Chapter 15

Performance Based Seismic Engineering

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Abstract: Performance based seismic engineering is the modern approach to earthquake resistant design. Rather than being based on prescriptive mostly empirical code formulations, performance based design is an attempt to predict buildings with predictable seismic performance. Therefore, performance objectives such as life-safety, collapse prevention, or immediate occupancy are used to define the state of the building following a design earthquake. In one sense, performance based seismic design is limit-states design extended to cover the complex range of issues faced by earthquake engineers. This chapter provides a basic understanding of the promises and limitations of performance based seismic engineering. The state-of-the-art methodologies and techniques embodied in the two leading guidelines on this subject (ATC-40 and FEMA 273/274) are introduced and discussed. Numerical examples are provided to illustrate the practical applications of the methods discussed.
15.1 INTRODUCTION

The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance. To turn this promise into a reality, a comprehensive and well-coordinated effort by professionals from several disciplines is required.

Performance based engineering is not new. Automobiles, airplanes, and turbines have been designed and manufactured using this approach for many decades. Generally in such applications one or more full-scale prototypes of the structure are built and subjected to extensive testing. The design and manufacturing process is then revised to incorporate the lessons learned from the experimental evaluations. Once the cycle of design, prototype manufacturing, testing and redesign is successfully completed, the product is manufactured in a massive scale. In the automotive industry, for example, millions of automobiles which are virtually identical in their mechanical characteristics are produced following each performance-based design exercise.

What makes PBSE different and more complicated is that in general this massive payoff of performance-based design is not available. That is, except for large-scale developments of identical buildings, each building designed by this process is virtually unique and the experience obtained is not directly transferable to buildings of other types, sizes, and performance objectives. Therefore, up to now PBSE has not been an economically feasible alternative to conventional prescriptive code design practices. Due to the recent advances in seismic hazard assessment, PBSE methodologies, experimental facilities, and computer applications, PBSE has become increasing more attractive to developers and building officials as well as engineers and earth-scientists. These are very positive developments which are bound to improve the quality of earthquake resistant construction.

In order to utilize PBSE effectively and intelligently, one need to be aware of the uncertainties involved in both structural performance and seismic hazard estimations. We discuss these issues first before exploring the philosophies and detailed requirements of the two most prominent PBSE guidelines available today. These guidelines are generally referred to by their short names: ATC-40\(^{15-1}\) and FEMA-273/274\(^{15-2,15-3}\).

15.2 UNCERTAINTIES IN SEISMIC DESIGN AND PERFORMANCE

Every structural system is designed to have a seismic capacity that exceeds the anticipated seismic demand. Capacity is a complex function of strength, stiffness and deformability conjectured by the system configuration and material properties of the structure.

A key requirement of any meaningful PBSE exercise is the ability to assess seismic demands and capacities with a reasonable degree of certainty. The recent popularity of PBSE has brought many state-of-the-art analysis and design techniques into the mainstream of earthquake engineering practice. Furthermore, it has opened the door for a multi-disciplinary approach to seismic design which involves developers and building officials as well as engineers and earth-scientists. These are very positive developments which are bound to improve the quality of earthquake resistant construction.

The mere desire to produce structures with predictable seismic performance does not by itself, however, turn PBSE into a reality. Many uncertainties and gaps of knowledge have to be dealt with before PBSE turns from a promise into a reality. Structural engineering practice has been able to produce structures which with a few notable exceptions (i.e., welded steel moment frame structures during the 1994 Northridge earthquake) generally exceed performance expectations postulated by routine design analysis. Our capability to estimate the ultimate seismic capacities and failure loads associated with a structure, however, at least
outside the academic research settings is fairly limited and not up to the standards needed for a reliable prediction of seismic performance.

For example, following the Northridge earthquake, the Applied Technology Council conducted a survey of 530 buildings which were located within 300 meters of strong-motion recording sites\(^{15-4}\). From the total of 530 buildings which were located in the areas of strong shaking (San Fernando Valley, Santa Monica, and West Los Angeles) with peak ground acceleration in their vicinity ranging from 0.15\(g\) to 1.78\(g\), only 10 (less than two-percent) showed heavy damage, a total of 78 buildings (about 15-percent) showed moderate damage and 340 (64-percent) were marked by insignificant damage (Figure 15-1). If response of these buildings were predicted by standard design analysis techniques, a far worse picture would have been predicted.

Crandell\(^{15-5}\) performed a similar statistically-based study of the seismic performance of residential buildings located within a 10-mile radius of the Northridge earthquake epicenter (Figure 15-2). Three hundred forty one of the 375 randomly selected homes were surveyed and although more than 90 percent of the homes in the sample were old and built prior to the 1971 San Fernando Valley earthquake the cases of moderate to high damage were infrequent (less than 2-percent). Most occurrences of serious damage were located in foundation systems and were associated with localized site conditions such as liquefaction, fissuring, and hillside slope failures. Here again, design analysis would have predicted much larger damage percentage than the 2-percent number reported by Crandell.

Large uncertainties also exist in our estimates of design ground motion. For example, median estimates of spectral accelerations for a magnitude 7.0 event at rupture distance of 10 km obtained from various attenuation relations can vary by as much as 50 percent\(^{15-6}\). If the uncertainties associated with other source and regional variables are also considered, the variance could be significantly larger. Most attenuation relations are updated every few years (Figure 15-3), indicating that there are still many things to be learned about the generation and propagation of earthquake ground motion.
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Figure 15-2. Description of Damage During the 1994 Northridge Earthquake to Single Family Dwellings Within a 10 Miles Radius of the Epicenter (data from Crandell, 1997)

Figure 15-3. Evolution of a Typical Attenuation Relation (Spectral velocity estimates are shown for a magnitude 7.0 event at 5.0 km for a strike-slip fault)
Another source of uncertainty is critical shortage of recorded earthquake ground motion where they are needed most. Despite the tremendous growth in the number of earthquake records during the past decade, the number of recordings from large earthquakes close by. Figure 15-3 shows a bivariate histogram of horizontal components recorded in north and central America categorized by magnitude and epicentral distance, indicating practically no record of M >7.5 at distances less than 20 km. All of the data for M>8 records come from a single event (Mexico, 1985). Clearly, this is one of the areas where more information is needed for performance based design.

Since PBSE is inherently multi-disciplinary in nature, further educational efforts are also of vital importance in bringing PBSE to fruition by developing a common understanding of issues and a common PBSE language and vocabulary. Only a broad multi-disciplinary approach can succeed in reduction of uncertainties, knowledge gaps, and common misunderstandings.

Figure 15-4. Distribution of Magnitude and Distance among Available Earthquake Records for North and Central America, 1933-1994 (M>5.5; PGA>0.05g)
15.3 ATC-40

15.3.1 Introduction

Seismic Evaluation and Retrofit of Concrete Buildings\(^{15-1}\) commonly referred to as ATC-40 was developed by the Applied Technology Council (ATC) with funding from the California Safety Commission. Although the procedures recommended in this document are for concrete buildings, they are applicable to most building types. This document provides a practical guide to the entire evaluation and retrofit process using performance-based objectives. Although it is not intended for the design of new buildings, the analytical procedures described in this document are certainly applicable.

ATC-40 recommends the following steps for the entire process of evaluation and retrofit:

1. Initiation of a Project: Determine the primary goal and potential scope of the project.
2. Selection of Qualified Professionals: Select engineering professionals with a demonstrated experience in the analysis, design and retrofit of buildings in seismically hazardous regions. Experience with PBSE and non-linear procedures is also needed.
3. Performance Objective: Choose a performance objective from the options provided for a specific level of seismic hazard.
5. Alternatives for Mitigation: Check to see if the non-linear procedure is appropriate or relevant for the building under consideration.
6. Peer Review and Approval Process: Check with building officials and consider other quality control measures appropriate to seismic evaluation and retrofit.
7. Detailed Investigations: Perform a non-linear static analysis if appropriate.
8. Seismic Capacity: Determine the inelastic capacity curve also known to pushover curve. Convert to capacity spectrum.
9. Seismic Hazard: Obtain a site specific response spectrum for the chosen hazard level and convert to spectral ordinates (ADRS\(^{15-8,15-9,15-10}\), see Section 15.3.6) format.
10. Verify Performance: Obtain performance point as the intersection of the capacity spectrum and the reduced seismic demand in spectral ordinates (ADRS) format. Check all primary and secondary elements against acceptability limits based on the global performance goal.
11. Prepare Construction Documents: Detail retrofit to conform to code requirements and get analysis and design peer-reviewed and submit for plan check.


15.3.2 Performance Objectives

A performance objective has two essential parts – a damage state and a 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 objective.

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.
Structural Performance Levels are defined as:

- **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.

- **Damage Control (SP-2):** A placeholder for a state of damage somewhere between Immediate Occupancy and Life Safety.

- **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.

- **Limited Safety (SP-4):** A placeholder for a state of damage somewhere between Life Safety and Structural Stability.

- **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.

- **Not Considered (SP-6):** Placeholder for situations where only non-structural seismic evaluation or retrofit is performed.

Non-structural Performance Levels are defined as:

- **Operational (NP-A):** Non-structural elements are generally in place and functional. Back-up systems for failure of external utilities, communications and transportation have been provided.

- **Immediate Occupancy (NP-B):** Non-structural elements are generally in place but may not be functional. No back-up systems for failure of external utilities are provided.

- **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.

- **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.

- **Not Considered (NP-E):** Non-structural elements, other than those that have an effect on structural response, are not evaluated.

Combinations of Structural and Non-structural Performance Levels to obtain a Building Performance Level are shown in Table 15-1.

### 15.3.3 Nonlinear Static Procedures

In Nonlinear Static Procedure, the basic demand and capacity parameter for the analysis is the lateral displacement of the building. The generation of a capacity curve (base shear vs roof displacement Figure 15-5) defines the capacity of the building uniquely for an assumed force distribution and displacement pattern. It is independent of any specific seismic shaking demand and replaces the base shear capacity of conventional design procedures. If the building displaces laterally, its response must lie on this capacity curve. A point on the curve defines a specific damage state for the structure, since the deformation for all components can be related to the global displacement of the structure. By correlating this capacity curve to the seismic demand generated by a specific earthquake or ground shaking intensity, a point can be found on the capacity curve that estimates the maximum displacement of the building the earthquake will cause. This defines the performance point or target displacement. The location of this performance point relative to the performance levels defined by the capacity curve indicates whether or not the performance objective is met.
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Thus, for the Nonlinear Static Procedure, a static pushover analysis is performed using a nonlinear analysis program for an increasing monotonic lateral load pattern. An alternative is to perform a step by step analysis using a linear program. The base shear at each step is plotted against roof displacement. The performance point is found using the Capacity Spectrum Procedure\(^{[15-8,15-9,15-10]}\) described in subsequent sections. The individual structural components are checked against acceptability limits that depend on the global performance goals. The nature of the acceptability limits depends on specific components. Inelastic rotation is typically one of acceptability parameters for beam and column hinges. The limits on inelastic rotation are based on observation from tests and the collective judgement of the development team.

### 15.3.4 Inelastic Component Behavior

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, do not need to be included in the analysis.

![Building Capacity Curve](Image)

**Figure 15-5. Building Capacity Curve**

In concrete buildings, the effects of earthquake shaking are resisted by vertical frame elements or wall elements that are...
connected to horizontal elements (diaphragms) at the roof and floor levels. The structural elements may themselves comprise of an assembly of elements such as columns, beam, wall piers, wall spandrels etc. It is important to identify the failure mechanism for these primary structural elements and define their non-linear properties accordingly. The properties of interest of such elements are relationships between the forces (axial, bending and shear) and the corresponding inelastic displacements (displacements, rotations, drifts). Earthquakes usually load these elements in a cyclic manner as shown in Figure 15-6a. For modeling and analysis purposes, these relationship can be idealized as shown in Figure 15-6b using a combination of empirical data, theoretical strength and strain compatibility.

Using the component load-deformation data and the geometric relationships among components and elements, a global model of the structure relates the total seismic forces on a building to it overall lateral displacement to generate the capacity curve. 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 document provides acceptability limits for component deformations depending on the specified performance level.

15.3.5 Geotechnical effects

The deformation and movement of the foundations of a building can significantly affect the seismic response and performance of structures. As the structural components are represented by non-linear load-displacement relationships, analogous techniques compatible and consistent with the general methodology should be used for the effects of the foundations.

The response parameters of foundation elements are dependent on structural as well as geotechnical components. Spread footings elements, for example, might consist of a rigid structural plate component model of the concrete footing bearing on soil represented by geotechnical components with appropriate force-displacement properties. Some generic models for typical foundation elements and acceptance criterion for structural components of the foundations are provided in ATC-40.

There is a large degree of uncertainty associated with both strength and stiffness of the geotechnical components. Thus, ATC-40 recommends enveloping analysis to determine the sensitivity of seismic performance to foundation behavior (See Figure 15-8). Guidance in provided for representative properties of normally encountered soil materials that are based on limited initial investigations in ATC-40. If the analysis shows sensitivity to foundation behavior than more detailed investigations and tests of geotechnical properties may be warranted.

Geotechnical properties are very ductile and failure is rarely encountered. Thus, deformation limits of geotechnical components are not explicitly defined. However, deformation of geotechnical components may affect the deformation and acceptability of components in the superstructure. It should also be noted that geotechnical components tend to accumulate residual displacements. This tendency may affect the acceptability of a structure for higher performance objectives such as Immediate Occupancy. Soil structure interaction also has beneficial affects such as lower demands on structural members due to base rotation, lower forces due to uplift and damping effects that reduce demand on the superstructure.
(a) Backbone curve from actual hysteretic behavior

(b) Idealized component behavior from backbone curves

Figure 15-6. Idealized Component Force-Deformation Relationships
Distributed Vertical Geotechnical Properties
- Vertical bearing properties of soil
- Component spacing along footing length

Horizontal Geotechnical Component
- Passive properties against side of footing
- Friction properties at bottom of footing

b. Element Model for Analysis

*Figure 15-7. Shallow Foundation Model*\(^{(15-1)}\)

*Figure 15-8. Basic Force-Displacement Envelope for Geotechnical Components*\(^{(15-1)}\)
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15.3.6 Capacity Spectrum Method

One of the methods used to determine the performance point is the Capacity Spectrum Method\textsuperscript{[15-8,15-9,15-10]}, also known as the Acceleration-Displacement Response Spectra method (ADRS). The Capacity Spectrum Method requires that both the capacity curve and the demand curve be represented in response spectral ordinates. It characterizes the seismic demand initially using a 5% damped linear-elastic response spectrum and reduces the spectrum to reflect the effects of energy dissipation to estimate the inelastic displacement demand. The point at which the capacity curve intersects the reduced demand curve represents the performance point at which capacity and demand are equal.

To convert a spectrum from the standard $S_a$ (Spectra Acceleration) vs $T$ (Period) format found in the building codes\textsuperscript{[15-13]} to ADRS format, it is necessary to determine the value of $S_d$ (Spectral Displacement) for each point on the curve, $S_{ai}, T_i$. This can be done with the equation:

$$ S_{di} = \frac{T_i^2}{4\pi^2} S_{ai} g $$

(15-1)

Standard demand response spectra contain a range of constant spectral acceleration and a second range of constant spectral velocity, $S_v$. Spectral acceleration and displacement at period $T_i$ are given by:

$$ S_{ai} g = \frac{2\pi}{T_i} S_v, \quad S_{di} = \frac{T_i}{2\pi} S_v $$

(15-2)

The capacity spectrum can be developed from the pushover curve by a point by point conversion to the first mode spectral coordinates. Any point $V_i$ (Base Shear), $\delta$ (Roof Displacement) on the capacity (pushover) curve is converted to the corresponding point $S_{ai}, S_{di}$ on the capacity spectrum using the equations:

$$ S_{ai} = \frac{V_i}{\alpha_i} $$

(15-3)

$$ S_{di} = \frac{\delta_i}{(PF_1 \times \phi_{1\text{roof}})} $$

(15-4)

Where $\alpha_i$ and $PF_1$ are the modal mass coefficient and participation factors for the first natural mode of the structure respectively. $\phi_{1\text{roof}}$ is the roof level amplitude of the first mode. The modal participation factors and modal coefficient are calculated as:

$$ PF_1 = \left[ \frac{\sum_{i=1}^{n}(w_i\phi_i)}{\sum_{i=1}^{n}(w_i\phi_i^2)} \right] $$

(15-5)
\[ \alpha_i = \left( \frac{\sum_{i=1}^{n} (w_i \phi_i) / g}{\sum w_i / g} \right) \left( \frac{\sum (w_i \phi_i^2)}{\sum w_i / g} \right)^2 \] (15-6)

Where \( w_i \) is the weight at any level \( i \).

As displacement increase, the period of the structure lengthens. This is reflected directly in the capacity spectrum. Inelastic displacements increase damping and reduce demand. The Capacity Spectrum Method reduces the demand to find an intersection with the capacity spectrum, where the displacement is consistent with the implied damping.

\[ \beta_o = \frac{1}{4\pi} \frac{E_D}{E_{So}} \] (15-8)

Where \( E_D \) is the energy dissipated by damping and \( E_{So} \) is the maximum strain energy. The physical significance is explained in Figure 15-11.

The damping that occurs when the structure is pushed into the inelastic range can be viewed as a combination of viscous and hysteretic damping. Hysteretic damping can be represented as equivalent viscous damping. Thus, the total effective damping can be estimated as:

\[ \beta_{eff} = \lambda \beta_o + 0.05 \] (15-7)

Where \( \beta_o \) is the hysteretic damping and 0.05 is the assumed 5% viscous damping inherent in the structure. The \( \lambda \)-factor (called \( \kappa \)-factor in ATC-40) is a modification factor to account for the extent to which the actual building hysteresis is well represented by the bilinear representation of the capacity spectrum (See Table 15-3 and Figure 15-11).

The term \( \beta_o \) can be calculated using:

\[ \beta_o = \frac{1}{4\pi} \frac{E_D}{E_{So}} \] (15-8)

Where \( E_D \) is the energy dissipated by damping and \( E_{So} \) is the maximum strain energy. The physical significance is explained in Figure 15-11.

\[ E_D = \text{Energy dissipated by damping} \]
\[ = \text{Area of hatched triangle} \]
\[ = \text{Area enclosed by hysteresis loop} \]
\[ = \text{Area of shaded parallelogram} \]

Figure 15-11. Derivation of Energy dissipated by Damping (15-1)

To account for the damping, the response spectrum is reduced by reduction factors \( SR_A \) and \( SR_V \) which are given by

\[ SR_A = \frac{1}{B_y} = \frac{3.21 - 0.68 \ln(\beta_{eff})}{2.12} \] (15-9)
Both $SR_A$ and $SR_V$ must be greater than or equal to allowable values in Table 15-4.

The elastic response spectrum (5% damped) is thus reduced to a response spectrum with damping values greater than 5% critically damped (See Figure 15-12). Note, the limits of the spectral reduction factors are arbitrary and need farther study.

![Reduced Response Spectrum](Figure 15-12)

There are three procedures described in ATC-40 to find the performance point. The most transparent and most convenient for programming is Procedure A. To find the performance point using Procedure A the following steps are used:

1. A 5% damped response spectrum appropriate for the site for the hazard level required for the performance objective is developed and converted to ADRS format.
2. The capacity curve obtained from the non-linear analysis is converted to a capacity spectrum using Equations 15-3 and 15-4.
3. A trial performance point $S_{api}, S_{dpi}$ is selected. This may be done using the equal displacement approximation (See Figure 15-13) or on the basis of engineering judgement.
4. A bilinear representation of the capacity spectrum is developed such that the area under the capacity spectrum and the bilinear representation is the same. In the case of a saw-tooth capacity spectrum, the bilinear representation must be based on the capacity spectrum that makes up the portion of the composite capacity spectrum where the performance point $S_{api}, S_{dpi}$ occurs.
5. The spectral reduction factors $SR_A$ and $SR_V$ are computed using Equations 15-9 and 15-10 and the demand spectrum is reduced as shown in Figure 15-12. The reduced demand spectrum is plotted together with the capacity spectrum.
6. If the reduced demand spectrum intersects the capacity spectrum at $S_{d_{ii}}$, $S_{d_{ii}}$ or if the intersection point $S_{d_{ii}}$ is within 5% of $S_{d_{ii}}$, then this point represents the performance point.

7. If the intersection point does not lie within acceptable tolerance (5% of $S_{d_{ii}}$ or other) then select another point and repeat Steps 4 to 7. The intersection point obtained in Step 6 can be used as the starting point for the next iteration.

Procedure B is also an iterative method to find the performance point, which uses the assumption that the yield point and the post yield slope of the bilinear representation, remains constant. This is adequate for most cases, however, in some cases this assumption may not be valid. Procedure C is graphical method that is convenient for hand analysis.

### 15.3.7 Checking Performance at Expected Maximum Displacement

Once the performance point $S_{d_{ii}}$, $S_{d_{ii}}$ (which are in spectral ordinates) is found, the base shear ($V_p$) and roof displacement ($\delta_p$) at the performance point are found using Equation 15-3 and 15-4. The following steps should be used in the performance check:

1. For the global building response, verify
   a. The lateral force resistance has not degraded by more than 20% of the peak resistance.
   b. The lateral drift limits satisfy the limits given in the Table 15-5.

2. Identify and classify the different elements in the building in the following types: beam-column frames, slab-column frames, solid walls, coupled walls, perforated walls, punched walls, floor diaphragms 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.

5. The strength and deformation demands at the performance point should be equal to or less than the capacities detailed in Chapter 11 of ATC-40.

6. The performance of secondary elements (such as gravity load carrying members not part of the lateral load resisting system) are reviewed for acceptability for the specified performance level.

7. Non-structural elements are checked for the specified performance level.
### 15.3.8 Other Considerations

Other considerations that should be noted are:

1. **Torsion**: For 3D models, the lateral load should be applied at the center of mass of each floor and the displacement plotted on the capacity curve should be for the center of mass for the roof. Use of 2D models should be limited to building where the torsional effects are sufficiently small such that the maximum displacement at any point is not more than 120% of the displacement at the center of mass.

2. **For structures with long fundamental modes**, higher mode effects may be more critical. Pushover analysis should be performed for additional mode shapes using corresponding force distributions.

### Table 15-5. Deformation Limits

<table>
<thead>
<tr>
<th>Interstory Drift Limit</th>
<th>Performance Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Immediate Occupancy</td>
</tr>
<tr>
<td></td>
<td>Damage Control</td>
</tr>
<tr>
<td>Maximum Total Drift</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>0.01 – 0.002</td>
</tr>
<tr>
<td>Maximum inelastic</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>0.005 – 0.015</td>
</tr>
</tbody>
</table>

### 15.3.9 Example

An example is provided of the procedure to determine the performance point using the Capacity Spectrum Method. This example reworked from numbers provided in the ATC-40 document.

#### 15.3.9.1 Building Description

The example building is a seven-story reinforced concrete building. The total weight of the building is 10,540 kips. The pushover curve determined for the building is given in Table 15-6. The pushover (capacity) curve is converted into a capacity spectrum using Equation 15-3 and 15-4. The demand for the building for the performance level desired is determined to be Soil Type D with \( C_s = 0.44 \) and \( C_v = 0.64 \) respectively. The demand spectrum is converted to ADRS format using Equation 15-1.

The demand and capacity spectrum are plotted together as shown in Figure 15-14. Using an equal displacement approximation, the first trial performance point \( S_{ap1} \), \( S_{dp1} \) is selected. A bilinear representation is developed such that the area under the capacity spectrum is the same as the area under the bilinear curve. Thus:

\[
S_{ap1} = 0.36g \quad S_{dp1} = 5.5 \text{ in}
\]

\[
S_a = 0.31g \quad S_d = 2.35 \text{ in}
\]

\[
\beta_{eff} = \frac{63.7 \lambda (S_a \cdot S_{dp1} - S_d \cdot S_{ap1})}{S_{ap1} \cdot S_{dp1}} + 5
\]

\[
= 14.11\%
\]

A \( \lambda \) of 0.33 is used for structural behavior type C from Table 15-3. Thus, the spectral reduction factors are calculated from Equations 15-9 and 15-10 as:

\[
SR_a = \frac{3.21 - 0.68 \ln(14.11)}{2.12} = 0.665
\]

\[
SR_d = \frac{2.31 - 0.41 \ln(14.11)}{1.65} = 0.742
\]

### Table 15-6. Conversion of Pushover Curve to Capacity Spectrum

<table>
<thead>
<tr>
<th>Point</th>
<th>( V ) (kips)</th>
<th>( \delta ) (in)</th>
<th>( V/W )</th>
<th>( PF_i \cdot \phi_{roof} )</th>
<th>( \alpha_i )</th>
<th>( Sa ) (g)</th>
<th>( Sd ) (g)</th>
<th>( T ) (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2200</td>
<td>2.51</td>
<td>0.209</td>
<td>1.31</td>
<td>0.828</td>
<td>0.254</td>
<td>1.92</td>
<td>0.88</td>
</tr>
<tr>
<td>B</td>
<td>2600</td>
<td>3.60</td>
<td>0.247</td>
<td>1.28</td>
<td>0.800</td>
<td>0.309</td>
<td>2.81</td>
<td>0.96</td>
</tr>
<tr>
<td>C</td>
<td>2800</td>
<td>5.10</td>
<td>0.266</td>
<td>1.35</td>
<td>0.770</td>
<td>0.346</td>
<td>3.78</td>
<td>1.06</td>
</tr>
<tr>
<td>D</td>
<td>3000</td>
<td>10.90</td>
<td>0.285</td>
<td>1.39</td>
<td>0.750</td>
<td>0.380</td>
<td>7.84</td>
<td>1.45</td>
</tr>
</tbody>
</table>

\( PF_i \) and \( \alpha_i \) change because the mode shape is changing as yielding occurs.
Using the spectral reduction factors, the demand spectrum is reduced as per Figure 15-12. The reduced spectrum is plotted together with the capacity spectrum and the intersection point is found (See Figure 15-15). The demand spectrum intersects the capacity spectrum at a spectral displacement of 6.1 inches. As this displacement is not within 5% of the first trial displacement of 5.5 inches.

A new trial performance point must be chosen and the process repeated. The second trial point may be chosen as the intersection from the previous iteration. However, in this example, the second trial performance point is chosen by engineering judgement at a spectral displacement of 5.9 inches. A new bilinear representation is constructed and the process repeated:

\[
\begin{align*}
S_{d_p2} &= 0.365g \\
S_{d_y} &= 0.305g \\
S_{d_p2} &= 5.9 \text{ in} \\
S_{d_y} &= 2.3 \text{ in}
\end{align*}
\]

\[
\beta_{eff} = \frac{63.7\lambda(S_{a}, S_{d_p2} - S_{d_y}, S_{a})}{S_{a}S_{d_p2}} + 5 = 14.37\% 
\]

The new spectral reduction factors are calculated from Equations 15-9 and 15-10 as:

\[
\begin{align*}
SR_A &= \frac{3.21 - 0.68\ln(14.37)}{2.12} = 0.659 \\
SR_c &= \frac{2.31 - 0.41\ln(14.37)}{1.65} = 0.738
\end{align*}
\]

A new reduced demand spectrum is plotted and a new intersection point is obtained. As seen in Figure 15-17, the intersection point is at a spectral displacement of 6.0 inches. As this intersection is within 5% of the second trial point, the demand spectral displacement is 6.0 inches.
Figure 15-15. Determination of Intersection Point and Comparison with the First Trial Performance Point

Figure 15-16. Determination of Second Performance Point
The actual roof displacement at the performance point is calculated from Equation 15-4. The modal participation factor is used by linear interpolation from Table 15-6.

\[ \delta_i = PF_i \cdot \phi_{\text{ref}i} \times Sd_p \]
\[ = 1.35 \times 6.0 = 8.1 \text{ inches} \]

Similarly, the base shear can be found from the spectral acceleration at the performance point by using Equation 15-3. The modal mass coefficient can be found by linear interpolation from Table 15-6.

\[ V_p / W = \alpha_i \times Sa_p \]
\[ = 0.76 \times 0.365 = 0.277 \]

The element capacities are checked for the building at this performance point as detailed in Section 15.3.7.

15.3.10 Recent Advances in the Capacity Spectrum Method

In recent publications it has been reported by Chopra and Goel\(^{(15-14,15-15)}\) that the Capacity Spectrum Method as described in ATC-40 does not produce conservative estimates of inelastic peak displacements when compared to inelastic response spectrum analysis. It has also been reported that the ATC-40 procedures are deficient relative to even the elastic design spectrum in the velocity and displacement sensitive regions of the spectrum. An improved method has been suggest by Chopra and Goel\(^{(15-15)}\) which makes use of inelastic spectra using any of three \(R_{\mu} \cdot \mu \cdot T\) equations (Newmark and Hall\(^{(15-16)}\), Krawinkler and Nassar\(^{(15-17)}\) and Vidić, Fajfar and Fischinger\(^{(15-18)}\)). In this improved Capacity Spectrum Method, the capacity and the constant ductility design spectra are plotted in ADRS format. The capacity spectrum intersects the demand spectrum for several values of ductility \(\mu\). The
deformation at the performance point is given by the one intersection point where the ductility factor calculated from the capacity spectrum matches the value associated with the intersected demand spectrum.

Another method for determining the performance point is suggested by Fajfar \(^{(15-19)}\). Here the ductility demand is determined using the equal displacement rule and the inelastic design spectra. Another variant of the Capacity Spectrum method called the Yield Point Spectra \(^{(15,20)}\) has recently been suggested. Here the yield displacement is plotted on the abscissa instead of the spectral displacement and \(R_\mu-T\) relations or exact computations are used instead of equivalent viscous damping.

### 15.4 FEMA 273 and 274

#### 15.4.1 Introduction

**NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA-273)** \(^{(15,2)}\) and the associated commentary (FEMA-274) \(^{(15,3)}\) was developed by the Building Seismic Safety Council (BSSC) with subcontractors American Society of Civil Engineering (ASCE) and the Applied Technology Council (ATC) with the funding provided by the Federal Emergency Management Agency (FEMA). The primary purpose of FEMA-273 was to provide technically sound and nationally acceptable guidelines for the seismic rehabilitation of buildings. Although the document was written with the objective of performance based retrofit of existing structures, the procedures described therein are equally applicable for new design. Unlike the ATC-40 document, these recommendations are applicable to all building materials and define acceptability limits for linear as well as non-linear analysis.

The basic procedure is similar to that recommended in ATC-40. The owner decides the performance object that needs to be achieved. The engineer then designs the retrofit or new structure to achieve the performance objective. The definitions of the basic performance levels are similar to those defined in ATC 40 (See Section 15.3.2).

FEMA-273 defines ground motion hazard levels in a probabilistic basis. Four ground motion hazard levels are defined:

<table>
<thead>
<tr>
<th>Earthquake Probability</th>
<th>Mean Return Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50% in 50 years</td>
<td>72</td>
</tr>
<tr>
<td>20% in 50 years</td>
<td>225</td>
</tr>
<tr>
<td>BSE-1 10% in 50 years</td>
<td>474</td>
</tr>
<tr>
<td>BSE-2 2% in 50 years</td>
<td>2,475</td>
</tr>
</tbody>
</table>

Where BSE is the Basic Safety Earthquake. The broad range of performance objectives recommended for a given earthquake hazard levels are shown in Table 15-7

**Table 15-7. Rehabilitation Objectives** \(^{(15-2)}\)

<table>
<thead>
<tr>
<th>Earthquake Hazard Level</th>
<th>Operational Level (1-A)</th>
<th>Immediate Occupancy Level (1-B)</th>
<th>Life Safety Performance Level (3-C)</th>
<th>Collapse Prevention Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>50%/50 yrs</td>
<td>a</td>
<td>b</td>
<td>c</td>
<td>d</td>
</tr>
<tr>
<td>20%/50 yrs</td>
<td>e</td>
<td>f</td>
<td>g</td>
<td>h</td>
</tr>
<tr>
<td>BSE-1 10%/50 yrs</td>
<td>i</td>
<td>j</td>
<td>k</td>
<td>l</td>
</tr>
<tr>
<td>BSE-2 2%/50 yrs</td>
<td>m</td>
<td>n</td>
<td>o</td>
<td>p</td>
</tr>
</tbody>
</table>

\(k+p = \) Basic Safety Objective
\(k+p+\)any of a, c, i or m; or b, f, j, or n = Enhanced Objectives
\(o = \) Enhanced Objectives
\(k\) alone or p alone = Limited Objective
\(c, g, d, h = \) Limited Objectives

From Table 15-7, it is clear that FEMA-273 specifies a two-level design to achieve the Basic Safety Objective (BSO), Life Safety Performance Level for BSE-1 demands and Collapse Prevention Level for BSE-2 demands. However, for new structures it is possible to control ductility and configuration of the design.
to an extent that will permit those structures
designed to achieve Life Safety Performance
Level for a BSE-1 level earthquake to also
avoid collapse for much larger events.

Two sets of earthquake hazard maps are
distributed with FEMA-273 and 274. One set
provide key response acceleration for the
Maximum Considered Earthquake (MCE)
which in most areas represents a 2%/50 years
exceedence level. The other uses 10%/50 years
exceedence probability. Thus, it is possible to
obtain a BSE-1 and BSE-2 level spectra from
these maps.

15.4.2 Mathematical Modeling

FEMA-273 provides four analysis
procedures for systematic design and
rehabilitation of buildings. The Linear Static
(LSP) and Linear Dynamic Procedures (LDP)
are linearly elastic analysis, which may include
geometric non-linearity. Also some material
non-linearity is also introduced by use of
cracked properties for concrete and masonry
components even though the analysis is linear.
In the Nonlinear Static (NSP) and Nonlinear
Dynamic Procedures (NDP) material non-
linearity is included in the analysis.

15.4.2.1 Basic Assumptions

In general, a three dimensional analysis
consisting of an assembly of elements and
components is recommended. Three-
dimensional analysis is required when the
building has plan irregularities and when
torsional effects cannot be ignored or indirectly
captured.

For buildings with flexible diaphragms, the
diaphragms may be individually modeled and
analyzed as two-dimensional assemblies of
components and elements or three-dimensional
models with flexible elements.

Explicit modeling of connections is not
required if the connection is stronger than the
connected components or when the deflection
of the connection does not cause a significant
increase in the relative deformation between the
connected components.

15.4.2.2 Horizontal Torsion

In addition to the actual eccentricities
between the centers of mass and centers of
rigidity, a additional accidental torsional
moment should be included which may be
produced by including a horizontal offset in the
centers of mass equal to a minimum of 5% of
the horizontal dimension at a given floor level.

For buildings with rigid diaphragms, the
effects of torsion must be included when the
maximum displacement at any point in a
diaphragm exceeds the average displacement in
that diaphragm by more than 10%. For linear
analysis, the effect of accidental torsion is
amplified by a factor $A_x$:

$$A_x = \left( \frac{\delta_{\text{max}}}{1.2\delta_{\text{avg}}} \right)^2$$  (15-11)

Where $\delta_{\text{max}}$ and $\delta_{\text{avg}}$ are the maximum and
average displacements in a diaphragm. $A_x$ is
greater than 1 and not greater than 3.

If $\eta = \delta_{\text{max}}/\delta_{\text{avg}}$ is greater than 1.5, then a
three-dimensional analysis is required. For two-
dimensional analysis subject to this limitation,
the effect of torsion can included for LSP and
LDP by increasing the design forces and
displacement by $\eta$. For NSP, the target
displacement is increased by $\eta$ and for NDP the
amplitude of the ground acceleration record is
increased by $\eta$.

15.4.2.3 Primary and Secondary Elements

Primary elements are key parts of the
seismic framing system required in the design
to resist earthquake effects. These must be
evaluated to resist earthquake forces as well as
gravity loads if required. Secondary elements
are not designed to be part of the lateral force
resisting system but must be evaluated to ensure
they can simultaneously sustain earthquake
induced deformation and gravity loads.

For linear analysis procedures, the
secondary elements must not constitute more
than 25% of the total stiffness of the primary elements at any level and may not be included in the analysis. For nonlinear procedures, the stiffness of the primary as well as the secondary elements must be included in the model. Additionally, the stiffness of non-structural elements must not exceed 10% of the total lateral stiffness of any story. If this is exceeded, then the non-structural elements must be included in the model.

15.4.2.4 Deformation and Force Controlled Elements

Elements can be classified as either deformation controlled or force controlled. A deformation controlled element is one that has an associated deformation that is allowed to exceed yield value, that is, the maximum associated deformation of the element is limited by the ductility of the element. A force controlled element is one where the maximum associated displacement is not allowed to exceed yield value. Elements with limited ductility shall be considered to be force controlled. See Table 15-8 for calculation of element capacities used to compare with demands.

15.4.2.5 Stiffness and Strength Assumptions

Element and component stiffness properties and strength assumptions for most material types are provided in FEMA-273. Guidelines for structural and foundation elements are also provided. These are similar to those provided in ATC-40.

15.4.2.6 Foundation Modeling

Foundation modeling assumptions are similar to ATC-40 (See Section 15.3.5). The foundation system may be included in the model for analysis with stiffness and damping properties as defined in Chapter 4 of FEMA-273. Otherwise, unless specifically prohibited, the foundation may be assumed to rigid and not included in the model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Deformation Controlled</th>
<th>Force Controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Linear Procedures</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Existing Material Strength</td>
<td>Mean value with allowance for strain hardening</td>
<td>Lower bound (Mean – Std Dev)</td>
</tr>
<tr>
<td>Existing Capacity</td>
<td>$m \kappa Q_{CE}$</td>
<td>$\kappa Q_{CE}$</td>
</tr>
<tr>
<td>New Material Strength</td>
<td>Mean value</td>
<td>Specified value</td>
</tr>
<tr>
<td>New Capacity</td>
<td>$Q_{CE}$</td>
<td>$Q_{CL}$</td>
</tr>
<tr>
<td><strong>Nonlinear Procedures</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deformation Capacity – Existing Element</td>
<td>$\kappa x$ Deformation limit</td>
<td>N/A</td>
</tr>
<tr>
<td>Deformation Capacity – New Element</td>
<td>Deformation limit</td>
<td>N/A</td>
</tr>
<tr>
<td>Strength Capacity – Existing Element</td>
<td>N/A</td>
<td>$\kappa Q_{CL}$</td>
</tr>
<tr>
<td>Strength Capacity – New Element</td>
<td>N/A</td>
<td>$\kappa Q_{CL}$</td>
</tr>
</tbody>
</table>

$\kappa = $ Knowledge factor

$m = $ Demand Modifier for expected ductility

$Q_{CE} = $ Expected Strength

$Q_{CL} = $ Lower Bound Estimate of Strength

15.4.2.7 Diaphragms

Diaphragms transfer earthquake induced inertial loads to the vertical elements of the seismic framing system. Connection between the diaphragms and the vertical elements of the lateral load resisting system must have sufficient strength to transfer the maximum calculated inertial loads. Diaphragms may be flexible, stiff or rigid. Flexible diaphragms are those where the maximum lateral deformation of the diaphragm is more than twice the average inter-story drift of the story below the diaphragm. Rigid diaphragms are those where the maximum lateral deformation of the diaphragm is less than half the average inter-story drift of the associated story. Diaphragms that are neither rigid nor flexible can be considered to be stiff.

Mathematical models of buildings with stiff or flexible diaphragms must consider the effect of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the
vertical lines seismic framing may be designed independently with seismic masses assigned on the basis of tributary areas.

15.4.2.8 P-Delta Effects

For linear procedures, at each story the quantity $\theta_i$ shall be computed for each direction of response as follows:

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \quad (15-12)$$

Where $P_i$ is the portion of the total weight of the structure including dead, permanent line and 25% of the transient live loads acting on the columns and load bearing walls. $V_i$ is the total calculated shear force, $h_i$ is the story height and $\delta_i$ is the lateral drift in the direction under consideration at story $i$.

For linear procedures, the story drifts $\delta_i$ must be increased by $1/(1-\theta_i)$ for evaluation of the stability coefficient, $\theta_i$. Therefore, the process is iterative. If the stability coefficient, $\theta_i$ is less than 0.1, the static P-Delta effects are small and can be ignored. If the stability coefficient, $\theta_i$ is greater than 0.33, the structure is unstable. If it lies between 0.1 and 0.33 than the seismic forces at level $i$ must be increased by $1/(1-\theta_i)$.

For non-linear procedures, these second order effects must be directly included in the model by use of geometric stiffness of all elements subject to axial loads. Dynamic P-Delta effects are included in the LSP and NSP by use of Coefficient $C_j$ (See Section 15.4.3.1 and 15.4.3.3).

15.4.2.9 Soil Structure Interaction

Soil Structure Interaction (SSI) may modify the seismic demand on the structure. To include SSI, one may use the effective fundamental period and effective damping ratios of the foundation-structure system to compute seismic demand or explicitly model SSI. SSI effects shall not be used to reduce component and element actions by more than 25%.

15.4.2.10 Multidirectional Effects

Buildings should be designed for seismic forces in any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act non-concurrently in the direction of each principle axis of the building. For buildings with plan irregularities and buildings with intersecting elements, multidirectional effects must be considered. An acceptable procedure is use of 100% of the seismic force in one horizontal direction and 30% of the seismic force in the perpendicular direction. Alternately SRSS may be used to combine forces in orthogonal directions.

Vertical excitation of horizontal cantilevers and pre-stressed elements must be considered. Vertical shaking characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum is acceptable where site-specific data is not available.

15.4.2.11 Load Combinations

The component gravity loads to be considered for combination with seismic loads are:

When effects of gravity and seismic loads are additive:

$$Q_g = 1.1(Q_d + Q_L + Q_S) \quad (15-13)$$

When the effects of gravity counteract seismic loads

$$Q_g = 0.9Q_d \quad (15-14)$$

Where $Q_d$, $Q_L$ and $Q_S$ are dead, live and snow loads respectively. Effective live loads may be assumed to be 25% of the unreduced live load but not less than measured live loads. Effective snow loads are 70% of the full design snow loads or an approved percentage by a regulatory agency.

Combination with earthquake loads is discussed in subsequent sections. Note such load combinations are relevant for linear analysis. Non-linear analysis is not conducive to checking both of the above load
combinations and therefore only the critical load combination (by inspection) may be used.

### 15.4.3 Analysis Procedures

#### 15.4.3.1 Linear Static Procedure

In this procedure a linear elastic model is used in the analysis with an equivalent damping that approximates values expected for loading near the yield point. A pseudo-lateral load is computed as shown in the following section and applied to the model. The resulting forces and displacements in the elements are then checked against capacities modified to account for inelastic response demands.

#### 15.4.3.1.1 Pseudo Lateral Load

To compute the pseudo lateral load, the fundamental period must be first determined. The period may be determined by one of the following methods:

1. Eigenvalue value analysis of the building. For buildings with flexible diaphragms, the model must consider representation of diaphragm flexibility unless it can be shown that the effects of the omission will not be significant.

2. Use of the following equation

\[
T = C_i h^3/4_i
\]  
(15-15)

Where \( T \) is the fundamental period in seconds under the direction under consideration and \( h_i \) is the height above the base to the roof.

- \( C_i = 0.035 \) for steel moment resisting frames.
- \( C_i = 0.030 \) for moment resisting frame system of concrete and eccentrically braced steel frames.
- \( C_i = 0.020 \) for all other framing systems.
- \( C_i = 0.060 \) for wood buildings.

3. For one-story buildings with flexible diaphragms:

\[
T = (0.1\Delta_w + 0.078\Delta_d)^{0.5}
\]  
(15-16)

Where \( \Delta_w \) and \( \Delta_d \) are in-plane wall and diaphragm displacements in inches due to a lateral loads in the direction under consideration equal to the weight tributary to the diaphragm. For multiple span diaphragms, a lateral load equal to the gravity weight tributary to the span under consideration can be applied to each span to calculate a separate period for each diaphragm span. The period so calculated that maximizes the pseudo lateral load is to be used for the design of all walls and diaphragm spans in the building.

The total pseudo lateral load, \( V \) in a given horizontal direction is determined as

\[
V = C_1 C_2 C_3 S_w W
\]  
(15-17)

Where

- \( C_1 \) = Modification factor to relate expected maximum inelastic displacements to displacements calculated for the linear elastic response. \( C_1 \) can be calculated as in Section 15.4.3.3.4 with the elastic base shear substituted for \( V_y \). Alternatively \( C_1 \) may be calculated as follows:
  - \( C_1 = 1.5 \) for \( T < 0.10 \) secs
  - \( C_1 = 1.0 \) for \( T \geq T_0 \) secs

Linear interpolation can be used to calculate \( C_1 \) for intermediate value of \( T \).

\( T \) = Fundamental period of the building in the direction under consideration. For SSI, the effective fundamental period should be used.

\( T_0 \) = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum

- \( C_2 \) = Modification factor to represent the effect of stiffness degradation and strength deterioration on the maximum displacement response. Values for different framing for different performance levels are listed in Table 15-9. Linear interpolation can be used to calculate \( C_2 \) for intermediate value of \( T \).

- \( C_3 \) = Modification factor to represent the increased displacement due to dynamic P-Delta effect. This effect is in addition to P-Delta
described in Section 15.4.2.8. For values of $\theta$ less than 0.1, $C_3$ may be set equal 1.0. For values of $\theta$ greater than 0.1, $C_3$ shall be calculated as $1+5(\theta-0.1)/T$. The maximum value of $\theta$ for all stories shall be used to calculate $C_3$.

$S_a =$ Response spectrum acceleration at the fundamental period and damping ratio of the building in the direction under consideration.

$W =$ Total dead load and anticipated live load as indicated below:
- In storage and warehouse occupancies, a minimum of 25% of the floor live load,
- The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater,
- The applicable snow load,
- The total weight of permanent equipment and furnishings.

Vertical distribution of the base shear $V$ is done by the following:

$$F_x = C_{xx} V \quad (15-18)$$

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>$T=0.1$ second</th>
<th>$T \geq T_0$ seconds</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate Occupancy</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Life Safety</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Collapse Prevention</td>
<td>1.5</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 15-9. Values of Modification Factor $C_2$ $^{(15-2)}$

Framing Type 1 = Structures in which more than 30% of the story shear any level is resisted by components or elements whose strength and stiffness deteriorate during the design earthquake. Such elements and components include: ordinary moment-resisting frames, concentrically braced frames, frames with partially restrained connections, tension only braced frames, unreinforced masonry walls, shear-critical walls and piers, or any combination of the above.

Framing Type 2 = All frames not assigned to Framing Type 1

Linear interpolation is used to estimate values of $k$ for intermediate values of $T$. $C_{xx}$ is the vertical distribution factor, $V$ is the pseudo lateral load from Equation 15-17, $w_i$ is the weight of level $i$, $w_x$ is the weight of the building of any level $x$, $h_i$ is height from the base to floor level $i$ and $h_x$ is height from the base to floor level $x$.

Floor diaphragms are designed to resist the inertial forces developed at the level under consideration and the horizontal forces resulting from offsets or changes in stiffness in the vertical seismic framing elements above and below the diaphragm. The diaphragm inertial force $F_{px}$ at level $x$ is given by

$$F_{px} = \frac{1}{C_1 C_2 C_3} \sum_{i=2}^{n} F_i \frac{w_i}{\sum_{i=2}^{n} w_i} \quad (15-20)$$

Where $F_i$ is the lateral load applied at floor level $i$ as given by Equation 15-18.

The base shear, vertical distribution and forces on the diaphragms for the LSP is not unlike current codes, however force levels and acceptance criterion for the elements in the lateral load resisting systems depend on the desired performance level.

15.4.3.1.2 Acceptance Criteria to satisfy Performance Point requirements

The design forces shall be calculated as per the following:

For Deformation-Controlled Elements -

$$Q_{UD} = Q_{ci} \pm Q_E \quad (15-21)$$

For Force-Controlled Elements -
\[ Q_{UD} = Q_G + \frac{Q_E}{C_1 C_2 C_3 J} \]  
(15-22)

\[ Q_{UF} = Q_G + \frac{Q_E}{C_1 C_2 C_3} \]  
(15-23)

Where \( Q_{UD} \) and \( Q_{UF} \) are the demands due to gravity and earthquake forces for deformation and force controlled elements respectively. \( Q_E \) is the demand due to the earthquake forces described in the previous section and \( J \) is the force delivery reduction factor given by:

\[ J = 1.0 + S_{xs} \]  
(15-24)

\( J \) cannot exceed 2 and \( S_{xs} \) is the short period spectral acceleration parameter for the design spectrum. Alternately, \( J \) can be taken as the smallest demand capacity ratio of the components in the load path delivering force to the component in question.

The capacities of elements must be checked against the demands as follows:

For Deformation-Controlled elements -

\[ m\kappa Q_{CE} \geq Q_{ID} \]  
(15-25)

For Force-Controlled elements -

\[ \kappa Q_{CL} \geq Q_{UF} \]  
(15-26)

Where \( Q_{CE} \) and \( Q_{CL} \) are the expected and lower bound strength of the element or component respectively. \( m \) is the demand modifier to account for the deformation associated with demand at the selected performance level. \( \kappa \) is the knowledge factor to account for uncertainty in capacity evaluations. A value of 0.75 is used for \( \kappa \) when only a minimum knowledge is available and a value of 1.0 can be used when comprehensive knowledge is available for the element or component in question.

The capacities that need to be checked against demands for each element type and material are listed in Chapters 5 to 8 in FEMA-273 together with the demand modifiers, \( m \), for each performance level.

### 15.4.3.2 Linear Dynamic Procedure

The basis, modeling approaches and acceptance criterion for the Linear Dynamic Procedure (LDP) is similar to those described for LSP. The main exception is that the response is obtained from either a linearly elastic response spectrum or a time-history analysis. As with LSP, LDP will produce displacements that are approximately correct, but will produce inertial forces that exceed those that would be obtained in a yielding building.

The response spectrum method uses peak modal responses calculated from an eigenvalue analysis of a mathematical model. The time history method involves a time-step by time-step evaluation of the building response using a discretized record or synthetic record as base motion input. In both the methods, only modes contributing significantly to the response need to be considered. In the response spectrum analysis, modal responses are combined using rational methods to estimate total building response quantities.

#### 15.4.3.2.1 Ground Motion

The ground motion can be characterized by either a linearly elastic response spectrum which may be site specific or a ground acceleration time history which may be recorded or synthesized. In both cases, the ground motion must be appropriately scaled to reflect the hazard level that is associated with the performance level desired (See Table 15-7).

#### 15.4.3.2.2 Response Spectrum Method

All significant modes must be included in the response spectrum analysis such that at least 90% seismic mass participation is achieved in each of the building’s principle directions. Modal damping must reflect the damping inherent in the building at the deformation levels less than yield deformation.

The peak member forces, displacements, story forces, shears and base reactions for each
mode should be combined using SRSS (square root sum of squares) or CQC (complete quadratic combination). It should also be noted that the directivity of the forces is lost in the response spectrum analysis and therefore the combination of forces must reflect this loss.

Multidirectional effects should also be investigated when using the response spectrum analysis.

15.4.3.2.3 Time History Method

All the requirements for response spectrum analysis are also identical for the time history analysis. Response parameters are computed for each time history analysis. If 3 pairs of time histories are used, the maximum response of the parameter of interest shall be used for the design. If seven or more pairs of time histories are used, the average response (of the maximum of each analysis) of the parameter of interest is to be used.

Multidirectional effects can be accounted by using a three dimensional mathematical model and using simultaneously imposed pairs of earthquake ground motions along each of the horizontal axes of the building.

15.4.3.2.4 Acceptance Criteria to satisfy Performance Point requirements

The acceptance criterion for LDP is similar to that described for LSP. However, all deformations and force demands obtained from either the response spectrum or the time history analysis must be multiplied by the product of the modification factors $C_1$, $C_2$ and $C_3$. Force demands on elements of the floor diaphragm need not be increased by these factors. The seismic forces on the diaphragm obtained in the analysis must not be less than 85% than those obtained in LSP (See Equation 15-20).

15.4.3.3 Nonlinear Static Procedure

In the Nonlinear Static Procedure (NSP) the nonlinear load-deformation characteristics of individual elements and components are modeled directly. The mathematical model of the building is subjected to monotonically increasing lateral load until a target displacement is reached or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The nonlinear effects are directly included in the model and therefore the calculated inertial forces are reasonable approximations of those expected during the design earthquake.

The target displacement can be calculated by any procedure that accounts for nonlinear response on displacement amplitude as well as damping effects at the performance point. One such procedure called the Displacement Coefficient Method is described in FEMA 273. ATC-40 also includes this method as an alternative method of finding the performance point. The advantage of this method over the Capacity Spectrum procedure is its simplicity.

The modeling requirements for NSP are similar to those described in ATC-40. The pushover analysis is performed and a curve relating the base shear force and the lateral displacement of the control node are established between 0 and 150% of the target displacement, $\delta$. Acceptance criterion is based on the forces and deformation corresponding to the displacement of the control node equal to $\delta$.

The analysis model must be sufficiently discretized to represent the load-deformation response of each element or component. Particular attention needs to be paid to identifying locations of inelastic action along the length of element or component. Thus, local models of elements or assemblages of elements need to be studied before embarking on the global models.

15.4.3.3.1 Control Node

The control node is usually the center of mass of the roof of the building. The top of the penthouse should not be considered to be the roof. As the displacement of the control node is compared with the target displacement, the choice of the control node is very important.

15.4.3.3.2 Lateral Load Patterns

The lateral load should be applied to building in profiles that approximately bound
the likely vertical and horizontal distribution of
the inertial force in an earthquake. At least two
vertical distributions of lateral loads must be
considered with NSP. Note use of only one load
pattern may not identify potential deficiencies
in the building.

The two lateral load patterns that are
recommended are
1. Uniform Load Pattern: Here the lateral load
may be represented by values of $C_{xi}$ as
given by Equation 15-19.

2. Modal Pattern: Here the lateral load pattern
is consistent with story shear distribution in
a response spectrum analysis where there is
at least 90% mass participation and the
appropriate ground motion is used.

Other appropriate load patterns substantiated by
rational analysis may be substituted for the
above.

15.4.3.3 Period Determination

The effective fundamental period, $T_e$ in the
direction considered can be computed using the
pushover curve obtained in the NSP. A bilinear
representation of the pushover curve is
constructed to estimate the effective lateral
stiffness, $K_e$, and the yield strength of the
building, $V_y$. The effective lateral stiffness can
be taken as the secant stiffness calculated at a
base shear force equal to 60% of the yield
strength (See Figure 15-18).

The effective fundamental period, $T_e$ is
computed as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$  \hspace{1cm} (15-27)

Where $T_i$ and $K_i$ are the initial elastic
fundamental period in seconds and initial
stiffness of the building in the direction under
considered.

It is obvious that to determine the effective
fundamental period, $T_e$, and the target
displacement, $\delta_t$, the pushover curve for the
building is needed.

\[ \delta_t = C_0 C_1 C_s S_y \frac{T^2}{4\pi^2} g \]  \hspace{1cm} (15-28)

Where

$C_0$ = Modification factor to relate the spectral
displacement and likely building roof
displacement. $C_0$ can be calculated using one of
the following

1. The first modal participation factor at the
level of the control node.

2. The modal participation factor at the level
of the control node calculated using a shape
vector corresponding to deflected shape of
the building at the target displacement.

3. The appropriate value from Table 15-10.

$C_1$ = Modification factor to relate maximum
inelastic displacements to displacements
calculated for linear elastic response. $C_1$ may be
calculated as follows:

<table>
<thead>
<tr>
<th>Number of Stories</th>
<th>Modification Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>1.2</td>
</tr>
<tr>
<td>3</td>
<td>1.3</td>
</tr>
<tr>
<td>5</td>
<td>1.4</td>
</tr>
<tr>
<td>10+</td>
<td>1.5</td>
</tr>
</tbody>
</table>

1. Linear interpolation should be used to calculate
intermediate values.
$C_1 = 1.0$ for $T_e \geq T_0$

$C_1 = [1.0 + (R-1) T_0/T_e]/R$ for $T_e < T_0$

Values for $C_1$ need not exceed those given for LSP (See Section 15.4.3.1.1) and in no case is $C_1$ taken less than 1.0.

$T_0 =$ Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

$R =$ Ratio of the elastic strength demand to calculated yield strength coefficient. $R$ can be computed as

$$R = \frac{S_a}{V_y/W} C_0$$

(15-29)

Where $W$ is the dead weight and anticipated live as computed for LSP (See Section 15.4.3.1.1) and $V_y$ is the yield strength determined from the bilinear representation of the pushover curve (See Figure 15-18).

$C_2 =$ Modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values of $C_2$ can be obtained from Table 15-9.

$C_3 =$ Modification factor to represent increased displacements due to dynamic P-Delta effects. For buildings with positive post-yield stiffness, $C_3$ can be set equal to 1.0. For buildings with negative post yield stiffness $C_3$ is given as

$$C_3 = 1.0 + \frac{\alpha(R-1)^{3/2}}{T_e}$$

(15-30)

Where $\alpha$ is the ratio of post-yield stiffness to effective elastic stiffness (See Figure 15-18). $C_3$ need not exceed values calculated for LSP (See Section 15.4.3.1.1).

$S_a =$ Response spectrum acceleration at the effective fundamental period, $T_e$ and damping ratio for the building in the direction under consideration.

For buildings with flexible diaphragms at each floor level, a target displacement can be calculated for each line of vertical framing. Equation 15-28 can be used to determine this target displacement using the effective fundamental period of the line of vertical framing. The general procedures described for NSP are to be used for each line of vertical framing with masses assigned to the mathematical model on the basis of tributary area.

For stiff diaphragms, which are neither rigid nor flexible, any rational procedure can be used to determine target displacements. An acceptable procedure is to multiply the target displacement obtained from Equation 15-28 by the ratio of the maximum displacements at any point on the roof to the displacements of the center of mass of the roof, both computed by a response spectrum analysis of a 3-D model of the building using a design response spectrum. The target displacement thus computed may not be less than those obtained from Equation 15-28 assuming rigid diaphragms. No vertical line of framing can have displacements less than the target displacement. The target displacement should also be modified as per Section 15.4.2.2 to account for system torsion.

Diaphragms are designed for forces computed in LSP (See Section 15.4.3.1.1) or LDP (See Section 15.4.3.2.4)

15.4.3.3.5 Acceptance Criteria to satisfy Performance Point requirements

For deformation-controlled elements, the maximum deformation demand must be less than expected deformation capacity. Procedures for computing expected deformation capacity are specified in Chapters 5 to 8 of FEMA-273 for various elements and materials.

For force-controlled elements, the maximum design forces must be less than the lower bound strengths $Q_{CL}$. Procedures for computing the lower bound strengths are also specified in Chapters 5 to 8 of FEMA-273 for various elements and materials.

15.4.3.4 Nonlinear Dynamic Procedure

The Nonlinear Dynamic Procedure (NDP) uses a dynamic time history analysis of a nonlinear mathematical model. The basis,
modeling approaches and acceptance criterion for the NDP are similar to those of the NSP. With the NDP the design displacements are not established using a target displacement, but determined directly through the dynamic time history analysis. As the analysis can be very sensitive to characteristics of individual ground motions, it is advisable to perform the analysis with more than one ground motion. Ground motions used for the analysis and the analysis procedure should be similar to those used in LDP (See Section 15.4.3.2).

It should be noted that the volume of data generated in NDP is enormous and it is difficult to condense the data to useful performance based design information. Sensitivity analysis to various parameters is also a prerequisite for NDP analysis. Thus, NDP must only be used with caution for very important, irregular and unusual structures.

15.4.4 Example

An example is provided of an analysis of existing building using NSP.

15.4.4.1 Building Description

The example building is a reinforced concrete structure located in California. The building was constructed circa 1962. The structure is irregular in plan, with a footprint similar to a compressed "H". The structure has been divided into the East, West, and Central Wings, as illustrated in Figure 15-19.

The building is situated on a site that slopes to the west. The structure has a total of seven levels, plus two small penthouses. The sloping site introduces significant complexities to the structure. The upper five levels are essentially above grade. The West Wing is a total of seven levels tall, two of which are partially below or below grade, depending on the slope of the site. The East Wing is five levels tall, with a partial basement. A portion of the first level is below grade, due to the sloping site.

Vertical loads are resisted by one-way concrete slabs spanning to reinforced concrete beams and girders. Thicker slabs are used in some areas, including a 17-inch thick "sonovoid" slab, a cast-in-place concrete slab with voids. The sonovoid slabs are located at the ground and first floor. The slabs, beams, and girders are supported by tied and spirally reinforced concrete columns and concrete bearing walls. The columns rest on spread footings, with continuous footings under the perimeter and interior walls.

There are some unusual features in the vertical load-carrying system. Along the north and south exterior walls and the Central Wing, vertical loads are carried by concrete columns outside the building envelope. At the second level, columns are discontinuous and are supported by transfer girders. At the First Floor, the Central Wing relies on massive concrete frames to resist vertical loads.

Figure 15-19. 3-D Linear Model of Example Building

The lateral force-resisting system of the example building consists of the concrete floor and roof slabs, acting as rigid diaphragms and reinforced concrete shear walls. The majority of the shear walls are concentrated around the elevator shafts and stair wells, with additional walls internally and on the building exterior. There are numerous vertical discontinuities in the interior shear walls, especially below the first floor. Most of the shear walls are in the East and West Wings.
15.4.4.2 Performance Objective

In keeping with project requirements, the linear as well as nonlinear analysis and rehabilitation design focused on the Basic Safety Objective. In the nonlinear static analysis, the building is pushed to the target displacement for the BSE-1 and BSE-2 level earthquakes.

15.4.4.3 Mathematical Modeling

The nonlinear analysis of the example building was performed using NLPUSH, the nonlinear module to SAP2000. The concrete shear walls were modeled using column elements. P-M interaction diagrams were generated for each column element. The column elements have stiffness in the strong axis computed based on the stiffness of the actual wall. Weak axis stiffness was assumed to be negligible. As NLPUSH requires the interaction surface to be input for both directions of bending, the wall is assumed to have the same moment capacity in both directions of strong axis bending. The gravity frames have been identified as secondary elements, and representative frames have been explicitly modeled to monitor the demands on the gravity load-carrying system. The diaphragms have been assumed to be rigid.

Potential failures in shear and flexure are considered in the analytical model. The wall and column elements have flexural hinges input at the top and bottom of the element at a distance of 0.05 times the element length from each end. Shear hinges are input at mid-height of the element. Because of numerical convergence problems, the column and wall elements had to be split into three segments with one hinge per segment. The flexure hinges are assigned to the top and bottom segments, and the shear hinge to the central segment. Wall elements with flanges are uncoupled and treated as separate walls, with the effective flange width assigned individually to the two walls.

Beams and coupling beams are modeled as frame elements with flexure or shear hinges depending which is the governing mode of failure. Full height walls spanning between walls or columns are connected by stiff unyielding elements.

Values for effective stiffness of the structural elements for the initial analysis are taken from Table 6-4 of FEMA 273. The stiffness for walls is the cracked stiffness, with a flexural rigidity of $0.5E_cI_c$. The columns are assumed to be in compression with a flexural stiffness $0.7E_cI_c$. The beams are non-prestressed and have an initial stiffness of $0.5E_cI_c$. The shear stiffness is included for columns, beams and walls as $0.4E_cA_w$.

The mathematical model of the building was subjected to monotonically increasing lateral forces until either the target displacement is reached or until the model became unstable. Because the building is not symmetric about any plane, the lateral loads were independently applied in both positive and negative directions.

The relationship between the base shear and lateral force was established for displacements ranging between 0 and 150% of $\delta_t$, where $\delta_t$ corresponds to the target displacement for the BSE-1 earthquake. Two lateral load patterns were applied to the structure. The uniform load pattern was applied using lateral loads that are proportional to the mass at each floor. The dynamic load pattern was applied, using a lateral load pattern similar to the story shear distribution calculated by combining the modal responses from a response spectrum analysis with sufficient number of modes to capture 90% of the mass. Foundation flexibility was not expected to be a significant factor in the nonlinear analysis of the building.

15.4.4.4 Target Displacement

The mapped short period response acceleration parameter, $S_5$ and the modified mapped response acceleration parameter at one second period, $S_1$, for the given site are obtained from the maps provided with FEMA 273. These maps are the Probabilistic Earthquake Ground Motion maps for California/Nevada for the 0.2 seconds and 1.0 second Spectral Response Acceleration (5% of Critical Damping) with 10% probability of exceedence in 50 years. The values obtained for the example site are:
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$S_5 = 1.5g$ and $S_1 = 0.75g$

These values adjusted for Site Class C from Tables 2-13 and 2-14 of FEMA-273 give the design short period spectral response acceleration parameter, $S_{X5}$ and design spectral response acceleration parameter, $S_{X1}$ as:

$S_{X5} = 1.5 \times 1.0 = 1.5g$

$S_{X1} = 0.75 \times 1.0 = 0.975g$

The period $T_0$ of the general response spectrum curve at an effective damping of 5% is:

$$T_0 = \frac{S_{X1} B_5}{S_{X5} B_1} = \frac{0.975}{1.5} = 0.65 \text{ seconds}$$

Where $B_5$ and $B_1$ are 1.0 from Table 2-15 of FEMA-273.
The period of the building is less than 0.65, thus the spectral acceleration, $S_a$, for the site falls in the constant acceleration part of the spectrum, and is equal to 1.5g.

The target displacement is calculated using:

$$C_0 = 1.3 \text{ from Table 15-10, as the lower level is very stiff compared with the rest of the structure.}$$

$$C_2 = 1.0 \text{ from Table 15-9 for framing Type 2.}$$

$$C_3 = 1.0 \text{ for positive post yield stiffness assumed.}$$

$W = 38,064 \text{ kips}$

$T_i = 0.65 \text{ seconds}$

$T_e = 0.41 \text{ seconds in East-West direction}$

$\delta_t = 5.37 \text{ inches}$

For the East-West Direction for $V_e = 7,200 \text{ lbs}$ from Figure 15-23:

$$R = \frac{S_a}{V_e/W} C_0 = \frac{1.5}{7,200/38,064} \times \frac{1}{1.3} = 6.1$$

$$C_1 = \left[ 1 + \frac{(R - 1)}{R} \frac{T_i}{T_e} \right] \frac{1}{R} = \left[ 1 + \frac{(6.1 - 1)}{6.1} \right] \frac{1}{6.1} = 1.49$$

This value is reduced to the maximum value of $C_1$ in Section 15.4.3.1.1, which is 1.28 (interpolated for $T_e = 0.41 \text{ seconds}$). Thus:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g = 1.3 \times 1.28 \times 1 \times 1.5 \times 0.41^2 \times 4\pi^2 \times g = 4.11 \text{ inches}$$

Similarly for the North-South direction:

$V_y = 6,400 \text{ lbs from Figure 15-24}$

$R = 6.86$

$C_1 = 1.33$

$\delta_t = 5.37 \text{ inches}$

Thus, using Equation 15-28, the target displacements for the North-South and East-West directions was determined to be 4.11 inches, and 5.37 inches respectively. The pushover analysis has been continued for 1.5 times the target displacements for collapse prevention.

15.4.4.5 Analysis Results

Pushover analyses were performed for the positive and negative North-South and East-West directions of the building. The pushover curves were not able to achieve the target displacement even for the Life Safety
acceptance criteria for BSE-1 in the East-West and North-South directions.

The maximum displacement reached and the type and number of hinges formed for the various pushover analyses performed was recovered. From the results of the pushover analyses, it was seen that the Modal pattern is more detrimental to this building as more number of hinges were formed for a given displacement level compared to the Uniform pattern. This also goes to show that the lower floors of this building are relatively stronger than the upper floors. However this building in its existing configuration was unable to achieve its target displacement. The building could only be pushed to a displacement of 2.8” in the negative East-West direction and 4.34” in the negative North-South direction.

The analyses also revealed a number of columns supporting walls above to have rotations beyond collapse. Many of the walls and beams also had plastic rotations beyond the Life Safety requirement at the target displacement. Some of the columns in the central wing had shear failures under the uniform pattern for push in the East-West direction. Clearly, this building does not meet the acceptance criteria of the basic safety objective, and therefore needs retrofit.

15.5 Conclusions

The principal advantage of PBSE is that the choice of performance goals lies with the owner who can decide the acceptable damage state. The engineer can also convey to the owner a better understanding of the expected damage state. PBSE does not eliminate the risks associated with uncertainties in ground motions, material properties, element behavior or geotechnical properties. However, it provides a new technique to remove unnecessary conservatism for some parameters and discover unidentified deficiencies for others. If implemented correctly and competently, PBSE can produce a design that is more reliable than traditional procedures.

One very useful characteristic of the ATC-40 and FEMA 273/274 documents is that they provide a step-by-step approach for PBSE. This is an important first step towards a building code implementations of performance based design.

There are some weaknesses that need to be addressed with additional research. Three broad areas need work:
1. A more reliable and conservative methodology, which is widely accepted, needs to be developed for establishing the performance point. More accurate equations need to be developed to find the effective damping or equivalent ductility used to reduce the design response spectra to levels consistent with observed structural behavior.
2. More sophisticated computer analysis programs are needed which can do nonlinear analysis of concrete/masonry/plywood shear walls, concrete and steel joints, confined concrete sections, etc. There is also a need to reduce the data to a finite number of parameters than can be used for design.
3. The element capacities and deformations limits for various performance levels are currently based on engineering judgment or relatively small number of experiments. More experimental and theoretical work is needed to establish reliable element capacities and deformation limits for given performance objectives.

REFERENCES


