu_{\mathbf{Y}_i} \cdot \mu_{\mathbf{Y_j}} \tag{F-4}$$

where $E$ is the expectation value operator and all other terms have been defined previously. For a sample calculation, return to Table F-3 and compute the $\text{cov}(\mathbf{Y}_1, \mathbf{Y}_2)$ as

$$
\begin{aligned}
\operatorname{cov}(\mathbf{Y}_1,\mathbf{Y}_2) &= E((\mathbf{Y}_1-\mu_{\mathbf{Y}_1})(\mathbf{Y}_2-\mu_{\mathbf{Y}_2})) \\
&= \frac{1}{n-1}\sum_{i=1}^{n}(\mathbf{Y}_{1,i}-\mu_{\mathbf{Y}_1})(\mathbf{Y}_{2,i}-\mu_{\mathbf{Y}_2}) \\
&= \frac{1}{10}((0.231-0.179)\times(0.372-0.625)..+(-0.041-0.179)\times(0.604-0.625)) \\
&= 0.0123
\end{aligned}
\tag{F-5}
$$

The correlation coefficient, $\rho_{1,2}$, is calculated as

$$
\rho_{1,2} = \frac{\operatorname{cov}(\mathbf{Y_1},\mathbf{Y_2})}{\sqrt{\operatorname{cov}(\mathbf{Y_1},\mathbf{Y}_1)\times\operatorname{cov}(\mathbf{Y}_2,\mathbf{Y}_2)}} = \frac{0.01226}{\sqrt{0.05910\times0.02209}} = 0.339 \tag{F-6}
$$

The matrix $\mathbf{L_Y}$ is the Cholesky decomposition of $\mathbf{R_{YY}}$, namely, $\mathbf{R_{YY}} = \mathbf{L_Y L_Y^T}$. Information on the Cholesky algorithm can be found in textbooks. The lower triangular matrix $\mathbf{L_Y}$ for the sample problem is given in Table F-6. (Note that the Matlab macro *chol* returns the upper triangular matrix, the transpose of which is presented in Table F-6.)

**Table F-6 Matrix $\mathbf{L_Y}$ for the Sample Problem**

| | | | | | | |
|---|---|---|---|---|---|---|
| 1.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 0.339 | 0.941 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| -0.019 | 0.704 | 0.710 | 0.000 | 0.000 | 0.000 | 0.000 |
| 0.375 | -0.510 | 0.708 | 0.314 | 0.000 | 0.000 | 0.000 |
| -0.022 | -0.678 | 0.540 | 0.377 | 0.325 | 0.000 | 0.000 |
| -0.193 | -0.699 | 0.594 | 0.085 | 0.315 | 0.120 | 0.000 |
| 0.145 | -0.451 | 0.761 | 0.185 | 0.242 | -0.101 | 0.305 |

All of the matrices required to generate the correlated vectors have been constructed. The third-to-last step in the process is to compute the 7×1 vector $\mathbf{U}$ of uncorrelated standard normal random variables, with a mean of 0 and a standard deviation of 1. The **randn** function in Matlab is used for this purpose. This process is repeated 199 times to construct the 200 realizations for loss assessment. The next step involves taking the exponential of each value in the 7×200 matrix to recover the demand parameters. Table F-7 presents the first 10 realizations of the 200.

As a last step, the statistics of the simulated demands should be compared with those of the demands calculated by response-history analysis. Figures F-2 and F-3 enable a partial comparison of demands. A rigorous comparison of the original and simulated demands is achieved by computing the ratios of the mean vectors and the correlation coefficient matrices. Table F-8 presents ratios of mean simulated response to mean response-history response—all values are close to 1.0. Table F-9 presents ratios of the simulated to response-history correlation coefficients—all values are close to 1.0 for those correlation coefficients in Table F-5 with an absolute value of greater than 0.5. The data of Tables F-8 and F-9 indicate that the *simulated* vectors of demand have the same underlying statistics as those of the *original* demand vectors.

**Table F-7 Matrix of Simulated Demand Parameters (first 10 vectors)**

| | $\delta_1$ (%) | $\delta_2$ (%) | $\delta_3$ (%) | $a_1$ (g) | $a_2$ (g) | $a_3$ (g) | $a_r$ (g) |
|---|---|---|---|---|---|---|---|
| 1 | 1.361 | 2.150 | 2.361 | 0.469 | 0.716 | 0.681 | 0.620 |
| 2 | 2.009 | 2.725 | 2.640 | 0.467 | 0.607 | 0.504 | 0.633 |
| 3 | 1.303 | 2.011 | 2.080 | 0.474 | 0.716 | 0.564 | 0.623 |
| 4 | 1.265 | 1.676 | 2.060 | 0.695 | 1.435 | 1.136 | 0.826 |
| 5 | 1.635 | 2.802 | 2.985 | 0.577 | 0.744 | 0.500 | 0.684 |
| 6 | 1.192 | 1.661 | 2.474 | 0.901 | 1.593 | 1.347 | 0.955 |
| 7 | 0.865 | 1.817 | 2.328 | 0.460 | 0.970 | 0.965 | 0.661 |
| 8 | 1.045 | 2.038 | 2.830 | 0.612 | 0.926 | 0.801 | 0.852 |
| 9 | 1.437 | 2.305 | 2.424 | 0.461 | 0.634 | 0.494 | 0.609 |
| 10 | 0.958 | 1.769 | 1.973 | 0.408 | 0.747 | 0.686 | 0.648 |

**Table F-8 Ratio of Simulated to Original Logarithmic Means**

| $\delta_1$ | $\delta_2$ | $\delta_3$ | $a_1$ | $a_2$ | $a_3$ | $a_r$ |
|---|---|---|---|---|---|---|
| 0.9462 | 0.9870 | 0.9822 | 1.0231 | 1.0715 | 1.0559 | 1.0220 |

**Table F-9 Ratio Of Entries in Simulated and Original $\mathbf{R}_{\mathbf{YY}}$ Matrices**

| | | | | | | |
|---|---|---|---|---|---|---|
| 1.000 | 0.987 | 2.128 | 0.761 | 2.205 | 1.161 | 1.265 |
| 0.987 | 1.000 | 0.908 | 1.338 | 1.077 | 1.084 | 1.146 |
| 2.128 | 0.908 | 1.000 | 0.916 | 0.783 | 1.119 | 0.926 |
| 0.761 | 1.338 | 0.916 | 1.000 | 1.047 | 1.069 | 1.045 |
| 2.205 | 1.077 | 0.783 | 1.047 | 1.000 | 1.007 | 1.000 |
| 1.161 | 1.084 | 1.119 | 1.069 | 1.007 | 1.000 | 0.964 |
| 1.265 | 1.146 | 0.926 | 1.045 | 1.000 | 0.964 | 1.000 |

### *F.2.4 Matlab Code*

The Matlab macro below can be used to generate correlated vectors of demand parameters. The vectors of demand parameters established by response-history analysis, DP.txt file, should be constructed per Table F-1, namely, one demand parameter per column, one simulation per row.

```
% Develop underlying statistics of the response-history analysis
X=load('D:\ATCandGroundmotions\DP_example\DP.txt');
Y=log(X);
 My=(mean(Y))';
n_My=length(My);
Dy=diag(std(Y));
Ryy=corrcoef(Y);
Ly=(chol(Ryy))';
% Generate correlated demand vectors using a Monte-Carlo technique
n=200; % the number of the generated realizations
U=[];
for i=1:n_My
   randn('state',i);
U1=randn(n,1);
U=[U U1];
end
%
My=diag(My)*ones(n_My,n);
 Z=Dy*Ly*U'+My;
W=exp(Z');
save D:\ATCandGroundmotions\DP_example\correlated_DP.txt W -ascii
-double -tabs;
%Check results
Mz=(mean(Z'))';
Rzz=corrcoef(Z');
 Mz./(mean(Y))'
Rzz./Ryy
% end
```

## F.3 Realizations for Assessment Using Simplified Nonlinear Analysis

For assessment using the simplified method of analysis per Chapter 6, PACT generates internally a set of 200 statistically consistent demand vectors, given the column vector of demand parameters, $\{\theta\}$ and a value of the dispersion, $\beta$, using the following two-step procedure: 1) randomly generate 200 values of $z$, where $z$ is normally distributed with a mean of 0 and a standard deviation of 1.0; and 2) compute $\{\theta\}\exp(z\beta)$ for each value of $z$. Each vector computed using step 2 is a realization; each realization is used to generate one value of loss.

## F.4 References

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# References


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Yang, T.Y., Moehle, J., Stojadinovic, B., Der Kiureghian, A., 2006, An Application of PEER Performance-Based Earthquake Engineering Methodology, *Proceedings*, 8th US National Conference on Earthquake Engineering, San Francisco, California.

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# PACT

## *Performance Assessment Calculation Tool*

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**ATC-58 Guidelines for Seismic Performance Assessment of Buildings**

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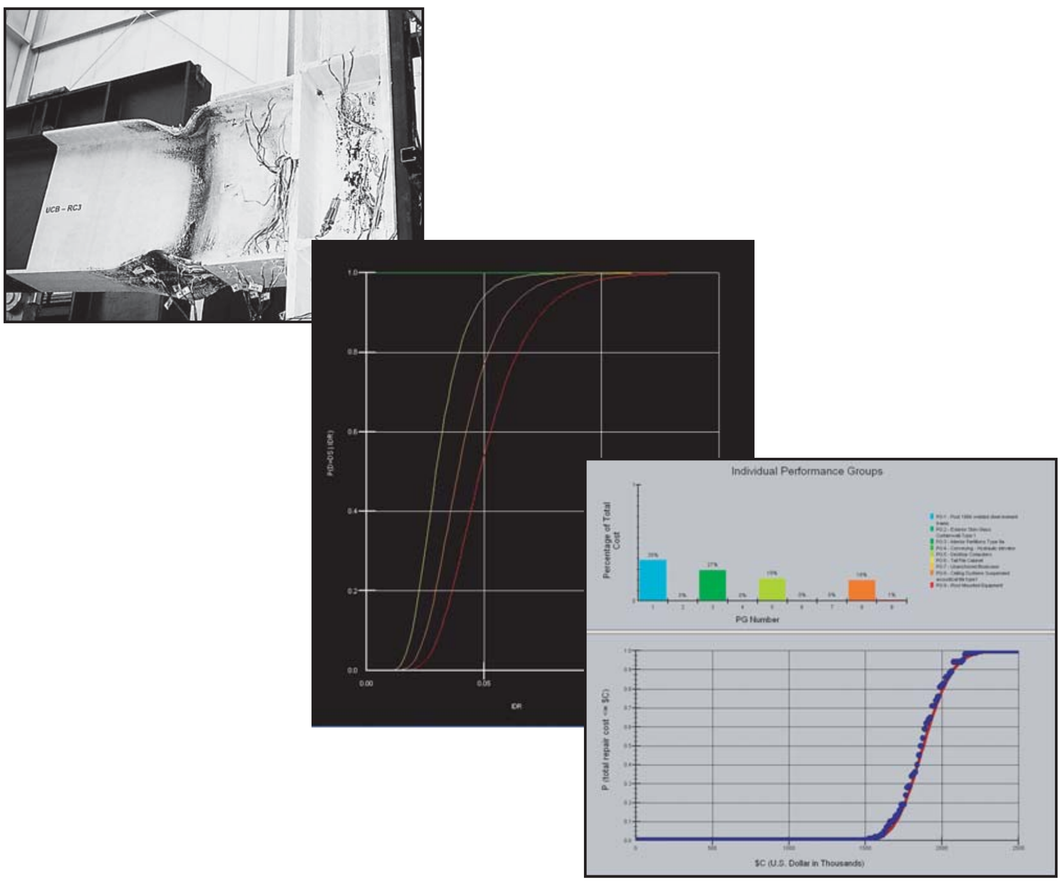


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