B. Detailed Procedures for Loss Estimation

B.1 Introduction

This appendix describes detailed loss estimation procedures for developing structural damage functions and related direct economic loss functions for welded, steel moment-frame (WSMF) buildings. These procedures are compatible with the *HAZUS* (NIBS, 1997a) methodology, a complex collection of modules that work together to estimate casualties, loss of function and economic impacts on a region due to a scenario earthquake. The *HAZUS* methodology was developed for the Federal Emergency Management Agency (FEMA) by the National Institute of Building Sciences (NIBS) and is documented in a three-volume Technical Manual (NIBS, 1997b). One of the main components of the methodology estimates the probability of various states of structural and nonstructural damage to buildings. Other modules of the methodology use the damage state probabilities to estimate various types of building-related losses. The *HAZUS* methodology is intended primarily for use in estimation of earthquake losses in regions with a large inventory of buildings represented by generic building types.

The procedures presented in this appendix utilize the results of WSMF building performance evaluations conducted in accordance with Chapter 3 of these *Recommended Criteria*, supplemented by default values of parameters provided in this appendix, to construct structural damage and loss functions. Specifically, structural analysis using the nonlinear static method must be performed as a precursor to the application of the loss estimation methods presented herein. Default values of damage and loss parameters are provided for typical 3-story, 9-story and 20-story WSMF buildings. Example loss estimates that illustrate application of the detailed methods are developed for typical 9-story WSMF buildings.

Commentary: To support mitigation efforts, FEMA funded NIBS to develop "Procedures for Development of HAZUS-Compatible Building-Specific Damage and Loss Functions" (Kircher, 1999). These procedures are an extension of the more general methods of HAZUS, but allow users to incorporate building-specific data including capacity and fragility values developed by nonlinear static (pushover) analysis of the building of interest. The purpose of such evaluations is to understand better the response behavior of the structure, the modes of structural damage and failure, and the amount of structural damage (e.g., connection damage) as a function of the level of earthquake ground shaking. These so-called "building-specific" methods provide the primary basis for the detailed loss-estimation procedures of this appendix.

Implementation of the detailed procedures requires users to have certain levels of expertise and knowledge. It is anticipated that users will be structural engineers:

1. *familiar with evaluation of the earthquake behavior of buildings,*
2. *experienced with nonlinear building analysis,*

3. *familiar with basic methods of statistical analysis, and*
4. *familiar with the HAZUS methodology and building-specific procedures.*

In addition to the HAZUS Technical Manual (NIBS, 1997b), further references on the HAZUS methodology may be found in papers contained in a 1997 special issue of Earthquake Spectra on loss estimation published by Earthquake Engineering Research Institute (EERI). Pertinent papers include Whitman et al. (1997), and Kircher et al. (1997a,b).

B.2 Scope

B.2.1 General

The scope of the detailed loss-estimation procedures is limited to steel moment-frame (WSMF) building damage caused by ground shaking. While ground shaking typically dominates earthquake loss, other hazards, such as ground failure, due to either liquefaction or land-sliding, and surface fault rupture, can also cause building damage. Although less prevalent, when building damage due to ground failure or surface fault rupture occurs it is typically more severe than building damage caused by ground shaking.

The scope of detailed loss-estimation procedures is further limited to damage to the structural system of WSMF buildings. While structural (connection-related) damage is the primary focus of this report, significant damage and loss can occur to nonstructural components and to building contents. Typically, at lower states of damage, nonstructural and contents losses are greater, by several times, than structural losses. This is due to the fact that damage usually begins to occur in nonstructural systems and can become severe before any damage occurs to the structural system. At higher states of damage, the structure becomes more important to economic loss estimation since damage to the structure can affect a complete loss of both structural and nonstructural systems (and contents), and cause long-term closure of the building (that is, loss of function).

The scope of detailed loss-estimation procedures is still further limited to direct economic losses associated with repair and replacement of damaged structural elements and to building loss of function.

Commentary: Other types of losses, such as casualties, may also be important to the user. In those cases for which users require loss estimates for hazards other than ground shaking, the HAZUS Technical Manual (NIBS, 1997b) should be used to develop appropriate loss models. In those cases for which users require loss estimates for building damage other than structural and loss types other than economic, Kircher (1999) should be used to augment the detailed procedures of this section.

B.2.2 Typical Welded, Steel Moment-Frame (WSMF) Buildings

Detailed loss-estimation methods permit the development of building-specific loss functions, based on the configuration and structural details of a specific building. In order to allow more general application, this appendix also presents a series of default loss functions, derived using these methods for use in prediction of damage to WSMF buildings of different height, different seismic force design and different connection type, without needing to resort to detailed structural analyses of individual buildings. Default values of various damage and loss parameters are provided for typical 3-story, 9-story and 20-story buildings. Default values are provided for buildings located in different regions (having different design codes and practice) and having different connection conditions, as identified in Table B-1.

Table B-1 Connections in Typical WSMF Buildings in Three Regions

Connection Condition	Los Angeles Region	Seattle Region	Boston Region
Pre-Northridge	X	X	X
Post-Northridge Special Moment Frame (SMF)	X	X	
Damaged Pre-Northridge	X		

A pre-Northridge connection condition assumes that the building has beam-column connections typical of buildings designed and built prior to the 1994 Northridge earthquake, but which have not been damaged by earthquake ground shaking. A post-Northridge connection condition assumes that the building has either new or retrofitted beam-column connections that comply with the recommendations of *FEMA-350 Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, as applied to Special Moment-Resisting Frame Systems. A damaged pre-Northridge connection condition assumes the building has beam-column connections that are typical of pre-Northridge buildings and that have sustained substantial earthquake damage, but have not been repaired.

B.3 Damage States

Structural damage is described by one of four discrete damage states: Slight, Moderate, Extensive and Complete. Of course, actual building damage varies as a continuous function of earthquake demand. Ranges of damage are used to describe damage, since it is not practical to have a continuous scale, and damage states provide users with an understanding of the structure’s physical condition. Descriptions of structural damage states for WSMF buildings (*HAZUS* model building type S1), based upon but modified from the *HAZUS Technical Manual* are indicated in Table B-2.

Table B-2 Descriptions of Structural Damage States

Damage State	Buildings with Pre-Northridge Connections	Buildings with Post-Northridge Connections
Slight structural damage	No permanent interstory drift. Minor deformations in some connection elements and fractures in less than 10% of the connections at any floor level.	No permanent interstory drift. Minor deformations in some connection elements. No fractures in connections.
Moderate structural damage	Permanent interstory drift as large as 0.5%. Perhaps as many as 25% of the connections on any floor level have experienced fracture.	Permanent interstory drift as large as 0.5%. Moderate amounts of yielding and distortion of some column panel zones. Minor buckling of some girders.
Extensive structural damage	Many connections have failed with a number of fractures extending into and across column panel zones. Some connections may have lost ability to support gravity load, resulting in partial local collapse. Large permanent interstory drifts occur in some stories.	Many steel members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some structural members or connections may have major permanent member rotations at connections, buckled flanges and failed connections. Some connections may have lost ability to support gravity load, resulting in partial local collapse.
Complete structural damage	A significant portion of the structural elements have exceeded their ultimate capacities and/or many critical structural elements or connections have failed resulting in dangerous permanent lateral displacement, partial collapse or collapse of the building. Approximately 15% (of the total square footage) of all WSMF buildings with complete damage are expected to have collapsed.	

General guidance to users regarding selection of damage parameters, taken from Kircher (1999), is provided in Table B-3. Additional steel moment-frame (WSMF) building-specific guidance is given in Table B-4 for determining the structural damage state based on the fraction of damaged connections.

Table B-3 General Guidance for Expected Loss Ratio and Building Condition in Each Damage State

Damage State	Likely Amount of Damage, Loss, or Building Condition			
	Range of Possible Loss Ratios	Probability of Long-Term Building Closure	Probability of Partial or Full Collapse	Immediate Postearthquake Inspection
Slight	0% - 5%	$P = 0$	$P = 0$	Green Tag
Moderate	5% - 25%	$P = 0$	$P = 0$	Green Tag
Extensive	25% - 100%	$P \cong 0.5$	$P \cong 0$[1]	Yellow Tag
Complete	100%	$P \cong 1.0$	$P > 0$	Red Tag

1. Extensive damage may include some localized collapse of the structure.

Table B-4 Specific Guidance for Selection of Damage State Based on Connection Damage

Fraction of All Connections Likely to be Damaged[1]		Damage State
Average Fraction	**Fraction Range**	
0.02	0.0 – 0.05	Slight
0.10	0.05 – 0.25	Moderate
0.50	0.25 – 0.75	Extensive
≅1.0	0.75 – 1.0	Complete

1. Connections having indications of flaws at the root of the Complete Joint Penetration (CJP) weld of beam flanges to columns are not considered as having damage.

B.4 Basic Approach

For the detailed procedures, maximum interstory drift is the basic parameter used to assess structural (i.e., connection) damage. Based on the calculated maximum interstory drift demand, the probability that a structure will be damaged sufficiently to be classified as conforming to each of the four damage states described in Section B.3, is determined. For example, at a maximum interstory drift demand of 3%, a structure may be found to have a low probability, only 10%, of having only slight damage, a 30% probability of moderate damage, a 40% probability of extensive damage and a 20% probability of complete damage. This probabilistic approach is taken in recognition of the fact that due to inherent uncertainties in the prediction of ground motion, structural response and structural damage, it is not possible to quantify precisely how much damage a structure will have for a given earthquake. In this methodology, the probabilistic relationship between structural damage and maximum interstory drift is termed a fragility function. Fragility functions are defined by median estimates of the maximum interstory drift at which a damage state will initiate in a structure (damage state medians) and a parameter b that represents the uncertainty associated with these estimates.

Maximum interstory drift is defined as the peak drift (throughout the duration of earthquake shaking) occurring in any story in the building. Maximum interstory drift is assumed to be about the same as the drift angle demand on nearby beam-column connections. On this basis, damage states of buildings with pre-Northridge connection conditions are related (and calibrated) to observed building response and damage. Similarly, users can define damage states (fragility medians) of buildings with post-Northridge connection conditions using the results of laboratory testing of connections.

In general, the maximum interstory drift in a structure will be greater than the average drift calculated over the height of the building due to various building characteristics (e.g., modes of vibration, nonlinearity, etc.) and the specific nature of the earthquake ground shaking. While response history analyses (of complex multi-degree-of-freedom nonlinear models) provide the most accurate and complete set of building response data, such analyses are rarely practical for engineering applications and are not required for this methodology.

The detailed procedures rely on nonlinear static (pushover) analysis to estimate peak interstory drift and damage. Height-dependent factors are used to adjust pushover drift results for higher-mode effects and other effects not explicitly included in the nonlinear static analysis. Similarly, other height-dependent modal factors are used to relate maximum interstory drift to spectral displacement demand, so that damage (fragility) functions may be expressed in terms of spectral displacement but still be based on the drift angle limits of connections at the story (or stories) experiencing the maximum drift.

The overall approach or process used to estimate economic loss involves a number of steps, as illustrated in the flowchart of Figure B-1. Users are expected to select an appropriate scenario earthquake and to develop the 5%-damped response spectrum of this earthquake using, for example, the generalized spectrum shape and soil amplification factors described in *FEMA-273* or *FEMA-302*.

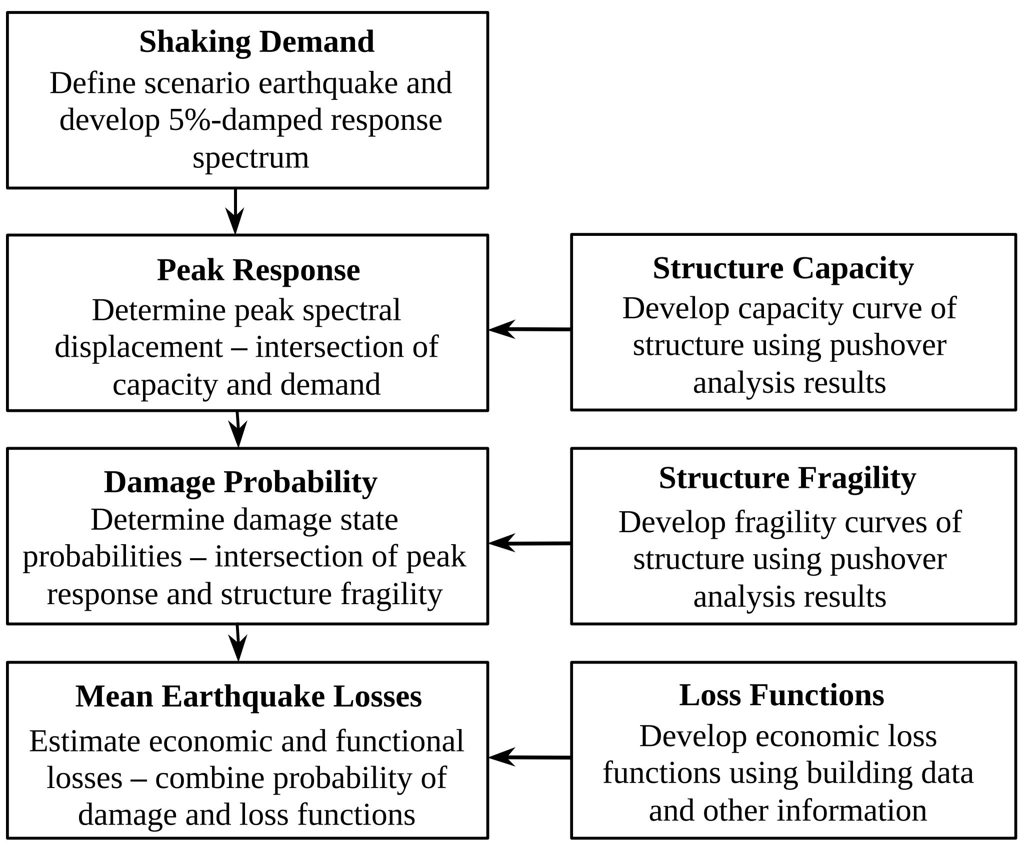

Figure B-1 Flowchart of Detailed Loss Estimation

Users are also expected to provide other information and data for the building. This can range from basic structural data obtained from the construction documents to results obtained from a nonlinear static analysis of the building, conducted in accordance with the guidelines of

Chapter 3. Section B.5 summarizes required input data to be supplied by the user. Subsequent sections provide guidance for developing structure capacity, structure response, structure fragility and building loss functions.

B.5 Required Data — User Input

The accuracy of loss estimates performed using the detailed methodology depends primarily on the extent and quality of the information provided by the user. While default data is provided and may be used if considered appropriate, the more effort the user puts into the determination of building data, the more reliable the results will be.

It is expected that the user will have seismic hazard data available. Although not required for development of damage and loss functions, seismic hazard data, including site soil conditions, are important and must be input by the user when developing loss estimates. It is also expected that, as a minimum, the user will have basic data on the building characteristics, such as the building size, occupancy (that is, use, rather than the number of occupants) and replacement cost.

Users are expected to calculate a pushover curve for the building at displacements up to complete failure of the structure. This may require pushing the building beyond the target displacement used in performance evaluation, as in Chapter 3, particularly if the evaluated performance objective was based on a low hazard level. The pattern of applied lateral loading should be based on the fundamental mode in the direction of interest and pushover results should represent both horizontal directions of building response (i.e., both principal axes of the building). If pushover results are significantly different for the two different directions, separate pushover curves should be developed and used to estimate losses for each direction. Three-dimensional models that permit rotation as well as translation should be used for pushover analysis of structures with plan irregularities that affect torsion.

Users are expected to have an understanding of the expected performance of the components of the structural system and the modes of failure as a function of building interstory drift. In addition to drift, Chapter 3 has identified other key performance parameters including column axial-load capacity and column tension-splice capacity that should be considered when determining at what drift level various failure modes and damage states are expected to occur.

Users are expected to provide the total replacement value of the structural system, expressed in terms of dollars/square foot. Although not required (default values are included in this appendix), users should also provide input on the repair of structural damage. That is, for each damage state, the user could review the associated damage to the structure and develop a cost and schedule for elements and components requiring repair. This may be done judgmentally, or more thoroughly by developing actual repair schemes, and obtaining estimates of, for example, construction costs, schedule, and building interruption.

B.5.1 Building Capacity Curve

The building capacity curve is derived from the pushover curve using modal properties for the building and a standard shape compatible with the *HAZUS* methodology. Specifically, the capacity curve is the pushover curve transformed from coordinates of base shear and roof displacement to coordinates of spectral acceleration (S_A) and spectral displacement (S_D). This coordinate transformation is accomplished on a point by point basis, by using the formulas:

$$S_{Di} = a_2 \Delta_i \tag{B-1}$$

$$S_{Ai} = \frac{V_i / W}{a_1} \tag{B-2}$$

where: a_1 = fraction of building weight effective in the fundamental mode in the direction under consideration (Equation B-3),
a_2 = fraction of building height at the elevation where the fundamental-modal displacement is equal to spectral displacement (Equation B-4),
D_i = displacement at point “i” on the pushover curve,
V_i = base shear force at point “i” on the pushover curve (kips),
W = building weight (kips),

and:

$$a_1 = \frac{\left(\sum_{i=1}^{N} (w_i f_{ip})/g\right)^2}{\left[\sum_{i=1}^{N} (w_i)/g\right]\left[\sum_{i=1}^{N} (w_i f^2_{ip})/g\right]} \tag{B-3}$$

$$a_2 = \frac{1}{PF_p f_{cp,p}} = \frac{\sum_{i=1}^{N} (w_i f^2_{ip})/g}{\left[\sum_{i=1}^{N} (w_i f_{ip})/g\right] f_{cp,p}} \tag{B-4}$$

where: w_i / g = mass assigned to the i^{th} degree of freedom,
f_{ip} = amplitude of modal shape at i^{th} degree of freedom,
$f_{cp,p}$ = amplitude of mode shape at control point,
N = number of degrees of freedom.

Some structural analysis software programs have the capability of automatically converting pushover curves to capacity curves using these formulas. As a simpler approximation to the formulas for a_1 and a_2 given above, these modal factors may be reasonably well estimated based only on the number of stories, N, using the following formula:

$$\frac{1}{a_1} \cong \frac{1}{a_2} \cong N^{0.14} \le 1.5 \tag{B-5}$$

In the *HAZUS* methodology, two control points define a standard shape for the capacity curve. These are the yield capacity control point and the ultimate capacity control point, as shown in Figure B-2. The yield point (normally designated by D_y, A_y) defines the limit of the elastic domain and the ultimate point (normally designated by D_u, A_u) defines the point along the curve where the structure is assumed to be fully plastic.

The user is expected to define capacity curve control points from the actual capacity curve using both judgment and the following rules:

- Yield capacity control point (D_y, A_y) is selected as the point where significant yielding is just beginning to occur (slope of capacity curve is essentially constant up to the yield point).
- The expected period, T_e, of the building, at or just below yield, should be the true "elastic" fundamental-mode period of the building:

$$T_e \cong 0.32\sqrt{\frac{D_y}{A_y}} \tag{B-6}$$

- The ultimate capacity control-point acceleration, A_u, is selected as the point of maximum spectral acceleration (maximum building strength), not to exceed the value of spectral acceleration at which the structure has just reached its full plastic capacity.
- The ultimate capacity control-point displacement, D_u, is selected as the greater of either the spectral displacement at the point of maximum spectral acceleration or the spectral displacement corresponding to Equation B-7:

$$D_u = 2D_y\frac{A_u}{A_y} \tag{B-7}$$

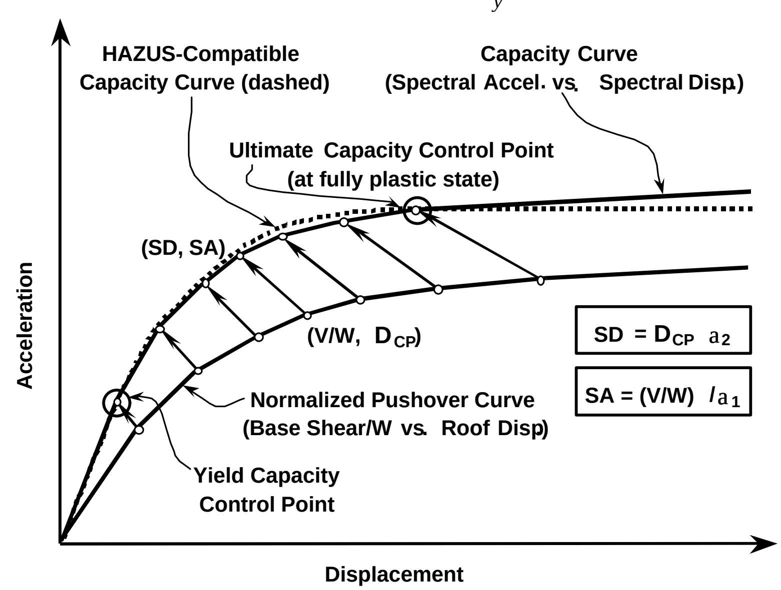

Figure B-2 Example Development of Standard (*HAZUS*-Compatible) Capacity Curve from a Normalized Pushover Curve

Commentary: The HAZUS definition of the elastic period T_e *is the same as the initial period, and must not be confused with the definition of the effective period* T_e *contained in FEMA-273. The effective period* T_e *of FEMA-273 is based on* *stiffness at 60% of the ultimate strength of the building and should not be used for loss estimation since it generally overestimates the displacement of the building.*

Table B-5 summarizes the elastic period and capacity curve control points for typical steel moment-frame buildings studied in this project. Capacity was derived from pushover analyses using modal properties based on Equation B-5. Building period and pushover properties were based on analyses reported in FEMA-355C and pertain to buildings conforming to the 1994 Uniform Building Code requirements. Individual buildings conforming to these same code provisions may be either stronger or weaker than those analyzed and buildings designed to other code requirements are likely to have substantially different characteristics than those indicated.

Table B-5 Capacity Curve Properties of Typical Welded Steel Moment-Frame Buildings

Capacity Parameter	Pre-Northridge Connections			Post-Northridge Connections		
	3-Story	9-Story	20-Story	3-Story	9-Story	20-Story
Buildings Located in Los Angeles						
Elastic Period (sec.)	1.01	2.24	3.74	1.02	2.21	3.65
Yield Point Disp. (in.)	2.6	8.0	11.7	2.7	7.7	11.1
Yield Point Accel. (g)	0.26	0.16	0.09	0.26	0.162	0.085
Ultimate Point Disp. (in.)	7.5	23	33	8.1	26	44
Ultimate Point Accel. (g)	0.37	0.23	0.12	0.40	0.27	0.167
Buildings Located in Seattle						
Elastic Period (sec.)	1.36	3.06	3.46	1.30	3.06	3.52
Yield Point Disp. (in.)	3.3	7.9	15.0	3.0	7.9	15.5
Yield Point Accel. (g)	0.18	0.09	0.13	0.18	0.086	0.128
Ultimate Point Disp. (in.)	9.3	22	43	12.0	25	48
Ultimate Point Accel. (g)	0.26	0.12	0.18	0.36	0.14	0.198
Buildings Located in Boston						
Elastic Period (sec.)	1.97	3.30	3.15	1.62	3.17	2.97
Yield Point Disp. (in.)	2.2	5.8	8.9	3.6	8.0	15.8
Yield Point Accel. (g)	0.058	0.054	0.091	0.140	0.082	0.183
Ultimate Point Disp. (in.)	7.1	20	33	10.2	29	47
Ultimate Point Accel. (g)	0.093	0.095	0.167	0.198	0.150	0.274

B.5.2 Structural Response

In the *HAZUS* methodology, structural response to ground motion is estimated based on elastic system properties modified using “effective” stiffness and damping properties of the structure to simulate inelastic response. Effective stiffness properties are based on secant stiffness at each displacement and effective damping is based on combined viscous and hysteretic measures of dissipated energy, assuming cyclic response of the structure to the given displacement. Effective damping greater than 5% of critical is then used to reduce spectral demand, in a manner similar to that followed in *ATC-40* (ATC, 1997).

Figure B-3 illustrates the process of developing an inelastic response (demand) spectrum from the 5%-damped elastic response (input) spectrum. The demand spectrum is based on elastic response divided by amplitude-dependent damping reduction factors (i.e., R_A at periods of constant acceleration and R_V at periods of constant velocity). In Figure B-3, the demand spectrum intersects the building's capacity curve at the point of peak building response (i.e., spectral displacement, *D*, and spectral acceleration, *A*). The amount of spectrum reduction typically increases for buildings that have reached yield and that dissipate hysteretic energy during cyclic response.

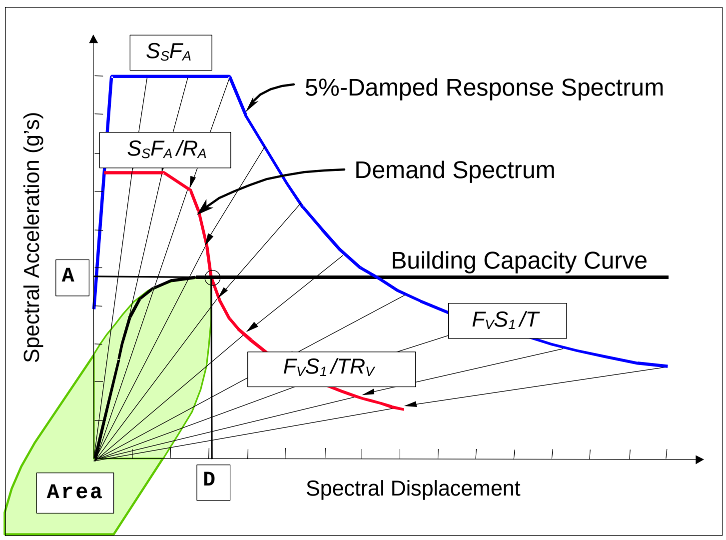

Figure B-3 Example Demand Spectrum Construction and Calculation of Peak Response Point (*D*, *A*)

Spectrum reduction factors are functions of the effective damping b_{eff} of the building as defined by Equations B-8 and B-9:

$$R_A = 2.12 / \left(3.21 - 0.68\, ln(\, b_{eff}\,)\right) \qquad \text{(B-8)}$$

$$R_V = 1.65 / \left(2.31 - 0.41\, ln(\, b_{eff}\,)\right) \qquad \text{(B-9)}$$

Effective damping b_{eff} is defined as the total energy dissipated by the building during peak earthquake response and is the sum of an elastic damping term b_E and a hysteretic damping term b_H associated with post-yield, inelastic response:

$$b_{eff} = b_E + b_H \qquad \text{(B-10)}$$

The elastic damping term b_E is assumed to be constant (i.e., amplitude independent) and represents response at, or just below, the yield point. For most steel moment-frame (WSMF) buildings the value of the elastic damping term should be taken as 5% of critical, assuming nonstructural components (e.g., cladding) help dampen the structure. The value of the elastic damping term should be taken as 3% of critical for bare steel frames or WSMF buildings with limited nonstructural damping.

The hysteretic damping term b_H is dependent on the amplitude of post-yield response and is based on the area enclosed by the hysteresis loop at peak building displacement D and acceleration A as shown in Figure B-3. Hysteretic damping b_H is defined in Equation B-11:

$$b_H = k\left(\frac{Area}{2p\,DA}\right) \tag{B-11}$$

where:
- *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, $\pm D$, assuming no degradation of components,
- D is the peak displacement response of the capacity curve,
- A is the peak acceleration response at the peak displacement, D
- k is a degradation factor that defines the fraction of the *Area* used to determine hysteretic damping.

The k (kappa) factor in Equation B-11 reduces the amount of hysteretic damping as a function of anticipated structure performance (e.g., connection condition) and shaking duration, to simulate degradation (e.g., pinching) of the hysteresis loop during cyclic response. Shaking duration is described qualitatively as either short, moderate or long, and is assumed to be primarily a function of earthquake magnitude, although proximity to fault rupture can also influence the duration of the level of shaking that is most crucial to building damage. For example, ground shaking close to the zone of fault rupture can be strong, but typically contains only a few strong pulses. Values of the degradation factor for typical WSMF buildings are suggested in Table B-6.

Table B-6 Values of the Degradation Factor k for Typical WSMF Buildings

Connection Condition	**Peak Response Amplitude and Post-Yield Shaking Duration**				
	At One-Half Yield	At or Below Yield	Post-Yield Shaking Duration		
			Short	Moderate	Long
Post-Northridge	1.0	1.0	1.0	0.9	0.7
Pre-Northridge	1.0	0.9	0.8	0.5	0.3
Damaged	1.0	0.7	0.6	0.3	0.1

As shown in Figure B-3, peak building displacement D is determined by the intersection of the capacity curve and the demand spectrum. The intersection requires either a graphical solution or a (spreadsheet) calculation that evaluates the area of the hysteresis loop as a function of amplitude. Alternatively, the target displacement of Section 3.4.5.3.1, divided by the modification factor α_2 calculated in accordance with Equation B-5 may used to estimate peak nonlinear spectral displacement of the building. In this case, the effective fundamental mode period, T_e, should be taken as equal to elastic fundamental-mode period T_i and the values of the coefficients C_1, R, C_2 and C_3 in Section 3.4.5 should be consistent with structural properties and the actual amount of nonlinear response corresponding to the target displacement.

B.5.3 Structure Fragility

Building fragility curves are lognormal functions that describe the probability of reaching, or exceeding, structural damage states, given deterministic (median) estimates of spectral displacement. These curves take into account the variability and uncertainty associated with structural response prediction, capacity curve properties, damage states and ground shaking. The fragility curves distribute damage among the Slight, Moderate, Extensive and Complete damage states. For any given value of spectral response, discrete damage-state probabilities are calculated as the difference of the cumulative probabilities of reaching, or exceeding, successive damage states. Discrete damage-state probabilities are used as inputs to the calculation of building-related losses.

Each fragility curve is defined by a median value of building response (i.e., spectral displacement) that corresponds to the threshold of that damage state and by the uncertainty associated with that damage state. The conditional probability of being in, or exceeding, a particular damage state ds, given the spectral displacement S_d (or other seismic demand parameter), is defined by Equation B-12:

$$\mathrm{P}\left[ds \middle| S_d\right] = \ddot{O}\left[\frac{1}{b_{ds}} \ln\left(\frac{S_d}{\hat{S}_{d,ds}}\right)\right] \tag{B-12}$$

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

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

F is the standard normal cumulative distribution function.

Development of damage-state medians requires users to:

- select specific values of maximum interstory drift of the structure that best represent the threshold of each of the discrete damage states (consistent with the descriptions of damage states provided in Section B.3), and
- convert damage-state threshold values (e.g., maximum interstory drift) to spectral displacement coordinates (i.e., same coordinates as those of the capacity curve).

Default values of maximum interstory drift that may be used for typical steel moment-frame buildings are provided in Table B-7. These values of drift are consistent with observations of damage and loss that occurred in the 1994 Northridge earthquake (pre-Northridge connection conditions) and with the interstory drift criteria of Section 3.6 for post-Northridge connection conditions. The values of drift given in Table B-7 do not necessarily reflect thresholds of damage states of buildings with significant plan or height irregularity. Buildings with a significant irregularity would be expected to have substantially smaller values of drift defining the thresholds of damage states.

Table B-7 Maximum Interstory Drift Values Defining Damage-State Thresholds of Typical WSMF Buildings

Connection Condition, Building Height and Location	Structural Damage State			
	Slight	Moderate	Extensive	Complete
Pre-Northridge – All Heights/Locations	0.01	0.015	0.025	0.04
Post-Northridge – 3-Story – Los Angeles	0.01	0.02	0.040	0.100
Post-Northridge – 9-Story – Los Angeles	0.01	0.02	0.040	0.080
Post-Northridge – 20-Story – Los Angeles	0.01	0.02	0.040	0.060
Post-Northridge – 3-Story – Seattle	0.01	0.0175	0.030	0.080
Post-Northridge – 9-Story – Seattle	0.01	0.0175	0.030	0.060
Post-Northridge – 20-Story – Seattle	0.01	0.0175	0.030	0.050
Post-Northridge – All Heights – Boston	0.01	0.015	0.025	0.04

Conversion of maximum interstory drift to damage-state medians is based on the building height and other factors and the following equation:

$$\hat{S}_{d,ds} = \frac{a_2 D_{ds} H_R}{a_3 a_{4,ds}} \qquad \text{(B-13)}$$

where:
$\hat{S}_{d,ds}$ = median spectral displacement value of damage state, *ds* (inches)
D_{ds} = maximum interstory drift ratio at the threshold of damage state *ds*, determined by user (e.g., typical building values of Table B-8)
H_R = height of building at the roof level (inches)
a_2 = pushover modal factor from Equation B-4 or Equation B-5
a_3 = higher-mode factor (Equation B-14)
$a_{4,ds}$ = mode-shape factor (Equation B-15)

The higher-mode factor, a_3, is the ratio of interstory drift due to all modes of vibration to the interstory drift of the fundamental (pushover) mode at the story with maximum fundamental-mode drift. The value of the higher mode factor may be determined by explicit calculation (e.g., ratio of peak drift values of response history and pushover analyses), or may be approximated based on the number of stories, *N*, and the following formula:

$$a_3 \cong N^{0.14} \leq 1.5 \qquad \text{(B-14)}$$

The mode-shape factor, $a_{4,ds}$, is the ratio of maximum fundamental-mode (pushover-mode) interstory drift to the average pushover-mode interstory drift (i.e., average drift over all stories). Maximum pushover-mode interstory drift is the value of drift of those stories contributing to the damage state of interest. For tall buildings with Slight structural damage of a localized nature, maximum pushover-mode interstory drift is simply the drift of the story with the most displacement. As the extent of the damage increases (with damage state) or the building height decreases, or both, the difference between maximum pushover-mode interstory drift and average pushover-mode interstory drift decreases. The value of the mode-shape factor is 1.0 for Complete damage, since damage would typically be pervasive throughout the building. The value of the mode-shape factor may be determined directly from the shape of the pushover mode or may be approximated based on the number of stories, *N*, the following formula:

$$a_{4,ds} \cong N^{0.10} \qquad \text{(B-15)}$$

Limits of $a_{4,S} \leq 1.5$ for Slight damage, $a_{4,M} \leq 1.25$ for Moderate damage, $a_{4,E} \leq 1.1$ for Extensive damage, and $a_{4,C} \leq 1.0$ for Complete damage are suggested.

Lognormal standard deviation (b) values describe the total uncertainty inherent in the fragility-curve damage states. Three primary sources contribute to the total uncertainty of any given state, namely, the uncertainty b_C associated with the capacity curve, the uncertainty b_D associated with the demand spectrum, and the uncertainty $b_{T,ds}$ associated with the discrete threshold of each damage state. Since the demand spectrum is dependent on building capacity, a convolution process is required to combine their respective contributions to total uncertainty. To avoid this rather complex calculation, the *Procedures for Developing HAZUS-Compatible Building-Specific Damage and Loss Functions* (Kircher, 1999) provides pre-calculated values of total damage-state uncertainty for different values of capacity, demand and damage state variability. Users may refer to this document when developing values of damage-state uncertainty or use the b values given in Table B-8 for typical steel moment-frame (WSMF) buildings.

Table B-8 Structural Damage-State Variability (b) Factors of Typical WSMF Buildings

Building Location	Pre-Northridge Connections			Post-Northridge Connections		
	3-Story	9-Story	20-Story	3-Story	9-Story	20-Story
Los Angeles	0.90	0.85	0.80	0.70	0.65	0.60
Seattle	0.95	0.90	0.85	0.75	0.70	0.65
Boston	0.95	0.90	0.85			

Commentary: The structural damage state uncertainty factors b given in Table B-8 include a large, dominant contribution to the total variability from the variability associated with ground shaking demand. A large amount of ground shaking variability is appropriate when the fragility functions are to be used to estimate damage and loss for a scenario earthquake characterized by median predictions of ground shaking. Ground shaking uncertainty accounts for the inherent differences between actual and median predictions of ground shaking. The structural damage state uncertainty factors b given in Table B-8 would not be appropriate for estimating damage when ground shaking is actually known, or for estimating probabilistic losses that include ground shaking variability directly in the hazard function.

B.5.4 Loss Functions

Loss functions convert damage to loss by taking the sum over all four damage states of the products of the probability that a building will be damaged within a given damage state multiplied by the expected loss given that the damage state is experienced. In the case of economic loss, the expected losses can be normalized by dividing by the total replacement value to obtain an estimate of the mean loss ratio.

As discussed in Section B.4, users are expected to provide economic loss data in terms of the value of the building (structure), and the costs and associated construction time that would be required to repair Slight, Moderate and Extensive damage. These loss parameters would most appropriately be based on estimated costs of repair schemes developed to correct Slight, Moderate and Extensive damage, as predicted by a performance evaluation (pushover analysis) of the structure. Alternatively, default economic loss ratios are provided at the end of this section for typical steel moment-frame (WSMF) buildings.

Repair and replacement costs are the expected dollar costs (per square foot) that would be required to repair (or replace) damaged structural elements. In general, the cost of the structural system (and related repairs) will vary based on building occupancy (for example, hospital structures cost more per square foot then industrial buildings).

Commentary: Some consideration should be given to prevailing codes and ordinances that would govern the repair work. Do prevailing regulations require strengthening as well as repair?

Replacement value is the preferred measure of direct economic loss, although other measures could be used, such as loss of market value. Market value would, in general, produce entirely different loss estimates. For example, an older building of no special importance or historical significance is to be vacated and completely renovated, but instead an earthquake occurs and destroys the structure. Should economic loss be based on the replacement value (e.g., cost of a new building of comparable size and function), on the near zero value of the existing building, or on the market value of the building (which would also *include value of the land)? These types of question are crucial to the estimation of economic loss, but are beyond the scope of this section. For steel moment-frame (WSMF) buildings, economic loss functions used here are based on repair and replacement value of the structure, consistent with HAZUS methodology.*

Table B-9 provides mean structural repair costs (loss ratios and corresponding loss rates) for damage states of typical WSMF buildings. These rates are based on a number of assumptions. First, typical WSMF buildings are assumed to have a total replacement value of $125/sq. ft. and the structure is assumed to be worth 20% of total building value ($25/sq. ft.).

Inspection costs of 5% of the cost of the structural system are included in the loss ratios and loss rates for buildings with Pre-Northridge connection conditions. The 5% value is based on an assumed inspection cost of $1,500 per connection and the assumption that on average about one-half of the connections of these types of buildings would be inspected following an earthquake. The cost of repair of damaged connections is assumed to be $20,000 per connection. On the basis of this amount, the cost of repairing all connections would be about one and one-half times the cost of a new structural system.

The cost of repair of Slight damage to buildings with Post-Northridge connection conditions is assumed to be zero on the basis that, for example, minor distortion of flanges, or other incidental structural damage would not require repair. The cost of repair of Moderate and Extensive structural damage of typical WSMF buildings is assumed to be 10% and 50% of the value of the structural system. However, the actual repair cost of a specific building could be very different due, for example, to the building's configuration, and the repair's interference with nonstructural systems and finishes.

Table B-9 Mean Structural Loss Ratios and Rates of Typical WSMF Buildings

Building Connection Condition	**Structural Damage State**			
	Slight	**Moderate**	**Extensive**	**Complete**
Mean Structural Loss Ratio (Repair Cost / Replacement Cost)				
Pre-Northridge	8%	20%	80%	100%
Post-Northridge	0%	10%	50%	100%
Mean Structural Loss Rates (Dollars per Square Foot)				
Pre-Northridge	$2.00	$5.00	$20.00	$25.00
Post-Northridge	$0.00	$2.50	$12.50	$25.00

Repair time is the time required for cleanup and construction to repair or replace damage to the structural system. Recovery time is the time required to make repairs, considering, for example, delays in decision-making, financing, and inspection, and typically takes much longer than the actual time of repair. Loss of function is the time that the facility is not available for use and is typically less than repair (recovery) time. Loss of function is less than repair time due to temporary solutions, such as the use of alternative space, or simply because buildings with Slight or Moderate damage can remain partially or fully operational while repairs are made. Table B-10 provides time for cleanup and construction, and loss of function multipliers for typical steel moment-frame (WSMF) buildings (mixed occupancy). The loss-of-function multipliers represent the fraction of the repair time for each damage state that the building would not be functional.

Table B-10 Cleanup and Construction Time and Loss-of-Function Multipliers for Typical WSMF Buildings

Building Connection Condition and Height	Structural Damage State			
	Slight	**Moderate**	**Extensive**	**Complete**
Mean Time of Repairs in Days (Cleanup and Construction)				
Pre-Northridge – 3-Story	5	30	90	180
Post-Northridge – 3-Story	0	20	90	180
Pre-Northridge – 9-Story	10	50	180	360
Post-Northridge – 9-Story	0	40	180	360
Pre-Northridge – 20-Story	15	75	240	480
Post-Northridge – 20-Story	0	60	240	480
Loss-of-Function Multipliers (Fraction of Building Cleanup and Construction Time)				
All Buildings	0.0	0.1	0.3	1.0

Commentary: The values given in Table B-10 are based on the default values of HAZUS adjusted for building height (size) and include time required for inspection of WSMF buildings with pre-Northridge connection conditions. HAZUS cleanup and repair times and the fractions of repair time that the building will not be functional vary widely, depending on the occupancy of the building. Values given in Table B-10 are considered appropriate for most commercial office buildings. In contrast to HAZUS default values, Slight structural damage was assumed to have no impact on building function (loss-of-function multiplier is equal to 0.0, in all cases), since structural inspections and repair of connections can typically be made while the building is in operation. The loss-of-function multiplier for Complete structural damage is 1.0, and assumes that the building is closed and that alternative space is not available.

B.6 Example Loss Estimates

This section develops example estimates of losses for typical 9-story Los Angeles buildings, designed to conform to the 1994 *Uniform Building Code*. Three building types are considered: (1) buildings with pre-Northridge connection conditions, (2) buildings with post-Northridge connection conditions, and (3) buildings with damaged pre-Northridge connection conditions. The example considers three levels of earthquake ground shaking that represent the Maximum Considered Earthquake (MCE), the Design Earthquake (DE) and one-half of the DE (½ DE) for regions of high seismicity (e.g., Los Angeles). The example first estimates peak building response (spectral displacement) as the intersection of building capacity curves and earthquake demand spectra. Building fragility damage and loss functions are then developed using default parameters of typical 9-story building properties provided in previous sections. Finally, mean building loss functions are developed as a function of building spectral displacement that illustrate a range of losses for MCE, DE, ½ DE, and other levels of spectral demand.

Commentary: The user is expected to have available an estimate of scenario earthquake ground shaking at the building site. Such an estimate may be obtained from site-specific hazard studies or from the 1997 USGS/NEHRP spectral contour design maps. For this example, 5%-damped response spectra were developed from the spectral contour maps representing a typical Los Angeles stiff soil site (Soil Profile Type D), not near an active fault. MCE ground shaking represents a sufficiently large magnitude event of long shaking duration that its approximate return period is between 1,000 to 2,500 years. The DE and ½ DE represent ground shaking of a large magnitude event and moderate magnitude event, respectively with approximate return periods of 500 and 100 years, respectively. Most of the steel moment-frame (WSMF) buildings damaged by the 1994 Northridge earthquake felt ground shaking that ranged between the ½ DE and DE levels illustrated in this example.

Figure B-4 shows the 5%-damped spectrum of the ½ DE, the capacity curves of buildings with pre-Northridge and post-Northridge connection conditions (solid symbols), and the demand curves of buildings with pre-Northridge, post-Northridge and damaged pre-Northridge connection conditions (open or shaded symbols).

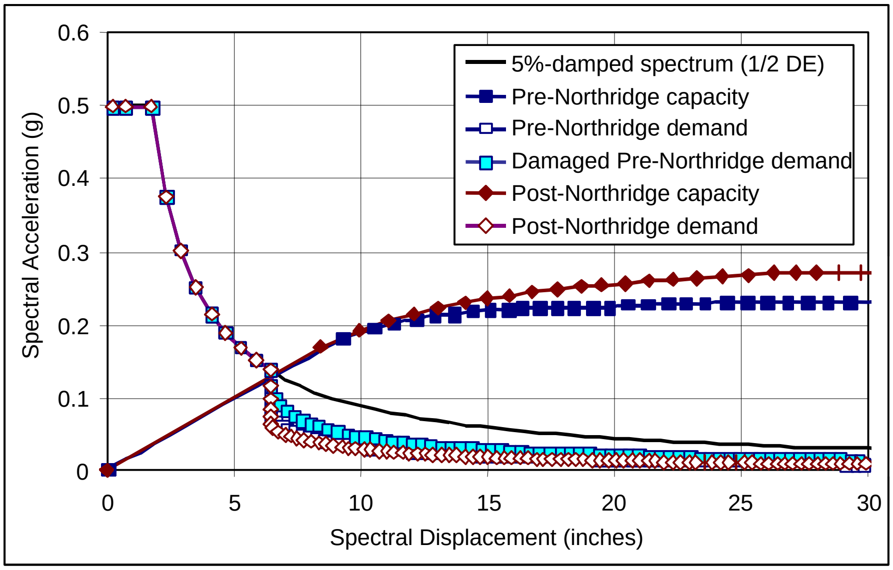

Figure B-4 Demand and Capacity of Typical 9-Story WSMF Buildings – Ground Shaking of ½ the Design Earthquake

The properties of the capacity curves are based on the yield and ultimate control points given in Table B-5. The demand spectra were constructed from the 5%-damped spectrum as described in Section B.5.2. The intersection points of demand and capacity curves indicate that spectral displacement of the building is about 6.5 inches for each building type. Figures B-5 and B-6 repeat the process and illustrate the determination of building spectral displacement for Design Earthquake (DE) and Maximum Considered Earthquake (MCE) ground shaking, respectively.

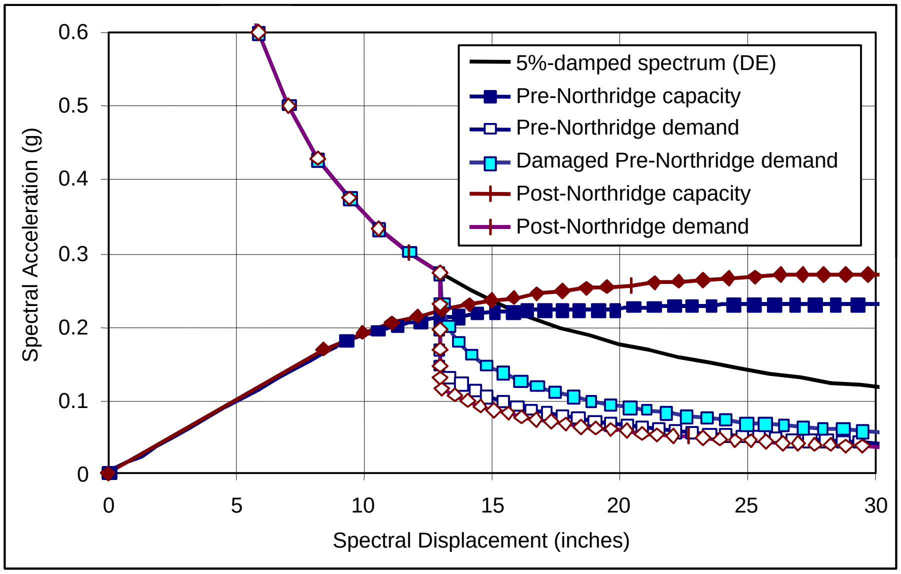

Figure B-5 Demand and Capacity of Typical 9-Story WSMF Buildings – Design Earthquake Ground Shaking

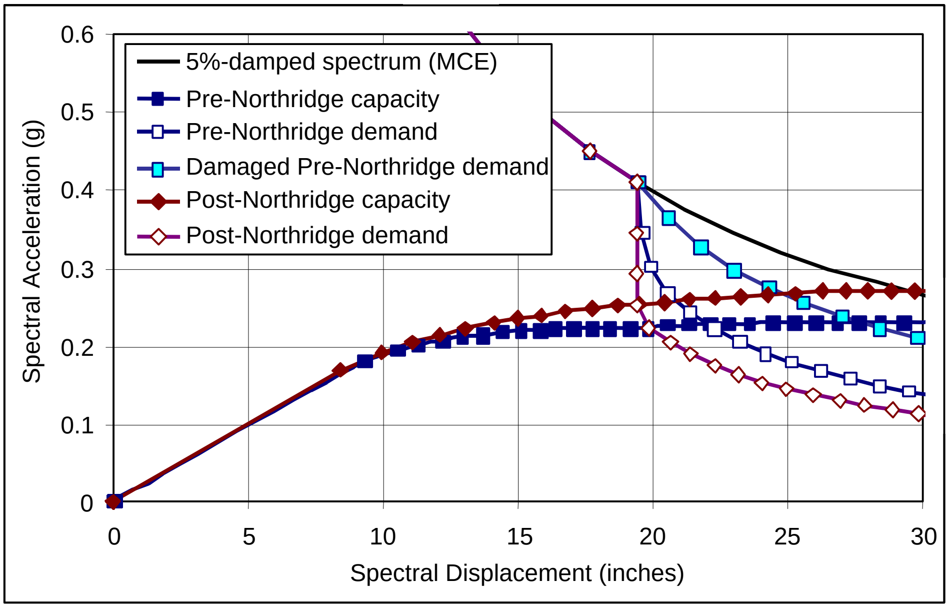

Figure B-6 Demand and Capacity of Typical 9-Story WSMF Buildings – Maximum Considered Earthquake Ground Shaking

Figure B-6 shows different Maximum Considered Earthquake (MCE) intersection points (i.e., different values of building spectral displacement) for the three building types. In particular, buildings with damaged pre-Northridge connection conditions are expected to degrade more than buildings with undamaged connections during the long duration of (post-yield) ground shaking associated with the MCE (k = 0.1, Table B-6) and hence are expected to displace farther.

Table B-11 provides a summary of the predicted peak building response parameters for each of the three earthquake ground shaking levels. Spectral displacement is used later in this section to estimate structural damage and loss. Table B-11 shows spectral displacement values converted to corresponding estimates of average interstory drift, 1st-mode only, average interstory drift including higher modes, and maximum interstory drift including higher modes. Average interstory drift applies to all stories over the height of the building; maximum interstory drift applies to the story experiencing the most displacement. Estimates of drift are based on the height of the building (H = 122 feet) and the factors a_2, a_3 and $a_{4,ds}$, defined in Section B.5.3.

Table B-11 Summary of Peak Response – Typical 9-Story WSMF Buildings

	Ground Shaking Level – Connection Condition				
	½ DE	DE	MCE – Long Duration		
Peak Response Parameter	**All**	**All**	**Post-NR**	**Pre-NR**	**Damaged**
Spectral Displacement (in.) SD	6.5	13	19.5	22	27.5
Average Interstory Drift - 1st-Mode (SD/H) x $1/\alpha_2$	0.006	0.012	0.018	0.020	0.026
Average Interstory Drift - All Modes (SD/H) x $1/\alpha_2$ x α_3	0.008	0.016	0.025	0.028	0.035
Maximum Interstory Drift – All Modes (SD/H) x $1/\alpha_2$ x α_3 x $\alpha_{4,S}$	0.010	0.021	0.031	0.035	0.043

Figure B-7 illustrates structural fragility curves for the example 9-story steel moment-frame (WSMF) Los Angeles buildings with post-Northridge connection conditions. These curves are constructed using Equation B-12 and the fragility parameters defined in Section B.5.3. Figure B-8 illustrates discrete damage-state probabilities for the same buildings. These curves are calculated as the difference in probability between adjacent damage-state fragility curves shown in Figure B-7. At each value of spectral displacement, the sum of discrete damage-state probabilities is equal to the probability of Slight or greater structural damage and the compliment of Slight or greater damage is the probability of no structural damage. The considerable overlap of discrete damage-state curves shown in Figure B-8 is a measure of the relatively large uncertainty in the prediction of damage and is due primarily to the inherent uncertainty in the prediction of ground shaking.

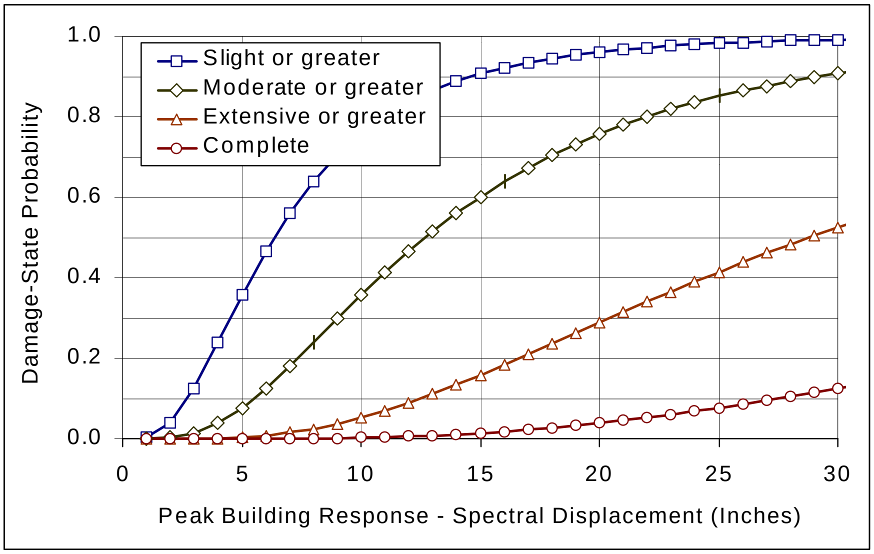

Figure B-7 Structural Fragility Curves – Typical 9-Story Los Angeles Buildings with Post-Northridge Connection Conditions

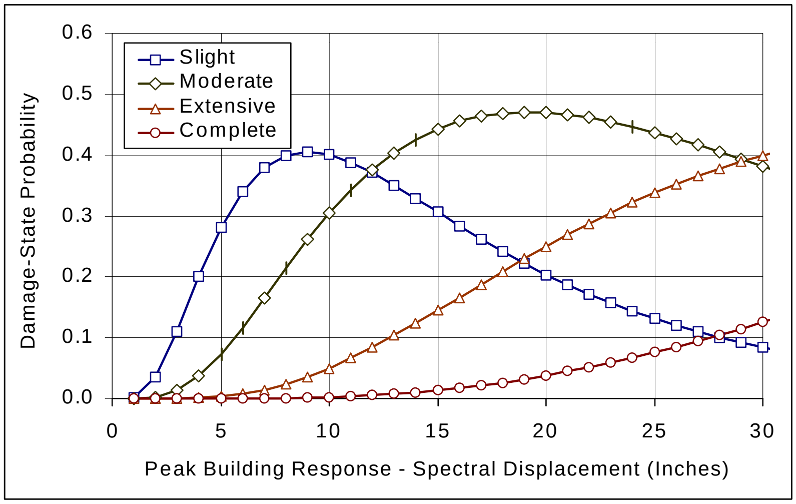

Figure B-8 Discrete Damage-State Probability Curves – Typical 9-Story Los Angeles Buildings with Post-Northridge Connection Conditions

Figure B-9 illustrates mean structural loss rates for the structural system of typical 9-story steel moment-frame (WSMF) Los Angeles buildings, expressed as a function of building spectral displacement. Structural loss ratios are shown for buildings with pre-Northridge and post-Northridge connection conditions to compare the typical reduction in postearthquake repair cost that would be expected for buildings with improved connections. Structural loss rates are the same for WSMF buildings with pre-Northridge connection conditions, with or without damage to connections, although buildings with damaged connections could, depending on the level and duration of ground shaking, experience larger spectral displacement and hence greater loss.

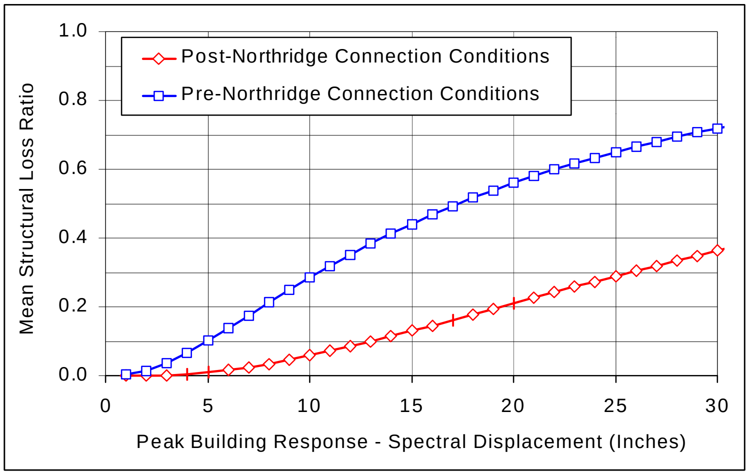

Figure B-9 Mean Structural Loss Ratio Curves – Typical 9-Story WSMF Los Angeles Buildings

Mean structural loss rate curves are constructed by first multiplying discrete damage-state probabilities, shown in Figure B-8, by the mean structural loss rates given in Table B-9, and then by summing the products over all damage states. Multiplying mean loss rates by the cost of the structural system produces mean estimates of the repair cost (including inspection cost for buildings with pre-Northridge connection conditions). For the typical 9-story buildings, the cost off the structural system is assumed to be about $5 million (i.e., 20% x $125/sq. ft. x 200,000 sq. ft.). Estimates of mean structural loss are made by finding the loss rate corresponding to the spectral displacement of the earthquake of interest (e.g., spectral displacement values given in Table B-11).

The results represent mean (or best) estimates of loss rates (rather than a complete distribution of loss), since loss rates represent mean (point estimates) of loss, given damage. Considering the rather large variability associated with damage estimates (which would only be made larger by considering loss uncertainty), actual loss for any given building could be significantly different than the mean estimate. The large uncertainty inherent in the fragility curves is reflected in the moderate slope of the curve for structural loss. At lower levels of loss, loss function tapers to zero gradually with decrease in building spectral displacement. Fragility uncertainty is primarily due to the uncertainty associated with median estimates of ground shaking. Actual ground shaking could be significantly higher (or lower) than the median and this uncertainty tends to broaden the loss functions, increasing estimates of loss at the low end and decreasing estimates of loss at the high end (which is typically beyond Maximum Considered Earthquake (MCE) demand).

The mean structural loss rate curves shown in Figure B-9 are plotted to spectral displacements of 30 inches, a displacement corresponding to an extremely rare level of earthquake ground shaking. Peak building spectral displacements likely to occur during the life of the building would not be expected to exceed the ½ DE level of ground shaking (i.e., about 6 inches, or less, of spectral displacement). Figure B-10 is a re-plot of Figure B-9 data to a spectral displacement of 10 inches. This figure shows that structural repair (and inspection) costs are likely not to exceed 13% of the cost of the structural system ($650,000 loss) on average during the life of the building. A loss of 13% is consistent with structural repair (and inspection) costs for steel moment-frame (WSMF) buildings damaged in the 1994 Northridge earthquake. The figure also shows that repair costs would likely not exceed 2% ($100,000 loss) on average for WSMF buildings with post-Northridge connection conditions. For comparison, a typical real estate transaction fee for a 200,000 square foot building, based only on the replacement value of the building (i.e., excluding the value of the land), would be in excess of $1 million each time the building is sold.

Figure B-11 illustrates mean functional loss (in days) due to damage of the structural system of typical 9-story WSMF Los Angeles buildings, expressed as a function of building spectral displacement. Functional loss is shown for buildings with pre-Northridge and post-Northridge connection conditions to compare the typical reduction in "downtime" for buildings with improved connections. Functional loss is the same for WSMF buildings with pre-Northridge connection conditions, regardless of connection damage, although buildings with damage connections could, depending on the level and duration of ground shaking, experience larger spectral displacement and hence greater loss.

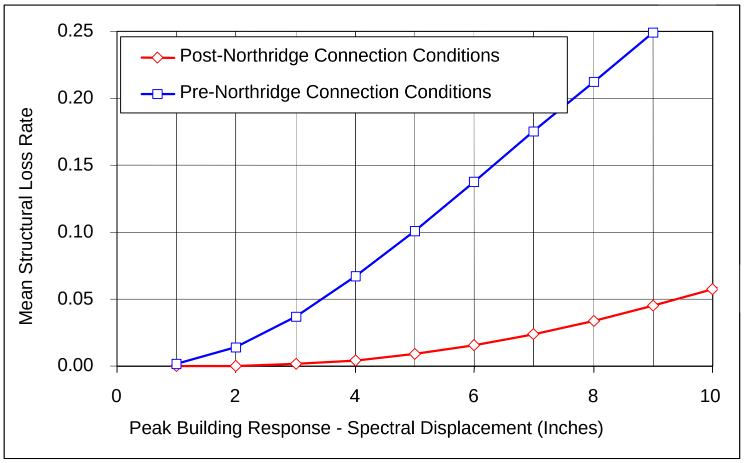

Figure B-10 Mean Structural Loss Rate Curves – Typical 9-Story WSMF Los Angeles Buildings

Mean loss of function curves are constructed by first multiplying discrete damage-state probabilities, shown in Figure B-8, by the product of the cleanup and construction time and the loss-of-function multipliers of Table B-10. Estimates of mean loss of function are made by finding the loss of time corresponding to the spectral displacement of the earthquake of interest (e.g., spectral displacement values given in Table B-11).

Mean loss of function (in days) is the probabilistic combination of short downtime due to Slight or Moderate structural damage, and long downtime due to Extensive or Complete structural damage. Complete damage is assumed to close the building for about the time it would take to build a new one (360 days for a 9-story WSMF building). Since the loss-of-function multipliers are very small for Slight or Moderate damage (repairs can usually be made while the building is in operation), loss function is dominated by the probability of Extensive or Complete structural damage that would likely close the building for an extended period of time. While mean estimates of loss of function are valid as the average of many buildings, actual downtime of specific building could range from no loss of function to long-term building closure. It may make more sense for users to convert mean loss of function (in days) to a probability of long-term building closure by dividing the mean days of downtime by maximum down time associated with Complete structural damage. For example, a building with post-Northridge connection conditions is expected to have about 18 days of downtime due to Design Earthquake (DE) ground shaking. Actual downtime would likely be considerably less, provided the building did not sustain damage sufficient to warrant long-term closure (e.g., a red tag). In this case, the probability of long-term closure is about 5% (i.e., mean loss estimate of 18 days divided by 360 days of loss associated with Complete damage).

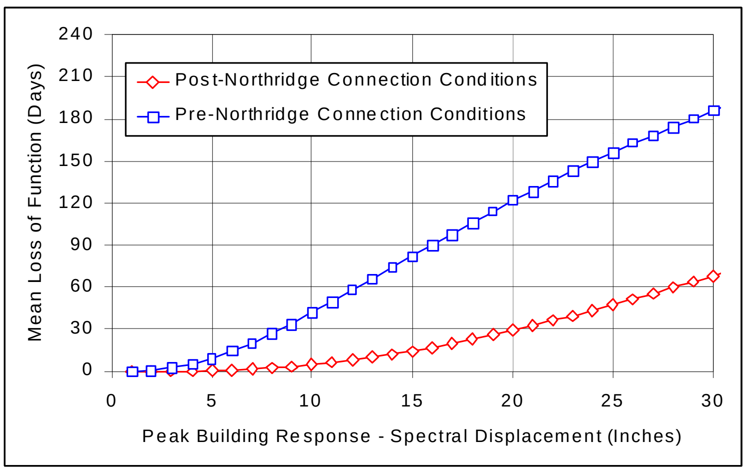

Figure B-11 Mean Loss of Function Curves – Typical 9-Story WSMF Los Angeles Buildings