Chapter 5
Direct Physical Damage - General Building Stock

5.1 Introduction

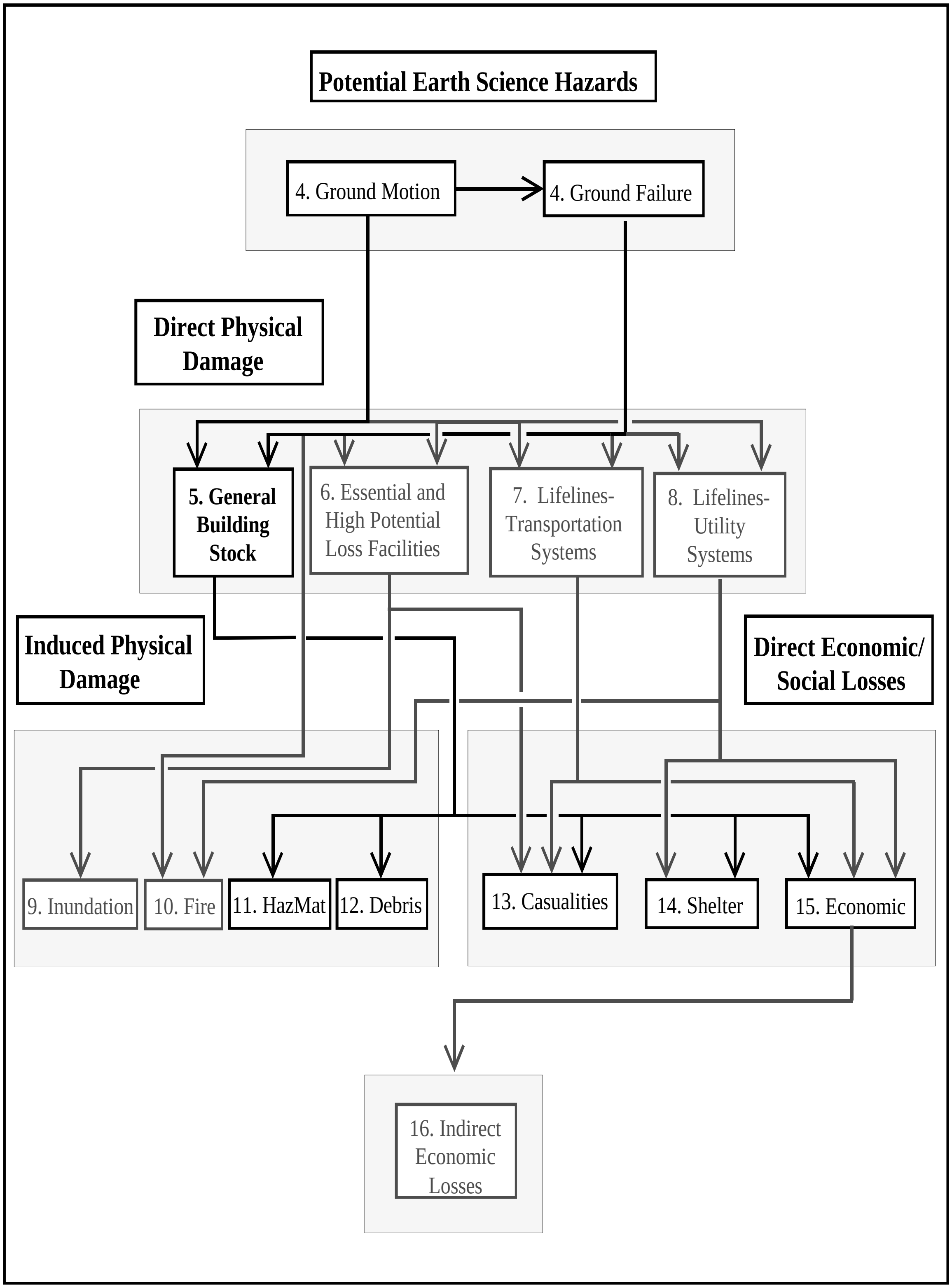
This chapter describes methods for determining the probability of Slight, Moderate, Extensive and Complete damage to general building stock. General building stock represents typical buildings of a given model building type designed to either High-Code, Moderate-Code, or Low-Code seismic standards, or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

5.1.1 Scope

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

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

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



Flowchart 5.1 Building Damage Relationship to Other Components of the Methodology

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

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

5.1.2 Input Requirements and Output Information

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

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

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

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

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

5.1.3 Form of Damage Functions

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

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

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

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

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

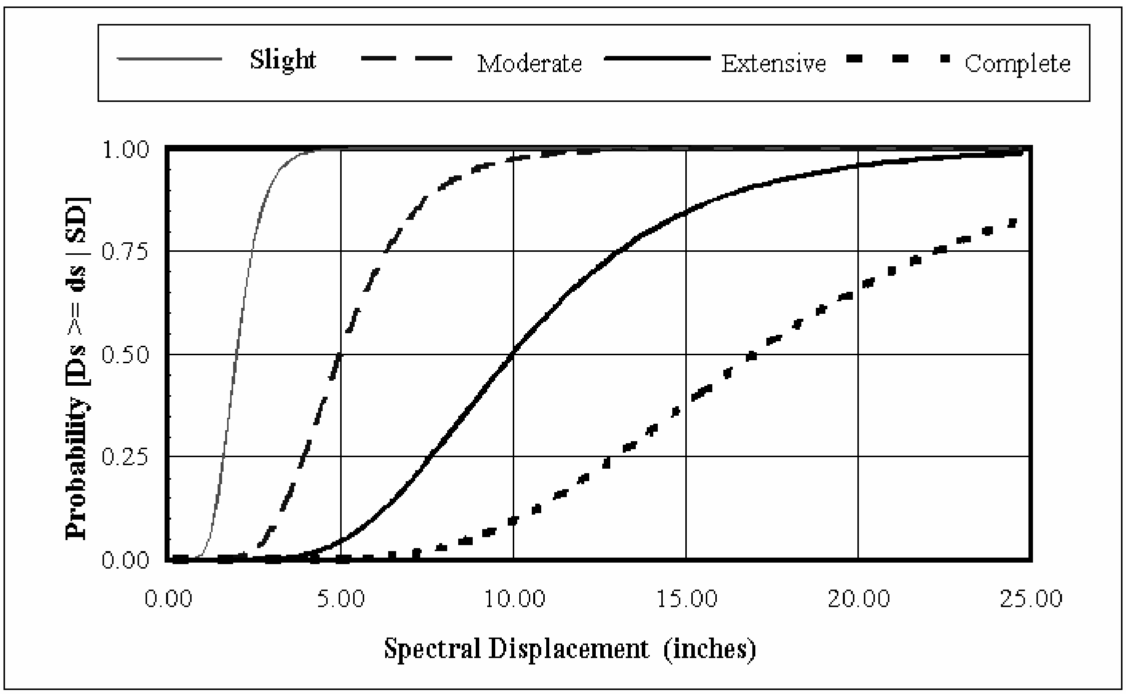


Figure 5.1 Example Fragility Curves for Slight, Moderate, Extensive and Complete Damage.

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

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

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

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

βds is the standard deviation of the natural logarithm of spectral displacement for damage state, ds, and

Φ is the standard normal cumulative distribution function.

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

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

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

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

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

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

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

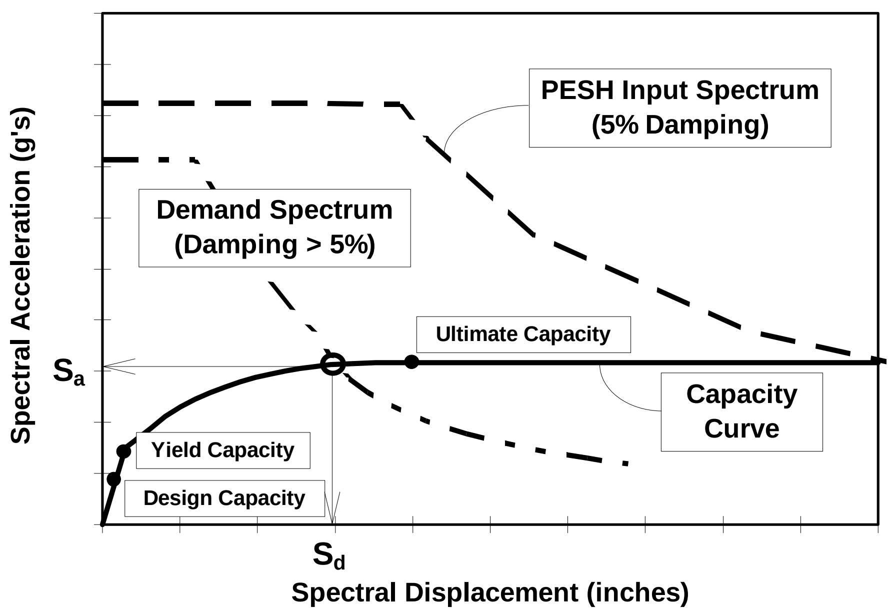


Figure 5.2 Example Building Capacity Curve and Demand Spectrum.

5.2 Description of Model Building Types

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

Table 5.1 Model Building Types

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

5.2.1 Structural Systems

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

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

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

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

Steel Braced Frame (S2):
These buildings are similar to steel moment frame buildings except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.

Steel Light Frame (S3):
These buildings are pre-engineered and prefabricated with transverse rigid frames. The roof and walls consist of lightweight panels, usually corrugated metal. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light steel plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by diaphragm elements, typically rod-braced steel roof framing bays. Tension rod bracing typically resists loads in the longitudinal direction.

Steel Frame with Cast-In-Place Concrete Shear Walls (S4):
The shear walls in these buildings are cast-in-place concrete and may be bearing walls. The steel frame is designed for vertical loads only. Diaphragms of almost any material transfer lateral loads to the shear walls. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections. In modern "dual" systems, the steel moment frames are designed to work together with the concrete shear walls.

Steel Frame with Unreinforced Masonry Infill Walls (S5):
This is one of the older types of buildings. The infill walls usually are offset from the exterior frame members, wrap around them, and present a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels, when they fully engage the surrounding frame members (i.e. lie in the same plane), may provide stiffness and lateral load resistance to the structure.

Reinforced Concrete Moment Resisting Frames (C1):
These buildings are similar to steel moment frame buildings except that the frames are reinforced concrete. There are a large variety of frame systems. Some older concrete frames may be proportioned and detailed such that brittle failure of the frame members can occur in earthquakes leading to partial or full collapse of the buildings. Modern frames in zones of high seismicity are proportioned and detailed for ductile behavior and are likely to undergo large deformations during an earthquake without brittle failure of frame members and collapse.

Concrete Shear Walls (C2):
The vertical components of the lateral-force-resisting system in these buildings are concrete shear walls that are usually bearing walls. In older buildings, the walls often are quite extensive and the wall stresses are low but reinforcing is light. In newer buildings, the shear walls often are limited in extent, generating concerns about boundary members and overturning forces.

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

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

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

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

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

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

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

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

5.2.2 Nonstructural Components

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

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

Table 5.2 List of Typical Nonstructural Components and Contents of Buildings

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

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

5.3 Description of Building Damage States

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

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

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

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

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

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

5.3.1 Structural Damage

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

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

Wood, Light Frame (W1):

Slight Structural Damage: Small plaster or gypsum-board cracks at corners of door and window openings and wall-ceiling intersections; small cracks in masonry chimneys and masonry veneer.
Moderate Structural Damage: Large plaster or gypsum-board cracks at corners of door and window openings; small diagonal cracks across shear wall panels exhibited by small cracks in stucco and gypsum wall panels; large cracks in brick chimneys; toppling of tall masonry chimneys.
Extensive Structural Damage: Large diagonal cracks across shear wall panels or large cracks at plywood joints; permanent lateral movement of floors and roof; toppling of most brick chimneys; cracks in foundations; splitting of wood sill plates and/or slippage of structure over foundations; partial collapse of “room-over-garage” or other “soft-story” configurations; small foundations cracks.
Complete Structural Damage: Structure may have large permanent lateral displacement, may collapse, or be in imminent danger of collapse due to cripple wall failure or the failure of the lateral load resisting system; some structures may slip and fall off the foundations; large foundation cracks. Approximately 3% of the total area of W1 buildings with Complete damage is expected to be collapsed.

Wood, Commercial and Industrial (W2):

Slight Structural Damage: Small cracks at corners of door and window openings and wall-ceiling intersections; small cracks on stucco and plaster walls. Some slippage may be observed at bolted connections.

Moderate Structural Damage: Larger cracks at corners of door and window openings; small diagonal cracks across shear wall panels exhibited by cracks in stucco and gypsum wall panels; minor slack (less than 1/8" extension) in diagonal rod bracing requiring re-tightening; minor lateral set at store fronts and other large openings; small cracks or wood splitting may be observed at bolted connections.

Extensive Structural Damage: Large diagonal cracks across shear wall panels; large slack in diagonal rod braces and/or broken braces; permanent lateral movement of floors and roof; cracks in foundations; splitting of wood sill plates and/or slippage of structure over foundations; partial collapse of "soft-story" configurations; bolt slippage and wood splitting at bolted connections.

Complete Structural Damage: Structure may have large permanent lateral displacement, may collapse or be in imminent danger of collapse due to failed shear walls, broken brace rods or failed framing connections; it may fall its foundations; large cracks in the foundations. Approximately 3% of the total area of W2 buildings with Complete damage is expected to be collapsed.

Steel Moment Frame (S1):

Slight Structural Damage: Minor deformations in connections or hairline cracks in few welds.

Moderate Structural Damage: Some steel members have yielded exhibiting observable permanent rotations at connections; few welded connections may exhibit major cracks through welds or few bolted connections may exhibit broken bolts or enlarged bolt holes.

Extensive Structural Damage: Most steel members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some of the structural members or connections may have exceeded their ultimate capacity exhibited by major permanent member rotations at connections, buckled flanges and failed connections. Partial collapse of portions of structure is possible due to failed critical elements and/or connections.

Complete Structural Damage: Significant portion of the structural elements have exceeded their ultimate capacities or some critical structural elements or connections have failed resulting in dangerous permanent lateral displacement, partial collapse or collapse of the building. Approximately 8%(low-rise), 5%(mid-rise) or 3%(high-rise) of the total area of S1 buildings with Complete damage is expected to be collapsed.

Steel Braced Frame (S2):

Slight Structural Damage: Few steel braces have yielded which may be indicated by minor stretching and/or buckling of slender brace members; minor cracks in welded connections; minor deformations in bolted brace connections.

Moderate Structural Damage: Some steel braces have yielded exhibiting observable stretching and/or buckling of braces; few braces, other members or connections have indications of reaching their ultimate capacity exhibited by buckled braces, cracked welds, or failed bolted connections.

Extensive Structural Damage: Most steel brace and other members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some structural members or connections have exceeded their ultimate capacity exhibited by buckled or broken braces, flange buckling, broken welds, or failed bolted connections. Anchor bolts at columns may be stretched. Partial collapse of portions of structure is possible due to failure of critical elements or connections.

Complete Structural Damage: Most the structural elements have reached their ultimate capacities or some critical members or connections have failed resulting in dangerous permanent lateral deflection, partial collapse or collapse of the building. Approximately 8%(low-rise), 5%(mid-rise) or 3%(high-rise) of the total area of S2 buildings with Complete damage is expected to be collapsed.

Steel Light Frame (S3):

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

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

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

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

Complete Structural Damage: Structure is collapsed or in imminent danger of collapse due to broken rod bracing, failed anchor bolts or failed structural members or connections. Approximately 3% of the total area of S3 buildings with Complete damage is expected to be collapsed.

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

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

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

Moderate Structural Damage: Most shear wall surfaces exhibit diagonal cracks; some of the shear walls have exceeded their yield capacities exhibited by larger diagonal cracks and concrete spalling at wall ends.
Extensive Structural Damage: Most concrete shear walls have exceeded their yield capacities; few walls have reached or exceeded their ultimate capacity exhibited by large through-the wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement. Partial collapse may occur due to failed connections of steel framing to concrete walls. Some damage may be observed in steel frame connections.
Complete Structural Damage: Structure may be in danger of collapse or collapse due to total failure of shear walls and loss of stability of the steel frames. Approximately 8%(low-rise), 5%(mid-rise) or 3%(high-rise) of the total area of S4 buildings with Complete damage is expected to be collapsed.

<u>Steel Frame with Unreinforced Masonry Infill Walls (S5):</u>

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

Slight Structural Damage: Diagonal (sometimes horizontal) hairline cracks on most infill walls; cracks at frame-infill interfaces.
Moderate Structural Damage: Most infill wall surfaces exhibit larger diagonal or horizontal cracks; some walls exhibit crushing of brick around beam-column connections.
Extensive Structural Damage: Most infill walls exhibit large cracks; some bricks may be dislodged and fall; some infill walls may bulge out-of-plane; few walls may fall off partially or fully; some steel frame connections may have failed. Structure may exhibit permanent lateral deformation or partial collapse due to failure of some critical members.
Complete Structural Damage: Structure is collapsed or in danger of imminent collapse due to total failure of many infill walls and loss of stability of the steel frames. . Approximately 8%(low-rise), 5%(mid-rise) or 3%(high-rise) of the total area of S5 buildings with Complete damage is expected to be collapsed.

<u>Reinforced Concrete Moment Resisting Frames (C1):</u>

Slight Structural Damage: Flexural or shear type hairline cracks in some beams and columns near joints or within joints.
Moderate Structural Damage: Most beams and columns exhibit hairline cracks. In ductile frames some of the frame elements have reached yield capacity indicated by larger flexural cracks and some concrete spalling. Nonductile frames may exhibit larger shear cracks and spalling.
Extensive Structural Damage: Some of the frame elements have reached their ultimate capacity indicated in ductile frames by large flexural cracks, spalled concrete and buckled main reinforcement; nonductile frame elements may have suffered shear failures or bond

failures at reinforcement splices, or broken ties or buckled main reinforcement in columns which may result in partial collapse.
Complete Structural Damage: Structure is collapsed or in imminent danger of collapse due to brittle failure of nonductile frame elements or loss of frame stability. Approximately 13%(low-rise), 10%(mid-rise) or 5%(high-rise) of the total area of C1 buildings with Complete damage is expected to be collapsed.

Concrete Shear Walls (C2):

Slight Structural Damage: Diagonal hairline cracks on most concrete shear wall surfaces; minor concrete spalling at few locations.
Moderate Structural Damage: Most shear wall surfaces exhibit diagonal cracks; some shear walls have exceeded yield capacity indicated by larger diagonal cracks and concrete spalling at wall ends.
Extensive Structural Damage: Most concrete shear walls have exceeded their yield capacities; some walls have exceeded their ultimate capacities indicated by large, through-the-wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement or rotation of narrow walls with inadequate foundations. Partial collapse may occur due to failure of nonductile columns not designed to resist lateral loads.
Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of most of the shear walls and failure of some critical beams or columns. Approximately 13%(low-rise), 10%(mid-rise) or 5%(high-rise) of the total area of C2 buildings with Complete damage is expected to be collapsed.

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

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

Slight Structural Damage: Diagonal (sometimes horizontal) hairline cracks on most infill walls; cracks at frame-infill interfaces.
Moderate Structural Damage: Most infill wall surfaces exhibit larger diagonal or horizontal cracks; some walls exhibit crushing of brick around beam-column connections. Diagonal shear cracks may be observed in concrete beams or columns.
Extensive Structural Damage: Most infill walls exhibit large cracks; some bricks may dislodge and fall; some infill walls may bulge out-of-plane; few walls may fall partially or fully; few concrete columns or beams may fail in shear resulting in partial collapse. Structure may exhibit permanent lateral deformation.

Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to a combination of total failure of the infill walls and nonductile failure of the concrete beams and columns. Approximately 15%(low-rise), 13%(mid-rise) or 5%(high-rise) of the total area of C3 buildings with Complete damage is expected to be collapsed.

Precast Concrete Tilt-Up Walls (PC1):

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

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

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

Complete Structural Damage: Structure is collapsed or is in imminent danger of collapse due to failure of the wall-to-roof anchorages, splitting of ledgers, or failure of plywood-to-ledger nailing; failure of beams connections at walls; failure of roof or floor diaphragms; or, failure of the wall panels. Approximately 15% of the total area of PC1 buildings with Complete damage is expected to be collapsed.

Precast Concrete Frames with Concrete Shear Walls (PC2):

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

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

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

Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of the shear walls and/or failures at precast frame connections. Approximately 15%(low-rise), 13%(mid-rise) or 10%(high-rise) of the total area of PC2 buildings with Complete damage is expected to be collapsed.

Reinforced Masonry Bearing Walls with Wood or Metal Deck Diaphragms (RM1):

Slight Structural Damage: Diagonal hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; minor separation of walls from the floor and roof diaphragms.
Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the shear walls have exceeded their yield capacities indicated by larger diagonal cracks. Some walls may have visibly pulled away from the roof.
Extensive Structural Damage: In buildings with relatively large area of wall openings most shear walls have exceeded their yield capacities and some of the walls have exceeded their ultimate capacities indicated by large, through-the-wall diagonal cracks and visibly buckled wall reinforcement. The plywood diaphragms may exhibit cracking and separation along plywood joints. Partial collapse of the roof may result from failure of the wall-to-diaphragm anchorages or the connections of beams to walls.
Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to failure of the wall anchorages or due to failure of the wall panels. Approximately 13%(low-rise) or 10%(mid-rise) of the total area of RM1 buildings with Complete damage is expected to be collapsed.

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

Slight Structural Damage: Diagonal hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings.
Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the shear walls have exceeded their yield capacities indicated by larger cracks.
Extensive Structural Damage: In buildings with relatively large area of wall openings most shear walls have exceeded their yield capacities and some of the walls have exceeded their ultimate capacities exhibited by large, through-the wall diagonal cracks and visibly buckled wall reinforcement. The diaphragms may also exhibit cracking
Complete Structural Damage: Structure is collapsed or is in imminent danger of collapse due to failure of the walls. Approximately 13%(low-rise), 10%(mid-rise) or 5%(high-rise) of the total area of RM2 buildings with Complete damage is expected to be collapsed.

Unreinforced Masonry Bearing Walls (URM):

Slight Structural Damage: Diagonal, stair-step hairline cracks on masonry wall surfaces; larger cracks around door and window openings in walls with large proportion of openings; movements of lintels; cracks at the base of parapets.

Moderate Structural Damage: Most wall surfaces exhibit diagonal cracks; some of the walls exhibit larger diagonal cracks; masonry walls may have visible separation from diaphragms; significant cracking of parapets; some masonry may fall from walls or parapets.
Extensive Structural Damage: In buildings with relatively large area of wall openings most walls have suffered extensive cracking. Some parapets and gable end walls have fallen. Beams or trusses may have moved relative to their supports.
Complete Structural Damage: Structure has collapsed or is in imminent danger of collapse due to in-plane or out-of-plane failure of the walls. Approximately 15% of the total area of URM buildings with Complete damage is expected to be collapsed.

Mobile Homes (MH):

Slight Structural Damage: Damage to some porches, stairs or other attached components.
Moderate Structural Damage: Major movement of the mobile home over its supports resulting in some damage to metal siding and stairs and requiring resetting of the mobile home on its supports.
Extensive Structural Damage: Mobile home has fallen partially off its supports, often severing utility lines.
Complete Structural Damage: Mobile home has totally fallen off its supports; usually severing utility lines, with steep jack stands penetrating through the floor. Approximately 3% of the total area of MH buildings with Complete damage is expected to be collapsed.

5.3.2 Nonstructural Damage

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

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

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

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

Partitions Walls

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

Suspended Ceilings

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

Exterior Wall Panels

Slight Nonstructural Damage: Slight movement of the panels, requiring realignment.

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

Electrical-Mechanical Equipment, Piping, Ducts

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

5.4 Building Damage Due to Ground Shaking

5.4.1 Overview

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

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

Table 5.3 Approximate Basis for Seismic Design Levels

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

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

5.4.2 Capacity Curves

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

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

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

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

- Design Capacity
- Yield Capacity
- Ultimate Capacity

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

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

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

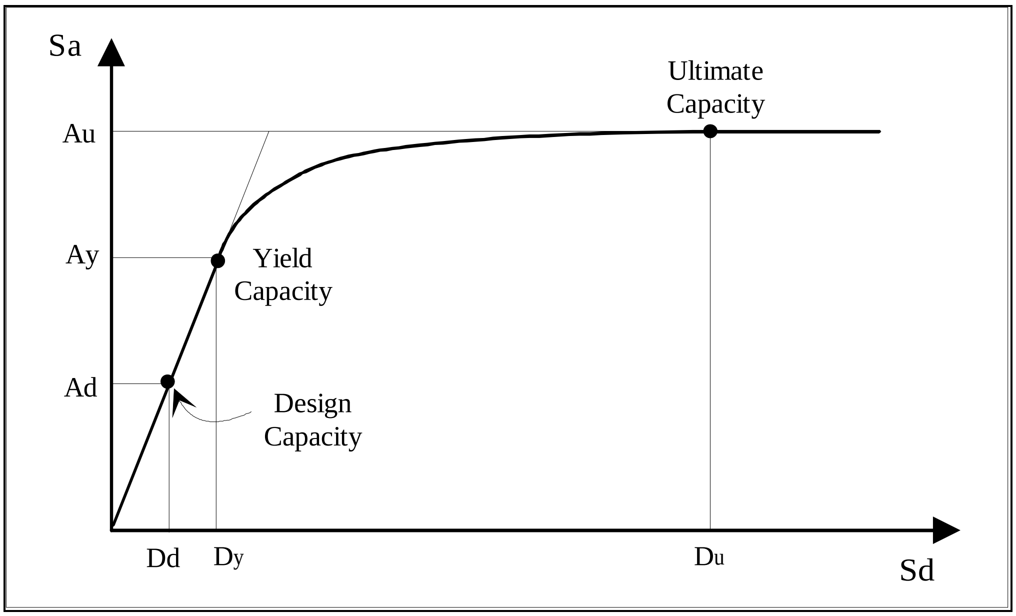


Figure 5.3 Example Building Capacity Curve.

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

- C_s design strength coefficient (fraction of building's weight),
- T_e true "elastic" fundamental-mode period of building (seconds),
- α_1 fraction of building weight effective in push-over mode,
- α_2 fraction of building height at location of push-over mode displacement,
- γ "overstrength" factor relating "true" yield strength to design strength,
- λ "overstrength" factor relating ultimate strength to yield strength, and

μ "ductility" factor relating ultimate displacement to λ times the yield displacement (i.e., assumed point of significant yielding of the structure)

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

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

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

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

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

Table 5.6 Code Building Capacity Parameter - Ductility (μ)

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

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

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

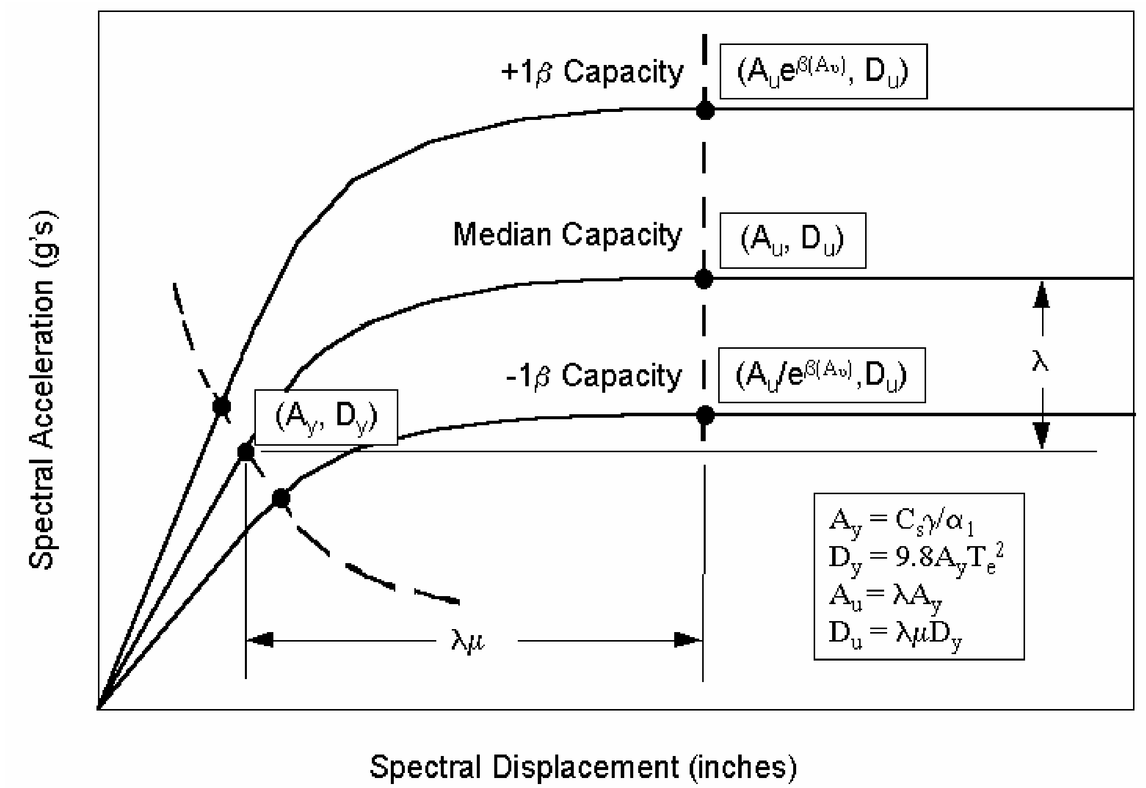


Figure 5.4 Example Construction of Median, +1β and -1β Building Capacity Curves.

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

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

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

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

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

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

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

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

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

5.4.3 Fragility Curves

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

5.4.3.1 Background

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

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

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

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

Φ is the standard normal cumulative distribution function.

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

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

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

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

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

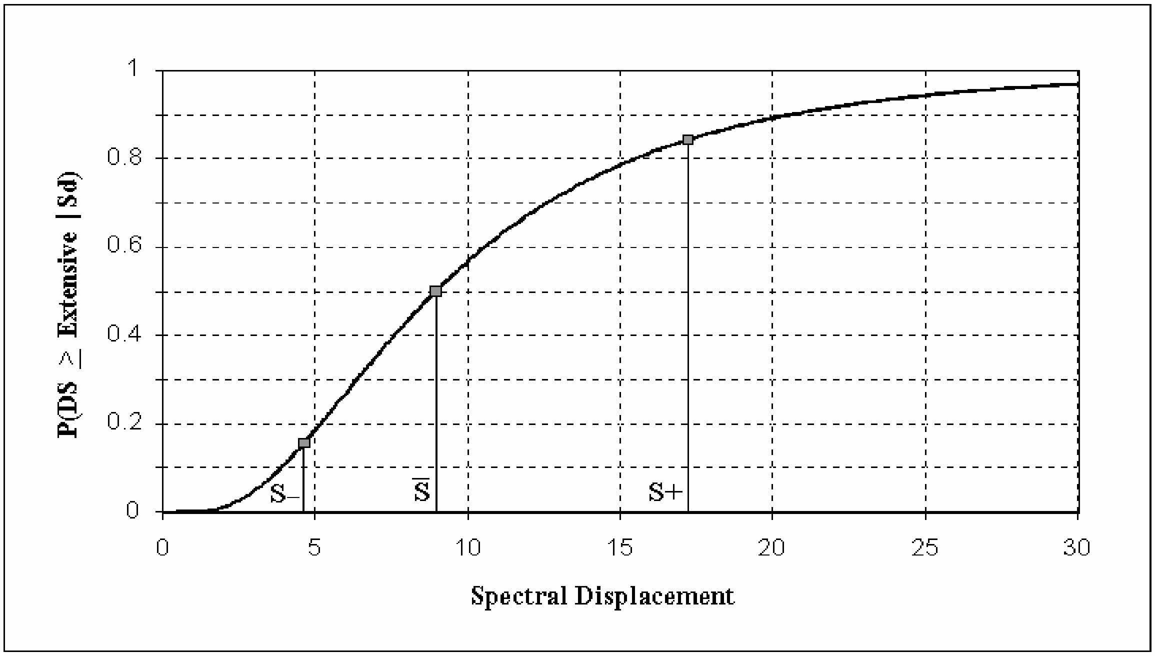


Figure 5.5 Example Fragility Curve - Extensive Structural Damage,

C1M Model Building Type, High-Code Seismic Design.

5.4.3.2 Development of Damage State Medians

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

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

$$\overline{S}_{d,Sds} = \delta_{R,Sds} \cdot \alpha_2 \cdot h \tag{5-4}$$

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

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

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

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

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

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

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

5.4.3.3 Development of Damage State Variability

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

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

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

where:

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

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

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

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

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

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

5.4.3.4 Structural Damage

Structural damage fragility curves for buildings are described by median values of drift that define the thresholds of Slight, Moderate, Extensive and Complete damage states. In general, these estimates of drift are different for each model building type (including height) and seismic design level. Table 5.8 summarizes the ranges of drift ratios used to define structural damage for various low-rise building types designed to current High-

Code seismic provisions. A complete listing of damage-state drift ratios for all building types and heights are provided for each seismic design level in Tables 5.9a, 5.9b, 5.9c and 5.9d, respectively.

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

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

In general, values of the drift ratio that define Complete damage to Moderate-Code buildings are assumed to be 75% of the drift ratio that define Complete damage to High-Code buildings, and values of the drift ratio that define Complete damage to Low-Code buildings are assumed to be 63% of the drift ratios that define Complete damage to High-Code buildings. These assumptions are based on the recognition that post-yield capacity is significantly less in buildings designed with limited ductile detailing. Values of the drift ratio that define Slight damage were assumed to be the same for High-Code, Moderate-Code and Low-Code buildings, since this damage state typically does not exceed the building's elastic capacity.

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

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

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

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

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

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

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

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

Tables 5.9a, 5.9b, 5.9c and 5.9d summarize median and lognormal standard deviation (β_{Sds}) values for Slight, Moderate, Extensive and Complete structural damage states High-Code, Moderate-Code, Low-Code and Pre-Code buildings, respectively. Note that for the following tables, shaded boxes indicate types that are not permitted by current seismic codes.

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

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

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

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

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

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

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

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

5.4.3.5 Nonstructural Damage - Drift-Sensitive Components

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

Table 5.10 Drift Ratios Used to Define Median Values of Damage for

Nonstructural Drift-Sensitive Components

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

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

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

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

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

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

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

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

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

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

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

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

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

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

5.4.3.6 Nonstructural Damage - Acceleration-Sensitive Components

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

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

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

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

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

- uncertainty in the damage-state threshold of nonstructural components ($\beta_{M(NSAds)}$ = 0.6, for all damage states and building types),
- variability in capacity (response) properties of the model building type that contains the nonstructural components of interest ($\beta_{C(Au)}$ = 0.25 for Code buildings, $\beta_{C(Au)}$ = 0.30 for Pre-Code buildings), and
- variability in response of the model building type due to the spatial variability of ground motion demand ($\beta_{D(A)}$ = 0.45 and $\beta_{C(V)}$ = 0.50).

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

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

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

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

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

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

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

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

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

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

5.4.4 Structural Fragility Curves - Equivalent Peak Ground Acceleration

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

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

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

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

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

- variability in response due to the spatial variability of ground motion demand ($\beta_{D(V)}$ = 0.5 for long-period spectral response).

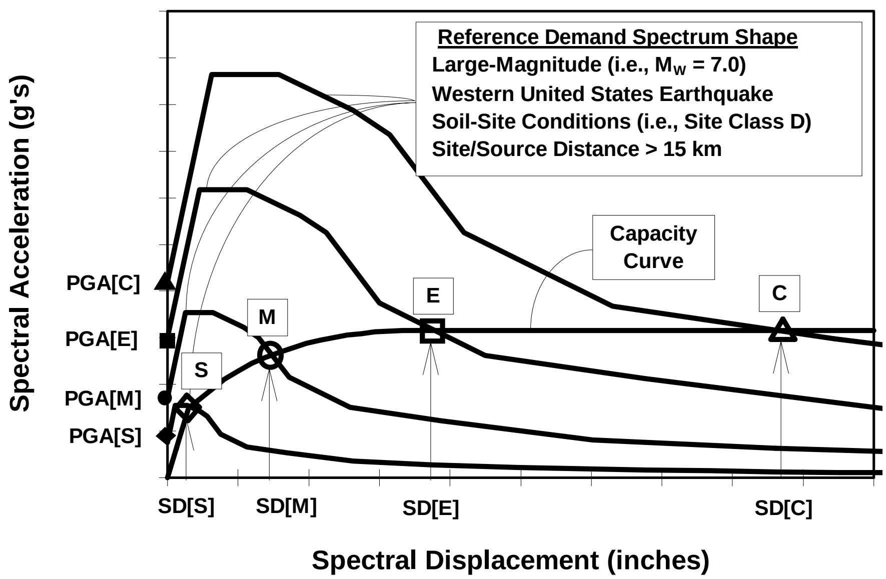


Figure 5.6 Development of Equivalent-PGA Median Damage Values.

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

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

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

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

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

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

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

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

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

where: $\overline{PGA}_{ds}$ is the median PGA of structural damage state, ds,

$\overline{PGA}_{R,ds}$ is the median PGA of structural damage state, ds, as given in Tables 5-13a through 5-13d for the reference spectrum shape

$R_{PGA/SA1}$ is the spectrum shape ratio, given in Tables 5.14 - 5.15, and

F_V is the soil amplification factor, given in Table 4.10

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

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

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

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

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

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

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

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

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

5.5 Building Damage Due to Ground Failure

5.5.1 Overview

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

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

5.5.2 Fragility Curves - Peak Ground Displacement

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

Table 5.17 Building Damage Relationship to PGD - Shallow Foundations

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

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

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

No attempt is made to distinguish damage based on building type, since model building descriptions do not include foundation type. Foundation type is critical to PGD performance and buildings on deep foundations (e.g., piles) perform much better than buildings on spread footings, if the ground settles. When the building is known to be supported by a deep foundation, the probability of Extensive or Complete damage is reduced by a factor of 10 from that predicted for settlement-induced damage of the same building on a shallow foundation. Deep foundations will improve building performance by only a limited amount, if ground spreads laterally. When the building is known to be supported by a deep foundation, the probability of Extensive or Complete damage is reduced by a factor of 2 from that predicted for spread-induced damage of the same building on a shallow foundation.

5.6 Evaluation of Building Damage

5.6.1 Overview

During an earthquake, the building may be damaged either by ground shaking, ground failure, or both. Buildings are evaluated separately for the two modes of failure the resulting damage-state probabilities are combined for evaluation of loss.

5.6.2 Damage Due to Ground Shaking

This section describes the process of developing damage state probabilities based on structural and nonstructural fragility curves, model building capacity curves and a demand spectrum. Building response (e.g., peak displacement) is determined by the intersection of the demand spectrum and the building capacity curve. The demand spectrum is based on the PESH input spectrum reduced for effective damping (when effective damping exceeds the 5% damping level of the PESH input spectrum).

5.6.2.1 Demand Spectrum Reduction for Effective Damping

The elastic response spectra provided as a PESH input apply only to buildings that remain elastic during the entire ground shaking time history and have elastic damping values equal to 5% of critical. This is generally not true on both accounts. Therefore, two modifications are made to elastic response spectra: (a) demand spectra are modified for buildings with elastic damping not equal to 5%, and (b) demand spectra are modified for the hysteretic energy dissipated by buildings “pushed” beyond their elastic limits. Modifications are represented by reduction factors by which the spectral ordinates are divided to obtain the damped demand spectra.

Extensive work has been published in the past two decades on the effect of damping and/or energy dissipation on spectral demand. The Methodology reduces demand spectra for effective damping greater than 5% based on statistically-based formulas of Newmark and Hall (1982). Other methods are available for estimating spectral reduction factors based on statistics relating reduction to ductility demand. It is believed that both methods yield the same results for most practical purposes (FEMA 273). Newmark and Hall provide formulas for construction of elastic response spectra at different damping ratios, B (expressed as a percentage). These formulas represent all site classes (soil types) distinguishing between domains of constant acceleration and constant velocity. Ratios of these formulas are used to develop an acceleration-domain (short-period) reduction factor, R_A, and a velocity-domain (1-second spectral acceleration) reduction factor, R_V, for modification of 5%-damped, elastic response spectra (PESH input). These reduction factors are based on effective damping, B_{eff}, as given in Equations (5-7) and (5-8) below:

$$R_A = 2.12 / \left(3.21 - 0.68 \ln(B_{eff})\right) \tag{5-7}$$

$$R_V = 1.65 / \left(2.31 - 0.41 \ln(B_{eff})\right) \tag{5-8}$$

for which effective damping is defined as the sum of elastic damping, B_E, and hysteretic damping, B_H:

$$B_{eff} = B_E + B_H \tag{5-9}$$

Elastic damping, B_E, is dependent on structure type and is based on the recommendations of Newmark & Hall for materials at or just below their yield point. Hysteretic damping, B_H, is dependent on the amplitude of response and is based on the area enclosed by the hysteresis loop, considering potential degradation of energy-absorption capacity of the structure during cyclic earthquake load. Effective damping, B_{eff}, is also a function of the amplitude of response (e.g., peak displacement), as expressed in Equation (5-10):

$$B_{eff} = B_E + \kappa \cdot \left(\frac{\text{Area}}{2\pi \cdot D \cdot A}\right) \tag{5-10}$$

where:

- B_E is the elastic (pre-yield) damping of the model building type
- Area is the area enclosed by the hysteresis loop, as defined by a symmetrical push-pull of the building capacity curve up to peak positive and negative displacements, $\pm D$
- D is the peak displacement response of the push-over curve,
- A is the peak acceleration response at peak displacement, D
- κ is a degradation factor that defines the effective amount of hysteretic damping as a function of earthquake duration, as specified in Table 5.18.

Table 5.18 Degradation Factor (κ) as a Function of Short, Moderate and Long Earthquake Duration

Building Type		High-Code Design			Moderate-Code Design			Low-Code Design			Pre-Code Design		
No.	Label	Short	Moderate	Long	Short	Moderate	Long	Short	Moderate	Long	Short	Moderate	Long
1	W1	1.00	0.80	0.50	0.90	0.60	0.30	0.70	0.40	0.20	0.50	0.30	0.10
2	W2	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
3	S1L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
4	S1M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
5	S1H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
6	S2L	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
7	S2M	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
8	S2H	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
9	S3	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
10	S4L	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
11	S4M	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
12	S4H	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
13	S5L	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
14	S5M	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
15	S5H	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
16	C1L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
17	C1M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
18	C1H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
19	C2L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
20	C2M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
21	C2H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
22	C3L	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
23	C3M	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
24	C3H	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
25	PC1	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
26	PC2L	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
27	PC2M	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
28	PC2H	0.70	0.50	0.30	0.60	0.40	0.20	0.50	0.30	0.10	0.40	0.20	0.00
29	RM1L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
30	RM1M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
31	RM2L	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
32	RM2M	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
33	RM2H	0.90	0.60	0.40	0.80	0.40	0.20	0.60	0.30	0.10	0.40	0.20	0.00
34	URML	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
35	URMM	0.50	0.30	0.10	0.50	0.30	0.10	0.50	0.30	0.10	0.40	0.20	0.00
36	MH	0.80	0.40	0.20	0.80	0.40	0.20	0.80	0.40	0.20	0.60	0.30	0.10

The Methodology recognizes the importance of the duration of ground shaking on building response by reducing effective damping (i.e., κ factors) as a function of shaking duration. Shaking duration is described qualitatively as either Short, Moderate or Long, and is assumed to be a function of earthquake magnitude (although proximity to fault rupture also influences the duration of ground shaking). For scenario earthquakes of magnitude $\mathbf{M} \leq 5.5$, effective damping is based on the assumption of ground shaking of Short duration. For scenario earthquakes of magnitude $\mathbf{M} \geq 7.5$, effective damping is based on the assumption of ground shaking of Long duration. Effective damping is based on the assumption of Moderate duration for all other earthquake magnitudes (including probabilistic, or other, analyses of unknown magnitude).

Construction of Demand Spectra

Demand spectral acceleration, $S_A[T]$, in units of acceleration (g) is defined by Equation (5-11a) at short periods (acceleration domain), Equation (5-11b) at long periods (velocity domain) and Equation (5-11c) at very long periods (displacement domain).

At short periods, $0 < T \le T_{AV\beta}$:

$$S_A[T] = S_{ASi} \big/ R_A[B_{eff}] = S_{ASi} \big/ \left(2.12 \big/ \left(3.21 - 0.68 \ln\left(B_{eff}\right)\right)\right) \quad (5\text{-}11a)$$

At long periods, $T_{AV\beta} < T \le T_{VD}$:

$$S_A[T] = \left(\frac{S_{A1i}}{T}\right) \Big/ R_V[B_{eff}] = \left(\frac{S_{A1i}}{T}\right) \Big/ \left(1.65 \big/ \left(2.31 - 0.41 \ln\left(B_{eff}\right)\right)\right) \quad (5\text{-}11b)$$

At very long periods, $T > T_{VD}$:

$$S_A[T] = \left(\frac{S_{A1i} \cdot T_{VD}}{T^2}\right) \Big/ R_V[B_{TVD}] = \left(\frac{S_{A1i} \cdot T_{VD}}{T^2}\right) \Big/ \left(1.65 \big/ \left(2.31 - 0.41 \ln\left(B_{TVD}\right)\right)\right) \quad (5\text{-}11c)$$

where: S_{ASi} is the 5%-damped, short-period spectral acceleration for Site Class i (in units of g), as defined by Equation (4-5),

S_{A1i} is the 5%-damped, 1-second-period spectral acceleration for Site Class i (units of g), as defined by Equation (4-6), times 1 second,

T_{AVi} is the transition period between 5%-damped constant spectral acceleration and 5%-damped constant spectral velocity for Site Class i (sec.), as defined by Equation (4-7),

B_{TVD} is the value of effective damping at the transition period, T_{VD}, and

B_{TAVB} is the value of effective damping at the transition period, T_{AVB}.

The transition period, T_{AVB}, between acceleration and velocity domains is a function of the effective damping at this period, as defined by Equation (5-12). The transition period, T_{VD}, between velocity and displacement domains is independent of effective damping, as defined by Equation (4-4).

$$T_{AVB} = T_{AVi}\left(\frac{R_A\left[B_{TAVB}\right]}{R_V\left[B_{TAVB}\right]}\right) = T_{AVi}\left(\frac{2.12 \big/ \left(3.21 - 0.68 \ln\left(B_{TAVB}\right)\right)}{1.65 \big/ \left(2.31 - 0.41 \ln\left(B_{TAVB}\right)\right)}\right) \quad (5\text{-}12)$$

Demand spectral displacement, $S_D[T]$, in inches, is based on $S_A[T]$, in units of g, as given on Equation (5-13):

$$S_D[T] = 9.8 \cdot S_A[T] \cdot T^2 \quad (5\text{-}13)$$

Figure 5.7 shows typical demand spectra (spectral acceleration plotted as a function of spectral displacement) for three demand levels. These three demand levels represent Short (κ = 0.80), Moderate (κ = 0.40) and Long (κ = 0.20) duration ground shaking, respectively. Also shown in the figure is the building capacity curve of a low-rise building of Moderate-Code seismic design that was used to estimate effective damping. The intersection of the capacity curve with each of the three demand spectra illustrates the significance of duration (damping) on building response.

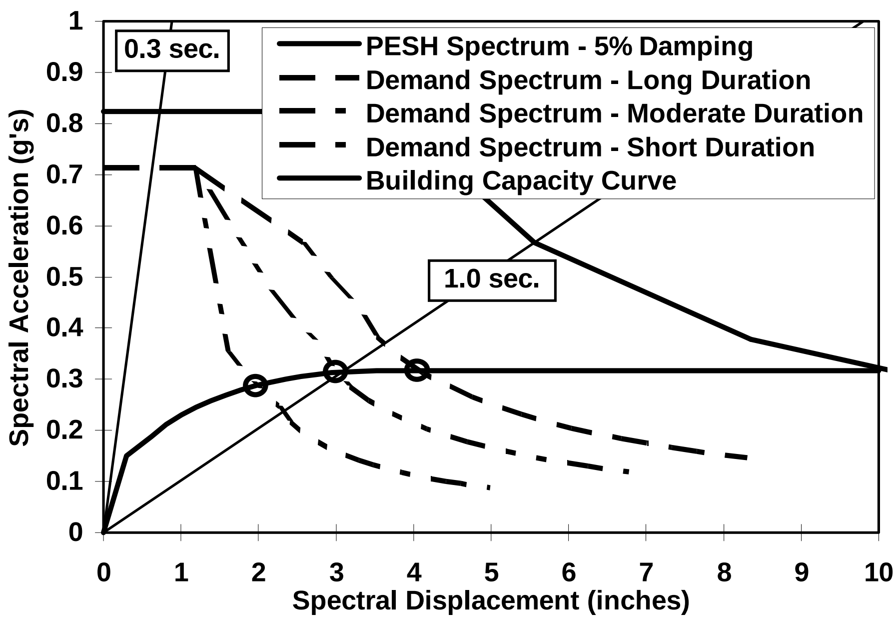


Figure 5.7 Example Demand Spectra - Moderate-Code Building

(M = 7.0 at 20 km, WUS, Site Class E).

5.6.2.2 Damage State Probability

Structural and nonstructural fragility curves are evaluated for spectral displacement and spectral acceleration defined by the intersection of the capacity and demand curves. Each of these curves describes the cumulative probability of being in, or exceeding, a particular damage state. Nonstructural components (both drift- and acceleration-sensitive components) may, in some cases, be dependent on the structural damage state (e.g., Complete structural damage may cause Complete nonstructural damage). The Methodology assumes nonstructural damage states to be independent of structural damage states. Cumulative probabilities are differenced to obtain discrete probabilities of being in each of the five damage states. This process is shown schematically in Figure 5.8.

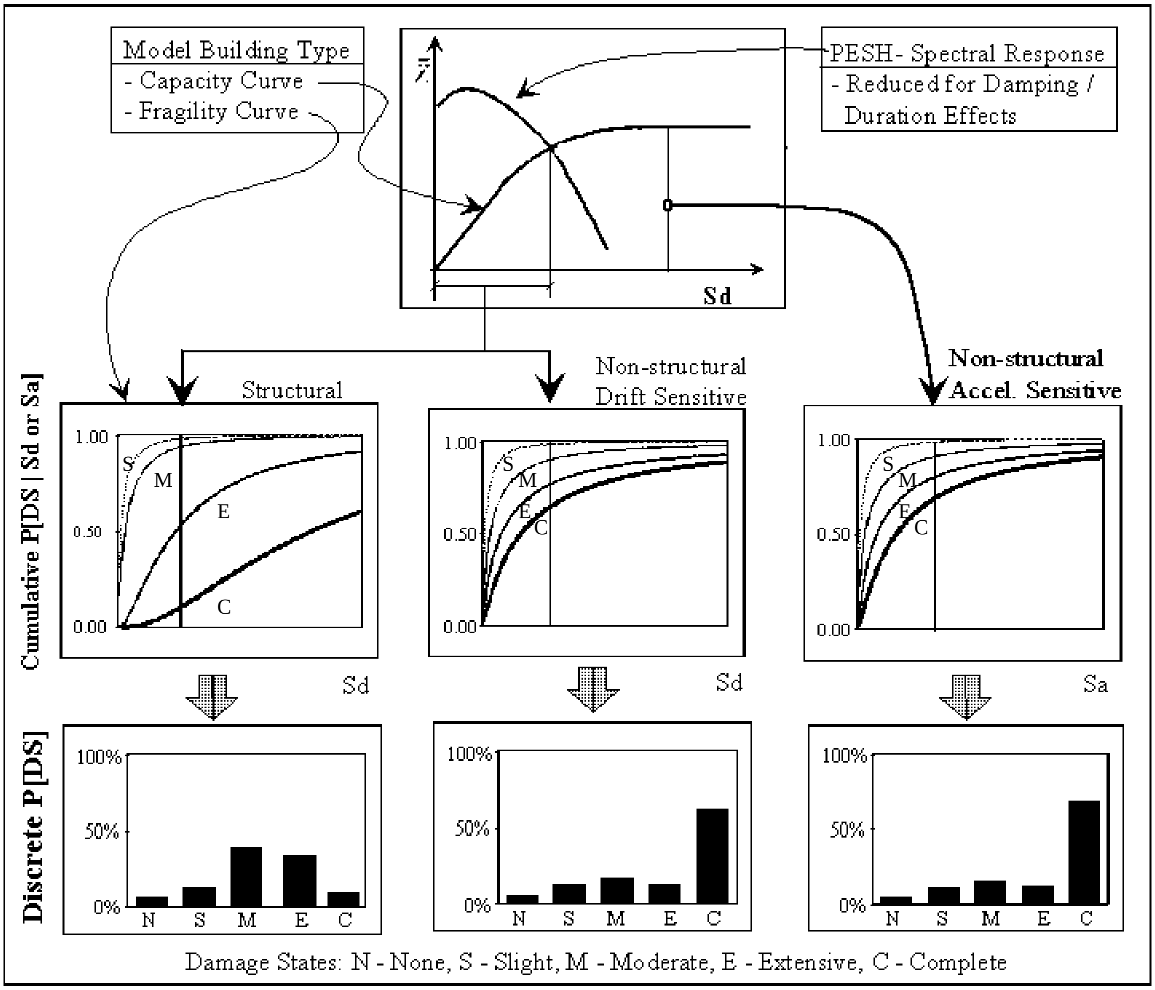


Figure 5.8 Example Building Damage Estimation Process.

It is also meaningful to interpret damage probabilities as the fraction of all buildings (of the same type) that would be in the particular damage state of interest. For example, a 30% probability of Moderate damage may also be thought of as 30 out of 100 buildings (of the same type) being in the Moderate damage state.

5.6.3 Combined Damage Due to Ground Failure and Ground Shaking

This section describes the combination of damage state probabilities due to ground failure (Section 5.5.2) and ground shaking (Section 5.6.2.2). It is assumed that damage due to ground shaking (GS) is independent of damage due to ground failure (GF). Ground failure tends to cause severe damage to buildings and is assumed to contribute only to Extensive and Complete damage states (refer to Section 5.5.1). These assumptions are described by the following formulas:

$$P_{GF}[DS \geq S] = P_{GF}[DS \geq E] \tag{5-14}$$

$$P_{GF}[DS \geq M] = P_{GF}[DS \geq E] \tag{5-15}$$

$$P_{GF}[DS \geq C] = 0.2 \times P_{GF}[DS \geq E] \tag{5-16}$$

The damage state probability (probability of being in or exceeding a given damage state) for GF is assumed to be the maximum of the three types of ground failure (liquefaction, landsliding, and spread). Thus, the combined (due to occurrence of GF or GS) probabilities of being in or exceeding given damage states are:

$$P_{COMB}[DS \geq S] = P_{GF}[DS \geq S] + P_{GS}[DS \geq S] - P_{GF}[DS \geq S] \times P_{GS}[DS \geq S] \tag{5-17}$$

$$P_{COMB}[DS \geq M] = P_{GF}[DS \geq M] + P_{GS}[DS \geq M] - P_{GF}[DS \geq M] \times P_{GS}[DS \geq M] \tag{5-18}$$

$$P_{COMB}[DS \geq E] = P_{GF}[DS \geq E] + P_{GS}[DS \geq E] - P_{GF}[DS \geq E] \times P_{GS}[DS \geq E] \tag{5-19}$$

$$P_{COMB}[DS \geq C] = P_{GF}[DS \geq C] + P_{GS}[DS \geq C] - P_{GF}[DS \geq C] \times P_{GS}[DS \geq C] \tag{5-20}$$

where DS is damage state, and the symbols: S, M, E, and C stand for Slight, Moderate, Extensive, and Complete damage, respectively. COMB indicates the combined probability for the damage state due to occurrence of ground failure or ground shaking. Note that the following condition must always be true:

$$1 \geq P_{COMB}[DS \geq S] \geq P_{COMB}[DS \geq M] \geq P_{COMB}[DS \geq E] \geq P_{COMB}[DS \geq C] \tag{5-21}$$

The discrete probabilities (probabilities of being in a given damage state) are given as:

$$P_{COMB}[DS = C] = P_{COMB}[DS \geq C] \tag{5-22}$$

$$P_{COMB}[DS = E] = P_{COMB}[DS \geq E] - P_{COMB}[DS \geq C] \tag{5-23}$$

$$P_{COMB}[DS = M] = P_{COMB}[DS \geq M] - P_{COMB}[DS \geq E] \tag{5-24}$$

$$P_{COMB}[DS = S] = P_{COMB}[DS \geq S] - P_{COMB}[DS \geq M] \tag{5-25}$$

$$P_{COMB}\left[DS = \text{None}\right] = 1 - P_{COMB}\left[DS \geq S\right] \quad (5\text{-}26)$$

5.6.4 Combined Damage to Occupancy Classes

The damage state probabilities for model building types (as estimated from Section 5.6.3) are combined to yield the damage state probabilities of the occupancy classes to which they belong. For each damage state, the probability of damage to each model building type is weighted according to the fraction of the total floor area of that model building type and summed over all building types. This is expressed in equation form:

$$POSTR_{ds,i} = \sum_{j=1}^{36}\left[PMBTSTR_{ds,j} \times \frac{FA_{i,j}}{FA_i}\right] \quad (5\text{-}27)$$

where $PMBTSTR_{ds,j}$ is the probability of the model building type j being in damage state ds. $POSTR_{ds,i}$ is the probability of occupancy class i being in damage state ds. $FA_{i,j}$ indicates the floor area of model building type j in occupancy class i, and FA_i denotes the total floor area of the occupancy class i (refer to Chapter 3 for floor area distributions of model building types by occupancy class). Similarly, the damage-state probabilities for nonstructural components can be estimated.

$$PONSD_{ds,i} = \sum_{j=1}^{36}\left[PMBTNSD_{ds,j} \times \frac{FA_{i,j}}{FA_i}\right] \quad (5\text{-}28)$$

$$PONSA_{ds,i} = \sum_{j=1}^{36}\left[PMBTNSA_{ds,j} \times \frac{FA_{i,j}}{FA_i}\right] \quad (5\text{-}29)$$

where $PMBTNSD_{ds,j}$ and $PMBTNSA_{ds,j}$ refer to the probabilities of model building type j being in nonstructural drift- and acceleration-sensitive damage state ds, respectively; and $PONSD_{ds,i}$ and $PONSA_{ds,i}$ refer to the probabilities of the occupancy class i being the nonstructural drift-sensitive and acceleration-sensitive damage state, ds, respectively. These occupancy class probabilities are used in Chapter 15 to estimate direct economic loss.

5.7 Guidance for Expert Users

This section provides guidance for users who are seismic/structural experts interested in modifying the building damage functions supplied with the methodology. This section also provides the expert user with guidance regarding the selection of the appropriate mix of design levels for the region of interest.

5.7.1 Selection of Representative Seismic Design Level

The methodology permits the user to select the seismic design level considered appropriate for the study region and to define a mix of seismic design levels for each model building type. The building damage functions provided are based on current-Code provisions and represent buildings of modern design and construction. Most buildings in a study region will likely not be of modern design and construction (i.e., do not conform to 1976 *UBC*, *1985 NEHRP Provisions*, or later editions of these model Codes). For many study regions, particularly those in the Central and Eastern United States, seismic provisions may not be enforced (or only adopted very recently). Building damage functions for new buildings designed and constructed to meet modern-Code provisions should not be used for older, non-complying buildings.

The building damage functions represent specific cells of a three by three matrix that defines three seismic design levels (High, Moderate and Low) and, for each of these design levels, three seismic performance levels (Inferior, Ordinary and Superior), as shown in Table 5.19. For completeness, cells representing Special buildings of Chapter 6 (Essential Facilities) are also included in the matrix.

Table 5.19 Seismic Design and Performance Levels of Default Building Damage Functions (and Approximate Structural Strength and Ductility)

Seismic Design Level	Seismic Performance Level		
	Superior[1]	**Ordinary**	**Inferior**
High **(*UBC* Zone 4)**	Special High-Code *Maximum Strength* *Maximum Ductility*	High-Code *High Strength* *High Ductility*	 *Moderate Strength* *Mod./Low Ductility*
Moderate **(*UBC* Zone 2B)**	Special Moderate-Code *High/Mod. Strength* *High Ductility*	Moderate-Code *Moderate Strength* *Moderate Ductility*	 *Low Strength* *Low Ductility*
Low **(*UBC* Zone 1)**	Special Low-Code *Mod./Low Strength* *Moderate Ductility*	Low-Code *Low Strength* *Low Ductility*	Pre-Code *Minimal Strength* *Minimal Ductility*

1. See Chapter 6 for Special High-Code, Moderate-Code and Low-Code building damage functions.

Table 5.19 also defines the approximate structural strength and ductility attributes of buildings occupying each of the nine cells. The design level is defined by Seismic Zones of the *Uniform Building Code* (*UBC*), since most buildings in the United States that have been designed for earthquakes used some version of the *UBC*. Table 5.20 relates *UBC* seismic zones to seismic design regions of the *NEHRP Provisions*.

Expert users may tailor the damage functions to their study area of interest by determining the appropriate fraction of each building type that conforms essentially to modern-Code provisions (based on age of construction). Buildings deemed not to conform to modern-Code provisions should be assigned a lower seismic design level, or defined as Pre-Code buildings if not seismically designed. For instance, older buildings located in High-Code seismic design areas should be evaluated using damage functions for either Moderate-Code buildings or Pre-Code buildings, for buildings that pre-date seismic codes. Table 5.20 provides guidance for selecting appropriate building damage functions based on building location (i.e., seismic region) and building age. The years shown as break-off points should be considered very approximate and may not be appropriate for many seismic regions, particularly regions of low and moderate seismicity where seismic codes have not been routinely enforced.

Table 5.20 Guidelines for Selection of Damage Functions for Typical Buildings Based on *UBC* Seismic Zone and Building Age

***UBC* Seismic Zone (NEHRP Map Area)**	**Post-1975**	**1941 - 1975**	**Pre-1941**
Zone 4 **(Map Area 7)**	High-Code	Moderate-Code	Pre-Code (W1 = Moderate-Code)
Zone 3 **(Map Area 6)**	Moderate-Code	Moderate-Code	Pre-Code (W1 = Moderate-Code)
Zone 2B **(Map Area 5)**	Moderate-Code	Low-Code	Pre-Code (W1 = Low-Code)
Zone 2A **(Map Area 4)**	Low-Code	Low-Code	Pre-Code (W1 = Low-Code)
Zone 1 **(Map Area 2/3)**	Low-Code	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)
Zone 0 **(Map Area 1)**	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)	Pre-Code (W1 = Low-Code)

The guidelines given in Table 5.20 assume that buildings in the study region are not designed for wind. The user should consider the possibility that mid-rise and high-rise buildings could be designed for wind and may have considerable lateral strength (though not ductility), even if not designed for earthquake. Users must be knowledgeable about the type and history of construction in the study region of interest and apply engineering judgment in assigning the fraction of each building type to a seismic design group.

5.7.2 Development of Damage Functions for Other Buildings

For a building type other than one of the 36 described in Table 5.1, expert users should select a set of building damage functions that best represents the type of construction, strength and ductility of the building type of interest. Such buildings include rehabilitated structures that have improved seismic capacity. For example, URM (Pre-Code) buildings retrofitted in accordance with Division 88, the Los Angeles City Ordinance to “reduce the risk of life loss,” demonstrated significantly improved seismic performance during the 1994 Northridge earthquake [SSC, 1995]. Structural damage to these buildings would be better estimated using either essential facility damage functions of either Low-Code or Moderate-Code RM1 buildings.

5.8 Building Damage References

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