Chapter 3 – Hazus Methodology

3.1 The Methodology of HAZUS

Hazards U.S. (HAZUS), is a standardized, nationally applicable earthquake loss estimation methodology that uses PC-based GIS software. HAZUS contains an extensive inventory of data that can help you conduct your loss estimation in a timely, cost-efficient manner.

Hazus manual describes procedures for developing building-specific damage ans loss functions with the Advanced Engineering Building Module (AEBM).

What is AEBM? AEBM are procedures to an extension of the more general methods of FEMA/NIBS earthquake loss estimation methodology (HAZUS) and provides damage and loss functions compatible with current HAZUS MH software.

Hazus damage and loss functions for generic model building types are considered to e reliable predictors of earthquake effects for large groups of buildings that include both median and below (an important advantage). A disadvantage, represents the fact that, they may not be very good predictors for a specific building or a particular type of building that it is known to have an inherent weakness or earthquake vulnerability.

For mitigation purposes, it is desirable that users may be able to create building-specific damage and loss functions that could be used to asses losses for an individual building or group of similar buildings. Building-specific damage and loss functions are based on the properties of a particular building. The particular building of interest could me either an individual building or a typical building representing a group of buildings of an archetype.

3.2 What is HAZUS?

HAZUS is the commonly name for FEMA/NIBS (Federal Emergency Management Agency/National Institute of Building Sciences), an earthquake loss estimation methodology.

HAZUS represents a complex set of components that work together to estimate casualties, loss of function and economic impacts on a region due to an scenario earthquake.

One of the main components of the methodology estimates the probability of various states of structural and non structural damage to buildings. Damage state probabilities are used by other components of the methodology to estimate various types of building-related loss. Currently , HAZUS includes building damage for 36 model building types . Each building type represents a "generic" group of buildings that share a common type of construction and a common seismic design level.

Damage and loss functions for generic building types are considered to be reliable predictors of earthquake effects for large group of buildings that include both above median and below (an important advantage). A disadvantage, represents the fact that, they may not be very good predictors for a specific building or a particular type of building that it is known to have an inherent weakness or earthquake vulnerability.

The HAZUS is intended for exclusive use within the US territory. This is a stand-alone software which bears the code with the mathematical algorithms for the calculations, but as a background to run a software Geographic Information Systems (Geographical Information System - GIS).

3.3 Overview of Methodology

The flow of the HAZUS methodology between those modules related to building damage and loss it is illustrated in the Figure 3.1 Inputs to the estimation of building damage include ground shaking an ground failure, characterized by permanent ground deformation (PGD) due to settlement and lateral spreading. Here it is described building-specific methods for estimating damage and loss due to ground shaking, typically the dominant contributor to building-related losses.

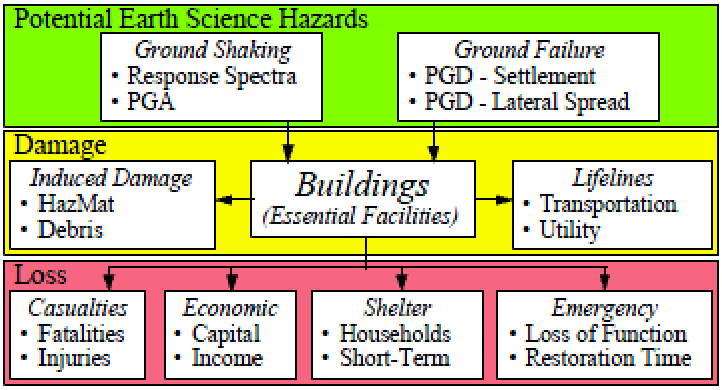


Fig 3.1 Building-Related Modules of the FEMA/NIBS Methodology

Most importantly, building damage is used as an input to a number of loss modules, including the estimation of casualties, direct economic losses, displace households and short–term shelter needs and loss of emergency facility function and the time require to restore functionality.

HAZUS damage functions for ground shaking have two basic components

1. Capacity curves and
2. Fragility curves.

The capacity curves are based on engineering parameters (yield an ultimate strength), that characterize the nonlinear (pushover) behavior of 36 different model types. For each of these building types, capacity parameters distinguish between different levels of seismic design and anticipated seismic performance.

The fragility curves describe the probability of damage to the building's :

- Structural system
- Nonstructural components sensitive to drift
- Nonstructural components (and contents) sensitive to acceleration.

For a given level of building response, fragility curves distribute damage between four physical damage state: Slight, Moderate, Extensive and Complete.

Earthquake loss due to building damage is based on the physical damage states that are deemed to e the most appropriate and significant contributors to that particular type of loss. As an example, deaths are based on the Complete state of damage, since partial or complete collapse of the building is assumed to dominate this type of loss. In contrast, direct economic loss, is accumulated from all the states of damage to both structural and nonstructural systems, since all are significant contributors to economic loss.

3.4 Building Classification

Buildings are classified both in terms of their use, or occupancy class, and in terms of their structural system, or model building type. Damage is predicted based on model building type, since the structural system is considered the key factor in assessing overall building performance, loss of function and casualties. Occupancy class is important in determining economic loss, since building value is primarily a function of building use (e.g., hospitals are more valuable than most commercial buildings, primary because of their expensive nonstructural systems and contents, not because of their structural systems).

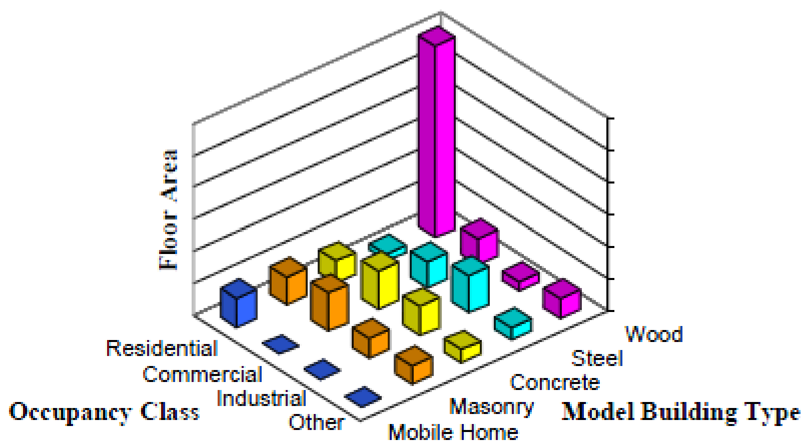


Fig3.2. Inventory Relationship of Model Building Type and Occupancy class

Thirty three occupancy classes are defined to distinguish among residential, commercial, industrial or other buildings; and 36 model building types are used to classify buildings within the overall categories of wood, steel, concrete, masonry or mobile homes. Building inventory data relate model building type and occupancy class on the basis of floor area, as illustrated in Fig3.2 so that for a given geographical area the distribution of the total floor area of model building types is known for each occupancy class.

No.	Label	Description	Height			
			Range		Typical	
			Name	Stories	Stories	Feet
1	W1	Wood, Light Frame (≤ 5,000 sq. ft.)		All	1	14
2	W2	Wood, Greater than 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	URM M		Mid-Rise	3+	3	39
36	MH	Mobile Homes		All	1	12

Table 3.1 Model Building Types of HAZUS

3.5 Structural and nonstructural systems and contents

Buildings are composed of both structural (load carrying) and nonstructural systems (architectural and mechanical components). While damage to the structural system is the most important measure of building damage affecting casualties and catastrophic loss of function (due to unsafe conditions), damage to nonstructural systems and contents tends to dominate economic loss. Typically, the structural system represents about 25% of the building's worth.

To better estimate different types of loss, building damage functions separately predict damage to:

- The structural system
- Drift-sensitive nonstructural components (partition walls, are primarily affected by building displacement)
- Acceleration-sensitive nonstructural components (suspended ceilings, affected by building shaking

Building contents are also considered to be acceleration sensitive. Distinguishing between drift and acceleration-sensitive nonstructural components and contents, permits more realistic estimates of damage considering building response. The below Table 3.2 shows a list of the typical drift-sensitive and acceleration-sensitive components and building components.

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

Tab.3.2 HAZUS Classification of Drift-Sensitive and Acceleration-Sensitive Nonstructural Components and Building Contents

3.6 Damage states

Damage states are defined separately for structural and nonstructural systems of a building. Damage is described by one of four damage states: Slight, Moderate, Extensive or

Complete and Collapse as a subset of complete structural damage. The actual building damage varies as a continuous function of earthquake demand.

Ranges of damage are used to describe building damage, since it is not practical to have a continuous scale and damage states provide the ser with an understanding of the building's physical condition. Loss functions relate the physical condition of the building to various loss parameters (direct economic loss, casualties, and loss of function).

The four damage states of the FEMA/NIBS methodology are similar to the damage states defined in "Expected Seismic Performance of Buildings", except that damage descriptions vary for each model building type based on the type of structural system and material. The Table 3.3 provides structural damage states for W1 buildings (light frame wood) typical of the conventional construction used for single-family homes.

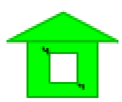
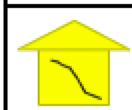
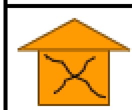
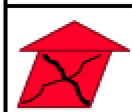
Damage State		Description
	Slight	Small plaster cracks at corners of door and window openings and wall-ceiling intersections; small cracks in masonry chimneys and masonry veneers. Small cracks are assumed to be visible with a maximum width of less than 1/8 inch (cracks wider than 1/8 inch are referred to as "large" cracks).
	Moderate	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	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.
	Complete	Structure may have large permanent lateral displacement or be in imminent danger of collapse due to cripple wall failure or failure of the lateral load resisting system; some structures may slip and fall off the foundation; large foundation cracks. Three percent of the total area of buildings with Complete damage is expected to be collapsed, on average.

Table 3.3 Example Damage States – Light-Frame Wood Building (W1)

3.7 Building Capacity Curve

A building capacity curve is a plot of a building's lateral load resistance as a function of a characteristic lateral displacements (for example force-displacement plot). It is derived from a plot of static-equivalent base shear versus building displacement at the roof, known commonly as a pushover curve. In order to facilitate direct comparison with spectral demand, base shear is converted to spectral acceleration and the roof displacement to spectral displacement using modal properties that represent pushover response.

Building capacity curves are constructed for each model building type and represent different levels of lateral force design and for a given loading condition, expected building performance. Each curve is defined by two controlled points:

1. The "yield" capacity and
2. The "ultimate" capacity

The yield capacity – represents the lateral strength of the building and accounts for design strength, redundancies in design, conservatism in code requirements and expected (rather than nominal) strength of materials. Design strengths of model building types are based on the requirements of current model seismic code provision (1994 UBC or NEHRP Provisions) or on an estimate of lateral strength for buildings not designed for earthquake loads. Certain buildings designed for wind, such as taller buildings located in zones of low or moderate seismicity, may have a lateral design strength considerably greater than those based on seismic code provisions.

The ultimate (plastic) capacity – represents the maximum strength of the building when the global structural system has reached a full mechanism. Typically, a building is assumed capable of deforming beyond its ultimate point without loss of stability, but its structural system provides no additional resistance to lateral earthquake force. Up to yield, the building capacity curve is assumed to be linear with stiffness based on an estimate of the expected period of the building. From yield 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 is show below with the help of the Figure 3.3

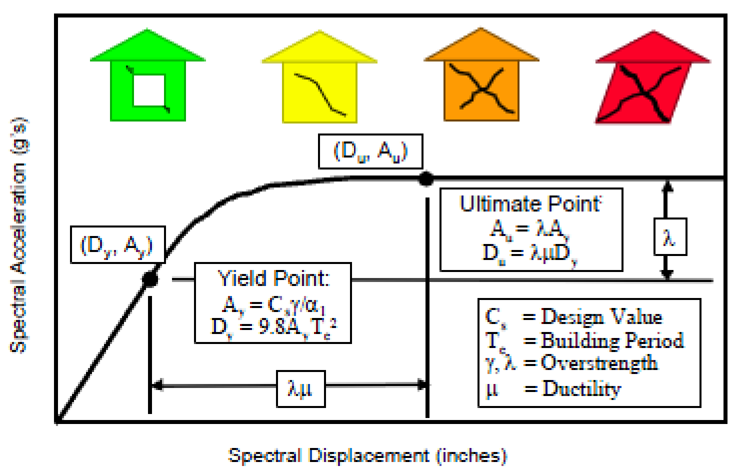


Fig.3.3 Example Building Capacity curve and Control Points

The following parameters define the yield point and the ultimate point of capacity curves as shown in Fig 3.3 :

C_s point of significant yielding of design strength coefficient (fraction of building's weight)

T_e expected "elastic" fundamental – mode period of building (seconds)

α_1 fraction of building weight effective in the pushover mode

α_2 fraction of building height at the elevation where pushover-mode displacement is equal to spectral displacement

γ "overstrength" factor relating "true" yield strength to design strength

λ "overstrength" factor relating ultimate strength to yield strength

μ "ductility" ratio relating ultimate displacement to λ times the yield displacement (assumed point of significant yielding of the structure)

3.8 Building Response Calculation

Building response is determined by the intersection of the demand spectrum and the building capacity curve. Intersections are illustrated in the figure,,, for three example demand spectra representing what can be considered as weak, medium and strong ground shaking and two building capacity curves representing weak and stronger construction, respectively. As show in the figure below (Fig. 3.4), stronger and stiffer construction displaces less than weaker and more flexible constructions for the same level of spectral demand and less damage is expected to the structural system and nonstructural components sensitive to drift. In contrast, stronger (and stiffer) construction will shake at higher acceleration levels, and more damage is expected to nonstructural components and contents sensitive to acceleration .

The demand spectrum is based on the 5% - damped response spectrum at the building's site (or center of a study area containing a group of buildings), reduced for effective damping when effective damping exceeds the 5% damping level of the input spectrum.

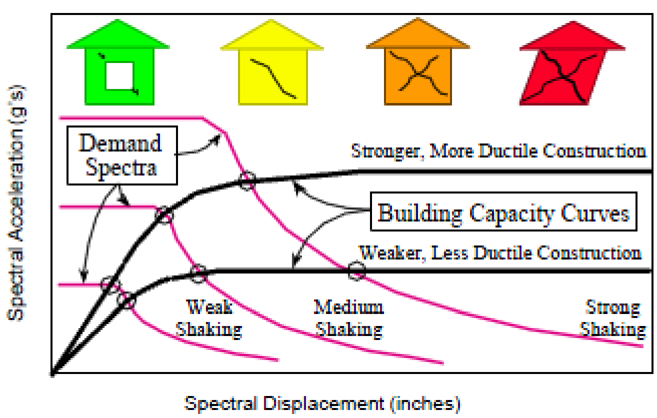


Fig 3.4 Example Intersection of Demand Spectra and building Capacity Curves

3.9 Building Fragility Curves

Building fragility curves are lognormal functions that describe the probability of reaching, or exceeding, structural or nonstructural damage states, given median estimates of spectral response, for example spectral displacement. These curves take into account the variability and uncertainty associated with capacity curve properties, damage states and ground shaking.

The below figure (Fig..3.5) provides an example of fragility curves for the four damage states used in the FEMA/NIBS methodology and illustrates differences in damage-state probabilities for three levels of spectral response corresponding to weak , medium, and strong earthquake ground shaking, respectively. The terms “weak”, “medium”, ”strong” are only for simplicity, in reality only quantitative values of spectral response are used.

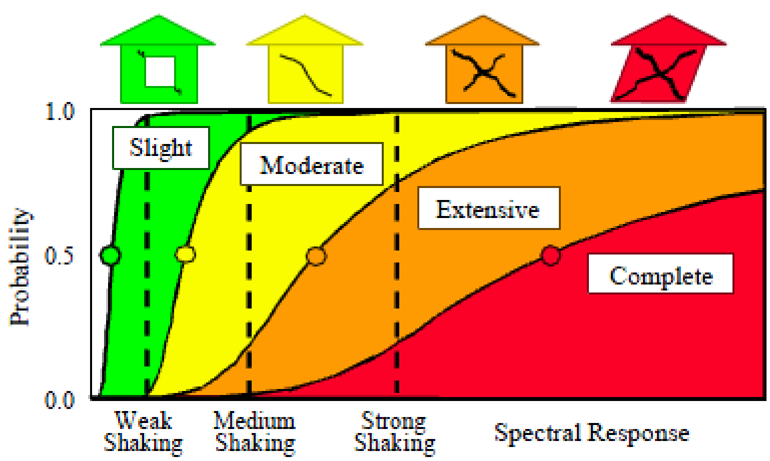


Figure 3.5 Example Fragility Curves for Slight, Moderate, Extensive and Complete Damage

The fragility curves distribute damage among 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. The probabilities of a building reaching or exceeding the various damage levels at a given response level sum to 100%. Discrete damage-state probabilities are used as inputs to the calculations of various types of building-related loss. Figure 3.6 provides an example of discrete damage state probabilities for the three levels of earthquake ground shaking.

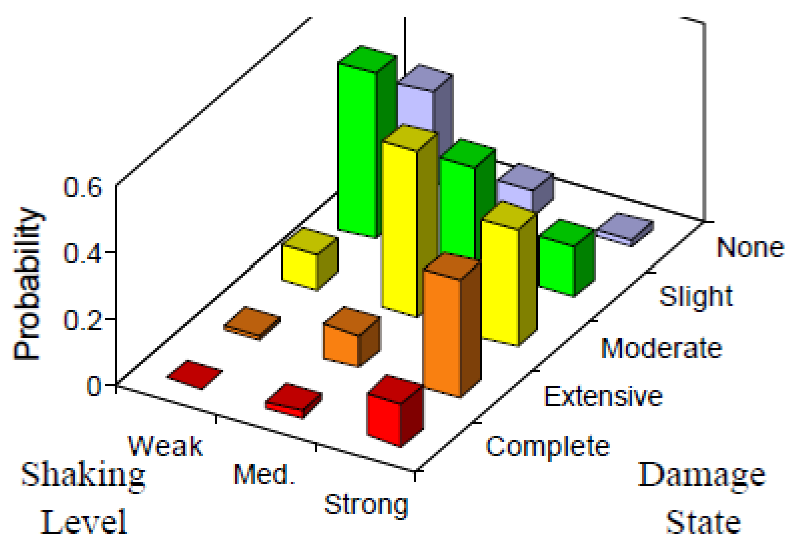
Each fragility curve is defined by a median value of the demand parameter (e.g, spectral displacement) that corresponds to the threshold of that damage state and by the variability associated with that damage state. For example, the spectral displacement, Sd, that defines the threshold of a particular damage state (ds) is given in Equation 3.1

$$S_d = \overline{S}_{d,ds}\ \varepsilon_{ds} \qquad \text{(Equation 3.1)}$$

Where:

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

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



Fig, 3.6 Example Damage –State Probabilities for Weak, Medium and Strong Shaking Levels

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 , respectively [Kennedy, et. Al.,1980] . In this formulation, uncertainty represents the component of the variability that could theoretically be reduced with improved knowledge; whereas, randomness represents the inherent variability (in response) that cannot be eliminated, even with perfect knowledge. Since it is not considered practical o separate uncertainty from randomness , the combined variability, β, is used to develop a composite "best-estimate" fragility curve.

The conditional probability of being in, or exceeding, a particular damage state, ds, given the spectral displacement, Sd. (or other seismic demand parameter) is defined by the Equation 3.2:

$$P[ds|Sd] = \Phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \quad \text{(Equation 3.2)}$$

$S_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, ds.

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

Φ is the standard normal cumulative distribution function.

3.10 Development of capacity curves and response parameters

3.10.1 Building model and pushover criteria

This sections gives guidance for the development of the capacity curves and related parameters that are used by Advanced Building Engineering Module (AEBM). It's purpose is to calculate the building response as a function of ground shaking at the building site.

The pushover analysis must appropriately represent the force-deflection and response characteristics of the building of interest. For use in developing fragility functions, the pushover analysis must appropriately capture the damage patterns of elements and components of the building.

Users must determine how many different pushover models are required for loss estimation. For complex buildings, a model could be developed for each horizontal direction of response (if response is different in different directions) and for separate structural segments of the building.

It is common for large buildings (in plain) to be composed of more than one structure, separated by construction joints . Each structure can have different capacity and response properties (and fragility and loss functions). For example, symmetrical buildings, a single pushover model would be likely to be sufficient to represent the building behavior. If a single pushover model is used to evaluate a complex and/or irregular building, then the model would

need to represent those modes of response and failure that are most likely to occur and cause damage and loss.

3.10.2 Pushover Analysis using SAP

The static pushover analysis is becoming a popular tool for seismic performance evaluation of existing and new structures. The expectation is that the pushover analysis will provide adequate information on seismic demands imposed by the design ground motion on the structural system and its components. The purpose of the paper is to summarize the basic concepts on which the pushover analysis can be based, assess the accuracy of pushover predictions, identify conditions under which the pushover will provide adequate information and, perhaps more importantly, identify cases in which the pushover predictions will be inadequate or even misleading.

Necessity of Non-Linear Static Pushover Analysis (NLSA)

The existing building can become seismically deficient since seismic design code requirements are constantly upgraded and advancement in engineering knowledge. Further, Indian buildings built over past two decades are seismically deficient because of lack of awareness regarding seismic behaviour of structures. The widespread damage especially to RC buildings during earthquakes exposed the construction practices being adopted around the world, and generated a great demand for seismic evaluation and retrofitting of existing building stocks.

3.10.3 What is Pushover Analysis?

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces.

A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. On a building frame, and plastic rotation is monitored, and lateral inelastic forces versus displacement response for the complete structure is analytically computed.

This type of analysis enables weakness in the structure to be identified. The decision to retrofit can be taken in such studies.
The seismic design can be viewed as a two step process. The first, and usually most important one, is the conception of an effective structural system that needs to be configured with due regard to all important seismic performance objectives, ranging from serviceability considerations. This step comprises the art of seismic engineering. The rules of thumb for the strength and stiffness targets, based on fundamental knowledge of ground motion and elastic and inelastic dynamic response characteristics, should suffice to configure and rough-size an effective structural system.

Elaborate mathematical/physical models can only be built once a structural system has been created. Such models are needed to evaluate seismic performance of an existing system and to modify component behavior characteristics (strength, stiffness, deformation capacity) to better suit the specified performance criteria.

The second step consists of the design process that involves demand/capacity evaluation at all important capacity parameters, as well as the prediction of demands imposed by ground motions. Suitable capacity parameters and their acceptable values, as well as suitable methods for demand prediction will depend on the performance level to be evaluated.
The implementation of this solution requires the availability of as set of ground motion records (each with three components) that account for the uncertainties and differences in severity, frequency characteristics, and duration due rapture characteristics distances of the various faults that may cause motions at the site. It requires further the capability to model adequately the cyclic load-deformation characteristics of all important elements of the three dimensional soil foundation structure system, and the availability of efficient tools to implement the solution process within the time and financial constraints on an engineering problem.

3.10.4 The purpose of Pushover Analysis

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest.

The evaluation is based on an assessment of important performance parameters, including global drift, interstory drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element connection forces (for elements and connections that cannot sustain inelastic deformations), The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behavior.

The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics:

- ✓ The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry walls, piers, etc.
- ✓ Estimates of the deformations demands for elements that have to form inelastically in order to dissipate the energy imparted to the structure.
- ✓ Consequences of the strength deterioration of individual elements on behavior of structural system.
- ✓ Identification of the critical regions in which the deformation demands are expected to be high and that we have to become the focus through detailing.
- ✓ Identification of the strength discontinuities in plan elevation that will lead to changes the dynamic characteristics in elastic range.
- ✓ Estimates of the inter story drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
- ✓ Verification of the completeness and adequancy of load path, considering all the elements of the structural system, all the connections, the stiff nonstructural elements of significant strength, and the foundation system.

The last item is the most relevant one as the analytical model incorporates all elements, whether structural or non structural, that contribute significantly to the lateral load distribution.

Load transfer through across the connections through the ductile elements can be checked with realistic forces; the effects of stiff partial-height infill walls on shear forces in columns can be evaluated; and the maximum overturning moment in walls, which is often limited by the uplift capacity of foundation elements can be estimated.

These benefits come at the cost of the additional analysis effort, associated with incorporating all important elements, modeling their inelastic load-deformation characteristics, and executing incremental inelastic analysis, preferably with three dimensional analytical models.

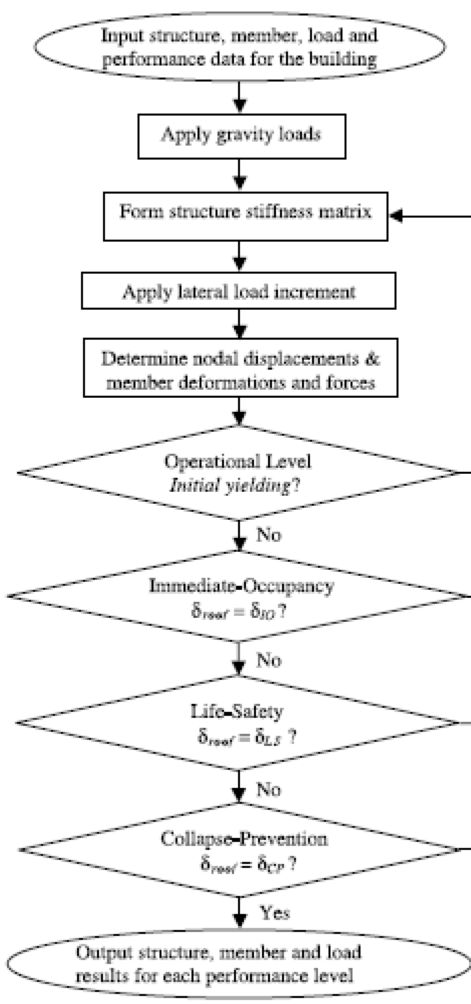

Input structure, member, load and performance data for the building
Apply gravity loads
Form structure stiffness matrix
Apply lateral load increment
Determine nodal displacements & member deformations and forces
Operational Level
Initial yielding?
Yes
No
Immediate-Occupancy
$\delta_{roof} = \delta_{IO}$?
Yes
No
Life-Safety
$\delta_{roof} = \delta_{LS}$?
Yes
No
Collapse-Prevention
$\delta_{roof} = \delta_{CP}$?
No
Yes
Update axial forces
Update plasticity-factors
Output structure, member and load results for each performance level

Fig 3.7 Pushover flow chart

3.10.5 Target displacement

The fundamental question in the execution of the pushover analysis is the magnitude of the target displacement at which seismic performance evaluation of the structure is to be performed.

The target displacement serves as an estimate of the global displacement of the structure is expected to experience in a design earthquake. It is the roof displacement at the center of mass of the structure.

In the pushover analysis it is assumed that the target displacement for the MDOF structure can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the SDOF domain through the use of a shape factor. This assumption, which is always an approximation, can only be accepted within limitations and only be accepted within limitations and only if great care is taken in incorporating in the predicted SDOF displacement demand all the important ground motion and structural response characteristics that significantly affect the maximum displacement of the MDOF structure. Inherent in this approach is the assumption that the maximum MDOF displacement is controlled by a single shape factor without regards to the higher mode effects.

Under the Non-linear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined.

The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake.

3.11 Adaptability of computer programs

It is well known fact the distribution of mass and rigidity is one of the major considerations in the seismic design of moderate to high rise buildings. Invariably these factors introduce coupling effects and non-linearities in the system, hence it is imperative to use non-linear static analysis approach by using specialized programs like SAP2000, ETABS, STAADPRO2005, IDARC, NISA-CIVIL etc, for cost effective seismic evaluation and retrofitting of buildings.

3.12 Nonlinear static analysis for buildings

Seismic analysis of buildings can be categorized depending upon the sophistication of modeling adopted for the analysis.

Buildings loaded beyond the elastic range can be analyzed using Non-Linear static analysis, but in this method one would not be able to capture the dynamic response, especially the higher mode effects.

This is pushover analysis. There is no specific code for NLSA. This procedure leads to the capacity curve which can be compared with design spectrum/DCR of members and one can determine whether the building is safe or needs strengthening and its extent.

The capacity of structure is represented by pushover curve. The most convenient way to plot the load deformation curve is by tracking the base shear and the roof displacement. The pushover procedure can be presented in various forms can be used in a variety of forms for the use in a variety of methodologies. As the name implies it is a process of pushing horizontally, with a prescribed loading pattern, incrementally, until the structure reaches the limit state. There are several types of sophistication that can be used over for pushover curve analysis.

Level 1 : It is generally used for **single storey building**, where at a single concentrated horizontal force equal to base shear applied at the top of the structure and displacement is obtained.

Level 2: In this level, lateral force in proportion to storey mass is applied at different floor levels, and story drift is obtained.

Level 3: In this method lateral force is applied in proportion to the product of storey masses and first mode shape elastic model of the structure. The pushover curve is constructed to represent the first mode response of structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This procedure is valid for tall buildings with fundamental period of vibration up to 1 sec.

Level 4: This procedure is applied to soft storey buildings, wherein lateral force in proportion to product of storey masses and first mode of shape of elastic model of the structure, until first yielding, the forces are adjusted with the changing the deflected shape.

Level 5: This procedure is similar to level 3 and level 4 but the effect of higher mode of vibration in determining yielding in individual structural element are included while plotting the pushover curve for the building in terms of the first mode lateral forces and displacements. The higher mode effects can be determined by doing higher mode pushover analysis. For the higher modes, structure is pushed and pulled concurrently to maintain the mode shape.

3.13 Case study on SAP

The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

The ATC-40 and FEMA-273 documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown in Figure 1, five points labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge and three points labeled IO, LS and CP are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA-273 documents.

Here are presented the steps used in performing a pushover analysis of a simple three-dimensional building. SAP2000, a state-of-the-art, general-purpose, three-dimensional structural analysis program, is used as a tool for performing the pushover. The SAP2000 static pushover analysis capabilities, which are fully integrated into the program, allow quick and easy implementation of the pushover procedures prescribed in the ATC-40 and FEMA-273 documents for both two and three-dimensional buildings.

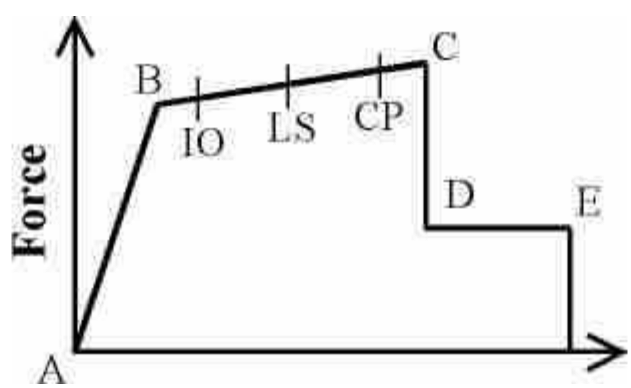


Fig. 3.9 Idealized Component Load versus Deformation Curve (from NEHRP Guidelines)

The following steps are included in the pushover analysis. Steps 1 through 4 discuss creating the computer model, step 5 runs the analysis, and steps 6 through 10 review the pushover analysis results.

1. Create the basic computer model (without the pushover data) in the usual manner using the graphical interface of SAP2000 makes this a quick and easy task.

2. Define properties and acceptance criteria for the pushover hinges as shown in Figure.

3. The program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members. These built in properties can be useful for preliminary analyses, but user-defined properties are recommended for final analyses. This example uses default properties.

4. Locate the pushover hinges on the model by selecting one or more frame members and assigning them one or more hinge properties and hinge locations.

5. Define the pushover load cases. In SAP2000 more than one pushover load case can be run in the same analysis. Also a pushover load case can start from the final conditions of another pushover load case that was previously run in the same analysis.

Typically the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final conditions of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a certain defined force level, or they can be displacement controlled, that is, pushed to a specified displacement.

Typically a gravity load pushover is force controlled and lateral pushovers are displacement controlled. SAP2000 allows the distribution of lateral force used in the pushover to be based on a uniform acceleration in a specified direction, a specified mode shape, or a user-defined static load case. Here how the displacement controlled lateral pushover case that is based on a user-defined static lateral load pattern named PUSH is defined for this example.

6. Run the basic static analysis and, if desired, dynamic analysis. Then run the static nonlinear pushover analysis.

7. Display the pushover curve . The File menu shown in this display window allows you to view and if desired, print to either a printer or an ASCII file, a table which gives the coordinates

of each step of the pushover curve and summarizes the number of hinges in each state as defined in Figure 1 (for example, between IO and LS, or between D and E).

8. Display the capacity spectrum curve. Note that you can interactively modify the magnitude of the earthquake and the damping information on this form and immediately see the new capacity spectrum plot. The performance point for a given set of values is defined by the intersection of the capacity curve (green) and the single demand spectrum curve (yellow). Also, the file menu in this display allows you to print the coordinates of the capacity curve and the demand curve as well as other information used to convert the pushover curve to Acceleration-Displacement Response Spectrum format.

9. Review the pushover displaced shape and sequence of hinge formation on a step-by-step basis . The arrows in the bottom right-hand corner of the screen allow you to move through the pushover step-by- step. Hinges appear when they yield and are color coded based on their state (see legend at bottom of screen).

10. Review member forces on a step-by-step basis . Often it is useful to view the model in two side-by-side windows with the step-by-step displaced shape in one window and the step-by-step member forces in the other. These windows can be synchronized to the same step, and can thus greatly enhance the understanding of the pushover
results.

11. Output for the pushover analysis can be printed in a tabular form for the entire model or for selected elements of the model. The types of output available in this form include joint displacements at each step of the pushover, frame member forces at each step of the pushover, and hinge force, displacement and state at each step of the pushover.

For buildings that are being rehabilitated it is easy to investigate the effect of different strengthening schemes. The effect of added damping can be immediately seen on the capacity spectrum form. You can easily stiffen or strengthen the building by changing member properties and rerunning the analysis. Finally you can easily change the assumed detailing of the building by modifying the hinge acceptance criteria and rerunning the analysis.

3.14 Limitations of Non-Linear Static Analysis

There are many unsolved issues that need to be addressed through more research and development. Examples of the important issues that need to be investigated are:

1. Incorporation of torsional effects (due to mass, stiffness and strength irregularities).
2. 3-D problems (orthogonality effects, direction of loading, semi-rigid diaphragms, etc)

3. Use of site specific spectra.
4. Cumulative damage issues.
5. Most importantly, the consideration of higher mode effects once a local mechanism has formed.

Since the pushover analysis is approximate in nature and is based on static loading, as such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that occur in a structure subjected to severe earthquakes, and it may significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

3.15 Conclusions and references

From the study of above model example discussed following conclusions can be obtained:

1. There are good reasons for advocating the use of the inelastic pushover analysis for demand prediction, since in many cases it will provide much more relevant information that an elastic static or even dynamic analysis, but it would be counterproductive to advocate this method as a general solution technique for all cases;

2. The pushover analysis is a useful, but not in fallible, tool for accessing inelastic strength and deformation demands and for exposing design weaknesses.

3. Its foremost advantage is that it encourages the design engineer to recognize important seismic response quantities and to use sound judgment concerning the force and deformation demands and capacities that control the seismic response close to failure, but it needs to be recognized that in some cases it may be provide a false feeling of security if its shortcomings and pitfalls are not recognized.

4. It must be emphasized that the pushover analysis is approximate in nature and is based on static loading. As such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that may occur in a structure subjected to severe earthquakes, and it may exaggerate others. Inelastic dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

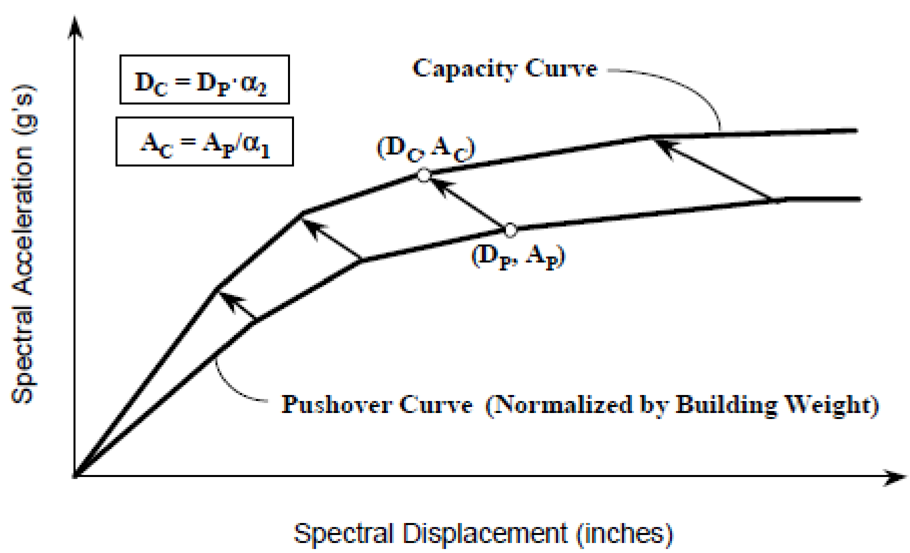
5. Thus performance of pushover analysis primarily depends upon choice of material models included in the study.

3.16 Development of capacity curve control points

3.16.1 Conversion of Pushover Curve to Capacity Curve

The first step in developing capacity curve control points it to convert pushover coordinates of base shear force and control point (for example the roof) displacement to spectral acceleration and displacement respectively.

The conversion of pushover curve is illustrated in Figure3.10. An example pushover curve (normalized by the building's weight W) is converted to capacity using pushover mode factors α_1 and α_2. Each point on the normalized pushover curve (D_p, A_p) is factorized by the pushover mode factors to create a corresponding point on the capacity curve (D_c, A_c). Provided the pushover curve was developed by using a push fore pattern based on the 1st mode shape of the building, then the initial (pre-yield) slope of the capacity curve is directly related to the building's elastic (pre-yield) period T_e, described by the equation A. Axes are labeled in terms of Spectral Acceleration and Spectral Displacement in Fig 3.10 recognizing that while pushover and capacity curves can have the same units, they are in different coordinate system.



Fig,3.10,Example Conversion of Pushover Curve to Capacity curve Using Pushover Mode factors

HAZUS defines the two pushover factors:

α_1 fraction of building weight effective in pushover mode

α_2 fraction of building height at the elevation where pushover-mode displacement is equal to spectral displacement.

Consistent with ATC-40 methods and terms, α_1 is defined by the distribution of building mass and pushover mode shape:

$$a_1 = \frac{\left[\sum_{i=1}^{N}(w_i\phi_{ip})/g\right]^2}{\left[\sum_{i=1}^{N}(w_i)/g\right]\left[\sum_{i=1}^{N}(w_i\phi^2_{ip})/g\right]} \qquad \text{(Equation 3.3)}$$

Where:

W_t/g represents the mass assigned to the i^{th} degree of freedom

φ_{ip} represents the amplitude of pushover mode at i^{th} degree of freedom.

Typically the shape of the pushover mode is based on the 1^{st} mode of the building in the direction of interest. In general, the pushover mode shape is amplitude dependent, after elements and components begin to yield. While the most appropriate pushover shape would be the amplitude-dependent shape at the amplitude of interest, the pre-yield(1^{st} mode) shape may be used to calculate $\boldsymbol{\alpha_1}$ without significant loss of accuracy. This statement does not apply to element/component demands that are directly related to the post-yield changes to pushover mode shape. The term “degree of freedom” is used herein, rather than the term “level” of ATC-40, to indicate that there may be more than one node (degree of freedom) per floor (e.g., buildings with flexible diaphragms would need several nodes to represent diaphragm response).

Consistent with ATC-40, the modal factor $\boldsymbol{\alpha_2}$, is defined by amplitude of the normalized pushover mode shape at the control point and the pushover mode participation factor:

$$a_2 = \frac{1}{PF_p f_{cp,p}} = \frac{\sum_{i=1}^{N} (w_i \phi^2_{ip})/g}{\left[\sum_{i=1}^{N} (w_i \phi_{ip})/g\right] f_{cp,p}}$$ (Equation 3.4)

Where:

W_t/g represents the mass assigned to the i^{th} degree of freedom

φ_{ip} represents the amplitude of pushover mode at i^{th} degree of freedom.

$\Phi_{cp,p}$ represents the amplitude of pushover mode at control point

Typically the roof is used as the location of the control point. The shape of the pushover mode is typically based on the 1^{st} mode of the building in the direction of interest and is in general amplitude dependent after elements and components begin to yield. As for the $\boldsymbol{\alpha}_1$ term, the most appropriate pushover shape would be the amplitude-dependent shape at the amplitude of interest, but the pre-yield (1^{st} mode) shape may be used to calculate $\boldsymbol{\alpha}_2$ in most cases without significant loss of accuracy.

The pushover mode factors are used directly to calculate capacity curve from the pushover curve where each point on the capacity curve is defined by a spectral displacement SD, and a spectral acceleration, SA:

$$SD = \alpha_2 \times \Delta_{cp}$$ (Equation 3.5)

$$SA = \frac{V/W}{\alpha_1}$$ (Equation 3.6)

Where:

Δ_{cp} Pushover control point (roof) displacement

V Pushover base shear force (kips)

W building weight (kips)

Certain structural analysis software programs (SAP2000 Nonlinear) automatically convert pushover curves to capacity curves using these formulas.

3.16.2 Yield and Ultimate Capacity Control Points

Capacity curve control points are determined from the 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 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:

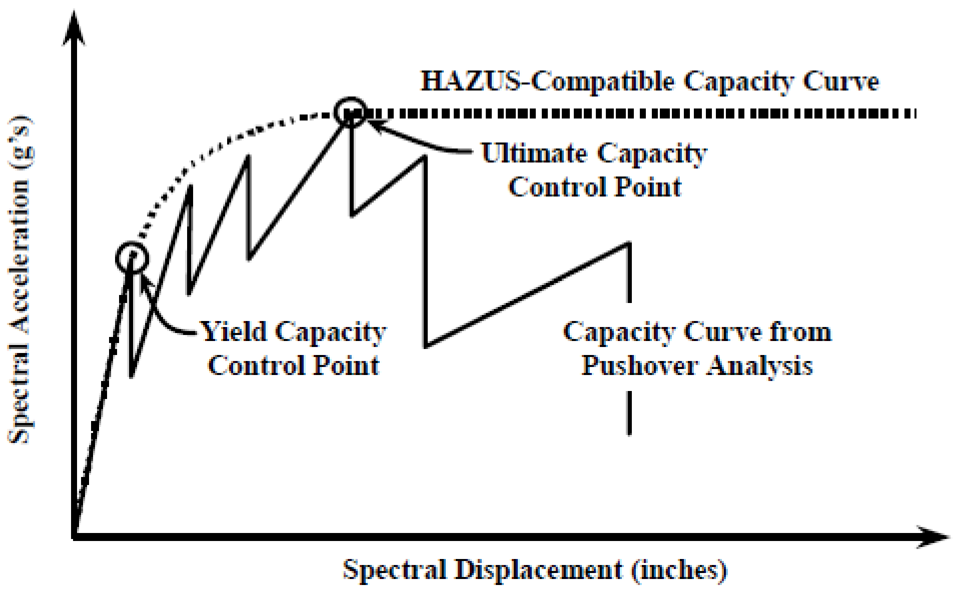
$$T_e \cong 0.32\sqrt{\frac{D_y}{A_y}} \qquad \text{(Equation 3.7)}$$

- 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 (i.e., ignore additional straining at the point at which the structure becomes a mechanism).
- 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 of the below equation (Eq.3.8)

$$D_u = 2 \cdot D_y \cdot \frac{A_u}{A_y} \qquad \text{(Equation 3.8)}$$

The HAZUS definition of the of the elastic period, T_e , is the same as the initial period, T_i, of the NEHRP *Guidelines* and should not be confused with the definition of the effective period, T_e , if the NEHRP *Guidelines.* The effective period T_e, of the NEHRP *Guidelines* is based on stiffness at 60% of the ultimate strength of the building and should not be used with HAZUS methods since it could significantly overestimate pre-yield displacement of the building.

Three sets of pushover capacity curves and the Control Points selected for each using the rules described above are shown in the Figures 3.11, 3.12 and 3.13 respectively. As shown in these figures, capacity curves typically extend beyond "ultimate" control point displacement, D_u, which defines the displacement at which the system is assumed to be fully plastic, but has not necessarily failed. The median values of fragility curves, described in the next section, define various states of damage along the HAZUS-compatible capacity curve.



Fig, 3 11 Example Development of the Capacity Curve for a Structure with Saw-Tooth Force-Deflection Behavior

In figure 3.11 , the first set of curves is for a structure that sustains shear failure and load reduction in a number of components at different levels of spectral displacement. The sequential shear failure of components creates a "saw-tooth" effect that is enveloped by the HAZUS capacity curve. In Figure 3.12, the second set of curves represents "brittle" force-deflection behavior and catastrophic failure of the structure. The Ultimate Capacity Control Point is actually selected to be past the point of failure. This is not inappropriate, since the ultimate point does not define the fragility of the building, only the plateau of the capacity curve.

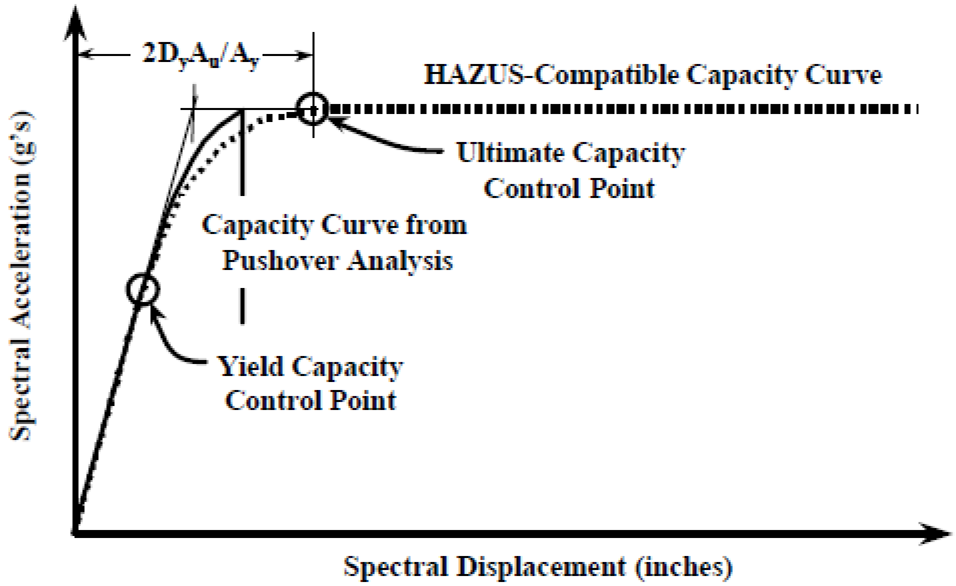


Fig.3.12 Example Development of the Capacity Curve f or a Structure with "Brittle" force-Deflection Behaviour

The third set of curves shown in the Figure 3.13, illustrate force-deflection behavior of a "ductile" building up the formation of a complete mechanism (fully plastic state). The pushover curve indicates some additional strength beyond the fully plastic state due to strain hardening assumptions.

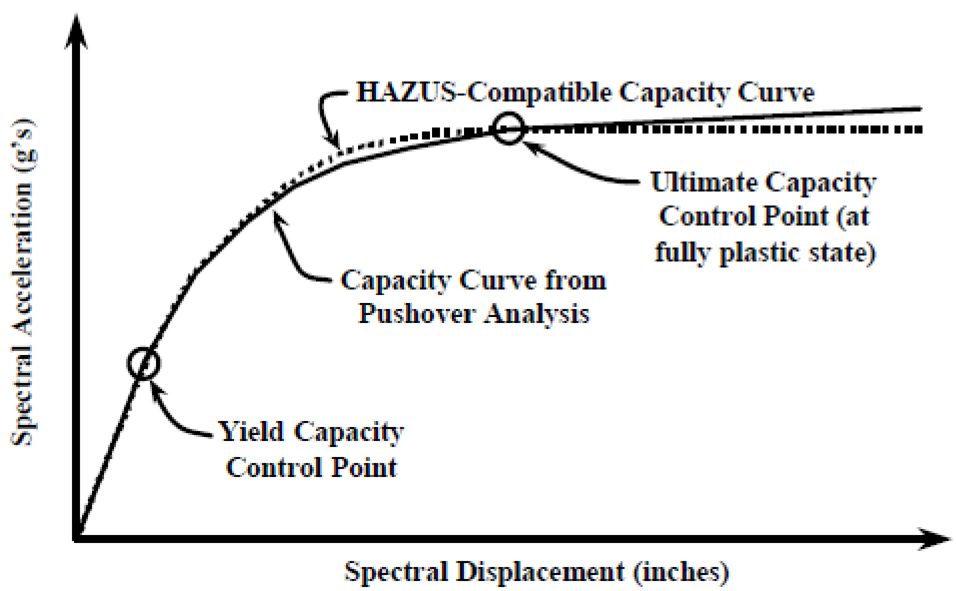


Fig 3.13 Example Development Of the Capacity Curve for a Structure with "ductile" Force-Deflection Behaviour

Both the initial stiffness (i.e. elastic period, T_e) and ultimate strength of the capacity curve will, in general, degrade with repeated cycles of post yield earthquake demand. The effects of degradation of stiffness and strength on capacity and response of the building are accounted for by degradation factors. Development of degradation factors is described in the next subsection.

3.16.3 Development of Response Parameters

Response parameters include Elastic Damping and degradation (Kappa) factors that reduce hysteretic damping and affect the intersection capacity and demand, and the fraction of non structural components at lower-floors (F_{NS}) which affects the calculation of demand on nonstructural-acceleration sensitive components. Background on the use of the elastic damping and degradation factors in the calculations of response is given in the following subsection.

3.16.4 Response calculations

Hazus characterizes ground shaking using a standard response spectrum shape, consistent with the format and parameters of the 1997 NEHRP *Provisions* and the NEHRP *Guidelines*. The standard shape consists in two primary parts:

1. A region of constant spectral acceleration at short periods and
2. A region of constant spectral velocity at long periods.

Short-period spectral acceleration Ss, is defined by 5% -damped spectral acceleration at a period of 0.3 seconds. The constant spectral velocity region has spectral acceleration proportional to 1/T and is anchored to 1-second, 5%-damped spectral acceleration S1. A region of constant spectral displacement exists at very long periods, although this region does not usually affects calculation of building damage. Amplification of ground shaking to account for local site condition is based on short period (F_A) and velocity domain (F_V) soil factors of the 1997 NEHRP *Provisions.*

HAZUS modifies elastic system properties to simulate inelastic response by use of "effective" stiffness and damping properties of the building. Effective stiffness properties are based on secant stiffness, and effective damping is based on combined viscous and hysteretic measures of dissipated energy. Effective damping greater than 5% of critical is used to reduce spectral demand in a manner similar to the capacity-spectrum method of *ATC-40*.

The below figure (Fig.3.14) 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., RA at periods of constant acceleration and RV at periods of constant velocity). The demand spectrum intersects the building's capacity curve at the point of peak response displacement, D, and acceleration, A. The amount of spectrum reduction typically increases for buildings that have reached yield and dissipate hysteretic energy during cyclic response.

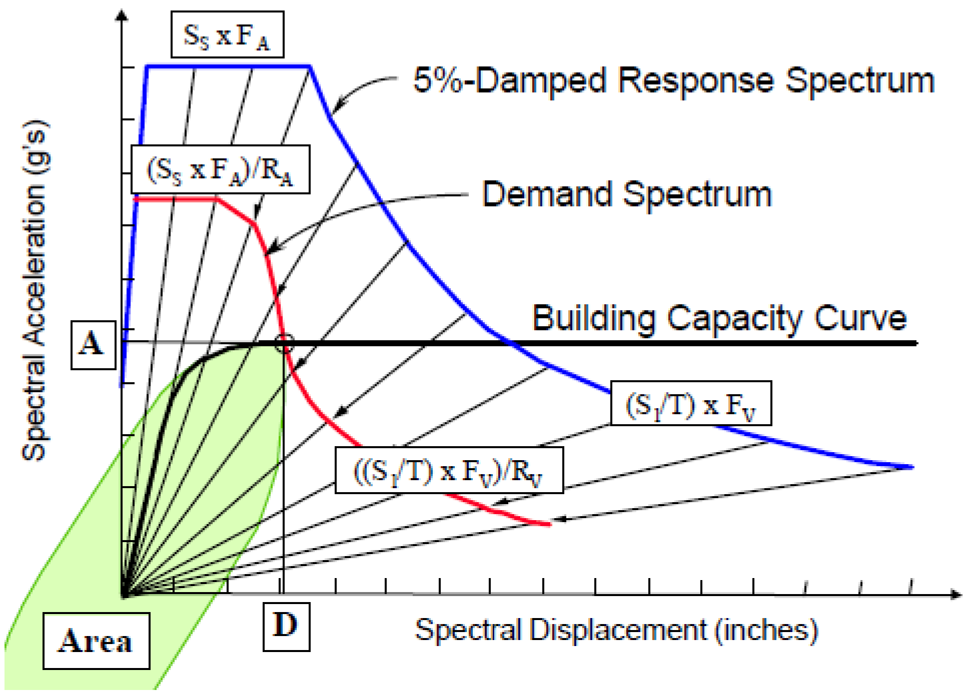


Fig.3.14 Example Demand Spectrum Construction and Calculation of Peak Response Point (D,A)

Spectrum reduction factors are a function of the effective damping of the building, beff, as defined by Equations 3.9 and 3.10:

$$R_A = \frac{2.12}{3.21 - 0.68\ln(\beta_{eff})} \qquad \text{(Equation 3.9)}$$

$$R_V = \frac{1.65}{2.31 - 0.41\ln(\beta_{eff})} \qquad \text{(Equation 3.10)}$$

These equations are based on the formulas given in Table 2 of Earthquake Spectra and Design [Newmark and Hall, 1982] for construction of elastic response spectra at different damping levels (expressed as a percentage of critical damping). The factors of Newmark and Hall represent all site classes (soil profile types), but distinguish between domains of constant acceleration and constant velocity. For either domain, the reduction factor is the ratio of 5%-damped response to response of the system with beff damping. Equations (3-8) and (3-9) yield reduction values of RA = 1.0 and RV = 1.0, respectively, for a value of βeff = 5% of critical.

Effective damping, βeff, is defined as the total energy dissipated by the building during peak earthquake response and is the sum of an elastic damping term, β_E, and a hysteretic damping term, β_H, associated with post-yield, inelastic response:

$$\beta_{eff} = \beta_E + \beta_H \qquad \text{(Equation 3. 11)}$$

The elastic damping term, β_E, is assumed to be a constant (i.e., amplitude independent) and follows the recommendations of Table 3 of Earthquake Spectra and Design for materials at or just below their yield points. The hysteretic damping term, β_H, is dependent on the amplitude of post-yield response and is based on the area enclosed by the hysteresis loop at peak response displacement, D, and acceleration, A, as shown in Figure 3.15 Hysteretic damping, β_H, is defined in Equation 3.12:

$$\beta_H = k\left(\frac{Area}{2\pi DA}\right) \qquad \text{(Equation 3.12)}$$

Where:

Area is the area enclosed by the hystereseis loop, as defined by a symmetrical push-pull of the building capacity curve up to peak positive and negative displacements, ±D, assuming no degradation of components.

D is the peak displacement response of the capacity curve

A is the peak acceleration response at peak displacement D

K is a degradation factor that defines the fraction of the Area used to determine hysteretic damping.

For a value of k=1, Eq 3.12 may be recognized as the definition of equivalent viscous damping, found in modern vibration textbooks [e.g. Chopra, 1995],] and traceable to the early work of Jacobsen [1930] and others. The k (Kappa) factor in Eq. 3.12, reduces the amount of hysteretic damping as a function of model building type, seismic design level 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.

Figure 3.15, shows a typical capacity curve and three example demand spectra for damping levels corresponding to short (k_S = 0.8), moderate (k_M = 0.5) and long (k_L= 0.3) duration ground shaking, respectively. In this example, building displacement due to long-duration ground shaking is more than twice that due to short-duration ground shaking (although building acceleration does not increase). Damage to the structural system and nonstructural, driftsensitive components and related losses increase significantly with increase in the duration of ground shaking for buildings that have reached yield.

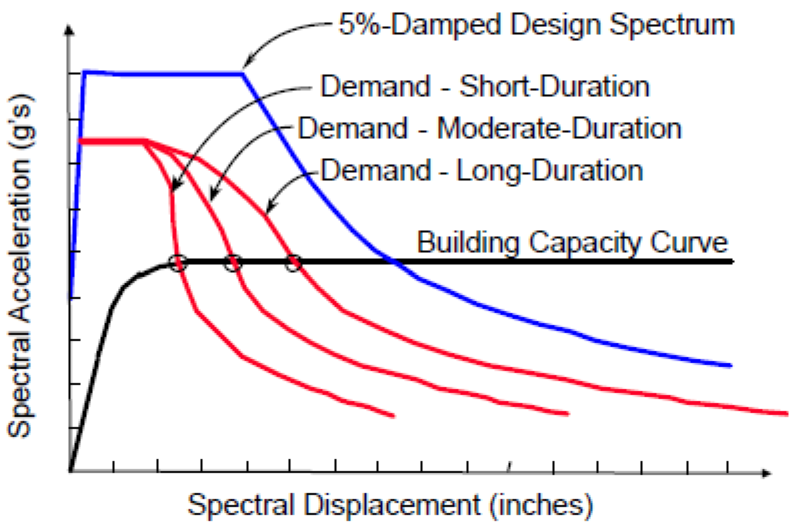


Fig 3.15 Example Demand Spectra-Post-Yield Response due to strong Ground Shaking of Either Short, Moderate or Long Duration

3.16.5 Elastic Damping Factors

As described in the preceding subsection, Elastic Damping factors estimate the damping of the building at or just below yield of the structural system. These values should be selected on the basis of the building type, reflecting the inherent differences in the damping behavior of different materials. In general, the Elastic Damping factors included in HAZUS for general building stock should be used without modification for building-specific applications. The below Table 3.4 summarizes the Elastic Damping values of HAZUS for different building types.

Building Type by Material	Damping (% of Critical)
Mobile Home	5%
Steel Buildings	5% - 7%
Reinforced-Concrete and Pre-cast Concrete Buildings	7%
Reinforced-Masonry Buildings	7% - 10%
Unreinforced-Masonry Bearing-Wall and In-Fill Buildings	10%
Wood Buildings	10% - 15%

Table 3.4 Suggested Elastic Damping Values

3.16.6 Degradation Factor (kappa)

Degradation (Kappa) factors are a function of the expected amplitude and duration (number of cycles) of post-yield building response. These parameters depend on the level of ground shaking, which is different for each building site and scenario earthquake. The default values of the **kappa** factor developed for generic building analysis assume that the building would have group shaking strong enough to effect significant post-yield response of the structure, and degradation os based on the magnitude of the scenario event. The larger the magnitude of the event, the longer the assumed duration of ground shaking. In this sense, earthquake magnitude became a surrogate indicator of the duration of post-yield response, assuming shaking was strong enough to push the structure beyond the yield point.. It should be recognized that if the ground shaking were not strong enough to yield the building, there would be little or no degradation, regardless of the magnitude of the scenario earthquake (or the type of structural system).

Kappa factors should be selected considering the extent to which brittle failure of the elements and components reduces the strength of the structural system. The capacity curve developed by pushover analysis provides some guidance on the selection of appropriate Kappa factors. If the capacity curve indicates a loss of strength at the ultimate capacity control point, then the Kappa factor should indicate a somewhat proportional reduction in hysteretic loop area. For example, in Figure 3.6 the capacity curve indicates about a 50% reduction in full strength, and a commensurate amount of degradation would be appropriate (e.g., $k_M = 0.50$ for a moderate duration of post- yield response). In Figure 3.8, the capacity curve indicates nearly complete (brittle) failure (at the ultimate capacity control point) and a very low value of the degradation factor would be appropriate (e.g., kM = 0.10 for a moderate duration of post-yield response). In Figure 3.11, the capacity curve indicates nearly fully ductile behavior, and a relatively high value of the degradation factor would be appropriate (e.g., $k_M = 0.90$ for a moderate duration of ground shaking).

Table 3.5 provides some general guidance on the selection of the degradation (Kappa) factor. The Kappa factors are shown as a function of the level of response (i.e., one- half yield, yield and post-yield levels of peak response) and for post-yield response as a function of post-yield shaking duration (i.e., short, moderate and long). The table also relates suggested values of Kappa factors to the seismic design level and quality of construction used to characterize generic building types of HAZUS.

Design Level and Construction Quality						Degradation (Kappa) Factor				
	Seismic Design Level[1]					At ½ Yield	At Yield	Post-Yield Shaking Duration		
	SHC	HC	MC	LC	PC			Short	Moderate	Long
QC[2]	S	S				1.0	1.0	1.0	0.9	0.7
		O	S			1.0	1.0	0.9	0.7	0.5
		I	O	S		1.0	0.9	0.7	0.5	0.3
			I	O	S	1.0	0.7	0.5	0.3	0.1
				I	O	1.0	0.5	0.3	0.1	0.0

1. Seismic Design Level Designation – Special High-Code (SHC), High-Code (HC), Moderate-Code (MC), Low-Code (LC) and Pre-Code (PC)
2. Construction Quality (QC) Designation – Superior (S), Ordinary (O) and Inferior (I)

Table 3.5 Suggested Values of the Degradation (kappa) Factor

The suggested values of Kappa factor given in table 3.5 do not apply to seismically rehabilitated buildings. If the user is developing damage functions for a building that been strengthened, or otherwise seismically improved, then the selection of Kappa's should be based on a seismic design level and quality of construction that reflects these improvements For example, substantial seismic rehabilitation of a Pre-Code building of Ordinary construction (i.e., older building constructed before seismic codes were adopted) might now be considered to be equivalent to a building of Moderate Code seismic design level of Superior construction quality. Of course, the amount by which the seismic design level and/or construction quality should be increased depends on the type and extent of the seismic improvements made to the structural system.

3.17 Development of Fragility Curves

3.17.1Building Response and performance criteria

This section guides users in the development of fragility curves parameters that are used by Advanced Engineering Building Module (AEBM) to calculate damage as a function of building response. It is assumed (and essential) that the user has already performed a detailed nonlinear static (pushover) analysis of the building that conforms essentially to the methods of the *NEHRP Guidance* (or ATC-40) and to certain other criteria as set forth in this section.

The pushover analysis must appropriately capture the damage patterns of elements and components of the building and evaluate modes of building failure (i.e., partial or full collapse of the structure). More than one pushover curve may be used to evaluate different modes if response and failure (e.g., of different building segments).

There are certain key aspects to the damage functions of which users must be aware when developing fragility parameters. First, the damage functions should predict damage without bias such as that inherent to the conservatism of seismic design codes and guidelines. In general, limit states of the *NEHRP Guidelines* (or *ATC-40*) will under-predict the capability of the structure, particularly for the more critical performance objectives, such as Collapse Prevention (CP). The *NEHRP Guidelines'* criteria for judging CP certainly do not intend that 50 out of 100 buildings that just meet CP limits would collapse. Most engineers would likely consider an acceptable fraction of CP failures (given that buildings just meet CP criteria) to be between 1 and 10 in every 100 buildings. In contrast, the median drift value of the Complete structural damage state of *HAZUS* is the amount of building displacement that would cause, on the average, 50 out of 100 buildings of the building type of interest to have Complete damage (e.g., full financial loss). In general, users should not derive median values of *HAZUS* damage states directly from the performance limits of the *NEHRP Guidelines* (and *ATC-40*).

Fragility parameters of the more extreme damage states are particularly difficult to estimate since these levels of damage are rarely observed even in the strongest ground shaking. In the 1995 Kobe earthquake, the worst earthquake disaster to occur in a modern urban region, only about 10 in every 100 mid-rise commercial buildings located close to fault rupture had severe damage or collapse. Typically, the fraction of modern buildings with such damage (e.g., Complete structural damage) is much less than 10 in 100. In selecting median values of damage states, users should be mindful that median values represent the 50 percentile (e.g., 50 in every 100 buildings have reached the state of damage of interest). Median values of spectral displacement (or spectral acceleration) for the more extensive states of damage may appear large relative to seismic code or guideline design criteria.

Calculation of damage-state probability is a step in the sequential process of estimating earthquake losses. Some leeway is available in determining building-specific fragility curves, since the building-specific loss functions will also be developed based on the fragility assumptions. What is essential is that the amount and type of damage associated with each damage state be consistent with the amount and type of damage assumed in the development of loss functions. For example, the one may have a choice of 4 inches, 5 inches or 6 inches of spectral displacement to represent Moderate structural damage to the building. In this example, these spectral displacements represent a range of plausible estimates resulting in “moderate” damage to elements and components, but with distinct differences in the cost of repair. That is, 6 inches of spectral displacement would cause more damage and cost more to repair than 4 inches of spectral displacement.

Fragility curves define boundaries between damage states. That is, the median value of the Damage State of interest defines the threshold of damage, and this state of damage is assumed to exit up to next state of damage. This description is illustrates in the figure 3.16 , which includes example fragility curves for Slight, Moderate, Extensive and Complete structural. In this illustration, a shaded region illustrated the probability – response space associated with Moderate damage. The boundary on the left on the shade region is defined by the fragility curve for Moderate (or greater) structural damage, and the boundary on the right of the shaded region is defined by the fragility curve for Extensive (or greater) damage. The probability of Moderate damage at a given level of spectral demand is calculated as the difference of the probability of Moderate(or greater) damages less the probability less the probability of extensive (or greater) damage – a probability of 0.40 at 6 inches of spectral displacement in the below example (Fig. 3.16)

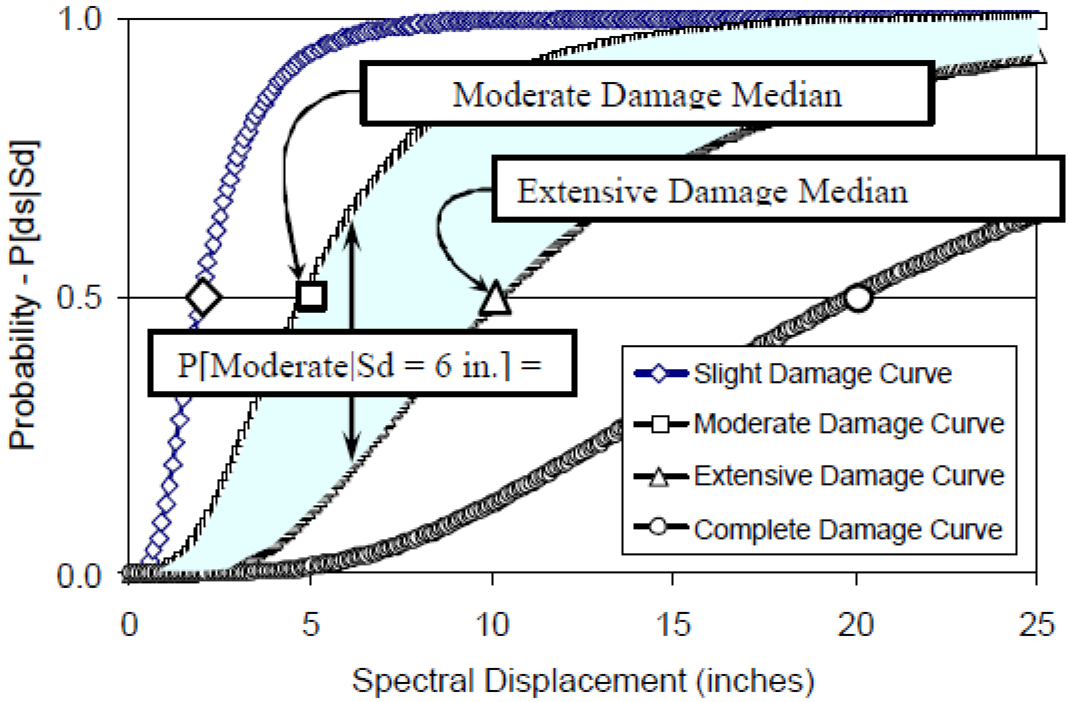


Fig 3.16 Example Fragility curves-Calculation of Damage – State Probability

The slope of the fragility curve is controlled by the lognormal standard deviation value (Beta). The smaller the value Beta, the less variable the damage state, and steeper the fragility curve. The larger the value if Beta, the more variable the damage state, and the flatter the fragility curve. The figure 3.17, illustrates this trend for this trend for fragility curves that share a common median (i.e., spectral displacement of 5 inches), but Beta values ranging from 0.4 to 1.2. This range of Beta values approximately covers the range of Beta values that could be used for building-specific fragility curves.

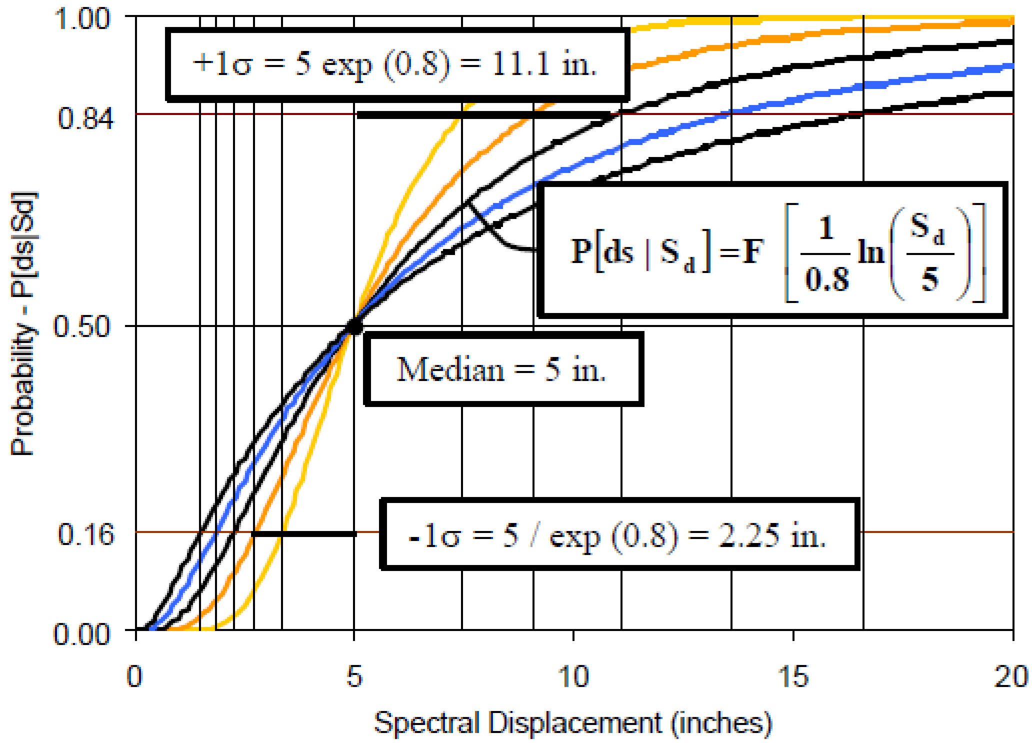


**Fig 3.17 Example Lognormal Fragility Curves (Beta=0.4,0.6,0.8,1.0,1.2)
and Calculation of ±σ Spectral Displacement**

The Figure3.17 illustrate the calculation of spectral displacement at ±1 standard deviation (±1σ) probability levels for a typical Beta value of 0.8. In this example, the +1σ level of spectral displacement is more than twice the median value (and the -1σ level of spectral displacement is less than one half the median value) for a Beta value of 0.8 which illustrates the large amount of variability typical HAZUS fragility curves.

3.17.2 Development of Damage-State Medians

Development of Damage-State Medians involves three basic steps:

- Develop a detailed understanding of damage to elements and components as a continuous function of building response(e.g., average inter-story or floor acceleration)
- Select specific values of building response that best represent the threshold of each discrete damage state.
- Convert damage-state threshold values (e.g., average inter-story drift) to spectral response coordinates (i.e., same coordinates as those of the capacity curve)

In general, the implementation of the three steps will be significantly different for structural and nonstructural systems. It is expected that detailed pushover analysis of the building will be the primary source of information regarding structural damage and selection of appropriate damage-state threshold values. In most cases generic-building fragility values of *HAZUS* would not be used for the structural system (but could provide a “sanity check” of building-specific results). In contrast, pushover analysis typically provides only minimal information of nonstructural system performance, and users will rely primarily on the generic-building fragility values of *HAZUS* to determine threshold values of nonstructural damage states.

3.17.3 Structural System

Selection of **Damage-State Medians** should be consistent for different model building types. Descriptions of damage in *HAZUS* are sufficiently vague to permit user selection of values that best fit the damage patterns of dominant elements and components of the structural system. In addition, general guidance is provided below in Table 3.6 regarding the selection of appropriate **Damage-State Medians** for the structural system.

Damage State	Likely Amount of Damage, Direct Economic Loss, or Building Condition			
	Range of Possible Loss Ratios	Probability of Long-Term Building Closure	Probability of Partial or Full Collapse	Immediate Post-Earthquake 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 local collapse (e.g., out-of-plane failure of URM infill walls).

Table .3.6 General Guidance for Selection of Structural Damage State-Median

Pushover analysis results typically express performance in terms of component ductility demand, rather than in terms of physical damage. It is expected that the results of the pushover analysis, whether expressed in terms of physical damage (e.g., crack size) or in terms of component ductility demand, will be sufficient to tabulate the type and sequence of damage (and failure) of elements and components.

Damage to elements and components of the structural system should be tabulated as a function of the lateral displacement of the building, qualified by the average inter-story drift ratio (i.e., roof displacement divided by building height. Of course, individual stores of multi-story building would not all be expected to have the same drift, nor would inter-story drift be the same at all locations on a given floor if there was diaphragm flexibility or a rotational component to the pushover mode shape. However, the average inter-story drift provides a convenient measure of building response that may be compared against default values of average inter-story drift that define damage states for generic building types of HAZUS.

Table 3.7 is used to relate deformation (deformation ratio) limit of the NEHRP Guidelines to average-inter story drift ratio of structural damage state. Table 3.7 provides two sets of criteria for each structural damage state. The first set of criteria establish damage states in terms of the fraction (by replacement value) of structural components reaching the control point "C" (or control point "E") on the idealized load versus deformation (backbone) curve. The second set of criteria establish an upper-bound on the average inter-story drift ratio of damage states by factors applied to the displacement at which 50% of structural components have reached their individual yield points (i.e., control point "B"). Figure 3.18 (taken from Figure 2-5 of the NEHRP Guidelines, illustrates points B, C and E on the idealized load versus deformation (backbone) curve.

Damage State	Component (Criteria Set No. 1)[1]			Component (Criteria Set No. 2)[1]		
	Fraction[2]	Limit[3]	Factor[4]	Fraction[2]	Limit[3]	Factor[4]
Slight	> 0%	C	1.0	50%	B	1.0
Moderate	≥ 5%	C	1.0	50%	B	1.5
Extensive	≥ 25%	C	1.0	50%	B	4.5
Complete	≥ 50%	E	1.0 - 1.5[5]	50%	B	12

Table 3.7 Guidance for Relating Component (or Element) Deformation to thevAverage Inter-Story Drift Ratios of Structural Damage-State Medians

1. The average inter-story drift ratio of structural damage state is lessor of the two drift ratios defined by Criteria Sets No. 1 and No.2, respectively.
2. Fraction defined as the repair or replacement cost of components at limit divided by the total replacement value of the structural system.
3. Limit defined by the control points of Figure 6-2 and the acceptance criteria of *NEHRP Guidelines*.
4. Factor applied to average inter-story drift of structure at deformation (or deformation ratio) limit to calculate average inter-story drift ratio of structural damage-state median.
5. Complete factor is largest value in range for which the structural system is stable.

As an example of the use of the 1st set of criteria of Table 3.7 (i.e., limits of 2nd criteria set are assumed not to govern), consider the development of damage-state medians for the “pushover” curve shown in Figure3.19. This pushover curve corresponds to the “saw-tooth” capacity curve except that curve is now shown in terms of base shear versus average inter-story drift ratio (i.e., roof displacement normalized by building height. This pushover curve is assumed to have been developed by nonlinear static analysis of the structure using the modeling and acceptance theory of the *NEHRP Guidelines*.

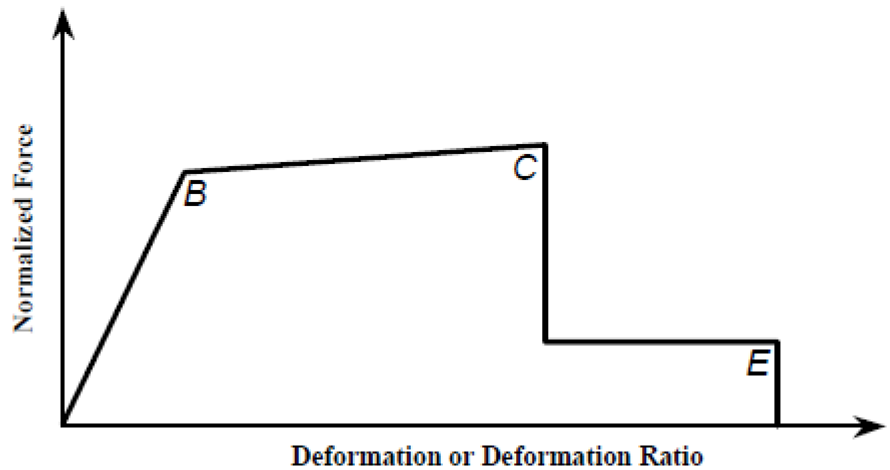


Figure 3.18 Idealized Component Load versus Deformation Curve (*NEHRP Guidelines)*

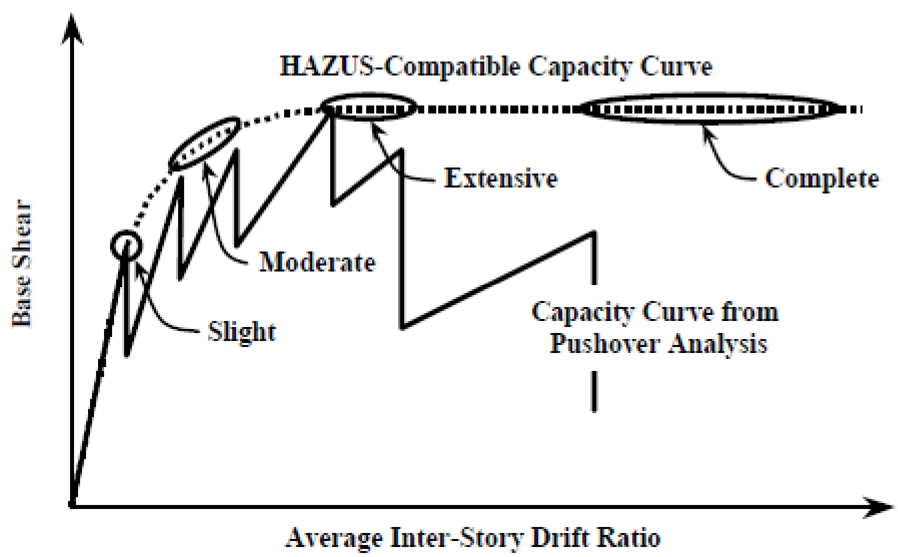


Figure 3.19 Damage-State Medians of "Saw-Tooth" Pushover Curve

Following the guidance of Table 3.7, the median of Slight damage is defined by the first structural component to reach control point C on its load deformation curve (i.e., point where component capacity of component drops, as illustrated in Figure 3.18). On a global basis, this point may be recognized as the first "tooth" of the capacity curve (i.e., point where structure capacity drops abruptly, as illustrated in Figure 3.19).

Moderate damage is defined by a median value for which a sufficient number of components have each reached control point C (on their respective load deformation curves) such that it will cost at least 5% of the replacement value of the structural system to repair (or replace) these components. Moderate damage is likely to be localized, since only a limited number of components can be repaired (or replaced) for 5% of the replacement value of the structural system. In Figure 3.19, an oval indicates that this extent damage might occur at the second or third “tooth” of the capacity curve, depending on type of repair, accessibility of damaged components and other factors that influence repair cost.

Extensive damage is defined by a median value similar to Moderate damage, except that damage repair now costs at least 25% of the value of the structural system. Extensive damage is likely to affect a number of components distributed throughout the building or affect all components at the most vulnerable story. Again, an oval indicates the sensitivity of the median to repair cost factors. The Extensive damage oval extends up to the point on the pushover curve for which there is a large drop in load capacity without significant recovery indicating (in this example) that a large number of elements would require repair or replacement at this level of response.

Complete damage is defined by a median value for which at least 50% (in terms of repair/replacement cost) of structural components have each lost full lateral capacity, as defined by control point E on their respective load deformation curves. Table 3.7 acknowledges the inherent conservatism in the values of control point E (as defined by the *NEHRP Guidelines*) and suggests that the median of the Complete damage state should be as much as 1.5 times greater than control point E, provided that the structure is not likely to collapse.

In Figure 3.19, a large oval indicates the range of possible median values for the Complete damage state. This range extends from 1.0 to 1.5 times the point of the last large drop in the load-carrying capacity of the pushover curve, indicating that most elements have reached their limit. The Complete damage state and related collapse failure modes are the most difficult to rationalize using engineering methods, even when evaluated using the sophisticated nonlinear methods of the *NEHRP Guidelines*. Correlation of predicted and observed damage and losses indicate that very liberal interpretations of engineering acceptance criteria are required to accurately predict Complete damage and the number of collapses that have actually occurred.

The average inter-story drift ratios of structural damage states of generic building types may be found in Table 3.8 of the *HAZUS-MH Technical Manual.* These tables provide drift ratios of each model building type for Special High-Code, High-Code, Moderate-Code, Low-Code and Pre-Code seismic design levels, respectively. These drift ratios are also summarized below in Table 3.8. The *HAZUS* drift ratios for generic buildings may be used as a “sanity check” of building-specific values, recognizing that generic-building damage state median values represent a typical building of the group and could be a factor of 2 or more greater (or less than) the medians of a specific building.

It should also be noted that Table 3.8 incorporates the effects of diaphragm flexibility (and other contributors to the overall flexibility of the structural system) in the values of average inter-story drift ratio that define the damage-state medians of generic buildings. In contrast, the control points and acceptance criteria of the *NEHRP Provisions* apply strictly to the component of interest. For structural systems with very stiff components (e.g., URM buildings), average inter-story drift ratios developed from pushover analysis using the modeling and acceptance criteria of the *NEHRP Guidelines* should also incorporate diaphragm (and other sources of) flexibility before comparison with the default values summarized in Table 3.8 for generic building types.

Model Building Type		Structural Damage States			
		Slight	Moderate	Extensive	Complete
Low-Rise Buildings – High-Code Design Level					
W1, W2		0.004	0.012	0.040	0.100
S1		0.006	0.012	0.030	0.080
C1, S2		0.005	0.010	0.030	0.080
C2		0.004	0.010	0.030	0.080
S3, S4, PC1, PC2, RM1, RM2		0.004	0.008	0.024	0.070
Low-Rise Buildings – Moderate-Code Design Level					
W1, W2		0.004	0.010	0.031	0.075
S1		0.006	0.010	0.024	0.060
C1, S2		0.005	0.009	0.023	0.060
C2		0.004	0.008	0.023	0.060
S3, S4, PC1, PC2, RM1, RM2		0.004	0.007	0.019	0.053
Low-Rise (LR) Buildings – Low-Code Design Level					
W1, W2		0.004	0.010	0.031	0.075
S1		0.006	0.010	0.020	0.050
C1, S2		0.005	0.008	0.020	0.050
C2		0.004	0.008	0.020	0.050
S3, S4, PC1, PC2, RM1, RM2		0.004	0.006	0.016	0.044
S5, C3, URM		0.003	0.006	0.015	0.035
Low-Rise (LR) Buildings – Pre-Code Design Level					
W1, W2		0.003	0.008	0.025	0.060
S1		0.005	0.008	0.016	0.040
C1, S2		0.004	0.006	0.016	0.040
C2		0.003	0.006	0.016	0.040
S3, S4, PC1, PC2, RM1, RM2		0.003	0.005	0.013	0.035
S5, C3, URM		0.002	0.005	0.012	0.028
Mid-Rise Buildings[1]					
All	Mid-Rise Building Types	2/3 * LR	2/3 * LR	2/3 * LR	2/3 * LR
High-Rise Buildings[1]					
All	High-Rise Building Types	1/2 * LR	1/2 * LR	1/2 * LR	1/2 * LR

1. Mid-rise and high-rise buildings have damage-state drift values based on low-rise (LR) drift criteria reduced by factors of 2/3 and 1/2, respectively, to account for higher-mode effects and differences between average inter-story drift and individual inter-story drift.

Table 3.8 *HAZUS* Average Inter-Story Drift Ratio (Δds) of Structural Damage States

As the final step in the development of **Damage-State Medians** for the structural system, average inter-story drift values for each damage state are converted to the corresponding amount of spectral displacement using the modal factor, α_2, and other terms:

$$S_{d,ds} = \Delta_{ds} H_R \ \alpha_2 \qquad \text{(Equation 3.12)}$$

Where:

$S_{d,ds}$ Median spectral displacement value of damage state, ds (inches)

Δ_{ds} Average inter-story drift ratio at the threshold of damage state, ds, determined by user (consistent with generic values of Table 3.8)

H_R Height of the building at the roof level (inches)

α_2 Pushover modal factor from Equation 3.4

3.17.4 Non-structural Components

In most applications, **Damage-State Medians** for nonstructural components may be based directly on the default values of *HAZUS*. Exceptions include buildings with nonstructural components or contents that are either significantly more rugged or significantly more vulnerable than the normal make-up of components of nonstructural systems in a typical commercial building. Examples of buildings with particularly vulnerable systems include certain manufacturing facilities (e.g., buildings with clean rooms), laboratories, computer facilities, historical buildings (architectural components), art museums and other buildings with special contents. Examples of buildings with particularly rugged systems include certain military, industrial or emergency facilities whose nonstructural systems and contents have been specially anchored or braced to resist earthquake shaking.

HAZUS default values for the drift ratio of the threshold of each damage state are summarized in Table 3.9 for drift-sensitive nonstructural components. These damage-state drift ratios are assumed to be the same for all building types and seismic design levels. The same values of drift ratio are also assumed to be appropriate for special buildings, such as emergency facilities, since drift-sensitive components (partitions) typically do not receive special design or detailing to accommodate building displacement.

Design Level	Nonstructural Damage States – All Building Types			
	Slight	Moderate	Extensive	Complete
Inter-Story Drift Ratio (Δ_{ds}) - Drift-Sensitive Components				
All	0.004	0.008	0.025	0.050
Peak Floor Acceleration ($A_{max,ds}$) - Acceleration-Sensitive Components/Contents (g's)				
Special High-Code	0.45	0.9	1.8	3.6
High-Code	0.30	0.6	1.2	2.4
Moderate-Code	0.25	0.5	1.0	2.0
Low-Code	0.20	0.4	0.8	1.6
Pre-Code	0.20	0.4	0.8	1.6

Table 3.9 HAZUS Damage-State Criteria for Nonstructural Systems and Contents

HAZUS default values of peak floor acceleration defining the threshold of each damage state are summarized in Table 3.8 for acceleration-sensitive nonstructural components (and contents). These damage-state accelerations are assumed to be the same for all building types, but to vary by seismic design level. Similarly, emergency or other facilities that have special anchorage and bracing requirements for nonstructural components and equipment (Special High-Code design level) have damage-state accelerations increased by a factor of 1.5.

Considering the importance to the estimates of certain types of loss, in particular estimates of direct economic loss, it would seem desirable to develop building-specific damage-state parameters for nonstructural components and contents, rather than rely on generic building data. However, rigorous development of nonstructural parameters would require detailed evaluation of component capacity, similar to that used to evaluate the structural system, only much more difficult to perform due to the complexity and variety of different nonstructural systems and components. Nonstructural systems and contents would need to be thoroughly inspected (detailed field survey). Capacity of anchorage and bracing would need to be evaluated (possibly requiring dynamic analysis of complex systems such as piping runs). Fragility values would then need to be developed based on the results of the analysis, available test data (e.g., of similar equipment), and/or experience data. This process is not practical for most applications and would likely be limited to a "walk-down" of nonstructural systems and building contents.

If the user has access to the building and is concerned that nonstructural components and/or contents are not "typical," then it is recommended that a building "walk-down" be performed using checklists and other guidance provided by *FEMA 74* [FEMA, 1994] or *FEMA 310* [FEMA, 1998]. These documents do not estimate damage or loss but are useful in spotting potential deficiencies in typical nonstructural systems. The user need not perform calculations, but may rely on judgment to estimate the approximate drift ratio (for drift-sensitive components) or peak floor acceleration (for acceleration-sensitive components) at which different nonstructural components would begin to fail and require repair or replacement.

Damage-State Medians for drift-sensitive nonstructural components must be converted from drift ratio to spectral displacement in a manner similar to that used for the structural system. Inter-story drift ratios for each damage state are converted to the corresponding amount of spectral displacement using the modal factor, α_2, and other terms:

$$S_{d,ds} = F_{\Phi p,ds} \cdot \Delta_{ds} \cdot H_R \cdot \alpha_2 \qquad \text{(Equation 3.13)}$$

Where:

$S_{d,ds}$ Median spectral displacement value of damage state, ds (inches)

$F_{\Phi p,ds}$ Factor relating average inter-story drift to the drift of the component at damage state, ds, as defined by the Equation.

. Δ_{ds} Component drift ratio corresponding to threshold of damage state (ds), to be determined (consistent with the generic values of table 3.8)

H_R Height of building at the roof level (inches)

α_2 Pushover modal factor from Equation 3.4

The factor $F_{\Phi p,ds,}$ is used is used to relate average inter-story drift to maximum inter-story drift to account for the effects of an uneven distribution of drift over the height of the building. Uneven distribution of drift causes damage to occur at certain stories sooner than at other stories. The factor $F_{\Phi p,ds}$, is based on both the shape of the pushover mode and damage-state loss ratio:

$$F_{fP,ds} = \left(\frac{f_{R,P}(1-NSD_{ds})}{H_R \Delta_{max,p}}\right) - NSD_{ds} \qquad \text{(Equation 3.14)}$$

Where:

$f_{R,P}$ Roof displacement of the pushover mode for damage state, ds (inches)

NSD_{ds} Nonstructural drift-sensitive component loss ratio of damage state, ds (expressed as a fraction)

H_R Height of building at the roof level (inches)

$\Delta_{max,p}$ Maximum inter-story drift ratio (considering torsion) over the height of the building corresponding to the roof displacement, $f_{R,P}$.

The factor, $F_{fP,ds}$, makes use of the results of the pushover analysis to better predict localized damage and loss for buildings that have a structural irregularity (e.g., soft story). When drift is uniformly distributed over building height, the value of the factor is equal 1.0. When drift is not uniformly distributed over building height, the factor reduces median values to reflect the lower thresholds of damage associated with accentuated drift of critical stories. The factor varies with the loss ratio of the damage state, effectively reducing the influence of localized damage on the more extensive states of damage (i.e., factor is 1.0 for Complete Damage).

Damage-State Medians for nonstructural acceleration-sensitive components (and contents) are developed in terms of peak floor acceleration. In general, medians expressed in terms of spectral acceleration are taken as equal to peak floor acceleration values since spectral acceleration (obtained by the intersection of pushover curve and spectral demand) is assumed to represent peak floor acceleration of a typical upper floor of the building. Demand on components (and contents) at ground level is based directly on peak ground acceleration and is also assumed to represent peak (ground) floor acceleration. The trivial equation summarizing conversion peak floor acceleration of each damage state to the corresponding amount of spectral acceleration is:

$$S_{a,ds} = A_{max,ds} \qquad \text{(Equation 3.15)}$$

$S_{a,ds}$ Median spectral acceleration value of damage state, ds (units of g)

$A_{max,ds}$ Peak floor acceleration of the threshold of damage state, ds (units of g) determined by user or based on generic values of Table 3.8.

The assumption that peak floor acceleration is the same as spectral acceleration demand ignores higher-mode shaking effects (not included in the pushover analysis) and the uneven distribution of floor acceleration over building height. Higher-mode effects can significantly increase upper floor accelerations, although they may not cause failure of systems that have some ductility. Users concerned about higher-mode response could reduce median values by a factor inversely proportional to the increase in (damaging) floor acceleration associated with higher-mode response.

Peak floor acceleration will vary over the height of the building, typically with the largest accelerations at the roof. The intersection of the pushover and demand spectrum corresponds to building response at a floor elevation of about a2 x HR. Users concerned that this location is not representative of a typical upper floor of nonstructural acceleration-sensitive components (e.g., all the equipment is on the roof) could modify median values based on the location of the components and the shape of the pushover mode. Such modification would have little effect on the prediction of damage for most buildings with well distributed nonstructural systems.

3.17.5 Development of Damage-State Variability

Lognormal standard deviation (**Beta**) values describe the total variability of fragility-curve damage states. Three primary sources contribute to the total variability of any given state, namely, the variability associated with the capacity curve, bC, the variability associated with the demand spectrum, bD, and the variability associated with the discrete threshold of each damage state, bT,ds, as described in Equation 3.16:

$$\beta_{ds} = \sqrt{\left(\mathrm{CONV}\left[\beta_C, \beta_D\right]\right)^2 + \left(\beta_{T,ds}\right)^2} \qquad \text{(Equation 3.16)}$$

Where:

β_{ds} is the lognormal standard deviation parameter that describes the total variability of damage state, ds,

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

β_D is the lognormal standard deviation parameter that describes the variability of the demand spectrum (values of $\beta_D = 0.45$ at short periods and $\beta_D = 0.50$ at long periods were used to develop Tables 3.10-12)

$\beta_{T,ds}$ is the lognormal standard deviation parameter that describes the total variability of the threshold of damage state, ds.

Since the demand spectrum is dependent on building capacity, a convolution process is required to combine their respective contributions to total variability. This is referred to as “CONV” in Equation 3.16. The third contributor to total variability, $\beta T,ds$, is assumed mutually independent of the first two variables and is combined with the results of the CONV process using the square root-sum-of-the squares (SRSS) method. Additional background on the calculation of **Damage-State Beta’s** is provided in the *HAZUS-MH Technical Manual.*

The variability of the demand spectrum (i.e., variability of ground shaking) is a key parameter in the calculation of damage-state variability. The values of demand variability, $\beta D = 0.45$ at short periods and $\beta D = 0.50$ at long periods, are the same as those used to calculate the default fragility curves of the *HAZUS-MH Technical Manual*. These value s are consistent with the variability (e.g., dispersion factor) of ground shaking attenuation functions used by *HAZUS* to predict response spectra for large-magnitude events in the Western United States (WUS). It may be noted that if there were no variability of demand (response spectrum is known exactly), then Equation 3.16 would become:

$$\beta_{ds} = \sqrt{(\beta_C)^2 + (\beta_{T,ds})^2} \quad \text{(Equation 3.17)}$$

This equation provides a lower-bound on the damage-state variability appropriate for use in probabilistic calculations of damage and loss that are based on the integration of the fragility with hazard functions that have already incorporated ground shaking variability in the hazard calculations. Similarly, Equation 3.17 also provides a lower-bound on damage-state variability for calculation of damage and loss using a response spectrum that is reasonably well known (i.e., response spectrum of recorded ground shaking). Arguably, there would always be some amount variability (uncertainty) in ground shaking demand, β_D, but such can be ignored in the calculation of total damage-state variability, β_{ds}, when substantially less than both capacity curve variability, β_C, and damage-state threshold variability, $\beta_{T,ds}$.

The convolution process involves a complex numerical calculation that would be very difficult for most users to perform. To avoid this difficulty, sets of pre-calculated values of **Damage- State Beta's** have been compiled in Tables 3.10 through 3.12 from which users may select appropriate values of variability for the structural system, nonstructural drift-sensitive components and nonstructural acceleration-sensitive components. The Beta values of these tables are a function of the following building characteristics and criteria:

- Building Height Group - Low-Rise Buildings (Table 3.10), Mid-Rise Buildings (Table 3.11) and High-Rise Buildings (Table 3.12)
- Post-Yield Degradation of the Structural System – Minor, Major and Extreme Degradation
- Damage-State Threshold Variability – Small, Moderate or Large Variability
- Capacity Curve Variability – Very Small, Small, Moderate or Large Variability.

The Beta values of the tables are applicable to all model building types. For example, a low-rise concrete-frame building (C1L) would have the same set of Beta's as a low-rise braced steel frame building (S2L), provided the two buildings have the same amount of capacity curve and damage-state threshold variability, and the same amount of post-yield degradation of the structural system.

Post-yield degradation of the structural system is defined by a Kappa factor, which is an direct measure of the effects of seismic design level and construction quality on the variability of response. Buildings that are seismically designed and/or have superior construction are less likely to degrade during post-yield earthquake shaking, and therefore have more predictable response, than buildings that are not seismically designed and/or have inferior construction.

To select a set of building-specific **Damage-State Beta's** (i.e., a structural Beta, a nonstructural drift-sensitive Beta and a nonstructural acceleration-sensitive Beta), users must first determine the building height group that best represents the specific building of interest. The height groups are defined by the same criteria as those used by *HAZUS* to define generic building types.

Tables 3.10 through 3.12 (referred to as the Beta tables) provide recommended sets of **Damage- State Beta's** for each of the three building height groups, respectively. In each of these tables, the Beta's are based on 36 possible combinations of capacity curve variability, damage threshold variability and the amount of post-yield degradation expected for the structural system.

Estimation of structural system degradation (minimum or maximum) is made on the basis of **Kappa** factors suggested by Table 3.5 (Section 5.3.3) and the degree of post-yield response expected for the damage state of interest. **Kappa** factors decrease with increase in response level (and damage). Slight damage corresponds to response between ½ yield and full yield; Moderate damage to response at or just beyond yield; and Extensive and Complete damage correspond to post-yield response for the duration of scenario earthquake shaking. Beta values are given in Tables 3.10 through 3.12 for $k \geq 0.9$ (minor degradation), $k = 0.5$ (major degradation) and $k \leq 0.1$ (extreme degradation) of the structural system; and linear interpolation may used to establish Beta's for other values of the **Kappa** factor.

Estimation of the variability of the capacity curve (β_C) and the variability of the threshold of the damage state ($\square_{T,ds}$) must be made by users on a judgmental basis (with some guidance provided herein). To assist the user, the Beta tables express capacity curve and damage threshold variability qualitatively (e.g., Small Variability) and in term of the numerical value used to develop the Beta's in the CONV process. Numerical values of variability (β_C and $\beta_{T,ds}$) are lognormal standard deviation parameters and may be used, as illustrated in Figure 3.7, to construct the distribution of capacity or damage threshold that they represent.

The variability of capacity curves and the damage-state thresholds are influenced by:

- Uncertainty in capacity curve properties and the thresholds of damage states, and
- Building population (i.e., individual building or group of buildings).

Relatively low variability of damage states would be expected for an individual building with well known properties (e.g., complete set of as-built drawings, material test data, etc.) and whose performance and failure modes are known with confidence. The taller the building the greater the variability in damage state due to uncertainty in the prediction of response and damage using pushover analysis. Relatively high variability of damage states would be expected for a group of buildings whose properties are not well known and for which the user has low confidence in the results (of pushover analysis) that represent performance and failure modes of all buildings of the group. The latter case essentially describes the original development of damage-state fragility curves for generic model building that were based on capacity variability, $\beta_C = 0.3$, and damage state threshold variability, $\beta_{T,ds} = 0.3$ (Structure), $\beta_{T,ds} = 0.5$ (NSD) and $\beta_{T,ds} = 0.6$ (NSA). The generic model building types represent large populations of buildings for which properties are not well known.

Building System[2]	Post-Yield Degradation of Structural System[3]								
	Minor Degradation ($\kappa >= 0.9$)			Major Degradation ($\kappa = 0.5$)			Extreme Degradation ($\kappa <= 0.1$)		
	Damage Variability[4] ($\beta_{T,ds}$)			Damage Variability[4] ($\beta_{T,ds}$)			Damage Variability[4] ($\beta_{T,ds}$)		
	Small (0.2)	Mod. (0.4)	Large (0.6)	Small (0.2)	Mod. (0.4)	Large (0.6)	Small (0.2)	Mod. (0.4)	Large (0.6)
Structural Systems with Very Small Capacity Curve Variability[5] ($\beta_C = 0.1$)									
Structure	0.70	0.80	0.90	0.85	0.90	1.00	0.95	1.00	1.10
NSD	0.65	0.75	0.90	0.85	0.90	1.00	0.95	1.00	1.10
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Small Capacity Curve Variability[5] ($\beta_C = 0.2$)									
Structure	0.70	0.80	0.90	0.85	0.90	1.00	0.95	1.05	1.15
NSD	0.70	0.75	0.90	0.85	0.90	1.00	0.95	1.00	1.10
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Moderate Capacity Curve Variability[5] ($\beta_C = 0.3$)									
Structure	0.75	0.80	0.95	0.85	0.95	1.05	1.00	1.05	1.15
NSD	0.70	0.80	0.90	0.85	0.95	1.05	1.00	1.05	1.15
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Large Capacity Curve Variability[5] ($\beta_C = 0.4$)									
Structure	0.80	0.85	0.95	0.90	1.00	1.10	1.05	1.10	1.20
NSD	0.75	0.85	0.95	0.90	1.00	1.05	1.00	1.05	1.15
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65

1. Building Systems include the Structure, Nonstructural Drift-Sensitive Components (NSD) and Nonstructural Acceleration-Sensitive (NSA) components.

Table 3.10 Low-Rise Building Fragility Beta's

Building System[2]	Post-Yield Degradation of Structural System[3]								
	Minor Degradation ($\kappa >= 0.9$)			Major Degradation ($\kappa = 0.5$)			Extreme Degradation ($\kappa <= 0.1$)		
	Damage Variability[4] ($\beta_{T,ds}$)			Damage Variability[4] ($\beta_{T,ds}$)			Damage Variability[4] ($\beta_{T,ds}$)		
	Small (0.2)	Mod. (0.4)	Large (0.6)	Small (0.2)	Mod. (0.4)	Large (0.6)	Small (0.2)	Mod. (0.4)	Large (0.6)
Structural Systems with Very Small Capacity Curve Variability[5] ($\beta_C = 0.1$)									
Structure	0.60	0.70	0.80	0.70	0.80	0.90	0.85	0.95	1.05
NSD	0.60	0.70	0.80	0.80	0.85	0.95	0.90	1.00	1.10
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Small Capacity Curve Variability[5] ($\beta_C = 0.2$)									
Structure	0.65	0.75	0.85	0.75	0.85	0.95	0.95	1.00	1.10
NSD	0.65	0.70	0.85	0.80	0.85	1.00	0.95	1.00	1.10
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Moderate Capacity Curve Variability[5] ($\beta_C = 0.3$)									
Structure	0.65	0.75	0.85	0.80	0.85	0.95	0.95	1.00	1.10
NSD	0.65	0.75	0.85	0.80	0.90	1.00	0.95	1.05	1.15
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Large Capacity Curve Variability[5] ($\beta_C = 0.4$)									
Structure	0.70	0.75	0.90	0.80	0.90	1.00	1.00	1.05	1.15
NSD	0.70	0.75	0.90	0.85	0.90	1.00	1.00	1.05	1.15
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65

1. Building Systems include the Structure, Nonstructural Drift-Sensitive Components (NSD) and Nonstructural Acceleration-Sensitive (NSA) components.

Table 3.11 Mid-Rise Building Fragility Beta's

Building System[2]	Post-Yield Degradation of Structural System[3]								
	Minor Degradation ($\kappa >= 0.9$)			Major Degradation ($\kappa = 0.5$)			Extreme Degradation ($\kappa <= 0.1$)		
	Damage Variability[4] ($\beta_{T,ds}$)			Damage Variability[4] ($\beta_{T,ds}$)			Damage Variability[4] ($\beta_{T,ds}$)		
	Small (0.2)	Mod. (0.4)	Large (0.6)	Small (0.2)	Mod. (0.4)	Large (0.6)	Small (0.2)	Mod. (0.4)	Large (0.6)
Structural Systems with Very Small Capacity Curve Variability[5] ($\beta_C = 0.1$)									
Structure	0.55	0.65	0.80	0.65	0.75	0.85	0.80	0.90	1.00
NSD	0.55	0.65	0.80	0.75	0.80	0.95	0.90	0.95	1.05
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Small Capacity Curve Variability[5] ($\beta_C = 0.2$)									
Structure	0.60	0.65	0.80	0.70	0.80	0.90	0.90	0.95	1.05
NSD	0.60	0.70	0.80	0.75	0.85	0.95	0.95	1.00	1.10
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Moderate Capacity Curve Variability[5] ($\beta_C = 0.3$)									
Structure	0.60	0.70	0.80	0.70	0.80	0.90	0.95	1.00	1.10
NSD	0.60	0.70	0.85	0.80	0.85	0.95	0.95	1.05	1.15
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65
Structural Systems with Large Capacity Curve Variability[5] ($\beta_C = 0.4$)									
Structure	0.60	0.70	0.85	0.75	0.80	0.95	0.95	1.00	1.10
NSD	0.60	0.70	0.85	0.80	0.90	1.00	1.00	1.05	1.15
NSA	0.35	0.50	0.65	0.35	0.50	0.65	0.35	0.50	0.65

1. Building Systems include the Structure, Nonstructural Drift-Sensitive Components (NSD) and Nonstructural Acceleration-Sensitive (NSA) components.

Table 3.12 High-Rise Building Fragility Beta's

http://www.architectjaved.com/nonlinear-static-pushover-analysis/conclusion-nonlinear-static-pushover-analysis.html