CHAPTER 4

PERFORMANCE BASED SEISMIC ANALYSIS AND DESIGN

4.1 PERFORMANCE OBJECTIVES

A performance objective specifies the desired seismic performance of the building. Seismic performance is described by designing the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion). A performance objective includes consideration of damage states to several levels of ground motion. General structural performance levels are as follows (**Fig. 4.1**):

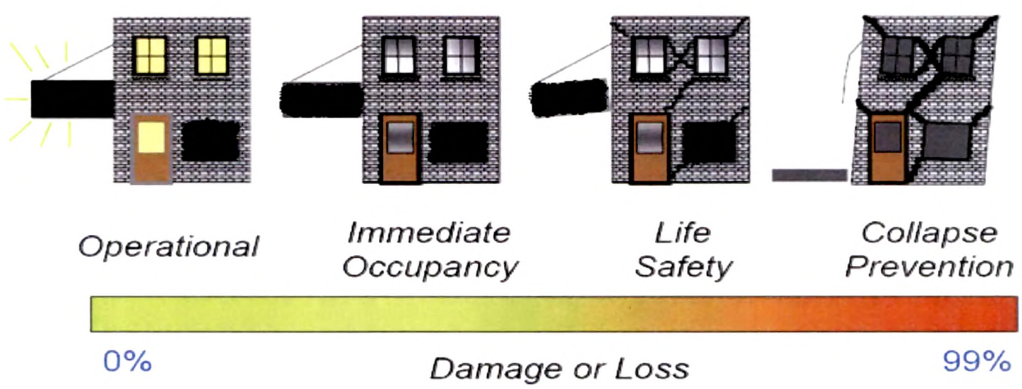


Fig. 4.1 Standard Structural Performance Levels

Operational Level

- Negligible structural and non-structural damage
- Occupants are safe during event
- Utilities are available
- Facility is available for immediate re-use (some cleanup required)
- Loss < 5% of replacement value.

Immediate Occupancy Level

- Negligible structural damage
- Occupants safe during event

- Minor non-structural damage
- Building is safe to occupy but may not function
- Limited interruption of operations
- Losses < 15%.

Life Safety Level

- Significant structural damage
- Some injuries may occur
- Extensive non-structural damage
- Building not safe for reoccupancy until repaired
- Losses < 30%.

Collapse Prevention Level

- Extensive (near complete) structural and non-structural damage
- Significant potential for injury but not wide scale loss of life
- Extended loss of use
- Repair may not be practical
- Loss >> 30%.

While performance based design specifically intends to limit the consequences of one or more perils (forces which causes disaster) to defined acceptable levels, one can limit the safety of the structure as per requirement like in immediate occupancy stage or life safety stage or collapse prevention stage etc. Perils addressed are wind, fire, snow, earthquake, live loads etc.

4.2 ATC-40 PROVISIONS

ATC stands for Applied Technology Council which is formed in the United States of America. The main objective of this council is seismic evaluation and retrofit of concrete buildings. Although the procedures recommended in this document are for concrete buildings, they are applicable to almost all types of buildings. This document provides guidelines to evaluate and retrofit concrete structures using performance based objectives. Following steps are recommended by ATC-40 [1] for evaluation and retrofitting:

1. Determine the primary goal of the project.
2. Select engineering professionals with experience in the analysis, design and retrofit of the buildings in seismically hazardous region.
3. Decide performance objectives for specific level of seismic hazard.
4. Do regular site visits and review drawings.
5. Check whether applied nonlinear procedure is appropriate or not for selected structure.
6. Check quality control measures.
7. Perform nonlinear static analysis if appropriate.
8. Determine inelastic curve known as pushover curve and convert to capacity spectrum.
9. Obtain response spectrum and convert to ADRS format.
10. Obtain performance point.
11. Prepare construction documents and
12. Monitor construction quality.

A performance objective has two essential parts – a damage state and level of seismic hazard. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion). A performance objective may include consideration of damage states for several levels of ground motion and would then be termed a dual or multiple level performance objectives.

The target performance objective is split into structural performance level (SP-n, Where n is the designated number) and non-structural performance level (NP-n, Where n is the designated letter). These may be specified independently. However, the combination of the two determines the overall building performance level.

4.2.1 Structural Performance Levels

Structural performance levels are defined as follows:

1. *Immediate Occupancy (SP-1)*: Limited structural damage with the basic vertical and lateral force resisting system retaining most of their pre-earthquake characteristics and capacities.
2. *Damage Control (SP-2)*: A placeholder for a state of damage somewhere between immediate occupancy and life safety.
3. *Life Safety (SP-3)*: Significant damage with some margin against total or partial collapse. Injuries may occur with the risk of life-threatening injury being low. Repair may not be economically feasible.
4. *Limited Stability (SP-4)*: A placeholder for a state of damage somewhere between life safety and structural stability.
5. *Structural Stability (SP-5)*: Substantial structural damage in which the structural system is on the verge of experiencing partial or total collapse. Significant risk of injury exists. Repair may not be technically or economically feasible.
6. *Not Considered (SP-6)*: Placeholder for situations where only non-structural seismic evaluation or retrofit is performed.

4.2.2 Non-structural Performance Levels

Non-structural performance levels are defined as follows:

1. *Operational (NP-A)*: Non-structural elements are generally in place and functional. Backup systems for failure of external utilities, communications and transportation have been provided.
2. *Immediate Occupancy (NP-B)*: Non-structural elements are generally in place but may not be functional. No backup system for failure of external utilities is provided.

3. *Life Safety (NP-C)*: Considerable damage to non-structural components and systems but no collapse of heavy items. Secondary hazards such as breaks in high pressure, toxic or fire suppression piping should not be present.
4. *Reduced Hazards (NP-D)*: Extensive damage to non-structural components but should not include collapse of large and heavy items that can cause significant injury to groups of people.
5. *Not Considered (NP-E)*: Non-structural elements, other than those that have an effect on structural response are not evaluated.

Table 4.1 shows combination of structural and non-structural performance levels to obtain a building's performance level.

Table 4.1 Structural and Non-Structural Performance

Building Performance Levels						
Non-Structural performance levels	Structural performance levels					
	SP-1 Immediate Occupancy	SP-2 Damage Control	SP-3 Life Safety	SP-4 Limited Safety	SP-5 Structural Stability	SP-6 Not Considered
NP-A Operational	1-A Operational	2-A	NR	NR	NR	NR
NP-B Immediate Occupancy	1-B Immediate Occupancy	2-B	3-B	NR	NR	NR
NP-C Life Safety	1-C	2-C	3-C Life Safety	4-C	5-C	6-C
NP-D Reduced Hazards	NR	2-D	3-D	4-D	5-D	6-D
NP-E Not Considered	NR	NR	3-E	4-E	5-E Structural Stability	Not Applicable

(white)	Commonly Referenced Building Performance Levels (SP-NP)
(blue)	Other possible combinations of SP-NP
(red)	Not recommended combinations of SP-NP

4.3 POTENTIAL PLASTIC HINGE ZONES

Location of plastic hinges in the structure is important because plastic hinges cause excessive deformation. In plastic hinge regions, rotation of the member is very high which leads to failure. Location of plastic hinges in beams must be clearly identified since special detailing requirements are needed in inelastic regions of beams of frames subjected to earthquake forces.

In capacity design of structures for earthquake resistance, distinct element of primary lateral force resisting systems are chosen and suitably designed and detailed for energy dissipation under several imposed deformations. So these critical regions are well detailed. In capacity design concept, potential plastic hinge regions within structure are clearly defined. These are designed to have dependable flexural strengths as close as practicable to the required strength. Subsequently, these regions are carefully detailed to ensure that estimated ductility demands in these regions can be reliably accommodated. This is achieved primarily by closely-spaced and well anchored transverse reinforcement.

Negative moment plastic hinge: Plastic hinges in the beams of frames, the design of which is dominated by seismic actions, commonly develop immediately adjacent to the sides of columns, as shown for the short span beams of the frame in **Fig. 4.2**. These hinges generally form in short span beams and adjacent to the face of the columns or at the maximum negative moment regions. As shown in **Fig. 4.2**, the rotation of the beams at the plastic hinges is θ and due to this rotation length is increased by Δl.

Positive moment plastic hinge: When the positive moments in the span become large because of the dominance of gravity loading, particularly in long-span beams, it may be difficult, if not impossible, to develop a plastic hinge at a face of a column. The designer may then decide to allow plastic hinge to form at some distance away from the column. Typical examples are shown for the long-span beams in **Fig. 4.3**.

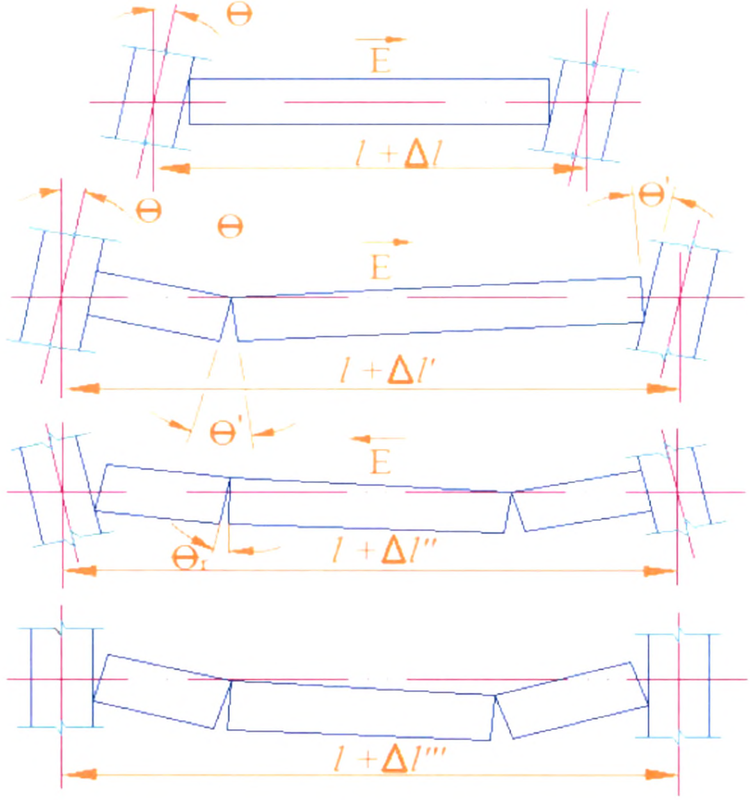


Fig. 4.2 Plastic Hinge Pattern by Seismic Actions

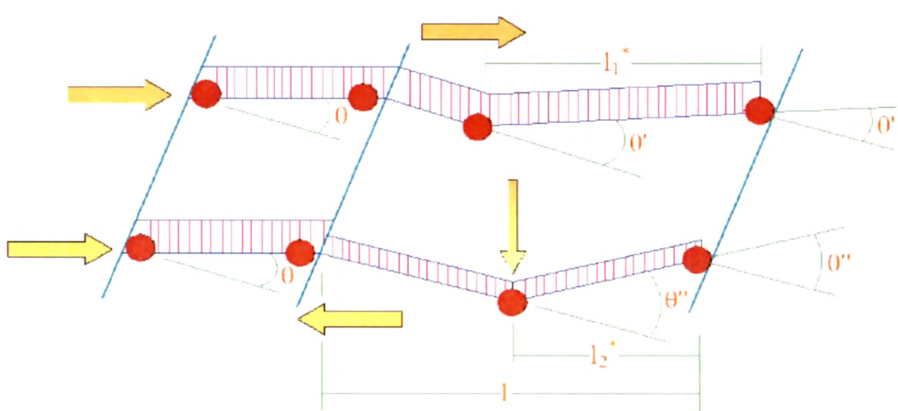


Fig. 4.3 Plastic Hinge Pattern Dominated by Gravity Loading

When earthquake forces act as shown, the positive moment plastic hinge in the top beam of **Fig. 4.3** will develop close to the inner column, at the location of the maximum moment. If plastic hinges develop at column faces, hinge plastic rotation would be θ, as seen in the short span. However, with a positive hinge forming a distance $l1^*$ from the right column, the hinge plastic rotations will increase to $\theta' = (l/l1^*)$.

It is also evident from **Fig. 4.3** that the farther the positive plastic hinge is from the left-hand column, the larger will be the hinge rotations. In the lower face of the long-span beams, the presence of a significant point load at mid span indicates that this will probably the location of maximum positive moment under the action of gravity load and seismic forces. If the plastic hinge is located there, the plastic rotations would increase to $\theta'' = (l/l2^*)\ \theta$.

4.4 EVALUATION OF PERFORMANCE

Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the analysis of existing concrete buildings. Elastic analysis methods include code static lateral force procedures, code dynamic lateral force procedures and elastic procedures using demand capacity ratios. The most basic inelastic analysis method is the complete nonlinear time history analysis. Simplified nonlinear analysis methods, referred to as nonlinear static analysis procedures, include the Capacity Spectrum Method (CSM) that uses the intersection of the capacity (pushover) curve and a reduced response spectrum to estimate maximum displacement; the displacement coefficient method that uses pushover analysis and a modified version of the equal displacement approximation to estimate maximum displacement; and the secant method that uses a substitute structure and secant stiffnesses.

Although an elastic analysis gives a good indication of the elastic capacity of structures and indicates where first yielding will occur, it cannot predict failure mechanism and account for redistribution of forces during progressive yielding. Inelastic analysis procedures demonstrate how building really behave by identifying modes of failure and the potential for progressive collapse. The use of inelastic procedures for design and evaluation is an attempt to help engineers better understand how structures will behave when subjected to major earthquakes, where it is assumed that the elastic capacity of the structure will be exceeded. This resolves some of the uncertainties associated with code and elastic procedures.

The capacity spectrum method, a nonlinear static procedure that provides a graphical representation of the global force-displacement capacity curve of the structure (i.e., pushover) and compares it to the response spectra representations of the earthquake demands, is a very useful tool in the evaluation and retrofit design of existing concrete buildings. The graphical representation provides a clear picture of how a building responds to earthquake ground motion, and, as illustrated in this chapter, it provides an immediate and clear picture of how various retrofit or safeguard strategies, such as adding stiffness or strength, will affect the building's response to earthquake demands.

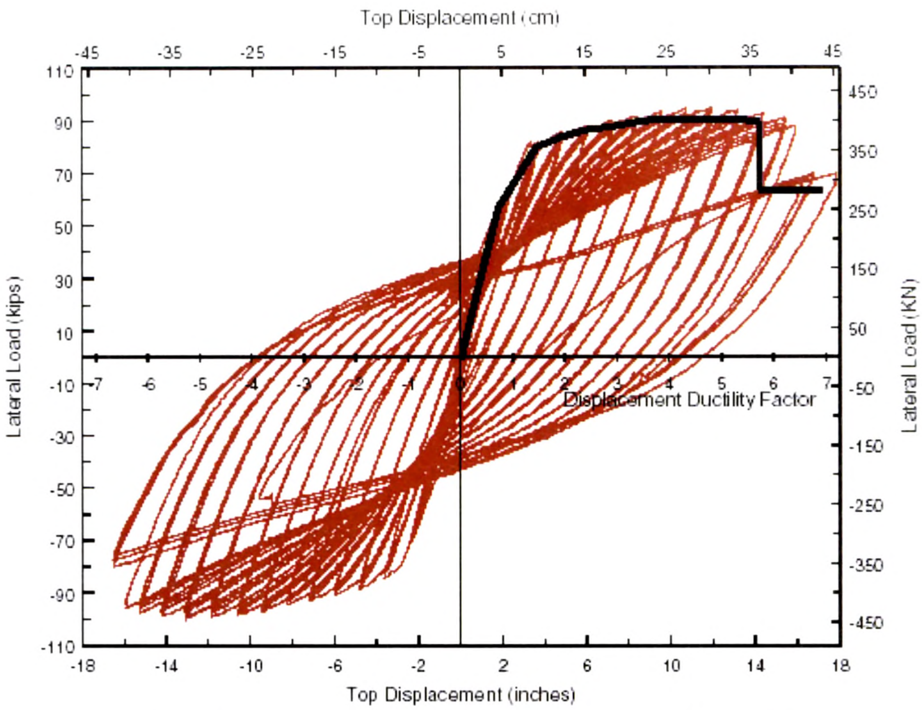
4.5 METHODS TO PERFORM NONLINEAR ANALYSIS

Two key elements of a performance-based design procedure are demand and capacity. Demand is a representation of the earthquake ground motion. Capacity is a representation of the structure's ability to resist the seismic demand. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design. The key step for the entire analysis is identification of the primary structural elements, which should be completely modeled in the non-linear analysis. Secondary elements, which do not significantly contribute to the building's lateral force resisting system, need not be included in the analysis.

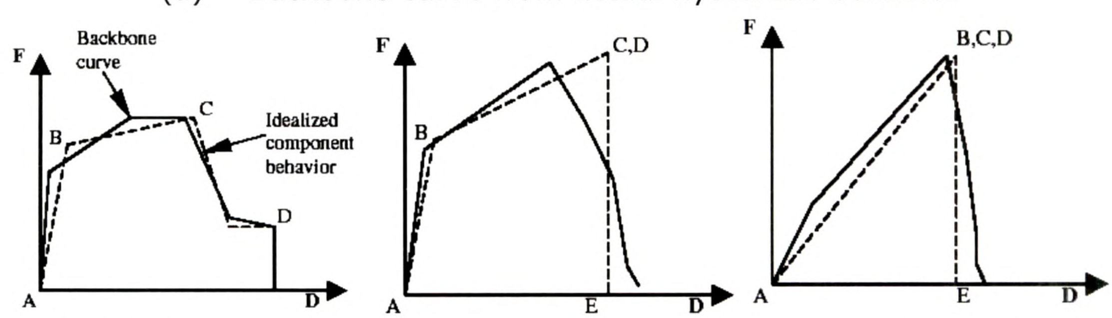
It is important to identify the failure mechanism for primary structural elements in RC structures and define their non-linear properties accordingly. Earthquakes usually load these elements in a cyclic manner. as shown in **Fig. 4.4 (a)**. For modeling and analysis purposes, these relationship can be idealized as shown in **Fig. 4.4 (b)** using a combination of empirical data, theoretical strength and strain compatibility.

During the pushover process of developing the capacity curve as brittle elements degrade, ductile elements take over the resistance and the result is a saw tooth shape that helps visualize the performance. Once the global displacement demand is estimated for a specific seismic hazard, the model is used to predict the resulting deformation in each component. The ATC-40 **[1]** document provides acceptability limits for component deformations depending on the specified performance level.

It may be noted here that the portion between points B and C shown in Fig. 4.4 (b) for ductile structures is further subdivided into three parts corresponding to the immediate occupancy, life safety and collapse prevention stages.



(a) Backbone curve from actual hysteretic behavior



Ductile (Disp. Controlled) Semi ductile Brittle(Force Controlled)

(b) Idealized component behavior from backbone curve

Fig. 4.4 Idealized Component Force-Displacement Relationship

Simplified nonlinear analysis procedures using pushover methods, such as the capacity spectrum method and the displacement coefficient method, require determination of three primary elements: capacity, demand (displacement) and performance. Each of these elements is briefly discussed below.

4.5.1 Capacity

The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits, some form of nonlinear analysis, such as the pushover procedure, is required. This procedure uses a series of sequential elastic analyses, superimposed to approximate a force-displacement capacity diagram of the overall structure. The mathematical model of the structure is modified to account for reduced resistance of yielding components. A lateral force distribution is again applied until additional components yield. This process is continued until the structure becomes unstable or until a predetermined limit is reached. For two-dimensional models, computer programs are available that directly model nonlinear behavior and can create a pushover curve directly. The pushover curve approximates how structures behave after exceeding their elastic limit.

4.5.2 Demand

Ground motions during an earthquake produce complex horizontal displacement pattern in structures that may vary with time. Tracking this motion at every time-step to determine structural design requirements is impractical. Traditional linear analysis methods use lateral forces to represent a design condition. For nonlinear methods it is easier to use a set of lateral displacements as a design condition. For a given structure and ground motion, the displacement demand is an estimate of the maximum expected response of the building during the ground motion.

4.5.3 Performance

Once a capacity curve and demand displacement is defined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacements implied by the displacement demand.

4.6 STEPS TO DETERMINE CAPACITY

Structure's capacity is represented by a pushover curve. The most convenient way to plot the force-displacement curve is by tracking the base shear and the roof displacement. The capacity curve is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This is generally valid for buildings with fundamental periods of vibration up to about one second. For more flexible buildings with a fundamental period greater than one second, the analyst should consider addressing higher mode effects in the analysis.

The steps for determining capacity can be listed as:

1. Create a computer model of the structure and the foundation, following the corresponding modeling rules.
2. Classify each element in the model as either primary or secondary.
3. Apply lateral story forces to the structure in proportion to the product of the mass and fundamental mode shape. This analysis should also include gravity loads.
4. Calculate member forces for the required combinations of vertical and lateral loads.
5. Adjust the lateral force level so that some element (or group of elements) is stressed to within 10 percent of its member strength. The element may be, for example, a joint in a moment frame, a strut in a braced frame, or a shear wall. Having reached its member strength, the element is considered to be incapable of taking addition lateral load. For structure with many elements, tracking and sequencing the

analysis at each and every element yield is time consuming and unnecessary. In such cases, elements should be grouped together at similar yield points. Most structures can be properly analyzed using less than 10 sequences, with many simple structures requiring only 3 or 4.

6. Record the base shear and the roof displacement. Also record member forces and rotations as they will be needed for the performance check.
7. Revise the model using zero (small) stiffness for the yielding elements.
8. Apply a new increment of lateral load to the revised structure such that another element (or group of elements) yields. The actual forces and rotations for elements at the beginning of an increment are equal to those at the end of the previous increment. However, each application of an increment of lateral load is a separate analysis, which starts from zero initial conditions. Thus, to determine when the next element yields, it is necessary to add the forces from the current analysis to the sum of those from the previous increments. Similarly, to add the rotations form the current analysis to the sum of those from the previous increments.
9. Add the increment of lateral load and the corresponding increment of roof displacement to the previous totals to give the accumulated values of base shear and roof displacement.
10. Repeat steps 7, 8 and 9 until the structure reaches an ultimate limit, such as: instability from P-Δ effect; distortions considerably beyond the desired performance level; an element (or group of elements) reaching a lateral deformation level at which significant strength degradation begins; or an element (or group of elements) reaching a lateral deformation level at which loss of gravity load carrying capacity occurs. **Fig. 4.5** represents a typical capacity curve.

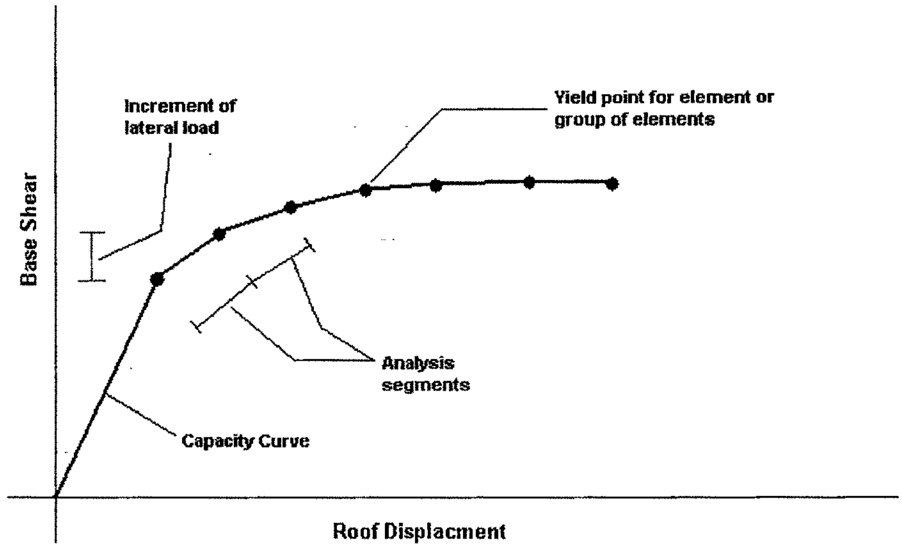


Fig. 4.5 Capacity Curve

11. Explicitly model global strength degradation. If the incremental loading was stopped in step 10 as a result of reaching a lateral deformation level at which all or a significant portion of an element's (or group of elements) load can no longer be resisted, that is, its strength has significantly degraded, then the stiffness of that element(s) is reduced, or eliminated. A new capacity curve is then created, starting with step 3 of this step-by-step process. Create as many additional pushover curves as necessary to adequately define the overall loss of strength. **Figure 4.6** illustrates the process, for an example where three different capacity curves are required.

Plot the final capacity curve to initially follow the first curve then transition to the second curve at the displacement corresponding to the initial strength degradation, and so on. This curve will have a "saw tooth" shape, as shown in **Fig. 4.7.**

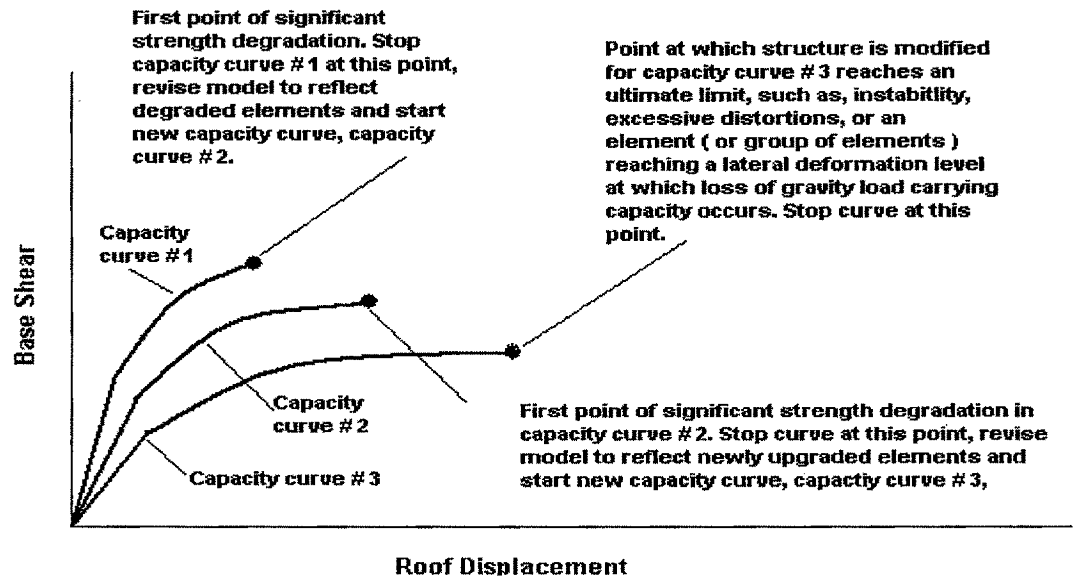


Fig. 4.6 Multiple Capacity Curve to Model Strength Degradation

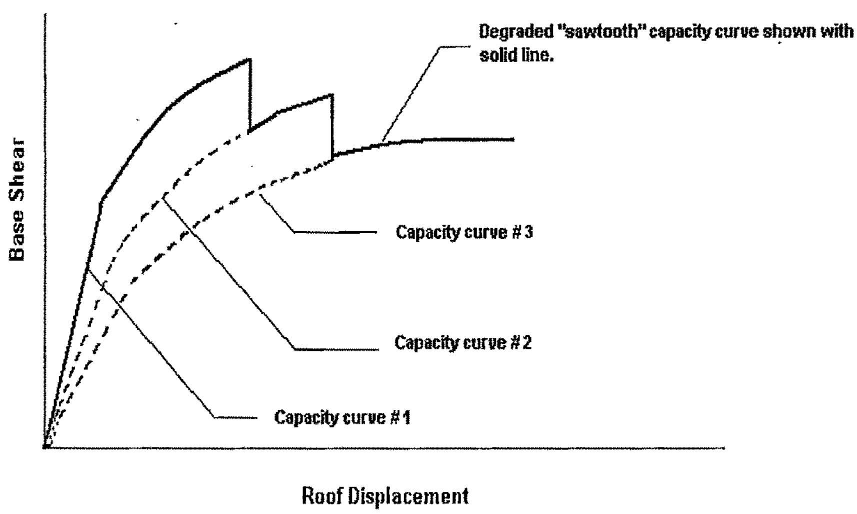


Fig. 4.7 Capacity Curve with Global Strength Degradation

Modeling global strength degradation requires considerable judgment. If strength degradation of over 20 percent is explicitly modeled, then the actual expected behavior of the degrading elements should be carefully reviewed. The sensitivity of the estimated demand displacement to the modeling assumptions should be checked by bounding the response with a range of assumptions.

4.7 STEPS TO DETERMINE DEMAND

Development of a capacity curve for an existing building, in itself, is extremely useful to the engineer, and will yield insights into the building's performance characteristics as well as methods of retrofit. However, to judge acceptability for a given performance objective, either for the existing condition or for a retrofit scheme, the probable maximum displacement for the specified ground motion must be estimated.

The capacity spectrum method is based on finding a point on the capacity spectrum that also lies on the appropriate demand response spectrum, reduced for nonlinear effects, and is most consistent in terms of graphical representation. The demand displacement in the capacity spectrum method occurs at a point on the capacity spectrum called the performance point. This performance point represents the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion.

The method used in FEMA-273 [2] is sometimes called the coefficient method. The coefficient method is based on statistical analysis of the results of time history analysis of single degree of freedom models of different types. The demand displacement in the coefficient method is called the target displacement. An estimate of the displacement due to a given seismic demand may be made using a simple technique called the equal displacement approximation. As shown in **Fig. 4.8**, this approximation is based on the assumption that the inelastic spectral displacement is the same as that, which would occur if the structure remained perfectly elastic. In some cases, particularly in the equal displacement approximation will usually yield results similar to the capacity spectrum and coefficient methods. In other cases, particularly in the short period, range (T< 0.5 second) the displacements obtained form the simple approximation may be significantly different from (less than) the results obtained using the capacity spectrum and coefficient methods. The target displacement obtained using the displacement coefficient method is equal to the displacement obtained using the equal displacement approximation modified by various coefficients.

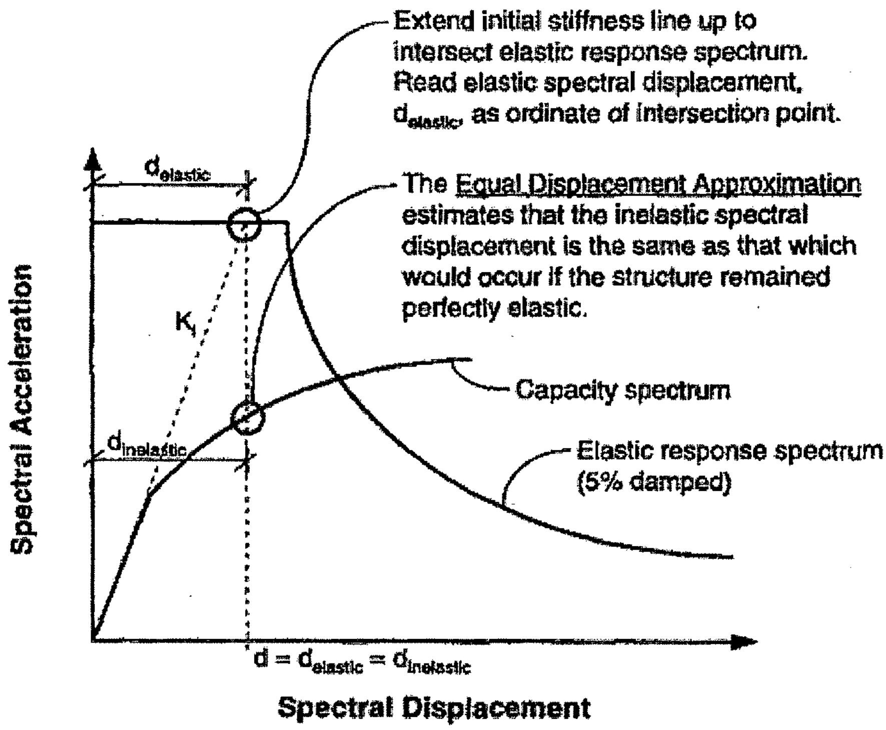


Fig. 4.8 Equal Displacement Approximation

4.7.1 Calculating Demand using the Capacity Spectrum Method

The location of the performance point must satisfy following two relationships: 1) lie on the capacity spectrum curve in order to represent the structure at a given displacement, and 2) the point must lie on a spectral demand curve, reduced from the elastic, 5 percent-demand curve (design spectrum), that represents the nonlinear demand at the same structural displacement. For this methodology, spectral reduction factors are given in terms of effective damping and are calculated based, on the shape of the capacity curve, the estimated displacement demand, and the resulting hysteresis loop. Probable imperfections in real building hysteresis loops, including degradation and duration effects, are accounted for by reduction in theoretically calculated equivalent viscous damping values.

In the general case, determination of the performance point requires a trial and error search for satisfaction of the two criterion specified above. However, this section contains three different procedures that standardize and simplify this iterative process. These alternate procedures are all based on the same concepts and mathematical relationships but vary in their dependence on analytical versus graphical technique.

A bilinear representation of the capacity spectrum is needed to estimate the effective damping and appropriate reduction of spectral demand. Construction of the bilinear representation requires definition of the point. This point is the trial performance point, which is estimated by the engineer to develop a reduced demand response spectrum. If the reduced response spectrum is found to intersect the capacity spectrum at the estimated a_{pi} , d_{pi} point, then that point is the performance point.

Refer to **Fig. 4.9** for an example of bilinear representation of a capacity spectrum. To construct the bilinear representation draw one line up from the origin at the initial stiffness of the building using element stiffnesses as shown in **Fig. 4.9**. Draw a second line back from the trial performance point, a_{pi}, d_{pi}. Slope the second line such that when it intersects the first line, at any point ay, dy, the area designated A1 in the figure is approximately equal to the area designated by A2. The intent of setting area A1 equal to area A2 is to have equal area under the capacity spectrum and its bilinear representation, that is, to have equal energy associated with each curve.

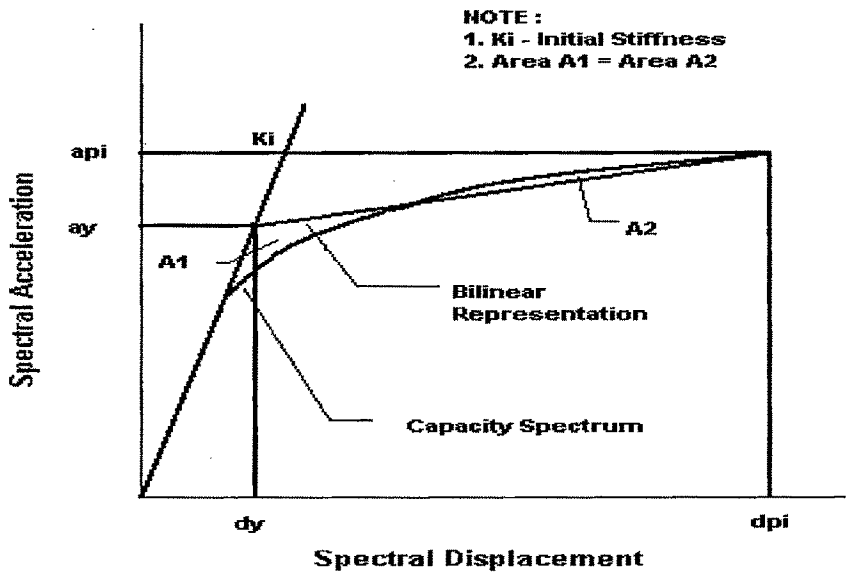


Fig. 4.9 Bilinear Representation of Capacity Spectrum

In the case of a "sawtooth" capacity spectrum, the bilinear representation should be based on the capacity spectrum curve, which describes behavior at displacement d_{pi}, as shown in **Fig. 4.10**.

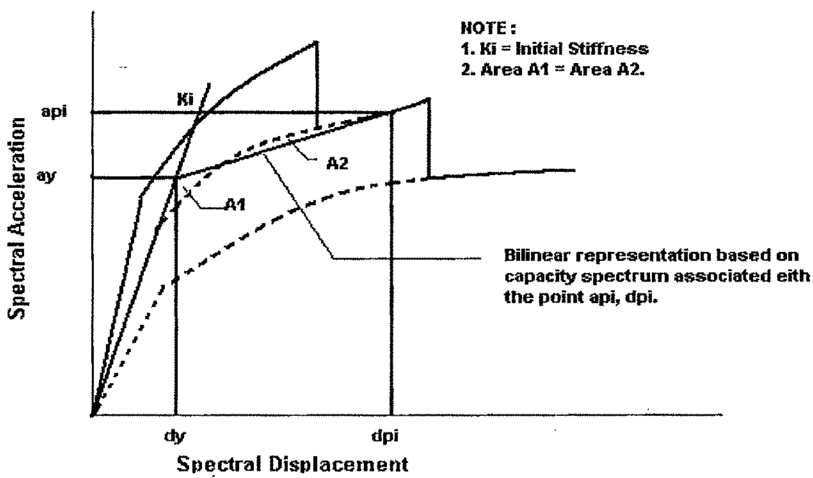


Fig. 4.10 Bilinear Representation for Degrading System

4.7.2 Estimation of Damping and Reduction of Response Spectrum

The damping that occurs when earthquake ground motion drives a structure into the inelastic range can be viewed, as a combination of viscous damping that is inherent in the structure and hysteretic damping.

Hysteretic damping is related to the area inside the loops that are formed when the earthquake force (base shear) is plotted against the structure displacement.

The equivalent viscous damping, β_{eq}, associated with a maximum displacement of d_{pi}, can be estimated form the following equation:

$$\beta_{eq} = \beta_o + 0.05 \qquad .....(4.1)$$

Where, β_o is hysteretic damping represented as equivalent viscous damping, 0.05 represents 5% viscous damping inherent in the structure (assumed to be constant).

The term β_o can be calculated as (Chopra 1995 **[73]**):

$$\beta_o = \frac{1}{4\pi}\frac{E_D}{E_{So}} \qquad .....(4.2)$$

where, E_D = energy dissipated by damping and E_{So} = maximum stain energy. The physical significance of the terms E_D and E_{So} in **Eq. 4.2** is illustrated in **Fig. 4.11**. E_D is the energy dissipated by the structure in a single cycle of motion, that is, the area enclosed by a single hysteresis loop. E_{So} is the maximum strain energy associated with that cycle of motion, that is, the area of the shaded triangle.

Referring to **Fig. 4.12**, the term E_D can be derived as follows:

$$\begin{aligned} E_D &= 4 \times (\text{shaded area in figures}) \\ &= 4\,(a_{pi}\,d_{pi} - 2A_1 - 2A_2 - 2A_3) \\ &= 4\,[(a_{pi}\,d_{pi} - a_y\,d_y - (p_{di} - d_y)\,(a_{pi} - a_y) - 2d_y\,(a_{pi} - a_y)] \\ &= 4\,(a_y\,d_{pi} - d_y\,a_{pi}) \qquad .....(4.3) \end{aligned}$$

Referring to **Fig. 4.11**, the term E_{So} can be derived as:

$$E_{So} = a_{pi}\,d_{pi} / 2 \qquad .....(4.4)$$

Note that E_{So} could also be written as $k_{effective}\,d_{pi}^2 / 2$

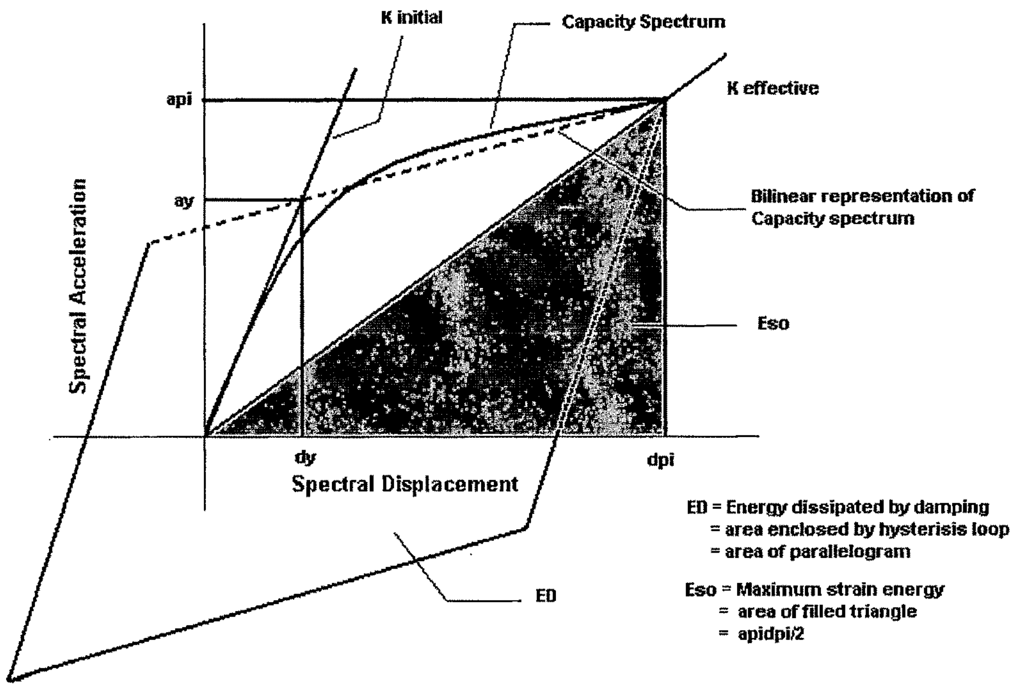


Fig. 4.11 Derivation of Damping for Spectrum Reduction

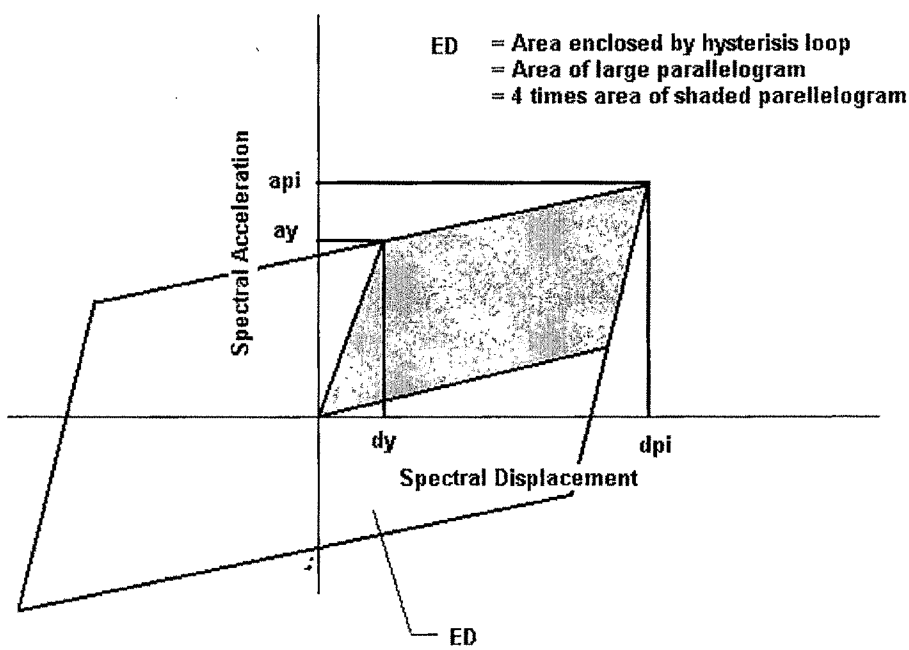


Fig. 4.12 Derivation of Energy Dissipated by Damping, E_D

Thus, β_o can be written as:

$$\beta_o = \frac{1}{4\pi}\,\frac{4\,(a_y\,d_{pi} - d_y\,a_{pi})}{a_{pi}\,d_{pi}\,/\,2} = \frac{2\,(a_y\,d_{pi} - d_y\,a_{pi})}{\pi\,a_{pi}\,d_{pi}}$$

$$\beta_o = \frac{0.637\,(a_y\,d_{pi} - d_y\,a_{pi})}{a_{pi}\,d_{pi}} \qquad \text{.....(4.5)}$$

and when β_o is written in terms of percent damping, the equation becomes:

$$\beta_o = \frac{63.7\,(a_y\,d_{pi} - d_y\,a_{pi})}{a_{pi}\,d_{pi}} \qquad \text{.....(4.6)}$$

Thus β_{eq} becomes:

$$\beta_{eq} = \beta_o + 5 = \frac{0.637\,(a_y\,d_{pi} - d_y\,a_{pi})}{a_{pi}\,d_{pi}} + 5 \qquad \text{.....(4.7)}$$

The equivalent viscous damping values obtained from **Eq. 4.7** can be used to estimate spectral reduction factors using relationships developed by Newmark and Hall **[74]**. As shown in **Fig. 4.13**, spectral reduction factors are used to decrease the elastic (5% damped) response spectrum to a reduced response spectrum with damping greater than 5% of critical damping. For damping values less than about 25 percent, spectral reduction factors calculated using the β_{eq} from **Eq. 4.7** and Newmark and Hall equation are consistent with similar factors contained in base isolation codes and in the FEMA guidelines. These factors are presented in these documents as the damping coefficient, B, which is equal to 1/SR. The spectra should not be reduced to this extent at higher damping values and one should judgmentally increase the coefficients starting at about 25 percent of the damping (increasing the damping coefficient B is the same as decreasing the spectral reduction factor SR, the net result is that the spectra are reduced less), as well as set an absolute limit on reductions at a β_{eq} of about 50 percent.

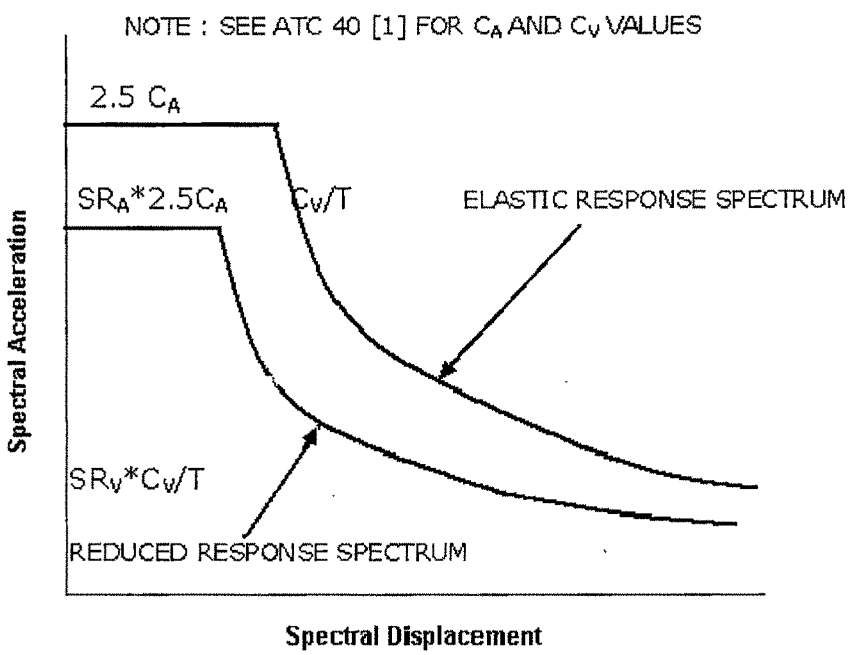


Fig. 4.13 Reduced Response Spectrum

The damping coefficient B, which is used to reduce the elastic (5% damped) spectrum, should not be confused with the damping β. The damping coefficient B is derived from a formula, which includes the variable β.

The idealized hysteresis loop shown in **Fig. 4.11**, is a reasonable approximation for a ductile detailed building subjected to relatively short duration ground shaking (not enough cycles to significantly degrade elements) and with equivalent viscous damping less than approximately 30%. For conditions other than these, the idealized hysteresis loops of **Fig. 4.11** lead to overestimates of equivalent viscous damping because the actual hysteresis loops are imperfect, that is, they are reduced in area, or pinched. ATC 40 **[1]** addresses existing reinforced concrete buildings that are not typically ductile structures. For such buildings, calculation of the equivalent viscous damping using **Eq. 4.9** and the idealized hysteresis loop in **Fig. 4.11** yields results that overestimate realistic levels of damping. Here, in order to be consistent with these previously developed coefficients, B, as well as to enable simulation of imperfect hysteresis loops (loops reduced in area), the concept of effective viscous damping using a damping modification factor, K, has been introduced. Effective viscous damping, β_{eff}, is defined by:

$$\beta_{eff} = K\,\beta_o + 5 = \frac{63.7\ K\,(a_y\,d_{pi} - d_y\,a_{pi})}{a_{pi}\,d_{pi}} + 5 \quad \text{.....}\ (4.8)$$

Note, that **Eq. 4.8** is identical to **Eq. 4.7** except that the k-factor has been introduced to modify the first (β_o) term.

The k-factor is a measure of the extent to which the actual building hysteresis is well represented by the parallelogram of **Fig. 4.11**, either initially, or after degradation. The k-factor depends on the structural behavior of the building, which in turn depends on the quality of the seismic resisting system and the duration of ground shaking. For simplicity, ATC 40 simulates three categories of structural behavior. Structural behavior Type A represents stable, reasonably full hysteresis loops most similar to **Fig. 4.11**, and is assigned a k of 1.0 (except at higher damping values as discussed above). Type B is assigned a basic k of 2/3 and represents a moderate reduction of area (k is also reduced at higher values of β_{eff} to be consistent with the Type A relationship). Type C represents poor hysteretic behavior with a substantial reduction of loop area (severely pinched) and is assigned as k of 1/3. For details of various types e.g. the ranges and limits for the values of k assigned to the three structural behavior types are given in **Table 4.2**

Table 4.2 Values for Damping Modification Value.

Structural Behavior Type	β0 (percent)	k
Type A	≤16.25	1.0
	≥16.25	$1.13-0.51(a_y d_{pi}-d_y a_{pi})/a_{pi} d_{pi}$
Type B	≤25	0.67
	≥25	$0.845-0.446(a_y d_{pi}-d_y a_{pi})/a_{pi} d_{pi}$
Type C	Any Value	0.33

4.8 STEPS FOR CHECKING PERFORMANCE

The following steps should be followed in the performance check as per ATC-40 **[1]**.

1. For the global buildings response verify the following:
   a. The lateral force resistance has not degraded by more than 20 percent of the peak resistance.
   b. The structure should satisfy the serviceability check i.e. the lateral drifts should not be more than the limits imposed.
2. Identify and classify the different elements in the building. Any type of the element may be present: beam-column frames, slab-column frames, solid walls, perforated walls, punched walls, floor diaphragm and foundations.
3. Identify all primary and secondary elements.
4. For each element type, identify the critical components and actions to check as detailed in Chapter 11 of ATC 40 **[1]**.
5. The strength and deformation demands at the structure's performance point shall be equal to or less than the capacities detailed in Chapter 11 of ATC40 considering all co-existing forces acting with the demand spectrum.
6. The performance of secondary elements (such as gravity load carrying members, not a part of the lateral load resisting system) shall be

reviewed for acceptability for the specified performance level.

7. Non-structural elements shall be checked for acceptability for the specified performance level.

4.9 CONVERSION OF DEMAND AND CAPACITY TO ADRS FORMAT

Application of the capacity-spectrum technique requires that both the demand response spectrum and structural capacity (or Pushover) spectrum be plotted in the spectral acceleration vs spectral displacement domain. Spectra plotted in this format are known as Acceleration Displacement Response Spectra (ADRS) after Mahaney et al. **[75]**.

4.9.1 Converting Demand Spectrum to ADRS Format

Every point on a response spectrum curve has associated with it, a unique spectral acceleration, S_a, spectral velocity, S_v, spectral dispacement, S_d and period, T. To convert a spectrum from the standard S_a versus T format found in the building code to ADRS format, it is necessary to determine the value of S_{di} of each point on the curve, (S_{ai}, T_i). This can be done using **Eq. 4.9**

$$S_{di} = S_{ai}T_i^2/4\,\pi^2 \qquad .....(4.9)$$

Standard demand response spectra contain a range of constant spectral acceleration and a second range of constant spectral velocity. Spectral acceleration and displacement at period T_i are given by **Eqs. 4.10 and 4.11**. This process of conversion is graphically shown in **Fig. 4.14.**

$$S_{ai} = 2\pi\, S_v / T_i \qquad .....(4.10)$$

$$S_{di} = T_i\, S_v / 2\pi \qquad .....(4.11)$$

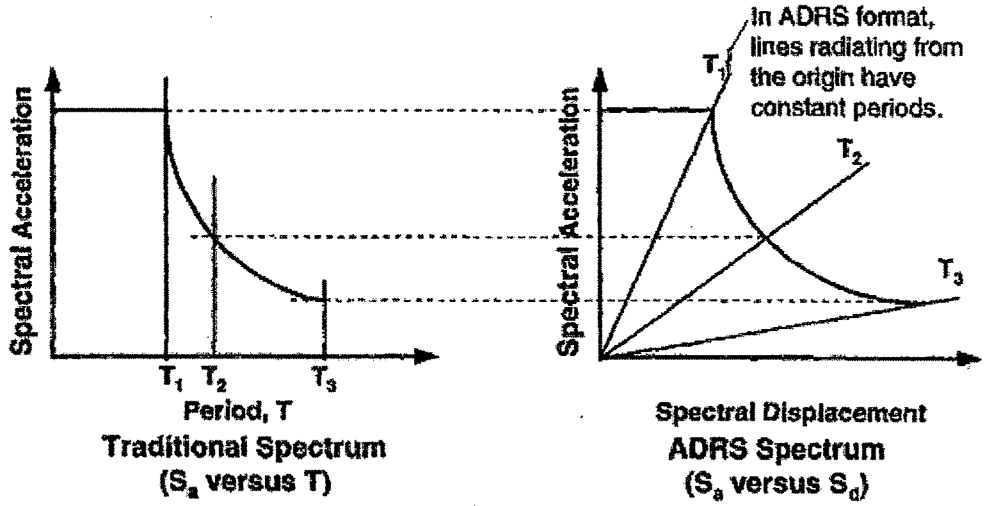


Fig. 4.14 Converting Demand Spectra to ADRS Format

4.9.2 Converting Capacity Spectrum to ADRS Format

In order to develop the capacity spectrum from the capacity (or Pushover) curve, it is necessary to do a point by point conversion to first mode spectral coordinates. Any point (V_i, Δ_{roof}) on the capacity curve is converted to the corresponding point (S_{ai}, S_{di}) on the capacity spectrum using the following two equations:

$$S_{ai} = V/(W*\alpha 1) \quad .....(4.12)$$

$$S_{di} = \Delta_{roof} / PF_1 \, \Phi_{1,roof} \quad .....(4.13)$$

where, V = Base Shear, W = Building dead load plus applicable live load, $\alpha 1$ = modal mass coefficient for the first natural mode, Δ_{roof} = roof displacement obtained from pushover curve, PF_1 = modal participation factor for the first natural mode and $\Phi_{1,roof}$ = roof level amplitude of the first mode. This conversion of capacity curve to capacity spectrum in ADRS format is represented in **Fig. 4.15**.

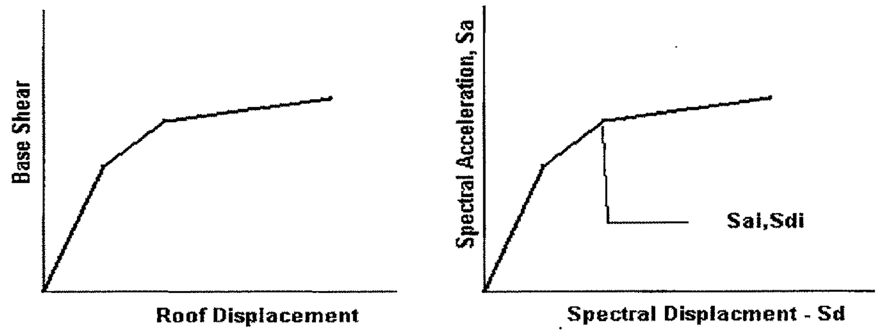


Fig. 4.15 Converting Capacity Spectrum to ADRS Format

4.9.3 Obtaining the Performance point

Performance point can be obtained by superimposing capacity spectrum and demand spectrum in the ADRS format and the intersection point of these two curves is the performance point. Simply stated, it is the point where capacity meets demand. **Figure 4.16** shows superimposing of demand spectrum and capacity spectrum.

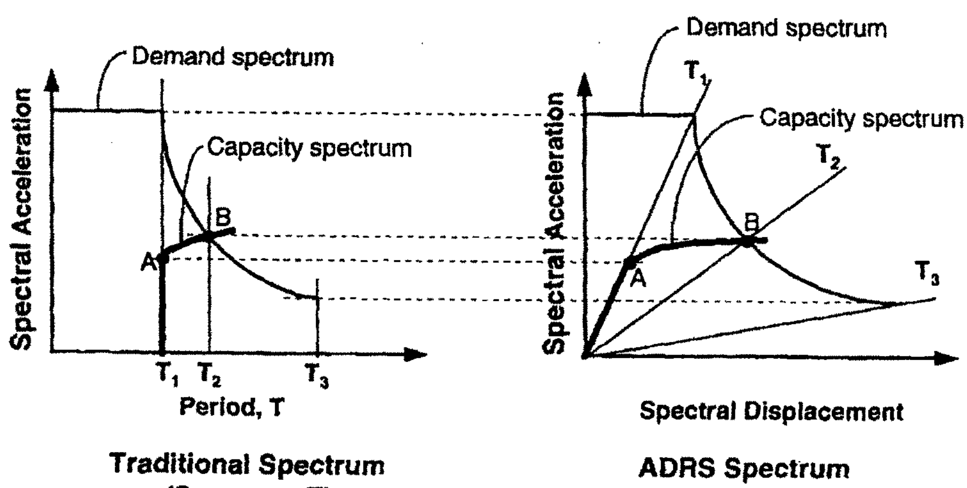


Fig. 4.16 Capacity Spectrum Superimposed over Demand Spectrum

Finally, check the performance level of the structure and plastic hinge formations at performance point. A performance check verifies that structural and non-structural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacements implied by the displacement demand.